Supporting Report (G)

Structural Design and Cost Estimate

PREPARATORY SURVEY
FOR
THE PROJECT ON THE DISASTER PREVENTION
AND
MITIGATION MEASURES FOR THE ITAJAI RIVER BASIN

FINAL REPORT

VOLUME III : SUPPORTING REPORT
ANNEX G : STRUCTURAL DESIGN AND COST ESTIMATE

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CHAPTER 1 INTRODUCTION

This chapter deals with a construction plan and cost estimate on the proposed projects at master plan study and feasibility study as the fowling structural measures.

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- Heightening of the Sul dam (Spillway section)
- Widening Dyke
- Basin Storage (small dams)
- New Flood Control dam
- Ring dyke
- Floodway
- Composite Section
- Floodgate

Feasibility Study as structural measures

- Heightening of the Oeste dam (Non-overflow and Spillway section)
- Heightening of the Sul dam (Spillway section)
- Floodgate

CHAPTER 2 PRELIMINARY DESIGN OF STRUCTURAL MEASURES OF THE MASTER PLAN

2.1 Flood Disaster Mitigation Measures

A preliminary structural design was carried out for the facilities proposed in the master plan. Due to the delay of ongoing topographical mapping with a scale of 1:10,000 by SDS and lack of geological information at the sites of facility, field investigation site conditions such as topography and geology for the design were assumed based on the field investigation as much as possible. As for the existing flood control dams, their structural dimensions were referred to the available old structural drawings. In addition, as no data is available on the geology of dam foundation, shear strength and bearing capacity of the foundation necessary for the design were determined based on the assumption that the current dams satisfy all of the stability conditions from the viewpoints of dam safety. The design criteria in Brazil titled “HYDROELECTRIC POWER PLANTS CIVIL DESIGN CRITERIA, October/2003, ELETROBRÁS” was applied to this preliminary structural design.

2.2 Heightening of Existing Flood Control Dams

(1) Selection of Heightening Method

The following table presents the criteria for setting dam height in Brazil.

Table 2.2.1 Criteria for Setting Dam Height in Brazil

<table>
<tr><th>Item</th><th>Condition</th><th>Dam Type/Flood</th><th>Criteria</th></tr>
<tr><td rowspan="4">Freeboard</td><td rowspan="2">Normal</td><td>Rock fill dam</td><td>The freeboard shall be defined to absorb wave height caused by wind. The wave height shall be estimated by the Saville method. At least 3.0 m shall be secured as the minimum freeboard.</td></tr>
<tr><td>Concrete dam</td><td>At least 1.5 m shall be secured as the minimum freeboard.</td></tr>
<tr><td rowspan="2">Flood</td><td>Rock fill dam</td><td>The minimum freeboard shall be secured 1.0 m above the maximum flood water level in reservoir.</td></tr>
<tr><td>Concrete dam</td><td>The minimum freeboard shall be secured 0.5 m above the maximum flood water level in reservoir.</td></tr>
<tr><td rowspan="2">Extraordinary flood</td><td>Normal</td><td>Probable maximum flood</td><td>For dam higher than 30 m, or there are permanent residents downstream and danger of dam failure</td></tr>
<tr><td>Small scale dam</td><td>1000-year flood</td><td>For dam lower than 30 m, or reservoir capacity of smaller than 50 million m^3 and there are no permanent residents downstream.</td></tr>
</table>

Source: Criteria for civil projects of Hydroelectric Power Plants, Eletrobrás – October/2003.

The Oeste dam shall be provided with the spillway to pass safely the 1000-year flood (=1,010 m^3/s), as its height is less than 30 m and there is no residents in the immediately downstream of the dam. On the other hand, the Sul dam shall be equipped with the spillway for passing the 10000 - year flood (=2,570 m^3/s) due to its height over 30 m.

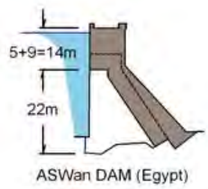
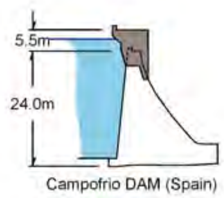
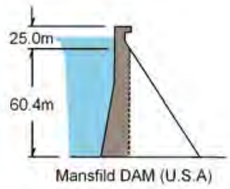
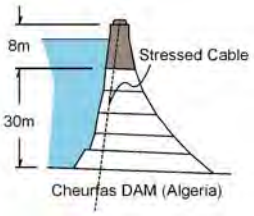
The Oeste dam is a concrete gravity dam, corresponding to the dam type to easy to be raised. As the dam is planned to be raised by 2 m at both the overflow and non-overflow portions, the form of existing spillway is just to be slid upward. The Sul dam is a rock fill dam of the zoned type. In case of heightening of the fill type dam, problems on the behavior of new and old joints of embankment have frequently occurred after the heightening, because it is difficult to ensure the quality of embankment materials. The fill type dam has generally smaller rock strength at the dam foundation compared to that of gravity dam, the maximum possible height for raising is therefore small for the fill type dam. Since it was difficult to confirm the conditions of foundation and embankment materials, it was decided not to raise the Sul dam. However, as the existing spillway allows to ensure sufficient freeboard for raising by 2 m, it was decided to raise the spillway (concrete structure).

(2) Heightening of Oeste Dam

1) Mode of Heightening of Concrete Dam

The table below presents the comparison of methods for heightening concrete gravity dams. As the planned heightening is as small as 2 m at the Oeste dam, raising the dam crest was selected.

Table 2.2.2 Heightening Method of Concrete Gravity Dam

Covering of New Dam	Raising of Dam Crest	Thickening of Upstream Dam Body	Anchoring
			
Placing new concrete on the downstream face of existing dam and forming unified dam body of the new and old concretes	Placing new concrete on the dam crest and forming unified dam body of the new and old concretes	Placing new concrete on the upstream face of the existing dam and forming unified body of the new and old concretes	Placing new concrete on the dam crest and connecting to the upstream dam foundation by stress cable

Source：JICA Survey Team

2) Design Condition

a. Criteria

The design criteria is applied for the "CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003 in Brazil".

b. Dimension of Oeste dam

The typical drawings of the Oeste dam is shown Figure 2.2.10. The dimensions of that was unclear that the several filed observation was implemented to deicide the dimensions.

c. Study Case

According to the Brazilian criteria, stability of dam shall be confirmed by the following four loading conditions:

Table 2.2.3 Loading Conditions for Dam Stability Analysis

Condition		Remarks
Normal	(CCN)	Normal
Excepcional	(CCE)	Normal＋Earthquake
Limite	(CCL)	Flood＋Earthquake
Construção	(CCC)	During Construction

Source：CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

d. Safety Factor

Safety factors for stability analysis vary according to the loading conditions as presented below.

Table 2.2.4 Safety Factors for Stability Analysis by Loading Condition

Condition	CCN	CCE	CCL	CCC
FSF (Uplift)	1.3	1.1	1.1	1.2
FST (Turnover)	3.0	2.0	1.5	1.3

FSD (Sliding)	c	3.0	1.5	1.3	2.0
	φ	1.5	1.1	1.1	1.3
σt (Bearing Capacity)		3.0	2.0	1.5	1.3

Source：CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

As mentioned earlier, as no data is available on the geology of dam foundation, shear strength and bearing capacity of the foundation necessary for the design were determined based on the assumption that the current dams satisfy all of the stability conditions from the viewpoints of dam safety. In addition, an internal friction angle was fixed in φ=45° as the design value of foundation rock.

The table below shows the combination of loads for respective stability analysis.

e. Equation of Stability of Calculation

The four(4) safety calculations are as the following equations

Lifting	$FSF = \dfrac{\Sigma V}{\Sigma U}$	Sliding	$FSD = \dfrac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} \geq 1.0$
Overturning	$FST = \dfrac{\Sigma M_e}{\Sigma M_t}$	Bearing Capacity	$e = \dfrac{L}{2} - \dfrac{M_e - M_t}{\Sigma V}$ $q_{(u,d)} = \dfrac{\Sigma V}{L} \cdot \left(1 \pm \dfrac{6 \cdot e}{L}\right)$

Source：CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

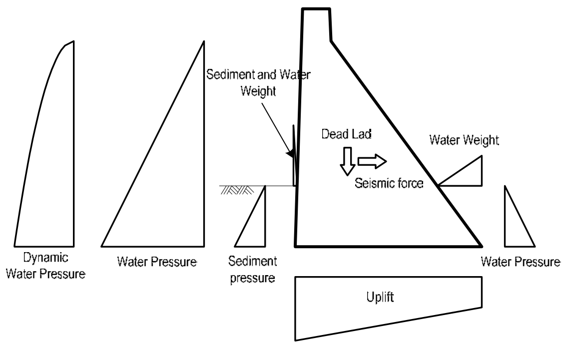
f. Combination of Loads Condition

The table below shows the combination of loads for respective stability analysis.

Table 2.2.5 Combination of Loads for Stability Analysis

Load	CCN	CCE	CCL	CCC
Own weight	Yes	Yes	Yes	Yes
Water weight	Yes	Yes	Yes	—
Dynamic pressure by earthquake	—	—	Yes	—
Earthquake force	—	—	Yes	—
Water pressure	Yes	Yes	Yes	—
Uplift pressure	Yes	Yes	Yes	—
Sediment weight	Yes	Yes	Yes	—
Sediment pressure	Yes	Yes	Yes	—

Source：JICA Survey Team



Source: JICA Survey Team

Figure 2.2.1 Load Diagram

g Basic Condition

- Unite Weight

The Physical property for stability analysis is normally decided in view of the local region characters. At moment, since there were neither calculation sheets nor the geological survey data, the typical figure is applied.

Table 2.2.6 Unite Weight

Item	Unit Weight (kN/m^3)	Remarks
Concrete	23.5	
Water	10.0	
Sediment(Under Water)	8.5	=17.5-9.0

Source：CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

- Seismic Factor

Seismic force is based on the formula in the below.

$Fh = 0.05 \cdot P$ (Horizontal)

$Fv = 0.03 \cdot P$ (Vertical)

Inertial force acting on the structure is based on the coefficient in the below table.

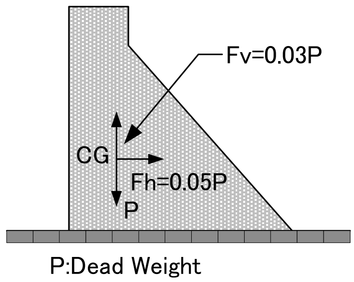


Figure 2.2.2 Diagram of Seismic Factor

Table 2.2.7 Seismic Coefficient

	Modulus	Remarks
horizontal	F_h = 0.05	
vertical	F_v = - 0.03	upper direction

Source：JICA survey team

- Rankine's Earth Pressure Coefficient

The earth pressure is calculated by Rankine's earth pressure factor. The sediment in the dam is supposed as the cohesive soil and the angle of internal friction is 25°.

$$Ka = \frac{1-\sin\phi}{1+\sin\phi} = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{25}{2}\right) \fallingdotseq 0.4$$

$$Pe = \frac{1}{2} \cdot Ka \cdot \gamma \cdot h^2 \ (kN/m)\ ,\ ye = \frac{h}{3}\ (m)$$

The height of the sediment at upstream is EL. 338.5 m as the height would be raised at the future.

- Dynamic Water Pressure

Dynamic water pressure acting on the structure is based on the formula below. Westergaard formula is applied.

$$p_d = \frac{7}{8} \cdot W_0 \cdot K_d \cdot \sqrt{H \cdot h} \ (kN / m^2)$$

$$P_d = \int \frac{7}{8} \cdot W_0 \cdot K_d \cdot \sqrt{H \cdot h} \cdot dh = \frac{7}{12} \cdot W_0 \cdot K_d \cdot \sqrt{H} \cdot h^{3/2} \ (kN / m)$$

$$yd = 0.4 \cdot h \ (m)$$

Notes:
P_d : dynamicwater pressure (kN)
W_0 : unit water weight (kN/m^3)
K_h : Seismic factor
H : Depth of the water reservoir at base point (m)
h : Depth of the water reservoir at any point (m)

y_d : Working point height (m)

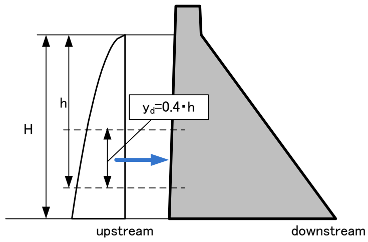


Figure 2.2.3 Diagram of Dynamic Water Pressure

- Water Pressure

Water pressure is based on the formula in the below.

$$P = W_0 \cdot h \ Y_w = \frac{1}{3} \cdot h$$

P:Waterpressure (kN/m^2), W_0:water unit weight, h:water level, Y_w: point of application

- Design Water Level

Water level is for stability analysis is two cased as below.

・ At the last point to start overflow

・ Ordinary discharge

The discharge at the last point to start overflow is the outflow discharge at water leverl EL.360.0 m. The ordinary discharge is calculated by the catchment area at point of Oeste times the specific discharge which is observed at Taio City.

Table 2.2.8 Design Water Level (Oeste Dam)

Load Condtion	Upstream WL.	Downstream WL.	Remarks
CCN	341.50 m	337.50 m	Q=28 m^3/s
CCE	341.50 m	337.50 m	
CCL	362.50 m	341.95 m	Q=163 m^3/s (EL 360.00)
CCC	---	---	

Souce: JICA Survey Team

(Ordinary Discharge)

The Ordinary discharge at the Oeste dam is calculated by converting the basin scale with the average of water level at Taio city (75 years data). The ordinary discharge is Q = 28.0 m^3/s.

(Water Level at originally)

The ordinary water level at downstream is EL. 337.50 m as the critical depth at the counter dam of the energy dissipater.

$$hc = \sqrt[3]{\frac{Q^2}{g \cdot B^2}} = \sqrt[3]{\frac{27.4^2}{g \cdot 100^2}} = 0.197 \cong 0.20m$$

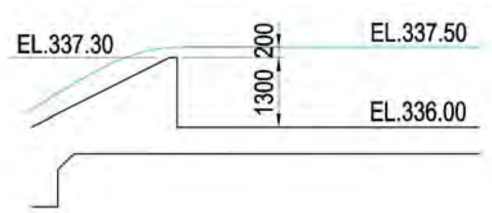


Figure 2.2.4 Water Level at Downstream (Oeste Dam)

(Flood Discharge)

The discharge curve of conduit for flood control is calculated as below equations.

Conduit for flood control (Existing) ;	$Q = 0.6667 \times 7 \times 1.7663 \cdot \sqrt{2 \cdot g \cdot (360 - 340.05)} = 163.0\ m^3/s$
Conduit for flood control (Heightening) ;	$Q = 0.6667 \times 7 \times 1.7663 \cdot \sqrt{2 \cdot g \cdot (362 - 340.05)} = 171.0\ m^3/s$

(Water Level at Flood)

The water level at flood is calculated by uniform flow with the calculated discharge.

Table 2.2.9 Result of Uniform Flow（Oeste River）

Oeste dam		Existing	Heightening
Grand Level	EL.m	336.00	336.00
Water Level	EL.m	338.00	338.05
River width	m	100	100
Water height	m	2.000	2.050
Side Slope (1:n)		1.00	1.00
Roughness Modules		0.0320	0.0320
Bed Slope (i)		1/3600	1/3600
Flow Area	m^2	204.00	209.20
Hydraulic Radius	m	1.93	1.98
Velocity	m/s	0.808	0.821
Discharge	m^3/s	164.7	171.7

Source: JICA Survey Team

- Uplift

The coefficient of uplift is 1/3 because of the foundation of dam is supposed to be the rock.

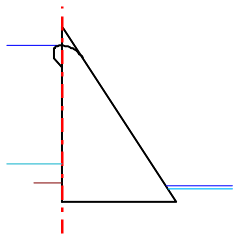
iii) Stability Analysis of Existing Oeste dam

As mentioned earlier, there are no geology date of dam foundation available, the case of existing dam is calculated to estimate the physical properties. The result of analysis, the angle of internal friction and shearing stress are φ=45° and c=50 kN/m^2 is satisfied the result. The definitive loading condition is CCL(Flood＋Earthquake). The critical bearing capacity of foundation ground is required qu=1900 kN/m^2.

（Calculation Condition）

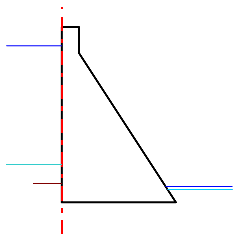
1.	Elevation of Top of Dam	H_O＝	363.000 m
2.	Downstream Slope	1 : n	0.750
3.	Dam base elevation	H_O ＝	335.500 m
4.	Crest width of non-overflow section	B＝	3.000 m
5.	Upper surface of the downstream slope	1 : o1	0.000
6.	Reservoir sediment level	H_D＝	338.500 m
7.	Reservoir water level (CCN: normal)	H_{W1}＝	341.500 m
8.	(CCE: Always + earthquake)	＝	341.500 m
9.	(CCL: flood + earthquake)	＝	360.000 m
10.	Downstream water level (CCN: normal)	H_{W2}＝	337.500 m
11.	(CCE: Always + earthquake)	＝	337.500 m
12.	(CCL: flood + earthquake)	＝	338.000 m
13.	Unit weight of concrete dams	γ_c＝	23.5 kN/m^3
14.	Weight of sediment in the water	γ_s＝	8.5 kN/m^3
15.	Unit weight of water	γ_w＝	10.0 kN/m^3
16.	Seismic Coefficient: Horizontal (kh)	K_h＝	0.050
17.	Seismic factor: vertical (kv)	K_v＝	0.030
18.	Coefficient of earth pressure (Rankine coefficient of earth pressure)	ka＝	0.40
19.	Uplift pressure coefficient	μ＝	1/3
20.	Shear strength of foundation	c＝	50.0 kN/m^2
21.	Friction angle of foundation	ϕ'＝	45.00 °
22.	Internal friction coefficient	f＝	1.00

Fig. Calculation Model



(Non Overflow Section)

Fig. Calculation Model



(Spillway Section)

(Result) Non-overflow Section

Table 2.2.10 Analysis Result of Non-overflow Section

	FSF	FST	FSD ≧ 1.0
[CCN]	12.41 > 1.30	113.84 > 1.50	25.81 ≧ 1.0
[CCE]	12.03 > 1.10	13.96 > 1.20	40.16 ≧ 1.0
[CCL]	5.21 > 1.10	1.18 > 1.10	1.62 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	629.85 ≤ 30M/3.0=10M	-21.80 ≥ -200
[CCE]	655.12 ≤ 30M/2.0=15M	-66.87 ≥ -200
[CCL]	133.67 ≤ 30M/1.5=20M	385.39 ≥ -200
[CCC]	669.67 ≤ 30M/1.3=23M	-9.74 ≥ -200

Source：JICA survey team

(Result) Spillway Section

Table 2.2.11 Analysis Result of Spillway Section

	FSF	FST	FSD ≧ 1.0
[CCN]	12.12 > 1.30	111.48 > 1.50	25.22 ≧ 1.0
[CCE]	11.76 > 1.10	14.67 > 1.20	41.27 ≧ 1.0
[CCL]	5.09 > 1.10	1.16 > 1.10	1.59 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	611.55 ≤ 30M/3.0=10M	-18.67 ≥ -200
[CCE]	634.73 ≤ 30M/2.0=15M	-61.19 ≥ -200
[CCL]	118.51 ≤ 30M/1.5=20M	385.84 ≥ -200
[CCC]	651.37 ≤ 30M/1.3=23M	-6.61 ≥ -200

Source：JICA survey team

iv) Stability for Heightening at Oeste dam

Only heightening the top of the dam is not enough for the stability in view point of raising 2.0 m water level. The countermeasure is proposed to lay the mat concrete at the foot of sloop. The detail figure showed on Figure 2.2.10.

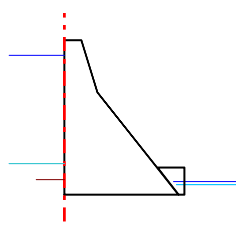
The definitive condition is that the angle of internal friction and shearing stress are φ=45° and c=50 kN/m^2 and loading condition is CCL(Flood＋Earthquake). The critical bearing capacity of foundation ground is requied qu=2,000 kN/m^2.

- Non – overflow section

（Calculation Condition）

1.	Elevation of Top of Dam	H_O =	365.000 m
2.	Downstream Slope	1 : n	0.750
3.	Dam base elevation	H_O =	335.500 m
4.	Crest width of non-overflow section	B =	3.000 m
5.	Upper surface of the downstream slope	1 : o1	0.300
6.	Reservoir sediment level	H_D =	338.500 m
7.	Reservoir water level (CCN: normal)	H_{W1} =	341.500 m
8.	(CCE: Always + earthquake)	=	341.500 m
9.	(CCL: flood + earthquake)	=	362.000 m
10.	Downstream water level (CCN: normal)	H_{W2} =	337.500 m
11.	(CCE: Always + earthquake)	=	337.500 m
12.	(CCL: flood + earthquake)	=	338.050 m
13.	Unit weight of concrete dams	γ_c =	23.5 kN/m^3
14.	Weight of sediment in the water	γ_s =	8.5 kN/m^3
15.	Unit weight of water	γ_w =	10.0 kN/m^3
16.	Seismic Coefficient: Horizontal (kh)	K_h =	0.050
17.	Seismic factor: vertical (kv)	K_v =	0.030
18.	Coefficient of earth pressure (Rankine coefficient of earth pressure)	ka =	0.40
19.	Uplift pressure coefficient	μ =	1/3
20.	Shear strength of foundation	c =	50.0 kN/m^2
21.	Friction angle of foundation	ϕ' =	45.00 °
22.	Internal friction coefficient	f =	1.00

Fig. Calculation Model



(Non Overflow Section)

(Result)

Table 2.2.12 Analysis Result of Non-overflow Section

	FSF	FST	FSD ≧ 1.0
[CCN]	13.04 > 1.30	134.35 > 1.50	28.46 ≧ 1.0
[CCE]	12.65 > 1.10	13.97 > 1.20	35.91 ≧ 1.0
[CCL]	5.16 > 1.10	1.11 > 1.10	1.53 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	655.51 ≤ 30M/3.0=10M	-13.52 ≥ -200
[CCE]	682.58 ≤ 30M/2.0=15M	-61.43 ≥ -200
[CCL]	94.97 ≤ 30M/1.5=20M	448.69 ≥ -200
[CCC]	693.50 ≤ 30M/1.3=23M	1.85 ≥ -200

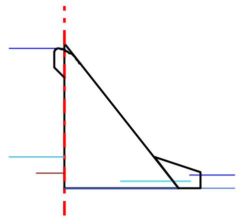
Source：JICA survey team

- Spillway Section

(Calculation Condition)

1.	Elevation of Top of Dam	H_O =	365.000 m
2.	Downstream Slope	1 : n	0.750
3.	Dam base elevation	H_O =	335.500 m
4.	Crest width of non-overflow section	B =	0.000 m
5.	Upper surface of the downstream slope	1 : o1	0.000
6.	Reservoir sediment level	H_D =	338.500 m
7.	Reservoir water level (CCN: normal)	H_{W1} =	341.500 m
8.	(CCE: Always + earthquake)	=	341.500 m
9.	(CCL: flood + earthquake)	=	362.000 m
10.	Downstream water level (CCN: normal)	H_{W2} =	337.500 m
11.	(CCE: Always + earthquake)	=	337.500 m
12.	(CCL: flood + earthquake)	=	338.050 m
13.	Unit weight of concrete dams	γ_c =	23.5 kN/m^3
14.	Weight of sediment in the water	γ_s =	8.5 kN/m^3
15.	Unit weight of water	γ_w =	10.0 kN/m^3
16.	Seismic Coefficient: Horizontal (kh)	K_h =	0.050
17.	Seismic factor: vertical (kv)	K_v =	0.030
	Coefficient of earth pressure		
18.	(Rankine coefficient of earth pressure)	ka =	0.40
19.	Uplift pressure coefficient	μ =	1/3
20.	Shear strength of foundation	C =	50.0 kN/m^2
21.	Friction angle of foundation	ϕ' =	45.00 °
22.	Internal friction coefficient	f =	1.00

Fig. Calculation Model



(Non Overflow Section)

（Result）

Table 2.2.13 Analysis Result of Spillway Section

	FSF	FST	FSD ≧ 1.0
[CCN]	11.08 > 1.30	139.09 > 1.50	27.26 ≧ 1.0
[CCE]	10.75 > 1.10	17.72 > 1.20	37.44 ≧ 1.0
[CCL]	4.38 > 1.10	1.12 > 1.10	1.47 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	568.24≤30M/3.0=10M	-30.72≥-200
[CCE]	581.22≤30M/2.0=15M	-61.46≥-200
[CCL]	120.40≤30M/1.5=20M	321.85≥-200
[CCC]	605.15≤30M/1.3=23M	-14.30≥-200

Source：JICA survey team

v) Conduit Pipes

Since the water level is raised 2.0 m, the conduit pipes is required to reinforce. The winch for gates is thought to replace the whole because of the hydraulic system.

		
Gate	Winch (hydraulic system)	Closing flange

Gates at Oeste dam

i) Design Conditions

According to the Brazilian criteria, stability of dam shall be confirmed by the following four loading conditions:

Table 2.2.14 Loading Conditions for Dam Stability Analysis

Condition	Remarks
Normal (CCN)	Normal
Exceptional (CCE)	Normal＋Earthquake
Limite (CCL)	Flood＋Earthquake
Construção (CCC)	During Construction

Source：CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

Safety factors for stability analysis vary according to the loading conditions as presented below.

Table 2.2.15 Safety Factors for Stability Analysis by Loading Condition

Condition		CCN	CCE	CCL	CCC
FSF (Uplift)		1.3	1.1	1.1	1.2
FST (Turnover)		3.0	2.0	1.5	1.3
FSD (Sliding)	c	3.0	1.5	1.3	2.0
	φ	1.5	1.1	1.1	1.3
σt (Bearing Capacity)		3.0	2.0	1.5	1.3

Source：CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

As mentioned earlier, as no data is available on the geology of dam foundation, shear strength and bearing capacity of the foundation necessary for the design were determined based on the assumption that the current dams satisfy all of the stability conditions from the viewpoints of dam safety. In addition, an internal friction angle was fixed in φ=45° as the design value of foundation rock. The table below shows the combination of loads for respective stability analysis.

Table 2.2.16 Combination of Loads for Stability Analysis

Load	CCN	CCE	CCL	CCC
Own weight	Yes	Yes	Yes	Yes
Water weight	Yes	Yes	Yes	—
Dynamic pressure by earthquake	—	Yes	Yes	—
Earthquake force	—	Yes	Yes	—
Water pressure	Yes	Yes	Yes	—
Uplift pressure	Yes	Yes	Yes	—
Sediment weight	Yes	Yes	Yes	—
Sediment pressure	Yes	Yes	Yes	—

Source : JICA Survey Team

ii) Results of Stability Analysis

The necessary critical bearing capacity of the dam foundation was estimated through stability analyses for two cases of the existing and heightened conditions as summarized below. The details of stability analysis are explained in Supporting Report. Structural drawing for dam heightening is shown in Figure 11.1.3.

Foundation condition assumed: Internal friction angle φ=45°, Shear stress c=50 kiN/m^2

Definitive loading condition: CCL (flood + earthquake)

Critical bearing capacity: q_u=1,900 kN/m^2 (existing condition), q_u=2,000 kN/m^2 (heightened condition)

(3) Heightening of Sul Dam

The heightening of Sul dam is the countermeasure against 50 year flood control.

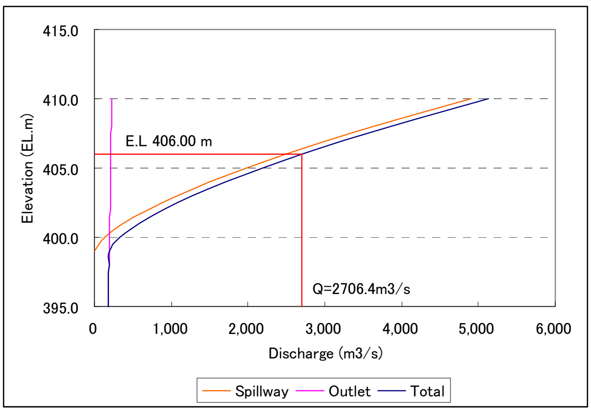
The elevation of the crest of spillway and dam body is 399.0 m and 410 m respectively.
The elevation difference is 11.0 m. The overflow height of spillway is maximum 7.0 m and the probable water lever is 406.m, and freeboard is estimated 4.0 m. The rockfill dam is required 1.0 m for freeboard, so that even if the dam was heighted 2.0 m, there was still a 2.0 m space for freeboard.

i) Sharpe of Spillway of Sul Dam

The typical sections of Sul dam is determined based on the actual topographical conditions through field investigation.

ii) The Relationship between capacity of overflow and the hegihte of bride

As indicated below, the Sul dam is able to release the 1,000-year flood with the

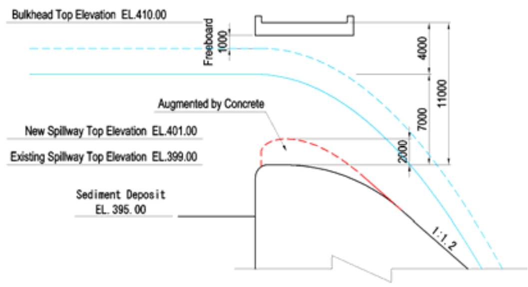


Source : JICA Survey Team

Figure 2.2.5 Spillway Capacity of Sul Dam

overflow depth of 7 m through the spillway.

Even if the girder of spillway bridge is assumed to be 1m, the current clearance over the spillway is 10 m (= (410.0-399.0) -1.0). Therefore, 1.0 m of freeboard can be secured against the 1,000-year flood when the dam is heightened by 2 m as illustrated below.



Source：JICA Survey Team

Figure 2.2.6 Overflow Condition at Sul Dam Spillway against 1000-year Flood

iii) Structure design of Heightening Overflow Section

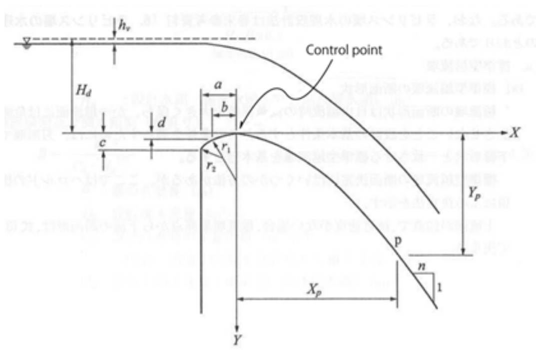
The shape of the crest spillway is basically required to keep the coefficient of discharge is high with free overflow and not to occur the suction at the overflow section. To meet those conditions is the shape of typical spillway.

Standard Shape of Overflow Spillway of Curve

$$y = \frac{x^{1.85}}{2 \cdot Hd^{0.85}}$$

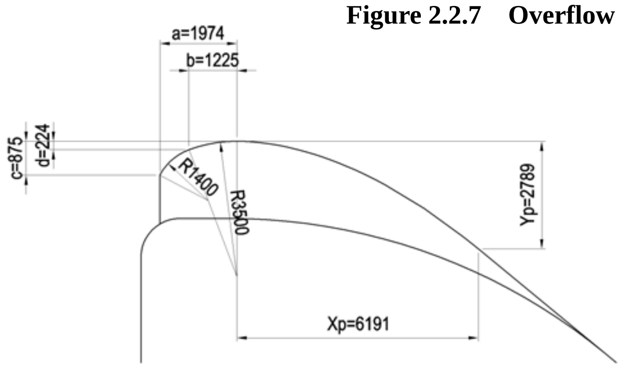
Hd=	7.000m
a=0.282*Hd →	1.974m
b=0.175*Hd →	1.225m
c=0.125*Hd →	0.875m
d=0.032*Hd →	0.224m
r1=0.5*Hd →	3.500m
r2=0.2*Hd →	1.400m

Xp=1.096*Hd*(1/n)^(1.176)
=6.191m
1/n= 1/1.2



Source :JICA survey team

Figure 2.2.7 Overflow Section（Typicall）



Source: JICA Survey Team

Figure 2.2.8 Overflow Section（Heightening by 2.0 m）

iv) Design Conditions

The same conditions applied to the Oeste dam is applied for the stability analysis.

Water Level Condition

The spillway of Sul dam is sloping to downstream and therefore the water level at downstream does not active to the stabilization of spillway.

Table 2.2.17 Design water level at downstream (Sul dam)

Load Condition	Water level (Existing)	Water level (Heighening)	Remarks
CCN (Normal)	387.00	387.00	The height of foundation
CCE1 (Flood)	406.00	408.00	Q=2,567m^3/s (1,000 year flood.)
CCE2 (Normal＋Earthquake)	387.00	387.00	The height of foundation
CCL (Flood＋Earthquake)	399.00	401.00	The crest of spillway
CCC (During Construction)	387.00	387.00	The height of foundation

Source: JICA Survey Team

v) Stability of existing Sul dam

As mentioned earlier, there are no geology date of dam foundation available, the case of existing dam is calculated to estimate the physical properties. The result of anayslis, the angle of internal friction and shearing stress are φ=45° and c=50 kN/m^2 is statisfied the result. The definitive loading condition is CCE(Flood, 1,000 year flood). The critical bearing capacity of foundation ground is requied qu=1,000 kN/m^2.

（Calculation Condition）

1.	Spillway crest elevation	H_1＝	399.000 m
2.	Elevation spillway foundation	H_2＝	387.000 m
3.	High Dam	H_3＝	12.000 m
3.	Base width	H_4＝	19.000 m
4.	Elevation of sediment	γ_s＝	17.5 kN/m^3
5.	Reservoir water level (CCE: flood)	H_{W1}＝	406.000 m
6.	(CCE: normal + earthquake)	＝	387.000 m
7.	(CCL: flood + earthquake)	＝	399.000 m
8.	Unit weight of concrete dams	γ_c＝	23.5 kN/m^3
9.	Weight of sediment in the air	γ_s＝	17.5 kN/m^3
10.	Weight of sediment in water	γ_s＝	8.5 kN/m^3
11.	Unit weight of water	γ_w＝	10.0 kN/m^3
12.	Seismic Coefficient: Horizontal (kh)	K_h＝	0.050
13.	Seismic factor: vertical (kv)	K_v＝	0.030
14.	Coefficient of earth pressure (Rankine coefficient of earth pressure)	ka＝	0.40
15.	Uplift pressure coefficient	μ＝	1/3
16.	Shear strength of foundation	C＝	50.0 kN/m^2
17.	Friction angle of foundation	ϕ'＝	45.00 °
18.	Internal friction coefficient	f＝	1.00

(Result)

Table 2.2.18 Analysis Result of spillway section

	FSF	FST	FSD ≧ 1.0
[CCN]-1	6.69 > 1.30	3.345 > 1.50	2.25 ≧ 1.0
[CCE]-2	∞ > 1.10	18.92 > 1.20	9.84 ≧ 1.0
[CCL]	10.27 > 1.10	6.38 > 1.10	3.67 ≧ 1.0
[CCN,CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]-1	127.77 ≤ 30M/3.0=10M	232.58 ≥ 200
[CCE]-2	291.08 ≤ 30M/2.0=15M	119.90 ≥ 200
[CCL]	204.99 ≤ 30M/1.5=20M	165.98 ≥ 200
[CCN,CCC]	327.58 ≤ 30M/1.3=23M	96.11 ≥ 200

Source：JICA survey team

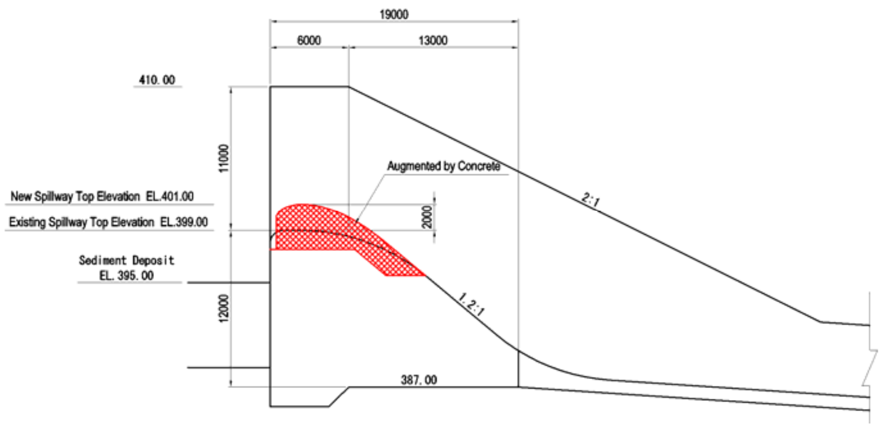
vi) Results of Stability Analysis (2.0 m heightening)

The analysis results are summarized below. The details of stability analysis are explained in Supporting Report. Structural drawing for dam heightening is shown in Figure 11.1.4.

Foundation condition assumed: Internal friction angle φ=45°, Shear stress c=50 kN/m^2

Definitive loading condition: CCE (1,000-year flood)

Critical bearing capacity: q_u=1,000 kN/m^2 (existing condition), q_u=1,200 kN/m^2 (heightened condition)



Source: JICA Survey Team

Figure 2.2.9 Typical cross section and spillway at Sul Dam

(Calculation Condition)

1.	Spillway crest elevation	H_1 =	401.000 m
2.	Elevation spillway foundation	H_2 =	387.000 m
3.	High Dam	H_3 =	14.000 m
3.	Base width	H_4 =	19.000 m
4.	Elevation of sediment	γ_s =	17.5 kN/m^3
5.	Reservoir water level (CCE: flood)	H_{W1} =	408.000 m
6.	(CCE: normal + earthquake)	=	387.000 m
7.	(CCL: flood + earthquake)	=	401.000 m
8.	Unit weight of concrete dams	γ_c =	23.5 kN/m^3
9.	Weight of sediment in the air	γ_s =	17.5 kN/m^3
10.	Weight of sediment in water	γ_s =	8.5 kN/m^3
11.	Unit weight of water	γ_w =	10.0 kN/m^3
12.	Seismic Coefficient: Horizontal (kh)	K_h =	0.050
13.	Seismic factor: vertical (kv)	K_v =	0.030
14.	Coefficient of earth pressure (Rankine coefficient of earth pressure)	ka =	0.40
15.	Uplift pressure coefficient	μ =	1/3
16.	Shear strength of foundation	C =	50.0 kN/m^2
17.	Friction angle of foundation	ϕ' =	45.00 °
18.	Internal friction coefficient	f =	1.00

(Result)

Table 2.2.19 Analysis Result of spillway section

	FSF	FST	FSD ≧ 1.0
[CCN]-1	6.52 > 1.30	2.43 > 1.50	1.92 ≧ 1.0
[CCE]-2	∞ > 1.10	17.65 > 1.20	10.11 ≧ 1.0
[CCL]	11.06 > 1.10	4.59 > 1.10	3.09 ≧ 1.0
[CCN,CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]-1	103.97 ≤ 30M/3.0=10M	281.85 ≥ 200
[CCE]-2	327.96 ≤ 30M/2.0=15M	114.19 ≥ 200
[CCL]	211.68 ≤ 30M/1.5=20M	190.47 ≥ 200
[CCN,CCC]	368.83 ≤ 30M/1.3=23M	86.99 ≥ 200

Source：JICA survey team

vii) Conduit Pipes

Since the water level is raised 2.0 m, the conduit pipes are required to reinforce.


Operation Room (downstream)


Upstream (Intake)

Operation room and intake(Sul dam)

v) Results of Stability Analysis

The analysis results are summarized below. The details of stability analysis are explained in Supporting Report. Structural drawing for dam heightening is shown in Figure 11.1.4.

Foundation condition assumed: Internal friction angle φ=45°, Shear stress c=50 kN/m^2

Definitive loading condition: CCE (1,000-year flood)

Critical bearing capacity: q_u=1,000 kN/m^2 (existing condition), q_u=1,200 kN/m^2 (heightening condition)

(4) Reinforcement of Existing Discharge Gates at Both Dams

As the hydraulic pressure will increase due to heightening by 2m at both dams , it is necessary to reinforce the existing discharge gates.

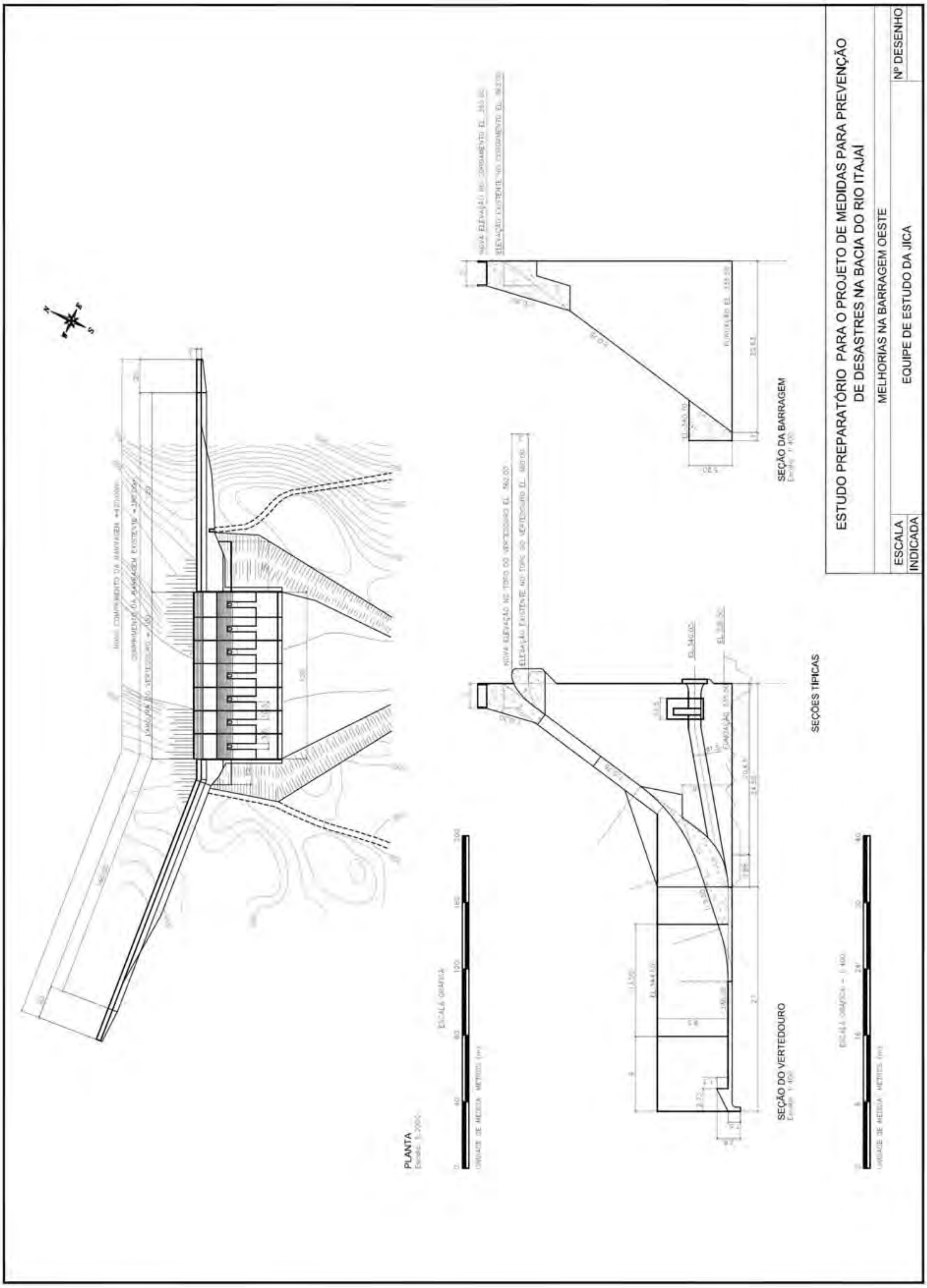


Figure 2.2.10 Drawing on Heightening of Oeste Dam

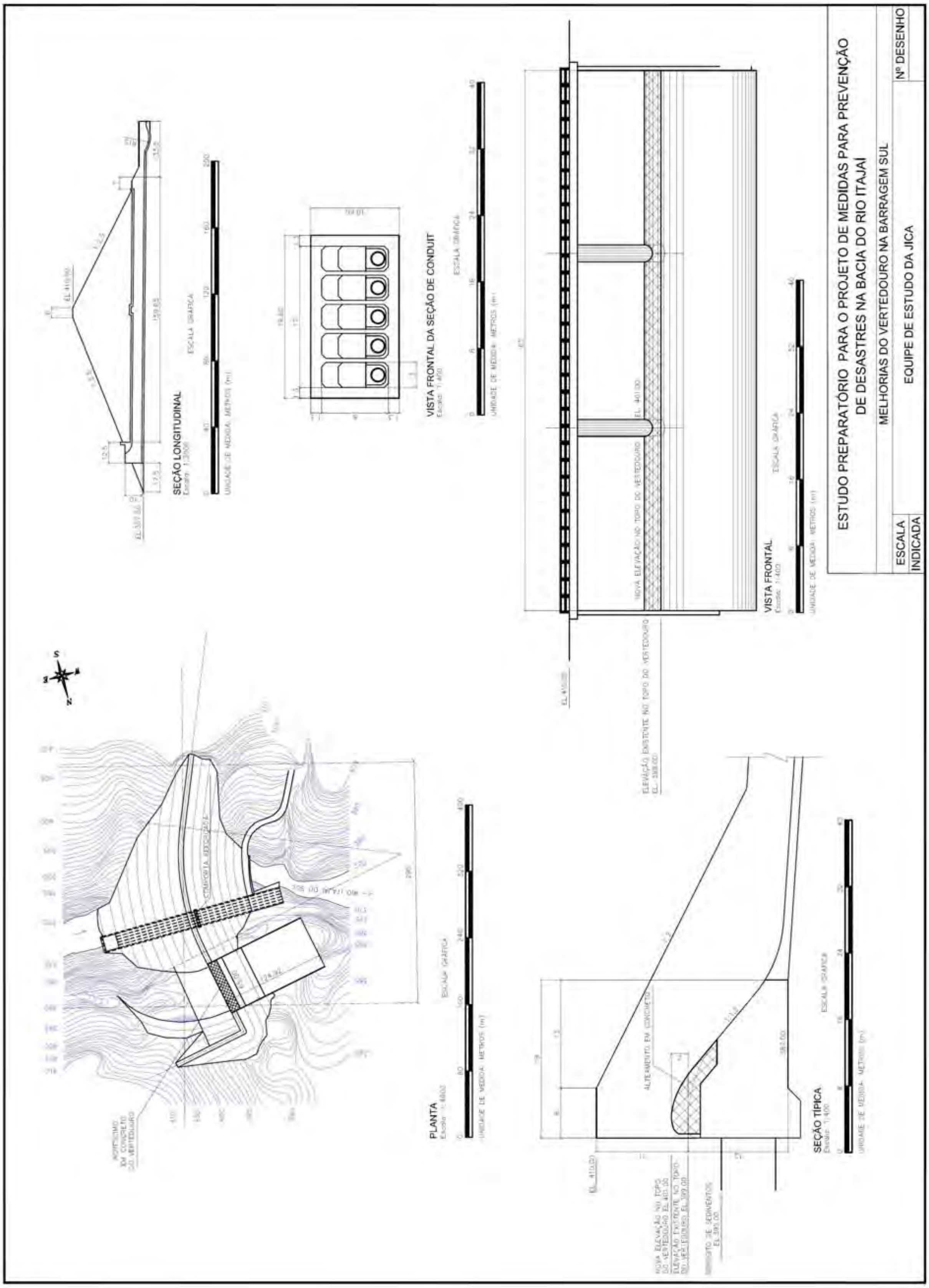


Figure 2.2.11 Drawing on Heightening of Sul Dam

2.3 River Improvement

The planned river improvement stretches by the probable floods are as follows:

Table 2.3.1 Planned River Improvement Stretch by Probable Flood

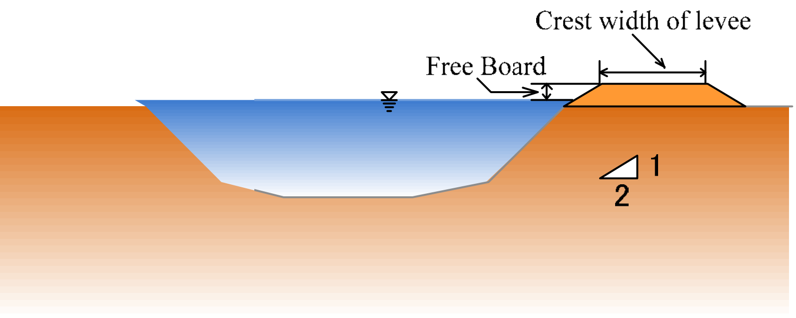
River / City (Safety Level)		5 year	10 year	25 year	50 year
Itajai River	Itajai		Dyke (3)* （L=12,830m）	Dyke (3)* （L=12,830m）	
	Ilhota			Ring Dyke (3)* （L=8,000 m）	Ring Dyke (3)* （L=8,000 m）
	Blumenau				Dyke (3)* （L=15,800m ）
	Rio do Sul			Channel Excavation （L=10,270m ）	Dyke (2)* （L=4,500m ）
Benedito River	Timbo			Channel Excavation （L=1,000m ）	Dyke (2)* Excavation （L=1,000m ）
Oeste River	Rio do Sul				Dyke (2)* （L=3,000m ）
	Taio			Channel Excavation （L=3,700m ）	Dyke (2)* （L=3,700m ）
Sul River	Rio do Sul				築堤(2)* （L=700m ）
Itajai Mirim River	Itajai	Dyke (1)* （L=950 m ）	Dyke (1)* （L=950 m ）	Dyke (2)* （L=950 m ）	Dyke (2)* （L=950 m ）

Remarks: (*) shows the category number in Figure 11.1.5. Source：JICA Survey Team

(1) Dyke and Ring Dyke

According to the information from DEINFRA, technical guidelines regarding the improvement of rivers have not yet established and almost no river improvement works have been undertaken. Under the current design, Japanese design criteria was applied. The design criteria for dyke are shown in Figure 2.3.1. As shown, freeboard and crest width of dyke vary according to the magnitude of design discharges. Regardless of the magnitude of discharges, stable dyke slope of 1:2 is applied for dyke design. Design condition of ring dyke is the same as the dyke design.

Dyke is provided to the river stretch in the urban area, where the flow capacity is smaller than the design discharge.



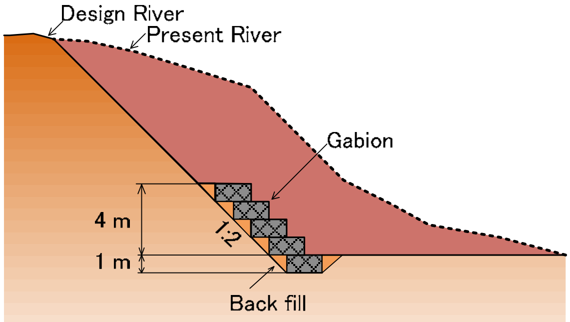
Category No.	Design Discharge (m3 s)	Free Board (m)	Crest width of levee (m)
1	200 ≦ Q < 500	0.8	3.0
2	500 ≦ Q < 2000	1.0	4.0
3	2000 ≦ Q < 5000	1.2	5.0

Source: JICA Survey Team

Figure 2.3.1 Design Conditions for Dyke

(2) River Widening and Channel Excavation

As for river widening and excavation of river channel, excavated slope is planned to be 1:2 as illustrated below. Gabions are to be placed to protect foot of the slope from scouring. The design river bed is set at the deepest riverbed of channel.



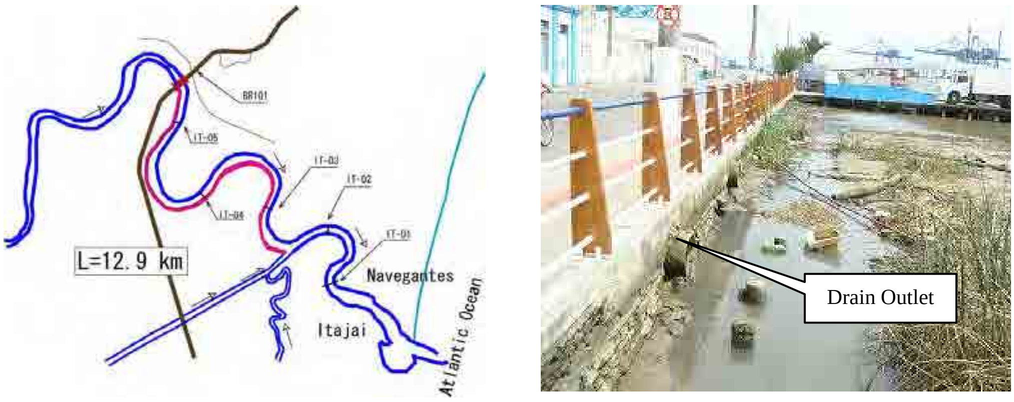
Source：JICA Survey Team

Figure 2.3.2 Design Conditions for Channel Excavation

(3) River Improvement Plan at Cities

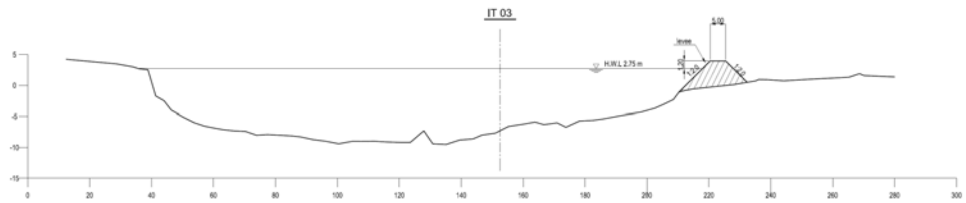
a) Itajai City, Itajai River

The river stretch subject to river improvement is on the right bank from the location 800 m downstream of the River Section IT-02 to the federal road BR 101 with a total length of 12.9 km. Although the low-lying area on the left bank (IT-03, IT-04) is below the design flood water level, this area will be unprotected by dyke considering that this area is subject to inundation and acts as a retarding basin. The river stretch to be improved is shown below.



Source：JICA Survey Team

Figure 2.3.3 River Improvement Stretch in Itajai City

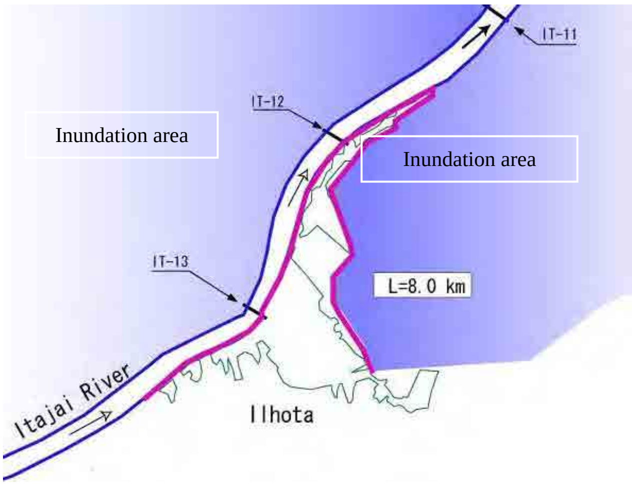


Source：JICA Survey Team

Figure 2.3.4 River Improvement Section in Lower Itajai River (Section IT-03, 25-year flood)

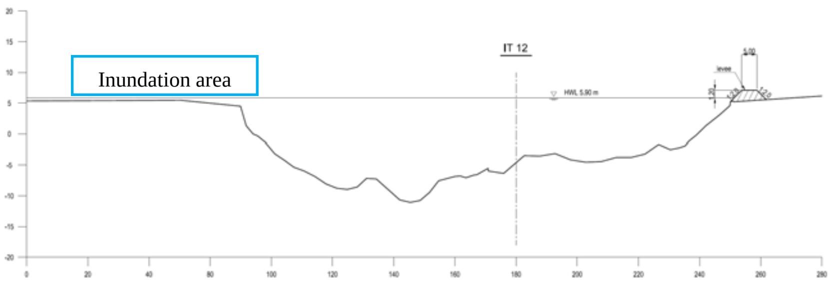
b) Ilhota City, Itajai River

As the flood inundation area spreading from Itajai city (BR 101) to Gaspar city is unprotected by dyke acting as a natural retarding basin, ring dyke is planned to protect Ilhota city from flood inundation. The existing road on the right bank along the Itajai River is heightened and the urban area of Ilhota city is surrounded by the dyke connecting to the location with higher elevation as illustrated below. The total length of ring dyke is 8.0km, comprising 4.4 km long heightening of the road and 3.6 km long dyke.



Source：JICA Survey Team

Figure 2.3.5 Ring Dyke Plan in Ilhota City

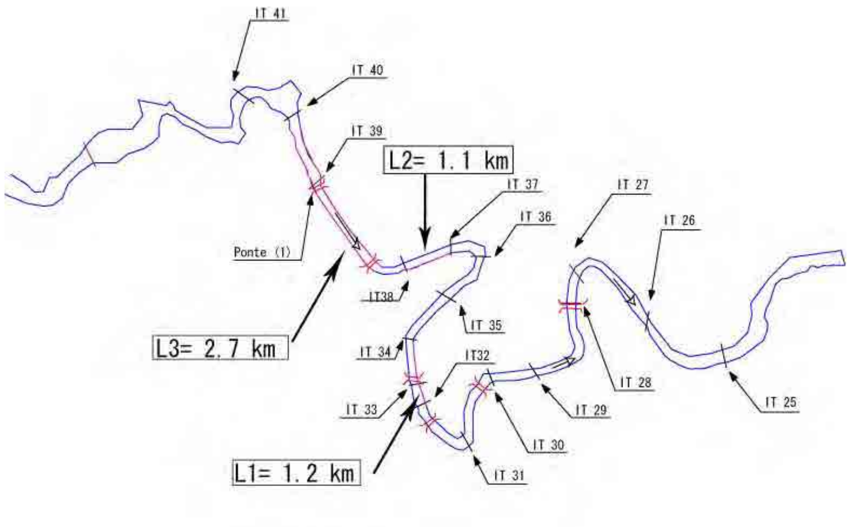


Source：JICA Survey Team

Figure 2.3.6 River Improvement Section in Ilhota City (Section IT-12, 25 year flood)

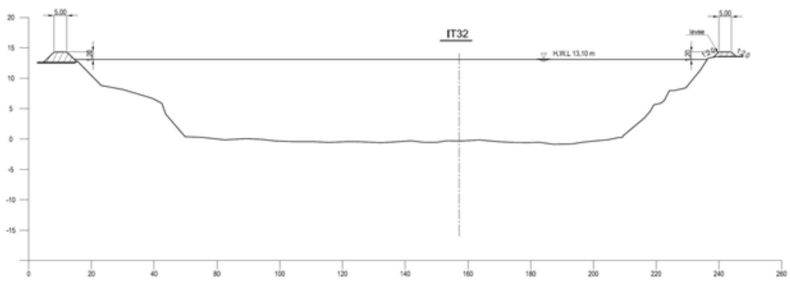
c) Blumenau City, Itajai River

River improvement in Blumenau city is proposed for the 50-year flood. The objective river stretches are 1.2 km long downstream stretch on the left bank (near sections IT-32 to IT-34), 1.1 km long stretch on the right bank from IT-37 to IT-38, and 2.7km long upstream stretch on the both banks from IT-40 as illustrated below. Relocation of residents along the river and reconstruction of one existing bridge are required as the associated works of river improvement.



Source：JICA Survey Team

Figure 2.3.7 River Improvement Stretch in Blumenau City



Source：JICA Survey Team

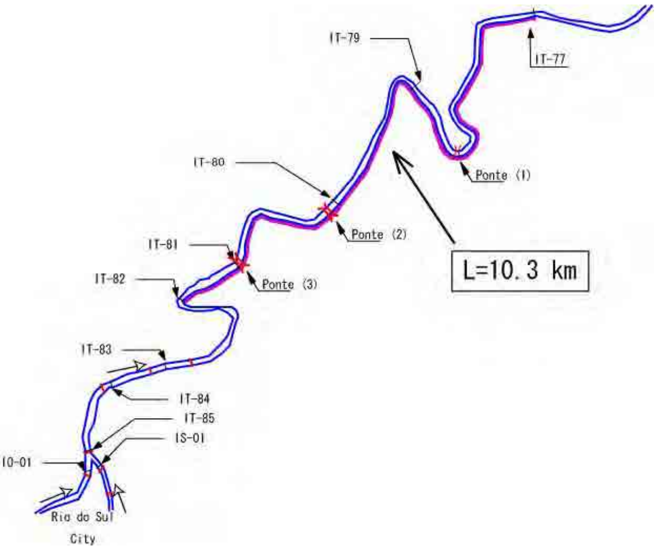
Figure 2.3.8 River Improvement Section in Blumenau City (Section IT-32, 50-year flood)

d) Rio do Sul City, Itajai River, Itajai do Oeste River, Itajai do Sul River

River improvement in Rio do Sul city is planned for both the 25-year and 50-year floods.

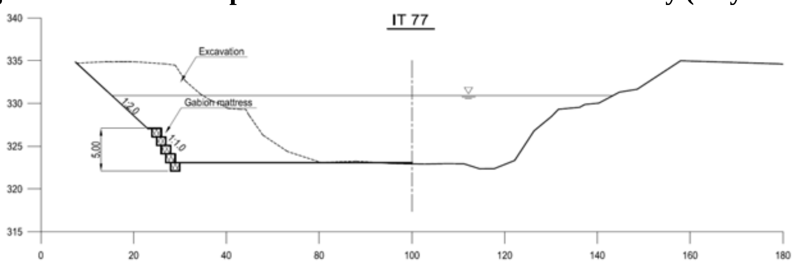
<u>River improvement for the 25-year flood:</u>

Both the Itajai do Oeste and Sul Rivers join each other in the urban area of Rio do Sul city. In order to lower river water level of the 25-year flood in Rio do Sul city, river widening in the downstream stretch is planned. The 10.3km long channel along the Itajai River is to be widened by around 10 m from the location approximately 4.5 km downstream of the confluence as illustrated below.



Source：JICA Survey Team

Figure 2.3.9 River Improvement Stretch in in Rio do Sul City (25-year flood)

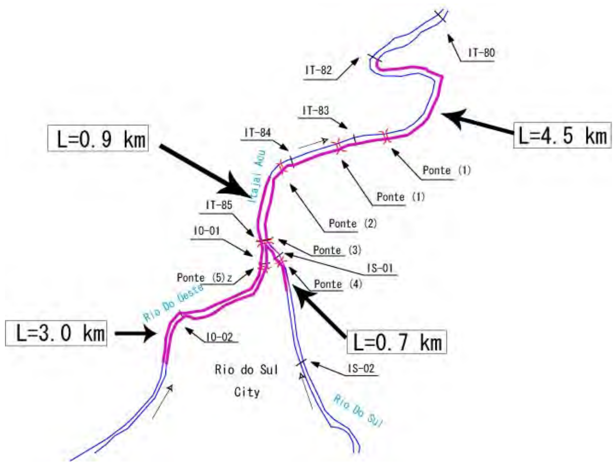


Source：JICA Survey Team

Figure 2.3.10 River Improvement Section in Rio do Sul City (Section IT-77, 25-year flood)

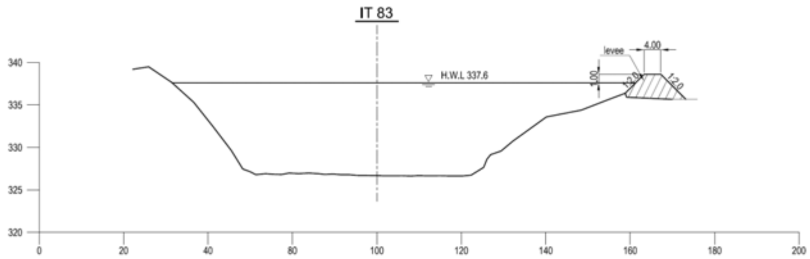
<u>River improvement for the 50-year flood:</u>

As illustrated below, three river stretches are improved by embankment; the Itajai River around 4.5 km long downstream of the confluence, the Itajai do Oeste River 3.0 km long upstream of the confluence, and the Itajai do Sul River 0.7 km long upstream of the confluence. Relocation of residents in the urban area and reconstruction of 5 existing bridges are required as the associated works of river improvement.



Source：JICA Survey Team

Figure 2.3.11 River Improvement Stretch in in Rio do Sul City (50-year flood)

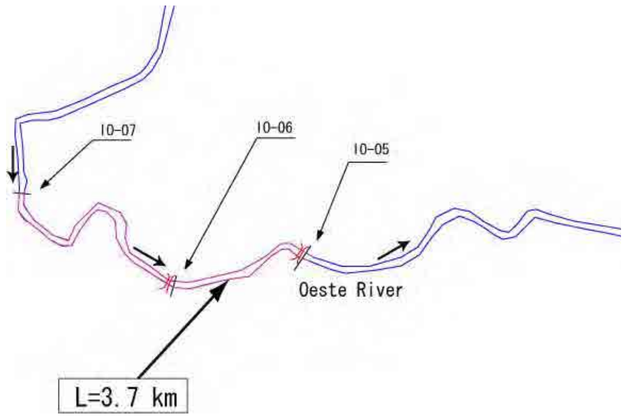


Source：JICA Survey Team

Figure 2.3.12 River Improvement Section in Rio do Sul City (Section IT-83, 50-year flood)

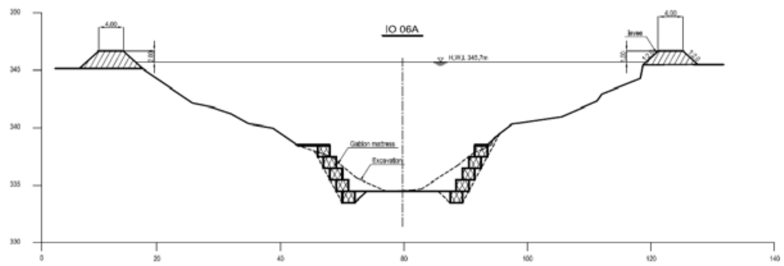
e) Taio City, Itajai do Oeste River

River improvement in Taio city is planned for both the 25-year and 50-year floods. River widening is proposed for the 25-year flood and combination of river widening and embankment is proposed for the 50-year flood. The objective river stretch is 3.7 km long in the urban area along the Itajai do Oeste River as shown below. The existing 2 bridges are necessary to be reconstructed due to river improvement.



Source：JICA Survey Team

Figure 2.3.13 River Improvement Stretch in Taio City

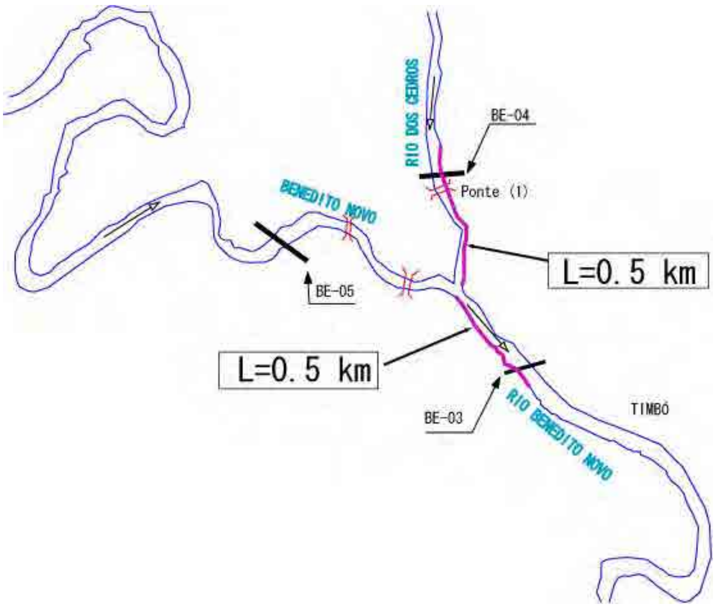


Source：JICA Survey Team

Figure 2.3.14 River Improvement Section in Taio City (Section IO-06a, 50-year flood)

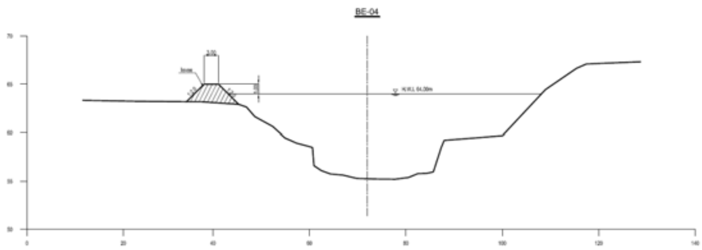
f) Timbo City, Cedros River

Timbo city is located at the junction of Benedito and Rio dos Cedros Rivers. As shown in Figure 11.1.20 below, part of urban area where the ground elevation is under the 50-year flood water level is to be protected by embankment. The objective stretches for improvement are 0.5 km on the left bank of Rio dos Cedros River upstream from the confluence and 0.5 km on the right bank of Benedito River downstream of the confluence as illustrated below. The existing bridge in the urban area is to be reconstructed due to implementation of river improvement.



Source：JICA Survey Team

Figure 2.3.15 River Improvement Stretch in Timbo City



Source：JICA Survey Team

Figure 2.3.16 River Improvement Section in Timbo City (Section BE-04, 50-year flood)

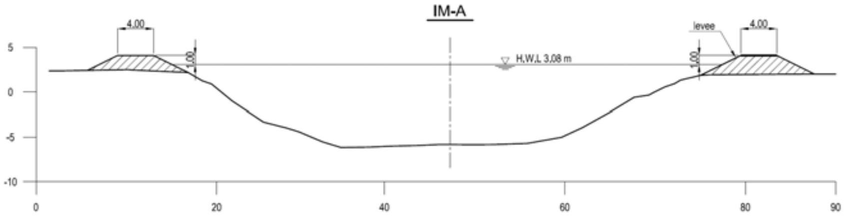
g) Itajai City, Itajai Mirim River

The objective stretch of the Itajai Mirim River subject to improvement is 950 m long stretch on its both banks between the confluence to the Itajai River and the junction of Canal and Old Mirim River as shown below. Residents along the stretch are to be relocated due to implementation of river improvement. Furthermore, the existing bridge is also to be reconstructed.

Upstream view of Itajai Mirim River from the bridge

Source : JICA Survey Team

Figure 2.3.17 River Improvement Stretch in Lower Itajai Mirim River



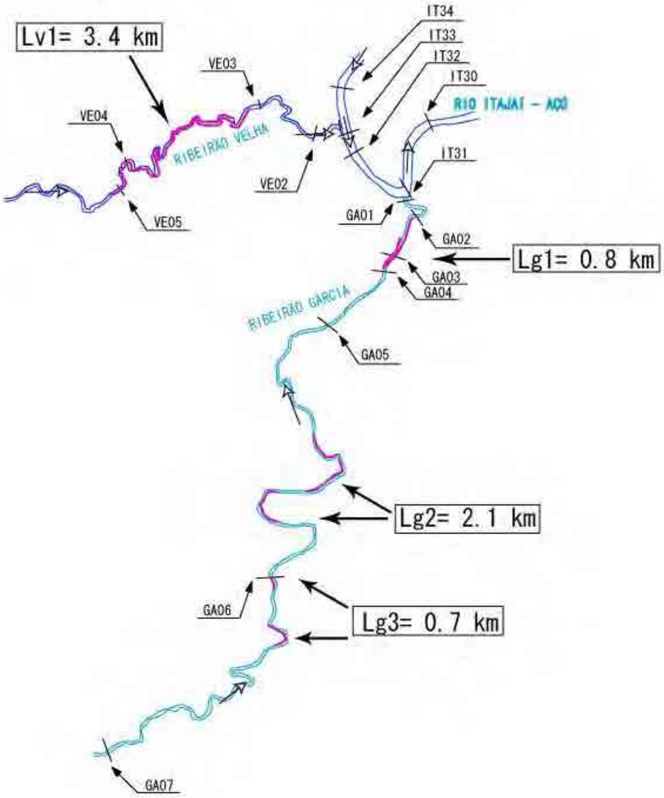
Source : JICA Survey Team

Figure 2.3.18 River Improvement Section in Itajai Mirim River (Section IM-A, 50-year flood)

h) Urban Rivers in Blumenau City

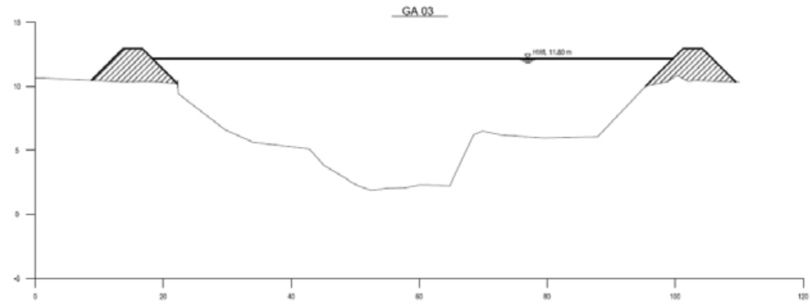
The Garcia River is seriously influenced by back water of the Itajai River in times of flood. Since the urban area along the Garcia River has been suffering from habitual flooding due to the back water effect, this area is planned to be protected by embankment against the 25-year flood. The stretches to be improved are 500 m on the right bank and 750 m on the left bank between river sections GA-02 and GA-04 as illustrated below. Furthermore, there are several channels in upper reaches, where the current flow capacities are insufficient to pass the 25-year flood. In these stretches, flow capacity is planned to be increased by means of excavation of the existing river channel with a total length of 2.8 km between sections GA-05 and GA-07 as shown below.

As for the Velha River, since no urban area is influenced by the backwater, river widening by excavation is planned to increase flow capacity in the 3.4 km long stretches between sections GA-03 and GA-05 as shown below.



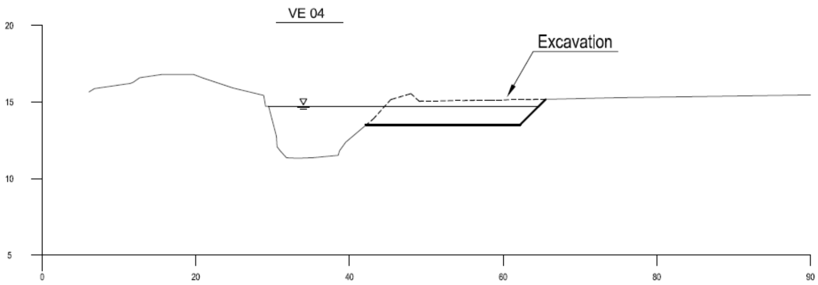
Source：JICA Survey Team

Figure 2.3.19 River Improvement Stretches of Urban Rivers in Blumenau City (Garcia and Velha Rivers)



Source：JICA Survey Team

Figure 2.3.20 River Improvement Section in Garcia River (Section GA-02, 25-year flood)

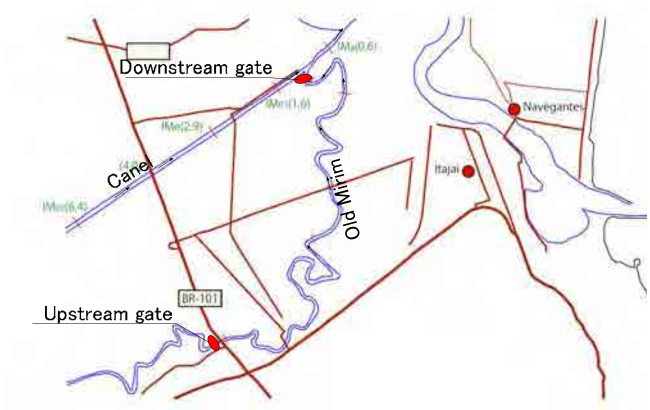


Source：JICA Survey Team

Figure 2.3.21 River Improvement Section in Velha River (Section VE-04, 25-year flood)

(4) Water Gates

The Old Mirim River has been suffering from frequent flooding on both banks due to small flow capacity. As shown in the figure below, two water gates are planned to be installed on the Old Mirim River to control flood inflow from the Mirim River into the Old Mirim River and the backwater intrusion from the Itajai River. The water gate is designed for respective probable floods. The crest elevation of flood gate is determined based on the probable flood water level estimated by the non-uniform flow calculation as well as freeboard. Table 2.3.2 shows structural dimensions of the designed water gate for respective probable floods.



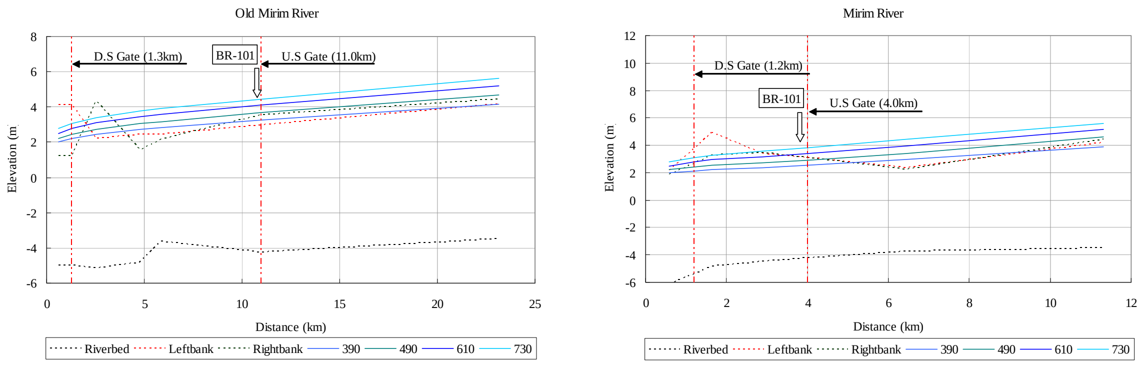
Source : JICA Survey Team

Figure 2.3.22 Location Map of Water Gates on the Old Mirim River

Table 2.3.2 Water Level Respective with Design Discharge

	5 year	10 year	25 year	50 year
Design Discharge	390 m^3/s	490 m^3/s	610 m^3/s	730 m^3/s
Downstream Gate Water Level	EL. 2.20 m	EL. 2.45 m	EL. 2.77 m	EL. 3.08 m
Upstream Gate Water Level	EL. 3.27 m	EL. 3.67 m	EL. 4.09 m	EL. 4.46 m

Source: JICA Study Team



Old Mirim River Mirim River

Source: JICA Study Team

Figure 2.3.23 Profile of Old Mirim River (left) and Mirim River (right)

2) Dimension of Floodgates per Scale of Probability

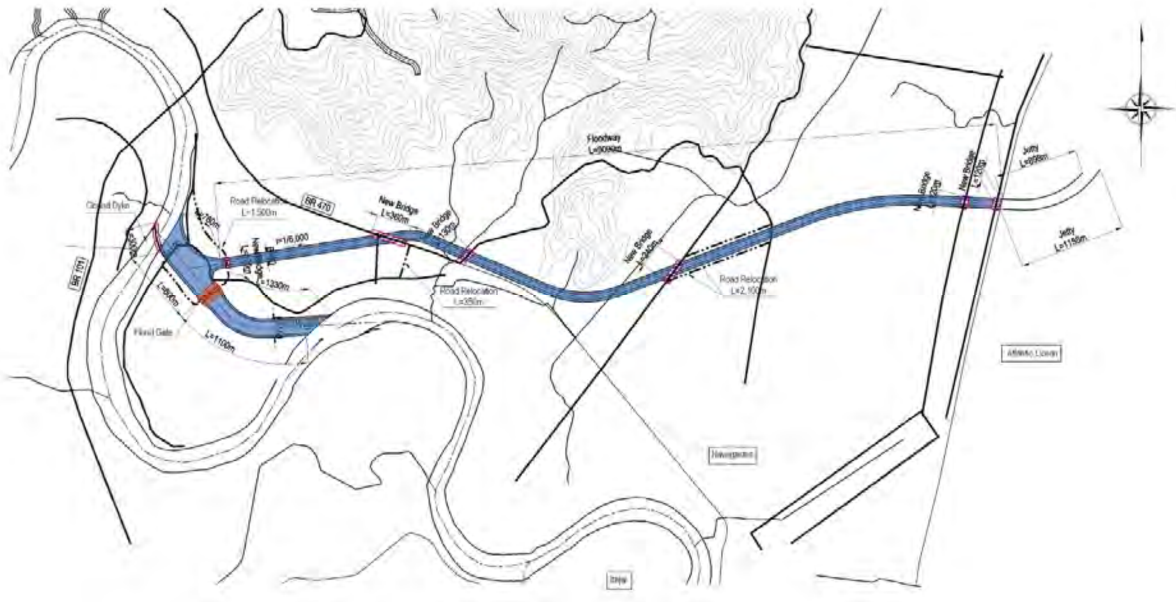
The design floodgates is required to closed in times of flood and after flood it is opened to drainage the own-basin discharge quickly. In addition, since the floodgates are under constant influence of the tide level, it is designed that the crest of floodgate is EL-0.50 m (=minimum tide level: -0.5 m) in order to minimize the floodgate size. As the water level below the crest of gate becomes dead water, a drain will be installed in the lower part of the floodgate. So that, it is immediately to drainage after the inundation for following the water level difference. The floodgates operation mechanism is just only open and close, not being equipped with flow adjustment functions. Figure 2.3.28 shows the structure dimensions per safety level in the control of floods.

3) Floodgates Structure

The foundation ground is supposed to be extremely soft since the site is near the river mouth and a pile foundation is proposed as the foundation of structure. On next study stage, it is required to survey the geological conditions and to design the diameter and length of the piles. In those analyses, a field study was conducted and the type of standard floodgate was defined.

(5) Floodway

Floodway is proposed to divert part of the 50-year flood discharge of the Itajai River to the Atrantic Ocean crossing Navegantes city from downstream reaches of the bridge of BR 101. The route of floodway route and the location of diversion weir are selected through field investigation confirming the current land use to minimize relocation of residents. As shown in the figure below, a gated diversion weir is to be installed on a new shortcut channel to divert the flood inflow smoothly into the floodway. The flood inflow into the lower reaches of Itajai River is controlled by the diversion weir so as not to cause overflowing into Itajai city.



Source：JICA Survey Team

Figure 2.3.24 Location Map of Floodway and Diversion Weir

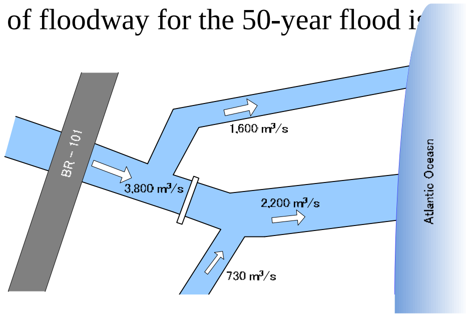
Table below presents the general features of the planned floodway and diversion weir.

Table 2.3.3 General Features of Floodway Plan

Floodway		B=50 m, h=12 m, L=9,000 m, 1:n=1:2.0、I=1/6000
Shortcut Channel	Upstream	B=190 m, h=12 m, L=600 m, 1:n=1:2.0
	Downstream	B=150 m, h=12 m, L=1,100 m, 1:n=1:2.0
Diversion Weir		Gate=20m × 9m × 8 nos., Width=190 m
New Bridge		6 nos.
Closure Dyke		L=300 m
Jetty		L=2,100 m（both banks）

Source：JICA Survey Team

Design discharge distribution of floodway for the 50-year flood is shown below.



Source：JICA Survey Team

Figure 2.3.25 Design Discharge Distribution of Floodway (50-year flood)

At the planning site for diversion weir, the water level is more than 10.0 m. Thus the construction with multiple-stage diversion is very difficult and the cost is very high. Under those conditions, constructing the diversion weir with dry condition is more advantage with making the short-cut channel in main stream. In addition, this site is considered in terms of the sure control of the discharge volume to downstream site, Itajai city.

(6) Jetty

A jetty is to be provided at the outlet of the floodway to prevent sediment deposition caused by the littoral drift at the outlet portion and also to prevent sandbar formation. The extent and magnitude of changes of coastal line, tidal current and diffusion of discharged turbid water at the Navegantes coast due to construction of the floodway and jetty should be examined and assessed from the socio-environmental viewpoints before implementation. Furthermore, detailed study on the angle of jetty to the coastal line and the length of jetty should be also carried out. The structural plan is shown in Figures 2.3.29.

(7) New Flood Control Dam on Itajai Mirim River

Regarding site selection for a new flood control dam, topographic maps with a scale of 1:10,000 are inevitably necessary. However, topographic mapping is still under preparation by SDS, site selection on the Itajai Mirim River was carried out based on the available topographical maps of 1:50,000. The dam site was selected in the upstream reaches of Brusque city.

The new dam was planned to be of a concrete gravity concrete type. The dam height is 34.2m considering the excavation of dam foundation by approximately 2 m. The dam is equipped with ungated spillway. . The energy dissipater was determined to be 20 m taking into consideration the current width of downstream river channel. The structural drawing is shown in Figure 2.3.30.

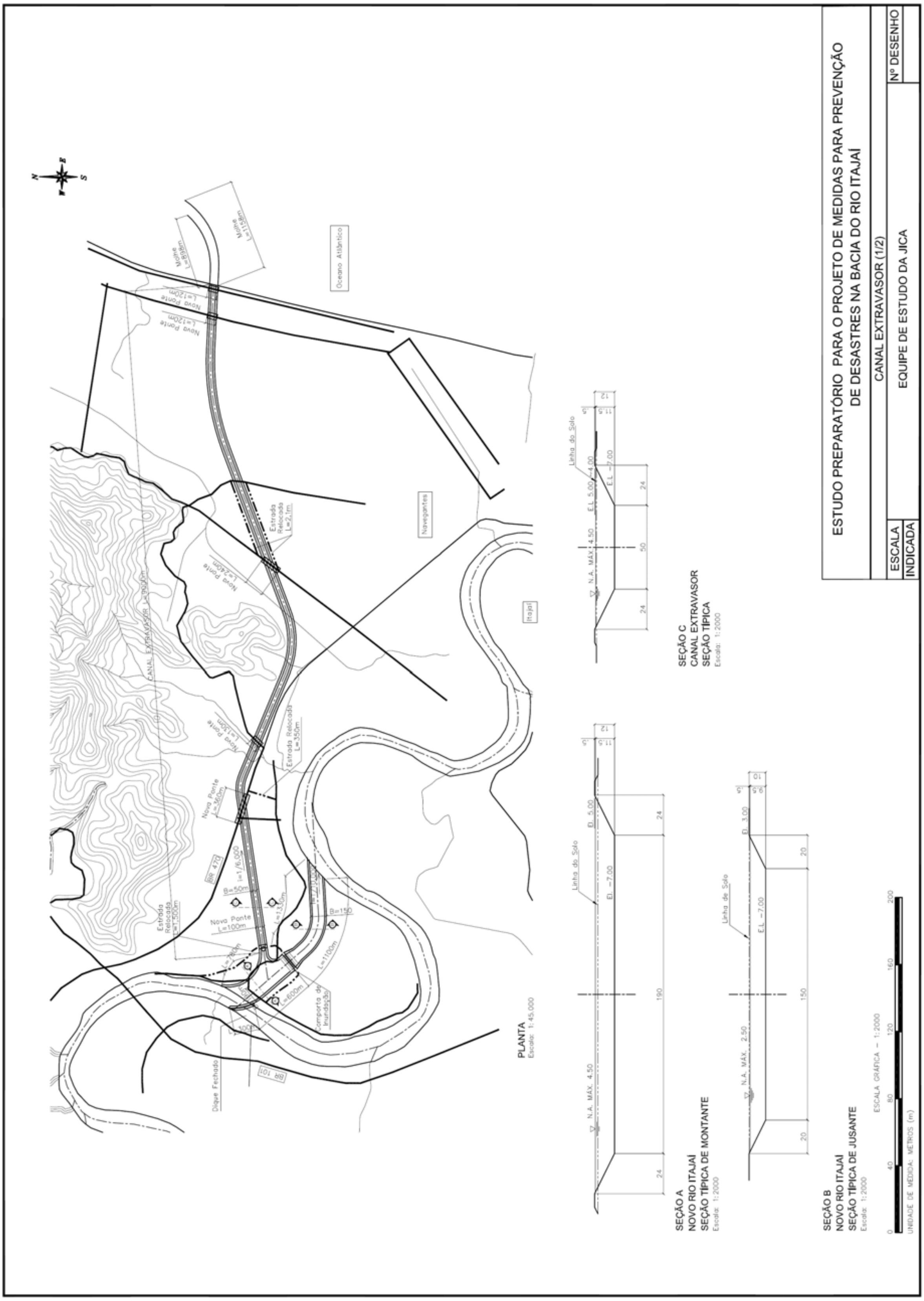


Figure 2.3.26 Structural Drawing of Water Gate on the Old Mirim River

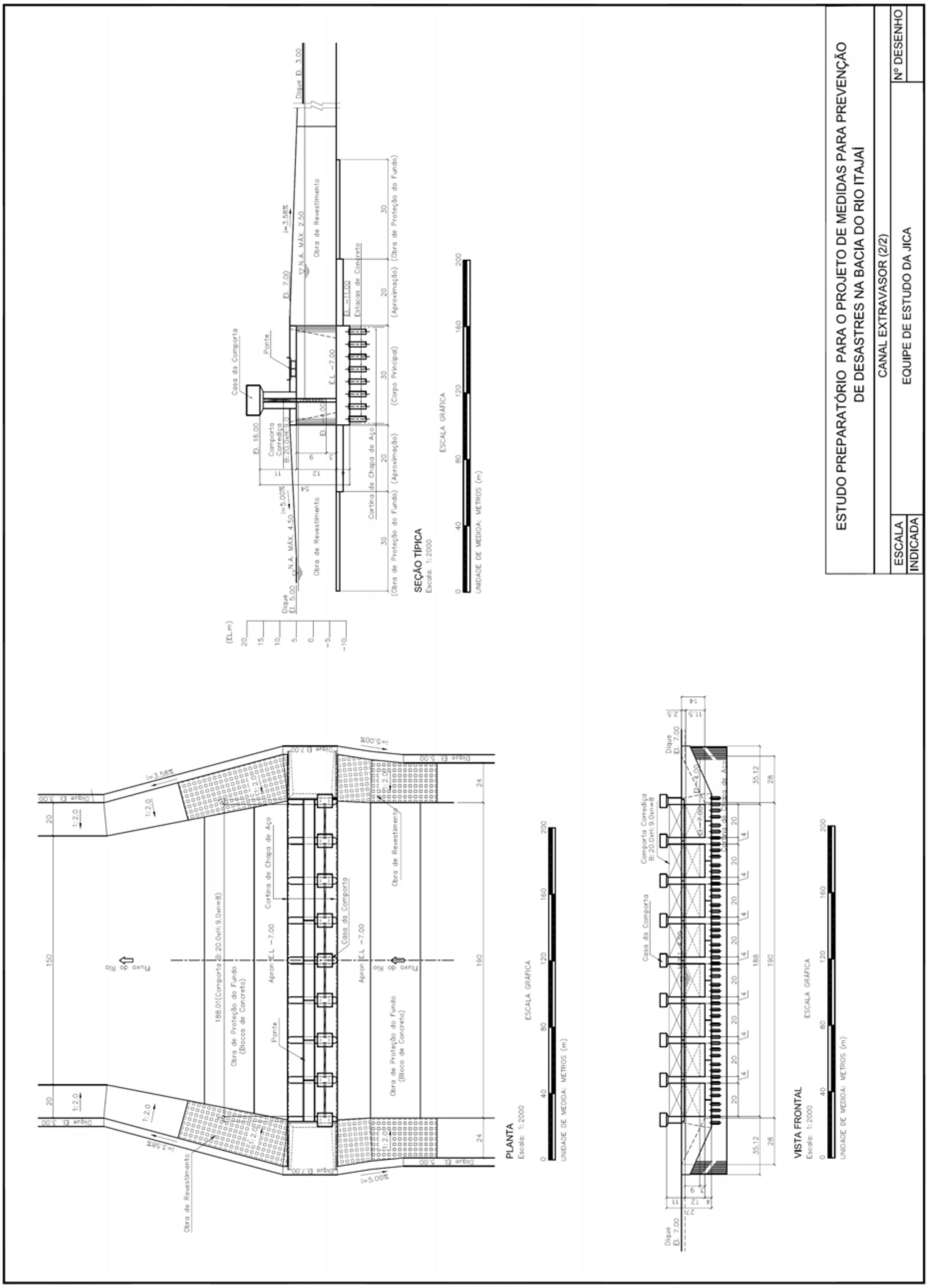
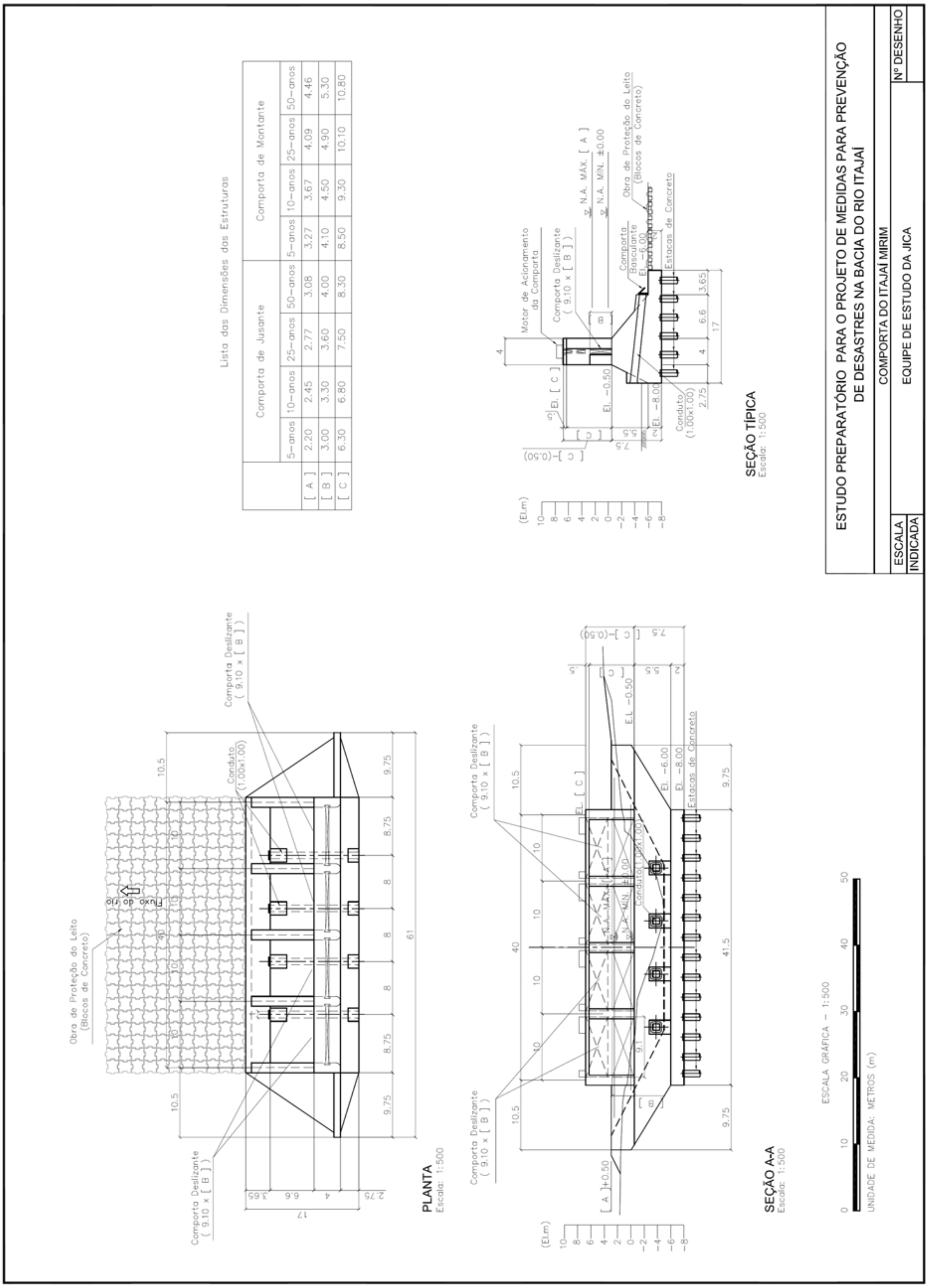


Figure 2.3.27 Structural Drawing of Floodway



	Comporta de Jusante				Comporta de Montante			
	5−anos	10−anos	25−anos	50−anos	5−anos	10−anos	25−anos	50−anos
[A]	2.20	2.45	2.77	3.08	3.27	3.67	4.09	4.46
[B]	3.00	3.30	3.60	4.00	4.10	4.50	4.90	5.30
[C]	6.30	6.80	7.50	8.30	8.50	9.30	10.10	10.80

Figure 2.3.28 Structural Drawing of Diversion Weir

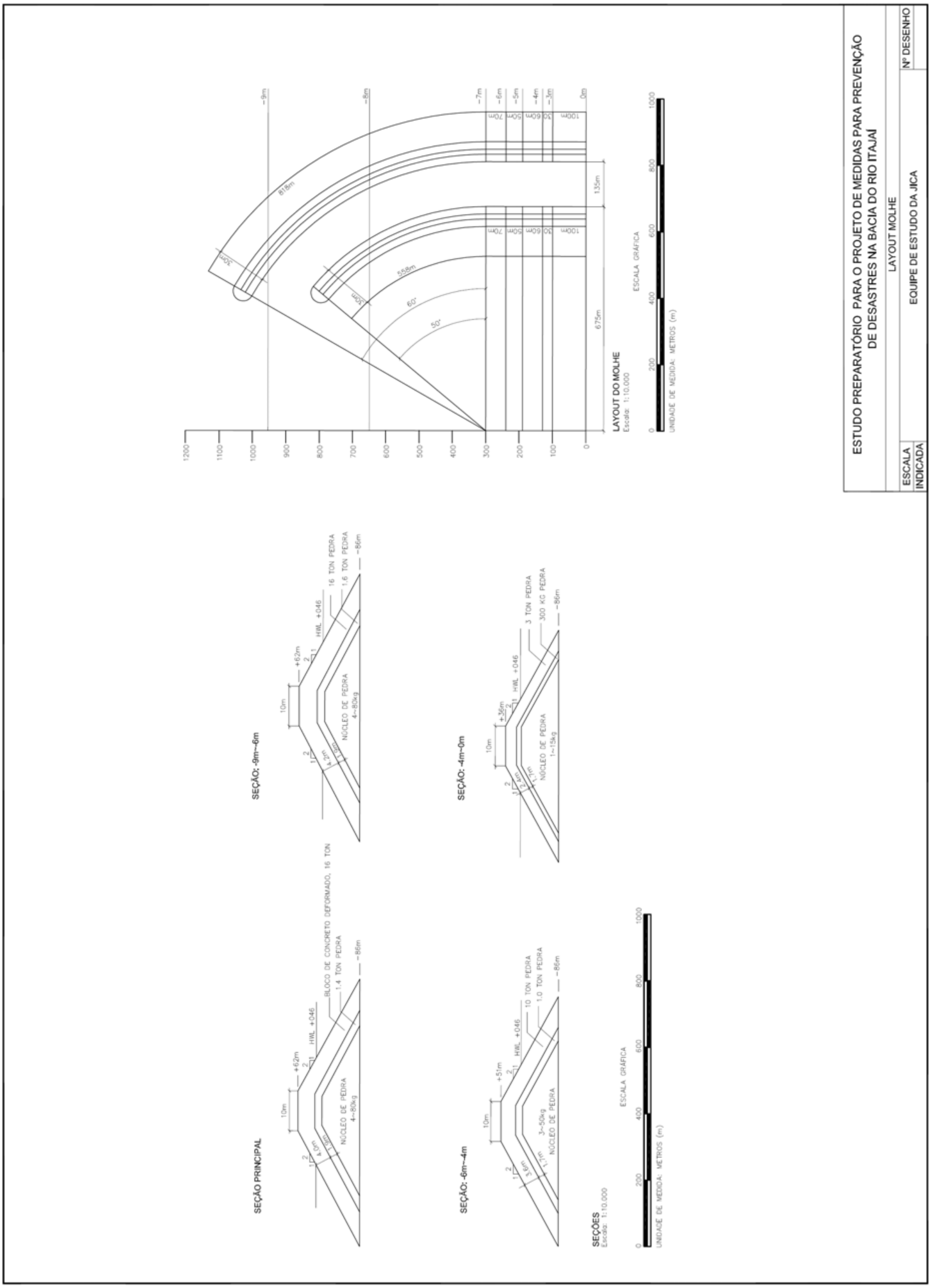


Figure 2.3.29 Layout of Jetty

(8) Small Dam (Small Water Storage Reservoir)

Likewise the site of a new flood control dam, the candidate sites for small dam are selected based on the topographic map with a scale of 1:10,000. The selected sites are the Trombudo and Trombudo Rivers as . The size of small dam is supposed to be about 3 million – 6 million m3/ pond. The number of small dam is required for flood control level is summarized as below table.

Table 2.3.4 the required numbers for flood control level

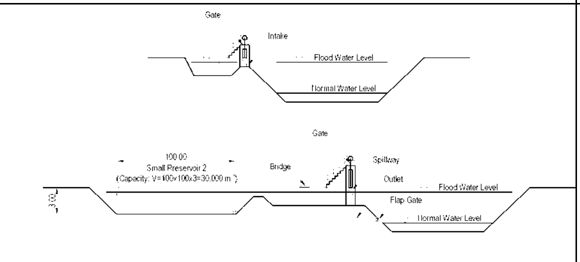
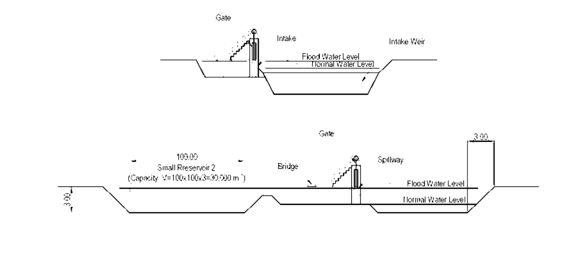
	5-year	**10-year**	**25-year**	**50-year**
nos	2	5	7	7

Source: JICA Survey Team

The required reservoir water level is expected to around 5 to 10 m in depth. The dam is designed as a homogeneous fill type dam because of relatively low dam height. In Brazil, retention wall of more than 15 m high is categorized as a dam. The structural drawing of small dam is shown in Figures 2.1.40 and 2.1.41. The typical shape of small dam is determined based on the actual topographical conditions through field investigation.

(9) Utilization for Agriculture's small dam

Agriculture's dams are used for flood control when it does not use for irrigation. When it occurs floods, those facilities are used to convey the raw water. The bottom of the small dam is not be able to design under river bed. So the depth is about 3.0 m or less. One small dam is thought to have the capacity 30,000 m3 (=100 m×100 m×3 m).

<table>
<tr><th>FREE INTAKE</th><th>WEIR INTAKE</th></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>- Intake facility is side overflow type and the overflow section is designed higher as much as possible to convey raw water at the flood.
- Countermeasure to avoid the high water is to design the spillway.
- Intake and spillway is equipped with the gate.</td><td>- Intake facility is afflux type.
- Countermeasure to avoid the high water is to design the spillway.
- Intake and spillway is equipped with the gate.
- To be equipped with drainage sluice-gate to drainage as soon as after flood passed.
- Sluice-gate at river side is equipped with flap-gate not to make reverse flow.</td></tr>
</table>

Source: : JICA Survey Team

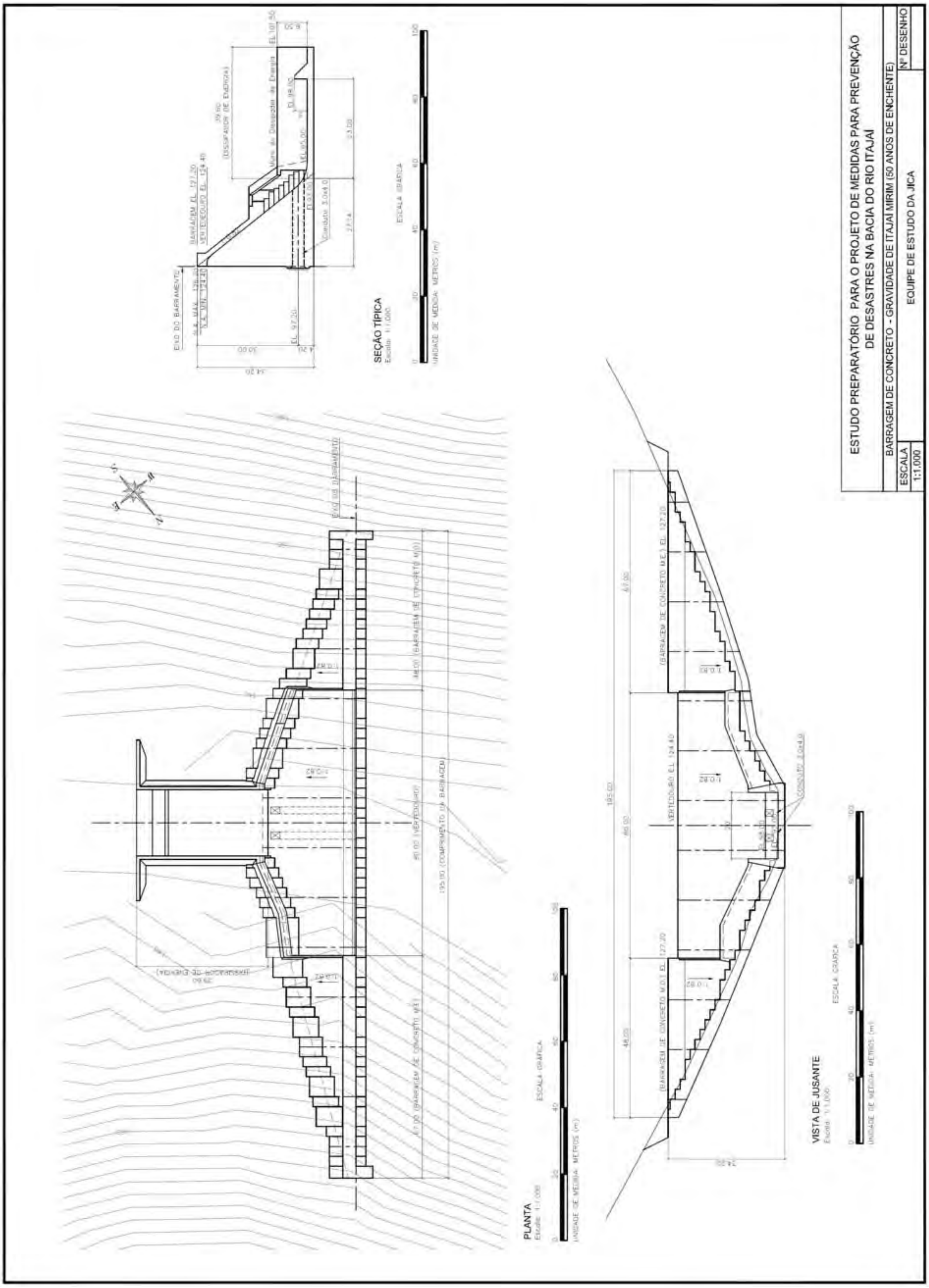


Figure 2.3.30 Utilization for Agriculture's small dam

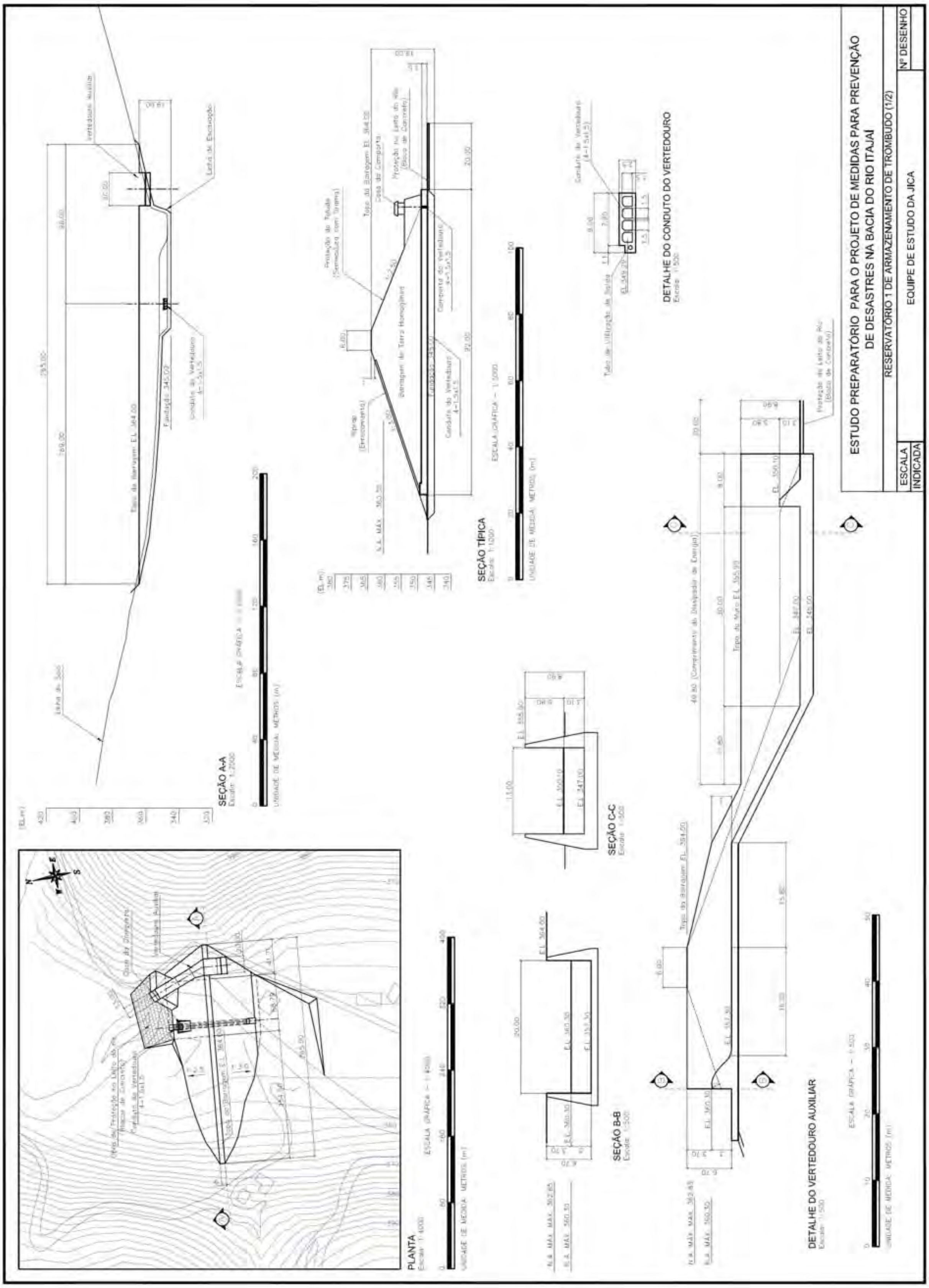


Figure 2.3.31 Structural Drawing of Small Dam (Site-1 on Trombudo River)

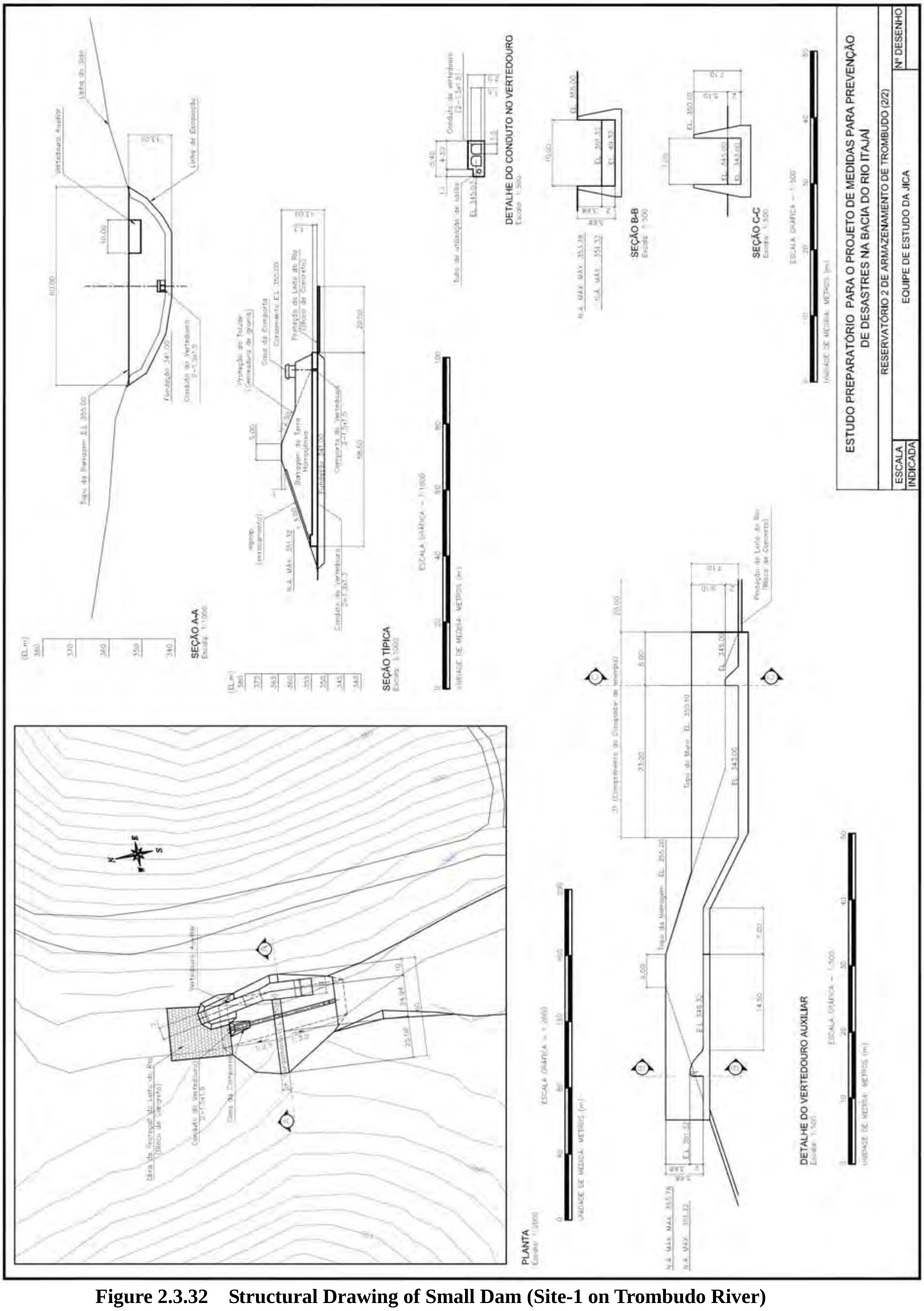


Figure 2.3.32 Structural Drawing of Small Dam (Site-1 on Trombudo River)

CHAPTER 3 COST ESTIMATE OF THE MASTER PLAN

3.1 Total Cost

Total cost for master plan consists of (1) Flood Disaster Mitigation Measure, (2) Landslide Disaster Mitigation Measure, (3) Flood Alarm and Alert System and (4) Alarm and Alert System for Flush Flood and Landslide Disaster. Besides, the landslide cost is mentioned and detailed in Annex B.

(1) Flood Disaster Mitigation Measure：

Classified total of items of each safety level of flood control and town respectively.

(2) Landslide Disaster Mitigation Measure：

Classified total of items of the target 67 areas.

(3) Flood Alarm and Alert System：

Classified total of items of the equipments for observation and communication, automatically calculation system of flood alarm and alert and the equipment for communication of alarm.

(4) Alarm and Alert System for Flush Flood and Landslide Disaster：

Classified total of items of the equipments for observation and communication and automatically calculation system of flood alarm and alert.

The Master plan’s total cost is as follow;

Table 3.1.1 Cost of Master Plan

（R$×$10^3$）

Safety Level of Flood Control	5 years	10 years	25 years	50 years
(1) Flood Disaster Mitigation Measure	202,000	541,000	1,025,000	1,996,000
(2) Landslide Disaster Mitigation Measure	54,000			
(3) Flood Alarm and Alert System	4,000			
(4) Alarm and Alert System for Flush Flood and Landslide Disaster	4,000			
Total	264,000	603,000	1,087,000	2,058,000

Source: JICA Study Team

The cost of the measure was estimated with base of the prices of 10/2010, in accordance with following exchange rate;

R$ 1.0 = JPY 47.87 = US$ 0.58.

The unit cost of the each work was estimated on the basis of the unit cost applied at the DEINFRA.

3.2 Cost Component

(1） Cost

The cost component is as follow;

i. Construction cost

ii. Land acquisition and compensation

iii. Government administration expenditure

iv. Engineering service

v. Physical contingency /Price escalation

(2) Construction cost

The construction cost was calculated based on the following conditions;

i. Construction Cost = Work Quantity x Unit Price

ii. Temporary work = 30% of major works

(3) Compensation

The Compensation cost was calculated on the basis of the land valuation's information of CREA, classifying into urban and rural area. The forest area at river margins was excluded of the extent of the compensation, considering that this land is in the public power. Besides, the compensation was calculated presupposing that each residential land has 100 m^2 of dimension.

Table 3.2.1 Detail of Cost of land Compensation

		Unit	Unit Cost (R$)
Cost of land compensation	Urban Area	m^2	0.5～3.0=1.75
	No Urban	m^2	950,00
Compensation cost for resettlements		Each Case	100 m^2×1,100 R$/$m^2$=111,000,00 (1,036～1,127,04 1.100R$/$m^2$)

Source: JICA Study Team

(4) Others Costs

The administrative expense was estimated as being 3% of the total construction costs and of land compensation and the consultants cost as being 10% of the construction direct cost. The physical Contingency was stipulated as being 10% of the total of the construction costs, compensation, administrative expenses and consultants. The price escalation was stipulated the readjustment of price of 5% on the amount of the physical Contingency.

3.3 Flood Disaster Mitigation Measure

3.3.1 Work Quantities

The amounts of the main works items listed in the Master Plan, are as follows;

Table 3.3.1 List of Works Amount for each Safety Level

Safety level of Flood Control	Construction Type	Unit.	5-year	10-year	25-year	50-year
Measure in river						
Heightening of dam						
Oeste dam	Heightening	Unit.	-	-	1	1
Sul dam	Heightening	Unit.	-	-	1	1
Improvement of river channel						
Taio	Dyke	m	-	-	3,682	3,682
Rio do Sul	Dyke	m	-	-	10,269	9,081
Timbo	Dyke	m	-	-	1,000	1,000
Blumenau	Dyke	m	-	-	-	8,667
Blumenau tributary	Dyke	m	7,300	7,300	7,300	7,300
Itajai	Dyke	m	-	12,828	12,828	-
Itajai Mirim	Dyke	m	950	950	950	950
Flood Gates (Itajai Mirim)	Gate	Unit.	2	2	2	2
	Bridge	Unit.	-	-	-	1
Floodway (Com Comporta)	Excavation	m	-	-	-	10,905
Ring dykes (Ilhota)	Dyke	m	-	-	8,000	8,000
New flood control dam	Dam	Unit.	-	-	1	1

Safety level of Flood Control	Construction Type	Unit.	5-year	10-year	25-year	50-year
Measure in Basin						
Rain water containment in rice fields		ha	22,000	22,000	22,000	22,000
Small-scale dams		Unit.	2	5	7	7

Source: JICA Study Team

The compensation area is as follow;

Table 3.3.2 Compensation Area for each Safety Level for Flood Control

(Unit:m^2)

Area	5 - year	10 - year	25 - year	50 - year
Urban Area	20,619	194,581	302,647	574,086
Rural Area	3,056,000	7,693,710	10,861,750	13,645,719
Total	3,076,619	7,888,291	11,164,397	14,219,805

Source: JICA Study Team

3.3.2 Unit Cost

The applied unit cost for the Cost estimate was of base in 10/2010.

3.3.3 Work Cost

The estimate costs of the construction works for each safety level are illustrated in Tables below. The measures of flood disaster mitigation are subdivided into three parts: measures in the river/basin and no-structural measure. And, because of that the no-structural measure is only the improvement of the operation method of the dams during the flood, therefore, this cost estimation was not considered of this extent.

Table 3.3.3 Construction Cost for each safety level (by each type of work)

Safety level of Flood Control	5-year	10-year	25-year	50-year
Measure in river	109,000	357,000	781,000	1,752,000
Heightening of dam				
Oeste dam	-	-	27,000	27,000
Sul dam	-	-	-	6,000
Improvement of river channel				
Taio	-	-	56,000	114,000
Rio do Sul	-	-	190,000	268,000
Timbo	-	-	21,000	21,000
Blumenau	-	-	-	267,000
Blumenau tributary	35,000	98,000	144,000	196,000
Itajai	-	181,000	197,000	-
Itajai Mirim	36,000	38,000	46,000	50,000
Flood Gates (Itajai Mirim)	38,000	40,000	42,000	44,000
Floodway (Com Comporta)	-	-	-	593,000
Ring dykes (Ilhota)	-	-	58,000	70,000
New flood control dam	-	-		95,000
Measure in Basin	93,000	184,000	244,000	244,000
Rain water containment in rice fields	33,000	33,000	33,000	33,000
Small-scale dams	60,000	151,000	211,000	211,000
TOTAL	202,000	541,000	1.025,000	1.996,000

Source: JICA Study Team

3.4 Flood Alarm and Alert System

3.4.1 Equipments

The monitoring and necessary communication equipments for the Flood alerts and alarm system are composed of the following;

- Automatic rain gage (Tippingbucket Rain Gauge)
- Automatic water level gage (radar system)
- Date logger (Registrations of data).
- Solar panel and battery (for the Guarantee of energy).
- Converter to send the observed data (system GPRS of cellular telephone)
- Receiving system and Base the Central Station(CEOPES)
- Communication (Internet) net communicated between the monitoring (Rio do Sul and Itajaí) stations.
- Communication (Internet) net communicated between the Headquarters of Monitoring (Florianópolis).
- Real Time Flood Situation monitoring System

3.4.2 Cost

The Cost for the installation of the flood alert and alarm system is as follow;

Table 3.4.1 Project Cost for Installation of Flood Alarm and Alert System

Items	Despesas (R$)
1 Observation equipments of alert and alarm system (FFWS)	2,350,000
2 River Inventory	938,000
3 Training	296,000
4 Consultants	416,000
Total	4,000,000

Source: JICA Study Team

CHAPTER 4 FLOODGATES AT MIRIM RIVER

4.1 Introduction

(1) General

Two(2) floodgates are proposed to be installed in the Old Mirim River as shown in Figure 4.1.1 below. The floodgates located downstream and upstream are called, in this report, "downstream floodgate" and "upstream floodgate" respectively.

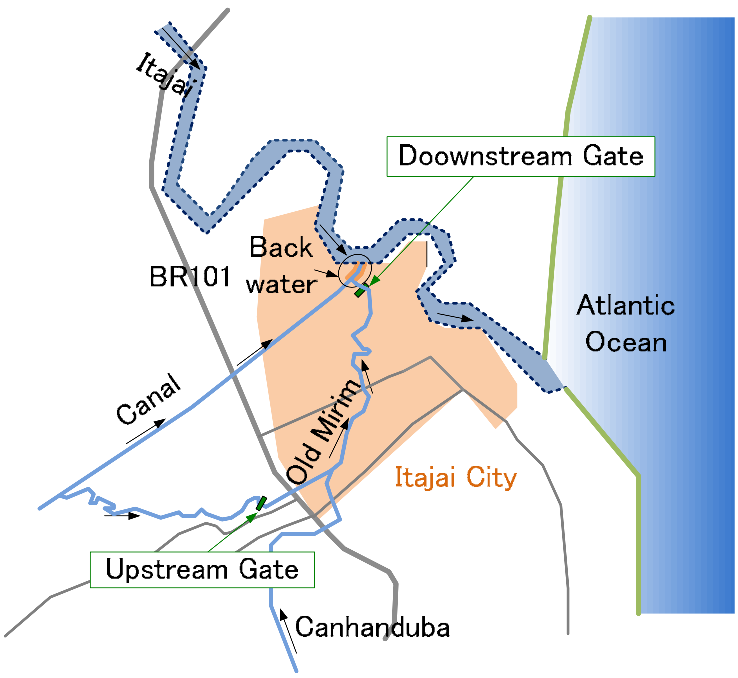
Both floodgates are proposed to be designed as 10-year flood control facilities in this study. However, the civil structure part of upstream floodgate (the main part of floodgate except the gate itself) is designed as a 50-year flood control facility due to the following reason:

In this study, the target is 10-year flood control. Generally, it is difficult for the civil structure to be extended -- 50-year flood control in this case. In contrast, it is not so difficult for the gate to be exchanged. Thus in this study the civil structure is designed as 50-year flood control and the gate is designed as 10-year flood control.

On the other hand, as for the downstream floodgate, the water level in the 50-year flood is less than that in the 10-year flood because the flood way is available in the Itajai River when the 50-year flood control plan is implemented. Thus the floodgate ability of 10-year flood control can cover that of 50-year flood control facility.

(2) Objective

Downstream gate:	Whole facility	10 - year flood control facility
Upstream gate:	Civil Structure	50 - year flood control facility
	Gate	10 - year flood control facility



Source: JICA Survey Team

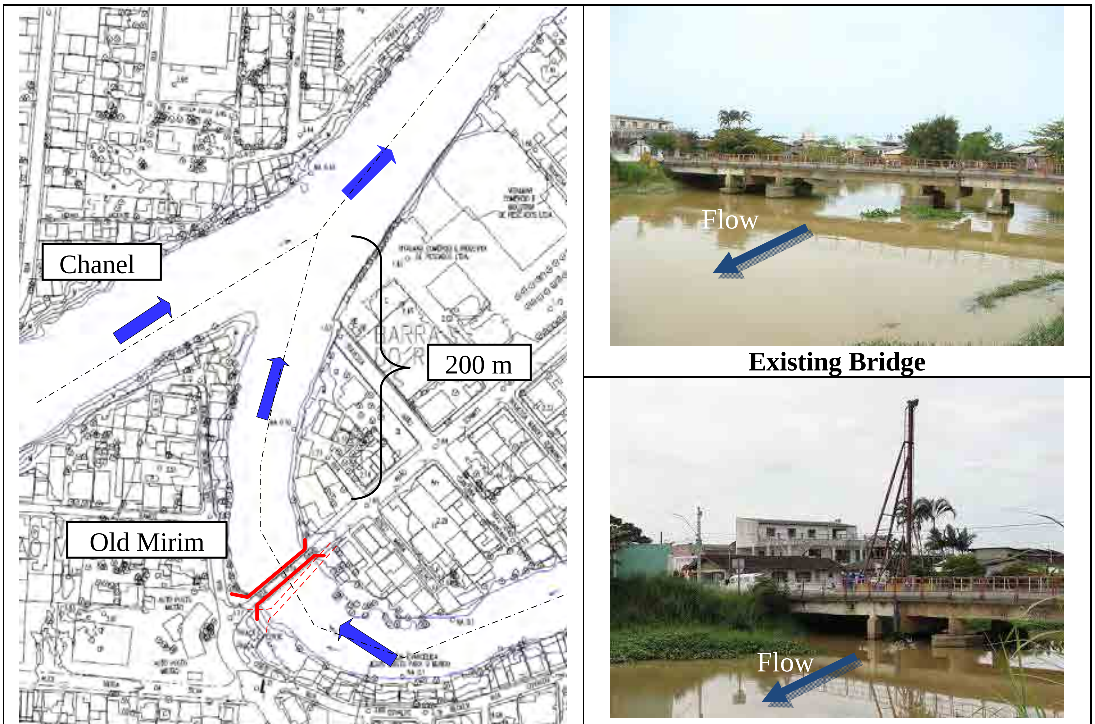
Figure 4.1.1 Location Map

4.2 Field Observation

4.2.1 Site property

(1) Downstream floodgate

The floodgate to be installed is located at the 200 m upstream from the point where the Canel and Old Mirim join. The planning point at the Master plan was downstream from the existing gate. But the new bridge is under construction since April, 2011.



Existing Bridge

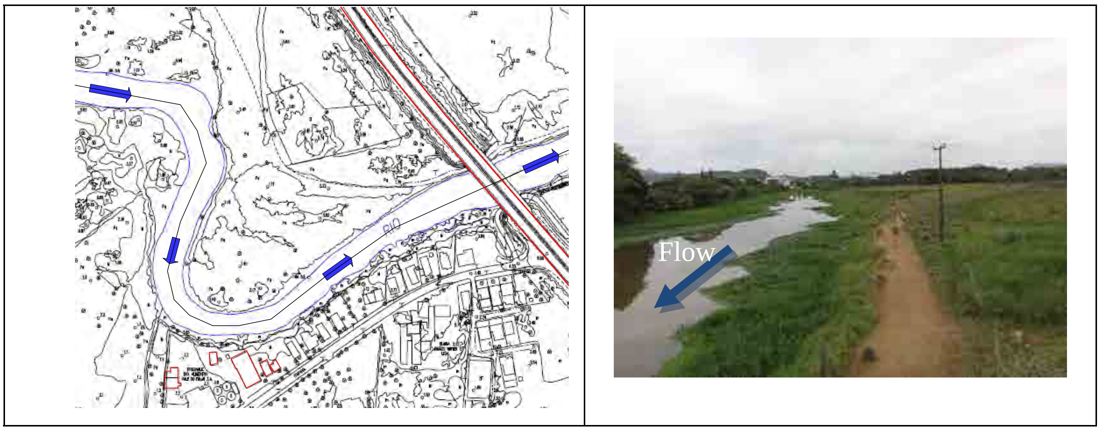
New Bridge (under construction)

Source: JICA Survey Team

Figure 4.2.1 Site at Planning Downstream Gate

(2) Upstream floodgate

The floodgate to be installed is located at the 250 m upstream from the point where BR101 and Old Mirim River crosses. There are few residences around there.



Source: JICA Survey Team

Figure 4.2.2 Site at Planning Upstream Gate

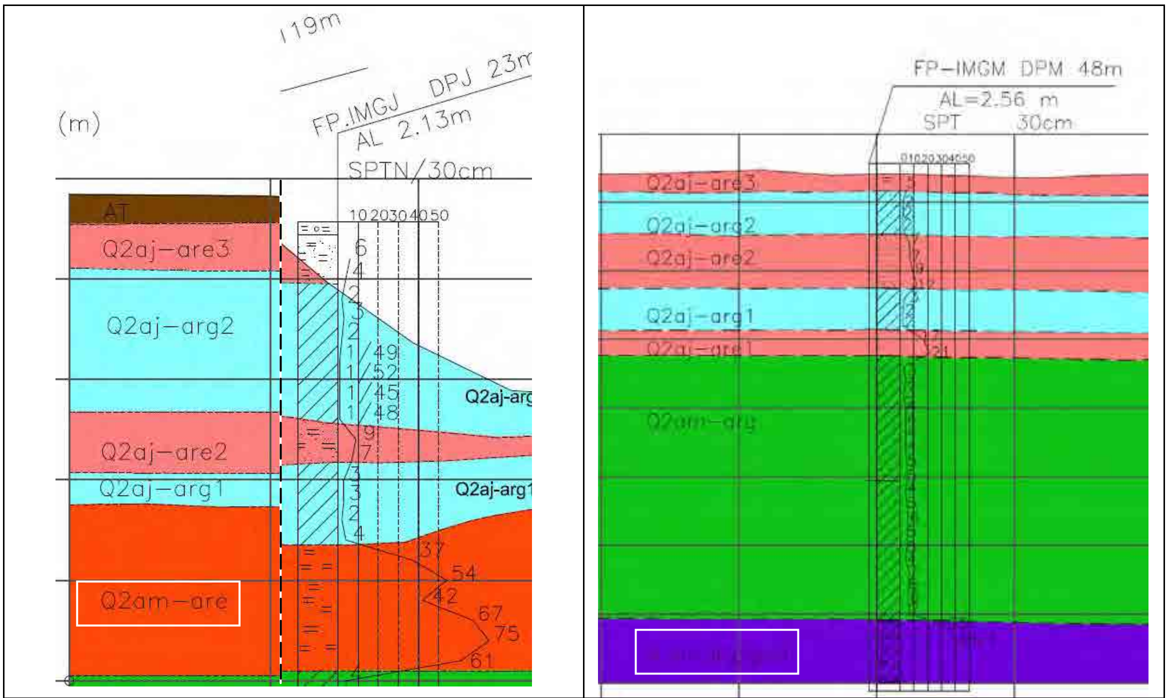
4.2.2 Geological

The geological conditions of both sites are poor and the foundations of structures are required to be the pile foundation as mentioned later in this report. The bearing layers of downstream and upstream gates is EL.-12.0 m and EL.-30.0 m respectively. As for the geological property, the details are shown in Supporting report C.

Table 4.2.1 Geological Property

Site	Layer	Type	Remarks
Downstream	Q2am-are	Middle Holocene sand 1	N=37, EL= －12 m～
Upstream	Q1a-are/ped	Pleistocene clay with Boulder	N=43, EL= －30 m～

Source: JICA Survey Team



downstream	upstream

Source: JICA Survey Team

Figure 4.2.3 Result of Geological Survey

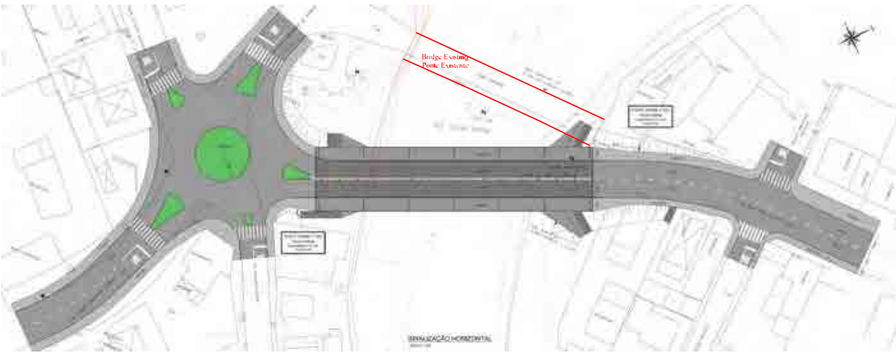
4.2.3 Environment and neighboring structure

(1) Bridge

The bridge which is controlled by Itajai city governor is now under construction. There is no information of the construction schedule but when the proposed floodgate would be constructed, the bridge must have been installed already. The type of bridge is the pretensioning system simple girder bridge.

(2) Gate

In Canal River, there is one(1) tide baffling gate. It is consists of eight(8) gates and the opening and closing system is rack system.



Source: Prefeitura Municipal de Itajaí

Figure 4.2.4 Constructing Bridge

(2) Gate

In Canal River, there is one(1) tide baffling gate. It is consists of eight(8) gates and the opening and closing system is rack system.

Source: JICA Survey Team

Existing Gate in Canal River

4.2.4 Construction Condition

(1) Downstream floodgate

As for the existing bridge, large vehicles can pass over the existing bridge. Thus there is no difficulty for vehicles to access the site.

The construction is required to avoid any impact on the new bridge. Also because there are residence near the planning site, it is necessary to consider the residents in terms of the vibration and noise.

(2) Upstream gate

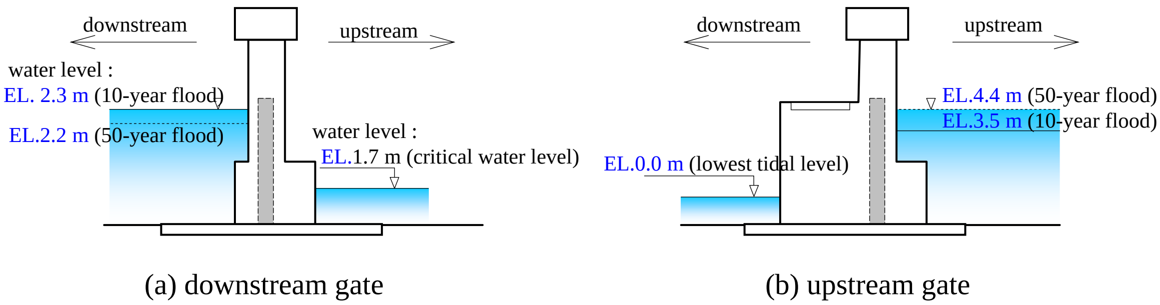
The access road to the site is available from BR101, so that it is not difficult for vehicles to transport. Also there are no residences around the planning site and the site for temporary diversion facility. So it is not necessary to consider the neighbors so far.

4.3 Basic Condition

4.3.1 Given Condition

(1) Water Condition

As mentioned in Supporting report B, the water condition is summarized as shown the Figure 4.3.1 below. At the downstream floodgate, the water level downstream(the Canel side) increases 2.3 m in 10-year flood while it increases 2.2 m in 50-year flood in times of flood. In the other hand, the upstream floodgate, the water level increase 3.5 m in 10-year flood and 4.4 m in 50-year flood.



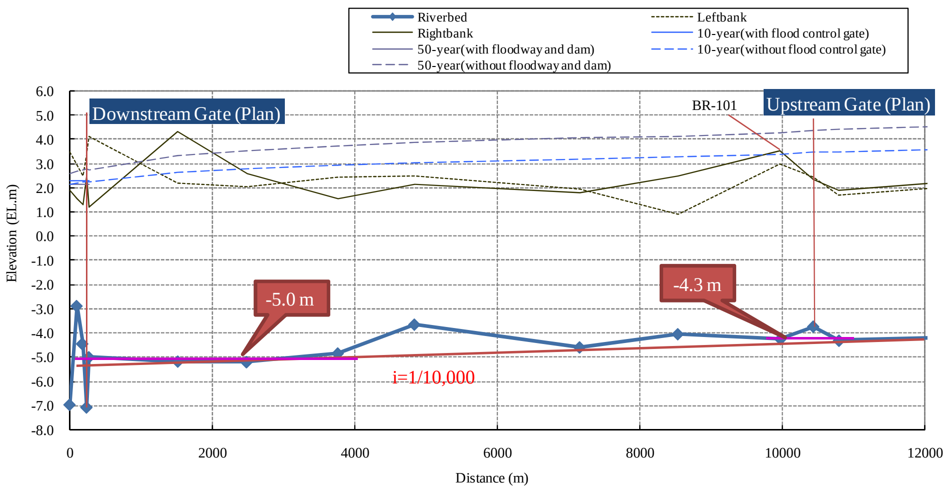
Source: JICA Survey Team

Figure 4.3.1 Design Water Levels of Floodgates

(2) River Condition

1) Slope of River Bed

The current condition in terms of the river bed is described as shown in Figure 4.3.2 below. The downstream and the upstream areas are almost flat, respectively - 4.3 m (downstream) and - 5.0 m (upstream).

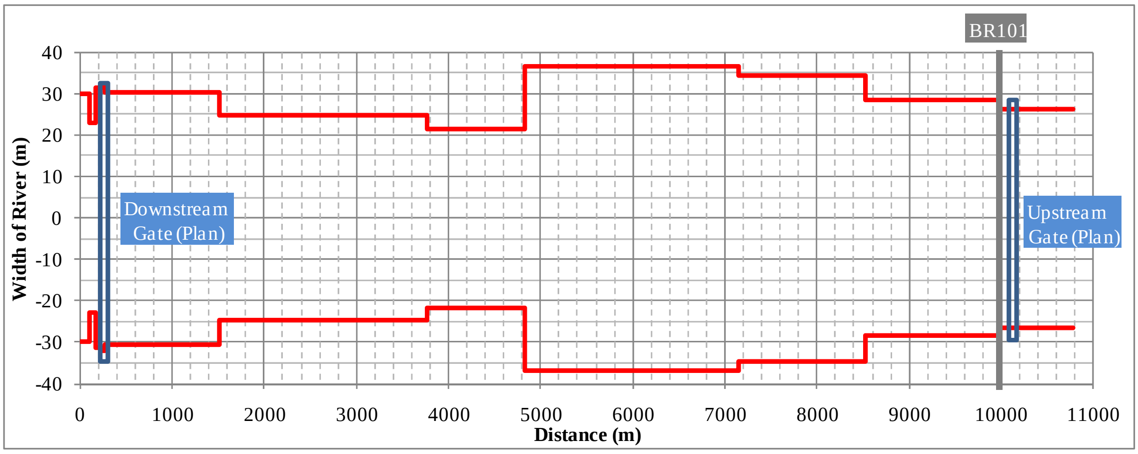


Source: JICA Survey Team

Figure 4.3.2 Profile of River Bed Sloop

2) Width of River

The current condition in terms of the width of the Old Mirim River is described as shown in Figure 4.3.3 below. The width of river ad downstream side and that of the upstream side are about 60.0 m (downstream) and 55.0 m (upstream) respectively.



Source: JICA Survey Team

Figure 4.3.3 Profile of River Width

4.3.2 Positioning the axis of water gate

Downstream gate

In the master plan phase, the axis of the floodgate was far from the existing bridge. In current state, the new bridge is now under construction and the existing bridge will be removed. Thus the axis of floodgate is at the existing bridge because of no land acquisition.

Upstream gate

With installing the floodgate, it needs the dike to connect the present roads (BR101 and Itaipava Avenue). The axis is proposed to be installed where the length of dike is more shorter and also consider the space to tuning flow when construction.

4.4 Design of water gate

4.4.1 Design of each structure

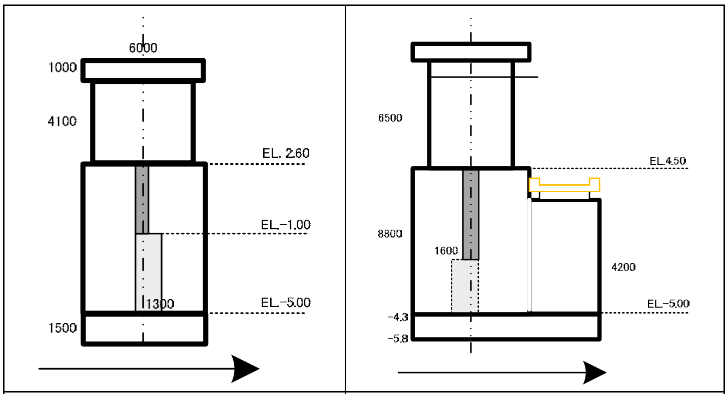
Main features of the designed floodgates are summarized in Table 4.4.1 below.

Table 4.4.1 Main Features of Floodgates

Gate	Downstream Gate	Upstream Gate
Nos. of Gate	3	3
Span of Gate	12.5 m	12.5 m
Foundation Elevation	EL.-5.0 m	EL.-4.3 m
Bottom Elevation of Gate	EL. -1.0 m	EL. -1.0 m
Main Structure	Separate slab and pier	Separate slab and pier
Gate Pier	EL. 7.70 m 6.00 m wide 14.20 m high	EL. 12.00 m 11.20 m wide 17.80 m high
Gate Operation System	On the top of pier	On the top of pier
Apron Length	6.0 m	8.0 m
Sheet Pile for Seepage	Downstream 2.0 m Upstream None	Downstream 2.5 m Upstream 5.5 m
Revetment	Downstream 10.0 m Upstream 10.0 m	Downstream 10.0 m Upstream none

Gate	Downstream Gate	Upstream Gate
Stair	Installed	Installed
Foundation	Pile foundation Pier :L=11.0 m ϕ 400 mm Slab :L=11.0 m ϕ 300 mm	Pile foundation Pier :L=27.0 m ϕ 400 mm Slab :L=27.0 m ϕ 300 mm

Source: JICA Survey Team



Downstream floodgate | **Upstream floodgate**

Source: JICA Survey Team

Figure 4.4.1 Profile of Gate

(1) Span Gates

The span gates are designed to be 12.5 m wide (required minimum size) as the following reasons. The number of gates is three (3) at both sites.

- To avoid the flow because the water gate is impediment of river flow.

- To avoid the case that the driftwood make the water gate close and lose its function.

- To make ship pass easily

(2) Foundation elevation

The foundation elevation is based on the present condition

Downstream

It was found that the part of a few areas was scoured by cross section survey, but the elevation of the foundation height is -5.0m to fit that of upstream and that of downstream.

Upstream

The elevation of the foundation height is EL.-4.3m to make the smooth flow from upstream to downstream.

(3) Bottom elevation of gates

It is supposed to avoid the impediment of river flow. Thus taking the following matters into consideration, the convex part (the under bed is higher than the other areas) is designed to be located the under bed at the point where the gate is closed/open.

(a) The gate is operated to open only in the normal flow, which means the flood (5-year or more flood) does not pass the gate.

(b) The normal flow is about 50 m^3/s; this value is equal to the flow capacity of the Old Mirim River.

(c) The height of the convex part (the under bed is higher than the other areas) is designed not to effect 50 m^3/s discharge.

(d) The downstream and upstream gate are located in estuary area(affecting tide). Thus the height of that is designed to be located under the lowest tide (EL.0.00 m).

(e) The space where ships can pass the gate is needed. Judging from the field survey, the draft of ships (the vertical distance between the waterline and the bottom of the hull) is EL. - 1.0 m.

(f) Neighbors and residence might worry about floods even thought the operation works well. Thus the convex shall be always under the water.

To satisfy these conditions, the elevation of foundation at under bed should be designed to be located EL.-1.00 m. The width of crest is requisite minimum size for open/close gates.

(4) Main Structure

The main structure is separated between the slab and pier for the following reasons.

-Span gates is 12.5 m and long.

-To reduce the number of piles for foundation

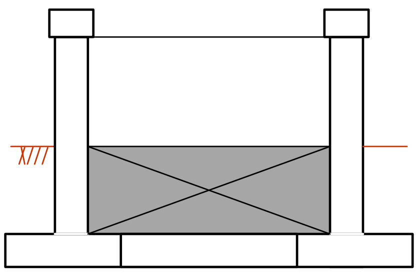


Figure 4.4.2 Image of Separate Type of Gate

(5) Length of main pier

The length of main pier is designed in terms of the structural stability.

<u>Downstream</u>

6.0 m

<u>Upstream</u>

8.0 m (including the bridge for maintenance)

(6) Gate Pier

1) Height

The height is designed in terms of the operation gates.

<u>Downstream</u>

EL.7.70 m

<u>Upstream</u>

EL.12.00 m

2) Width

<u>Downstream</u>

6.00 m

<u>Upstream</u>

11.20 m

3) Length

<u>Downstream</u>

14.20 m (+E.L. 7.70 m to -E.L. 6.50 m)

<u>Upstream</u>

17.80 m (+E.L.12.00 m to -E.L. 5.80 m)

(7) Gate operation system

Operation system is installed on the top of the gate pire.

(8) Apron and Bed Protection

It is supposed to lay the apron/bed protection to protect against scouring since the hydraulic jumping and rapid flow make flow instability. In case of this gate operating, the normal flow does not create hydraulic jumping and rapid flow basically. Thus it is not necessary to lay the apron/bed protection.

However the pier needs the length and width for the structural stability, so the slab is also equipped with the apron – the sam as the pire in length. And the bed protection at downstream side is for the normal flow as safety.

<u>Downstream Gate</u>

Apron 6.0 m (including the pier), Bed Protection 10.0 m (downstream), none (upstream)

<u>Upstream Gate</u>

Apron 8.0 m (including the pier), Bed Protection 10.0 m (downstream), none (upstream)

(9) Seepage Control Work

The length of seepage control work is calculated by the Lane's weighted creep theory. The equation is shown below.

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

Where, C:the rait of Creap (the table below) , L:the length of the main body and apron,

$\sum l$:the seepage vertical length, Δh : The maximum water difference.

Soil Type	C	Soil Type	C
fine sand or silt	8.5	coarse sand and gravel	4.0
fine sand	7.0	medium-gravel	4.0
medium sand	6.0	coarse sand and gravel with cobblestone	3.0
coarse sand	5.0	gravel with cobblestone	3.0

(11) Revetment

The length of river protextion is 10.0 m follow the neighbors’ structure.

(12) Stair

The operation system for gate is over the pier. Thus stairs are required to access the tops of both gates.

(14) Type of foundation

As mentioned in the next section, the foundation of both floodgates is the type of pile foundation.

<u>Downstream Gate</u>

Pier: L = 11.0 m (φ400 mm), Slab: L = 11.0 m (φ300 mm)

<u>Upstream Gate</u>

Pier: L = 27.0 m (φ400 mm), Slab: L = 27.0 m (φ300 mm)

(15) Dyke

<u>Downstream Gate</u>

Install the backwater dike

<u>Upstream Gate</u>

Install the closure dike.

4.4.2 Stability Analysis

Stability analysis about the pier and slab is estimated.

(1) Water Condition

Downstream floodgate : 1.7 m (Upstream)
2.3 m (Downstream)

Upstream floodgate : 4.4 m (Upstream)
0.0 m (Downstream)

(2) Stability Condition

1) Sliding and Overturning

Safety factor against Sliding and overturning is summarized as shown table below.

2) Bearing Capacity

Safety capacity is in normal condition.

Table 4.4.2 Stability Condition

	Sliding (Safety Factor)	**Overturning (e :Distance from the point of load acting)**	**Bearing Capacity**
Normal	Fs=1.5	$\|e\| \leq \frac{B}{6}$, B = base width	at normal condition
Construction	Fs=1.2	$\|e\| \leq \frac{B}{3}$, B = base width	--

Source : JICA survey team

(2) Analysis

(1) Stability Analysis

Downstream

1) Pier

Construction

	Vertical Force	x	N ・ x	Horizontal Force	y	N ・ y
	kN	m	kN ・ m	kN	m	kN ・ m
Pier 1	588.0	3.0	1764.0			
Pier 2	1255.6	3.0	3766.9			
Pier 3	2756.3	3.0	8268.8			
Removal Space	-105.8	3.0	-317.5			
Slab	882.0	3.0	2646.0			
Upper load	84.0	3.0	252.0			
Gate1	282.6	3.0	847.8			
Gate2	519.2	3.0	1557.6			
Σ	6261.8		18785.5			

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(18785.52 – 0)/ 6261.84 = 3.0 m

$e = B/2 - d = 0.0m < 6.00/6 = 1.0m$ (satisfied)

Subgrade Reaction : Q

$$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{6261.84}{6.00 \cdot 4.50}\left(1 \pm \frac{6 \cdot 0}{6.00}\right) = 231.92 \pm 0.0 \ \text{kN/m}^2$$

2) Flood

	Vertical Force	x	N ・ x	Horizontal Force	y	N ・ y
	kN	m	kN ・ m	kN	m	kN ・ m
Pier 1	588.0	3.0	1764.0			
Pier 2	1255.6	3.0	3766.9			
Pier 3	2756.3	3.0	8268.8			
Removal Space	-105.8	3.0	-317.5			
Slab	882.0	3.0	2646.0			
Upper load	84.0	3.0	252.0			
Water Pressure 1				990.0	2.4	2366.1
Water Pressure 2				357.2	6.4	2286.1
Water Pressure 3				-1093.5	2.1	-2329.2
Water Pressure 4				-489.3	6.6	-3206.5
Up lift	-1341.8	5.4	-7191.8			
Gate1	282.6	3.0	847.8			
Gate2	519.2	3.0	1557.6			
Σ	4836.1		11341.7	-235.6		-883.4

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(11341.74 – 883.40)/ 4836.09 = 2.16 m

$e = B/2 - d = 0.84m < 6.00/6 = 1.0m$ ok

Subgrade Reaction : Q

$$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{4836.09}{6.00 \cdot 4.50}\left(1 \pm \frac{6 \cdot 0.84}{6.00}\right) = 179.11 \pm 150.45 = 329.56 \text{ or } 28.66 \ \text{kN/m}^2$$

2) Slab

Construction

	Vertical Force	x	N・x	Horizontal Force	y	N・y
	kN	m	kN・m	kN	m	kN・m
Convex	1568.0	3.3	5213.6			
Slab	2744.0	3.0	8232.0			
Σ	4312.0		13445.6			

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(13445.6– 0)/ 4312 = 3.12 m

$e = B/2 - d = -0.12m < 6.00/6 = 1.0m$ (satisfied)

Subgrade Reaction : Q

$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right)$ = $\frac{4312}{6.00 \cdot 4.50}\left(1 \pm \frac{6 \cdot -0.12}{6.00}\right) = 159.7 \pm -19.16$ = 140.54 or 178.86 kN/m^2

2) Flood

	Vertical Force	x	N・x	Horizontal Force	y	N・y
	kN	m	kN・m	kN	m	kN・m
Convex	1568.0	3.3	5213.6			
Slab	2744.0	3.0	8232.0			
Water weight 1	196.0	1.0	196.0			
Water weight 2	235.2	4.8	1129.0			
Water Pressure 1				357.0	1.9	678.3
Water Pressure 2				-490.0	2.1	-1006.0
Up lift	-1341.8	5.4	-7191.8			
Σ	3401.5		7578.8	-133.0		-327.7

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(7578.78 – 327.67) / 3401.45 = 2.13 m

$e = B/2 - d = 0.87m < 6.00/6 = 1.0m$ ok

Subgrade Reaction : Q

$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right)$ = $\frac{3401.45}{6.00 \cdot 4.50}\left(1 \pm \frac{6 \cdot 0.87}{6.00}\right) = 125.98 \pm 109.60$ = 235.58 or 16.38 kN/m^2

Upstream

1) Pier

Construction

	Vertical Force	x	N・x	Horizontal Force	y	N・y
	kN	m	kN・m	kN	m	kN・m
Pier 1	686.0	3.5	2401.0			
Pier 2	1990.6	3.5	6967.2			
Pier 3	3773.0	3.5	13205.5			
Removal Space	-256.0	3.5	-896.1			
Pier 4	1762.2	9.1	16035.7			

Slab	1646.4	5.6	9219.85			
Bridge	231.5	9.1	2106.9			
Upper load 1	367.5	9.1	3344.3			
Upper load 2	98	3.5	343.0			
Gate1	412.1	3.5	1442.4			
Gate2	733.6	3.5	2567.6			
Σ	11444.9		56737.2			

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(56737.22 – 0)/ 11444.89 = 4.96 m

$e = B/2 - d = 0.64m < 11.20/6 = 1.9m$ (satisfied)

Subgrade Reaction : Q

$$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{11444.89}{11.20 \cdot 4.50}\left(1 \pm \frac{11.20 \cdot 0.64}{11.20}\right) = 227.08 \pm 77.86 = 304.94 \text{ or } 149.22 \text{ kN/m}^2$$

2) Flood (50 year)

	Vertical Force	x	N · x	Horizontal Force	y	N · y
	kN	m	kN · m	kN	m	kN · m
Pier 1	686.0	3.5	2401.0			
Pier 2	1990.6	3.5	6967.2			
Pier 3	3773.0	3.5	13205.5			
Removal Space	-256.0	3.5	-896.1			
Pier 4	1762.2	9.1	16035.7			
Slab	1646.4	5.6	9219.85			
Bridge	231.5	9.1	2106.9			
Water Pressure 1				1509.4	3.1	-4603.7
Water Pressure 2				1482.3	6.6	9832.3
Water Pressure 3				-900.7	2.1	-1864.4
Water Pressure 4				-441.0	5.8	-2557.8
Uplife	-2825.2	5.1	-14408.5			
Gate1	412.1	3.5	1442.4			
Gate2	733.6	3.5	2567.6			
Σ	11444.9		56737.2	1650.0		10013.8

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(38641.45 - 10013.78)/ 8154.19 = 5.97 m

$e = B/2 - d = -0.37m < 11.20/6 = 1.9m$ ok

Subgrade Reaction : Q

$$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{8154.19}{11.20 \cdot 4.50}\left(1 \pm \frac{6 \cdot -0.37}{11.20}\right) = 161.79 \pm 32.07 = 129.72 \text{ or } 193.86 \text{ kN/m}^2$$

3) Flood (10 year)

	Vertical Force	x	N · x	Horizontal Force	y	N · y
	kN	m	kN · m	kN	m	kN · m
Pier 1	686.0	3.5	2401.0			
Pier 2	1990.6	3.5	6967.2			
Pier 3	3773.0	3.5	13205.5			
Removal Space	-256.0	3.5	-896.1			
Pier 4	1762.2	9.1	16035.7			
Slab	1646.4	5.6	9219.85			
Bridge	231.5	9.1	2106.9			
Water Pressure 1				1248.1	2.7	3419.8
Water Pressure 2				992.3	6.3	6251.2

Water Pressure 3				-900.7	2.1	-1864.4
Water Pressure 4				-441.0	5.8	-2557.8
Uplife	-2345.6	5.4	-12666.0			
Gate1	412.1	3.5	1442.4			
Gate2	733.6	3.5	2567.6			
Σ	11444.9		40384.0	898.7		5248.8

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ = (40383.95 – 5248.82) / 8633.83 = 5.29 m

$e = B/2 - d = 0.31m < 11.20/6 = 1.9m$ ok

Subgrade Reaction : Q

$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{8633.83}{11.20 \cdot 4.50}\left(1 \pm \frac{6 \cdot 0.31}{11.20}\right) = 171.31 \pm 28.45$ = 199.76 or 142.86 kN/m^2

2) Slab

Construction

	Vertical Force	x	N ・ x	Horizontal Force	y	N ・ y
	kN	m	kN ・ m	kN	m	kN ・ m
Convex	1293.6	3.9	4980.4			
Slab	2744.0	5.6	15366.4			
Σ	4037.6		20346.8			

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ =(20346.76 – 0)/ 4037.6 = 5.04 m

$e = B/2 - d = -0.56m < 11.20/6 = 1.9m$ (satisfied)

Subgrade Reaction : Q

$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{4037.6}{11.20 \cdot 4.50}\left(1 \pm \frac{6 \cdot -0.56}{11.20}\right) = 80.11 \pm 24.03$ = 104.16 or 56.08 kN/m^2

Flood

	Vertical Force	x	N ・ x	Horizontal Force	y	N ・ y
	kN	m	kN ・ m	kN	m	kN ・ m
Convex	1293.6	3.3	4980.4			
Slab	2744.0	5.6	15366.4			
Water weight 1	2630.3	1.5	4024.4			
Water weight 2	4044.0	7.9	32048.5			
Water Pressure 1				2312.0	2.3	5317.6
Water Pressure 2				1503.8	1.5	-2255.7
Up lift	-2825.2	5.1	--14408.5			
Σ	7886.7		42011.1	808.2		3061.9

The distance from the point of resultant force from the center of slab : e

$d = (\sum N \cdot x - \sum H \cdot y)/\sum N$ = (42011.09 – 3061.9) / 7886.69 = 5.72 m

$e = B/2 - d = -0.12m < 11.20/6 = 1.9m$ (satisfied)

Subgrade Reaction : Q

$e = \frac{\sum N}{B \cdot L}\left(1 \pm \frac{6 \cdot e}{B}\right) = \frac{7886.69}{11.20 \cdot 4.50}\left(1 \pm \frac{6 \cdot -0.12}{11.20}\right) = 156.48 \pm 10.06$ = 146.42 or 166.54 kN/m^2

4.4.3 Foundation

(1) Mode of foundation

Downstream

The layer which is just below the slab is Cray-layer whose N-value is 2. The good quality layer is considered under 12.0 m or deeper. This floodgate is the type that the pier is above the ground. Thus the direct foundation is inappropriate.

Upstream

The layer which is just below the slab is Sand-layer whose N-value is 7. The good quality layer is considered under 30.0 m or deeper. This floodgate is the type that the pier is above the ground. Thus the direct foundation is inappropriate.

As mentioned above, since both sites are not suitable to the direct foundation, the foundation is pile foundation.

(2) Load bearing layer

The bearing layer of foundation is designed to set at the good quality layer. More detailed information of geology is mentioned on Supporting B.

Site	**Layer**	**Remarks**
Downstream	Qam-are2: Clay	N=37, EL= －12 m～
Upstream	Q1a-are/ped Clay with Boullder	N=43, EL= －30 m～

4.4.4 Designed sheet pile

(1) Calculation method

The design for sheet piles is calculated as right flow. This method for calculating the number of the pile is simplified equation. The detailed design requires to calculate as displacement method.

1) Design load to pile foundation plane section

2) Allowable bearing per one pile.

3) Set the number of piles and layout

4) Test the occurring the compressive stress

1) Design load to pile foundation plane section

Load condition is below.

Downstream floodgate : 6261.8 kN as the pier

4312.0 kN as the slab

Upstream floodgate : 11444.9 kN as the pier

4037.6 kN as the slab

2) Ultimate bearing capacity per one(1) pile

The calculation formula is below.

$$R_u = q_d \cdot A_p + A_s f \quad Q_u = \frac{R_u}{3}$$

R_u :ultimate bearing capacity

q_d :ultimate end bearing pressure

A_p :end bearing contact area

A_s :skin friction contact area

f :ultimate skin friction stress

3) Calculate the number of pile

$$P = \frac{V_0}{n} + \frac{e \times V_0}{\sum x_i^2} \cdot x_i \quad \text{(kN per One (1) pile)}$$

P : Maximum force to pile

V_0 : Subgrade reaction

e : Eccentricity force

n : Number of the pile

x_i : No. i moment of group of pile.

$\sum x_i^2$: Second moment of group of pile. (Nos. ・m^2)

$$P\max = \alpha \times P \le Q_u$$

(2) Calculation Result

The calculation sheets were shown below.

<u>Downstream</u>

The allowable bearing capacity is 627.98 kN/nos. as ϕ400, and 369.67 kN/nos as ϕ300. The length of piles is 11.0 .m.

The required number of sheet pile – ϕ400 is more than 10 nos for pier.

The required number of sheet pile – ϕ300 is more than12 nos for slab.

<u>Upstream</u>

The allowable bearing capacity is 588.94 kN/nos. as ϕ400, and 359.24 kN/nos. as ϕ300. The length of that is 27.0 .m.

The required number of sheet pile is more than 20 nos for pier.

The required number of sheet pile is more than 12 nos for slab.

Pile size: ϕ 300(Downstream)

1. Design Data

(1) Allowable capacity of pile

a) Condition of Pile

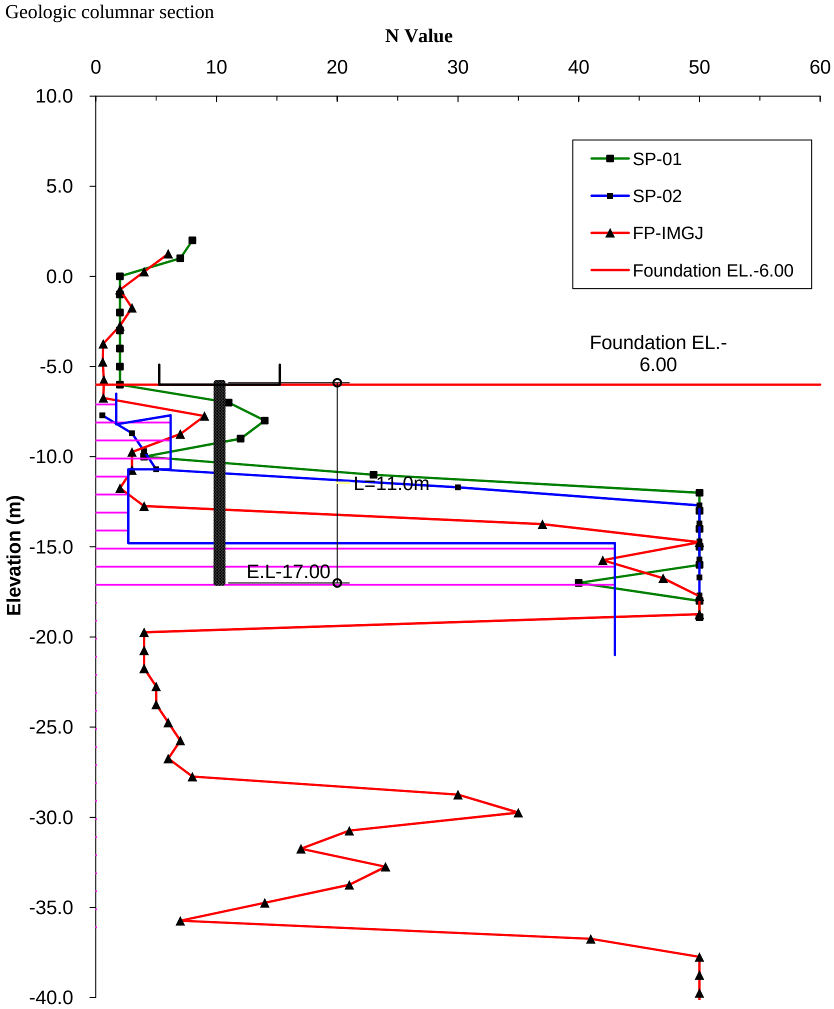
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ300	(mm)
	Thickness	60	(mm)

b) Allowable bearing capacity (Ra)

Data :	River bed (EL.)		-5.00	
	Footing Top Level (EL.)		-6.00	
	L	(m)	11.0	(length of pile)
	D	(m)	0.30	(width of pile)
	n		3	(safety factor: normal condition)
	n		2	(safety factor: seismic condition)
	Ap	(m2)	0.0707	(area of pile top effective in bearing)
	U	(m)	0.942	(peripheral length of pile)
	l	(m)	10.9	(embedded pile length)

2. Pile Arrangement of Longitudinal Direction

(1) Geologic columnar section



Result of Standard Penetration Test

3. Allowable bearing capacity

(1) Ultimate end Bearing Capacity

1) Compensation of N-value

N-value of the pile end ground for use in destaining.

N= 43 (N-value of the pile end)

2) Estimation of ultimate end bearing capacity

-For piles other than open tip steel pile

$$\frac{qd}{N} = (40*\frac{Df}{D} + 100) = (40*\frac{1.50}{0.30} + 100) = 300$$

where: D= 0.30 m

Df=5xD= 1.50 m

qd = 300 x 43 = 12900 kN/m^2

3) Estimation of the maximum skin friction power

The friction resistance contribution (Fs) was calculated as follows:

Fs (kN/m^2)

2.0

Table of friction resistance (Fs)

Foundation soil \ Cons. method	Precast
Sandy soil	2*N (≤ 100)
Cohesif soil	C (≤ 150)

N value	Layer thick(m)	Sandy 2*N	Cohesif	Fs (kN/m^2)	U (m)	U*Fs (kN)
---	1.0		11.0	11.00		
6.2	1.0	12.4		12.40	0.942	11.69
6.2	1.0	12.4		12.40	0.942	11.69
6.2	1.0	12.4		12.40	0.942	11.69
---	1.0		17.0	17.00		
---	1.0		17.0	17.00		
---	1.0		17.0	17.00		
---	1.0		17.0	17.00		
43.0	1.0	86.0		86.00	0.942	81.05
43.0	1.0	86.0		86.00	0.942	81.05
43.0	1.0				0.942	

	11.00				Total	197.17

Under normal condition ,Under flood condition

Ra = (qd.Ap + UlFs) / n

= (12900 x 0.0707 + 197.17) / 3 = 369.67 kN/nos

Pile size: ϕ 400(Downstream)

1. Design Data

(1) Allowable capacity of pile

a) Condition of Pile

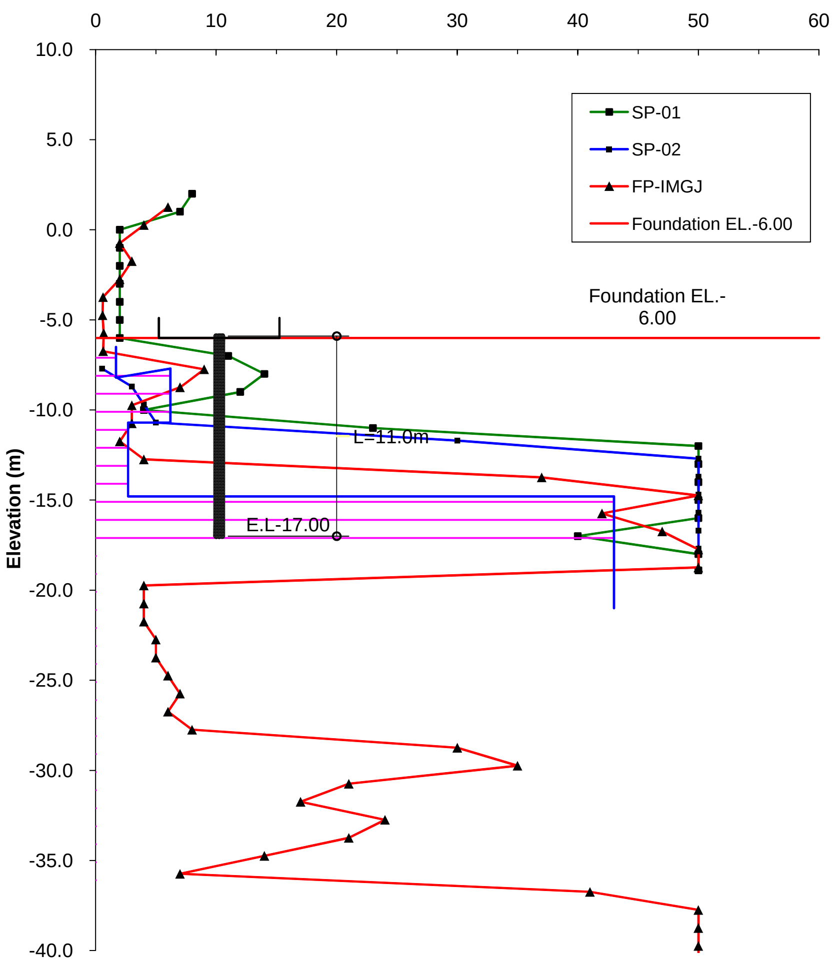
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ400	(mm)
	Thickness	75	(mm)

b) Allowable bearing capacity (Ra)

Data :	River bed (EL.)		-5.00	
	Footing Top Level (EL.)		-6.00	
	L	(m)	11.0	(length of pile)
	D	(m)	0.40	(width of pile)
	n		3	(safety factor: normal condition)
	n		2	(safety factor: seismic condition)
	Ap	(m2)	0.1257	(area of pile top effective in bearing)
	U	(m)	1.257	(peripheral length of pile)
	l	(m)	10.9	(embedded pile length)

2. Pile Arrangement of Longitudinal Direction

(1) Geologic columnar section



Result of Standard Penetration Test

3. Allowable bearing capacity

(1) Ultimate end Bearing Capacity

1) Compensation of N-value

N-value of the pile end ground for use in destaining.

N= 43 (N-value of the pile end)

2) Estimation of ultimate end bearing capacity

-For piles other than open tip steel pile

$$\frac{qd}{N} = (40*\frac{Df}{D} + 100) = (40*\frac{2.00}{0.40} + 100) = 300$$

where: D= 0.40 m

Df=5xD= 2.00 m

qd = 300 x 43 = 12900 kN/m^2

3) Estimation of the maximum skin friction power

The friction resistance contribution (Fs) was calculated as follows:

Fs (kN/m^2) 2.0

Table of friction resistance (Fs)

Foundation soil \ Cons. method	Precast
Sandy soil	2*N (≤ 100)
Cohesif soil	C (≤ 150)

N value	Layer thick(m)	Sandy 2*N	Cohesif	Fs (kN/m^2)	U (m)	U*Fs (kN)
---	1.0		11.0	11.00		
6.2	1.0	12.4		12.40	1.257	15.58
6.2	1.0	12.4		12.40	1.257	15.58
6.2	1.0	12.4		12.40	1.257	15.58
---	1.0		17.0	17.00		
---	1.0		17.0	17.00		
---	1.0		17.0	17.00		
---	1.0		17.0	17.00		
43.0	1.0	86.0		86.00	1.257	108.07
43.0	1.0	86.0		86.00	1.257	108.07
43.0	1.0				1.257	

	11.00				Total	262.89

<u>Under normal condition ,Under flood condition</u>

Ra = (qd.Ap + UlFs) / n

= (12900 x 0.1257 + 262.89) / 3 = 627.98 kN/nos

Pile size: ϕ 300(Upstream)

1. Design Data

(1) Allowable capacity of pile

a) Condition of Pile

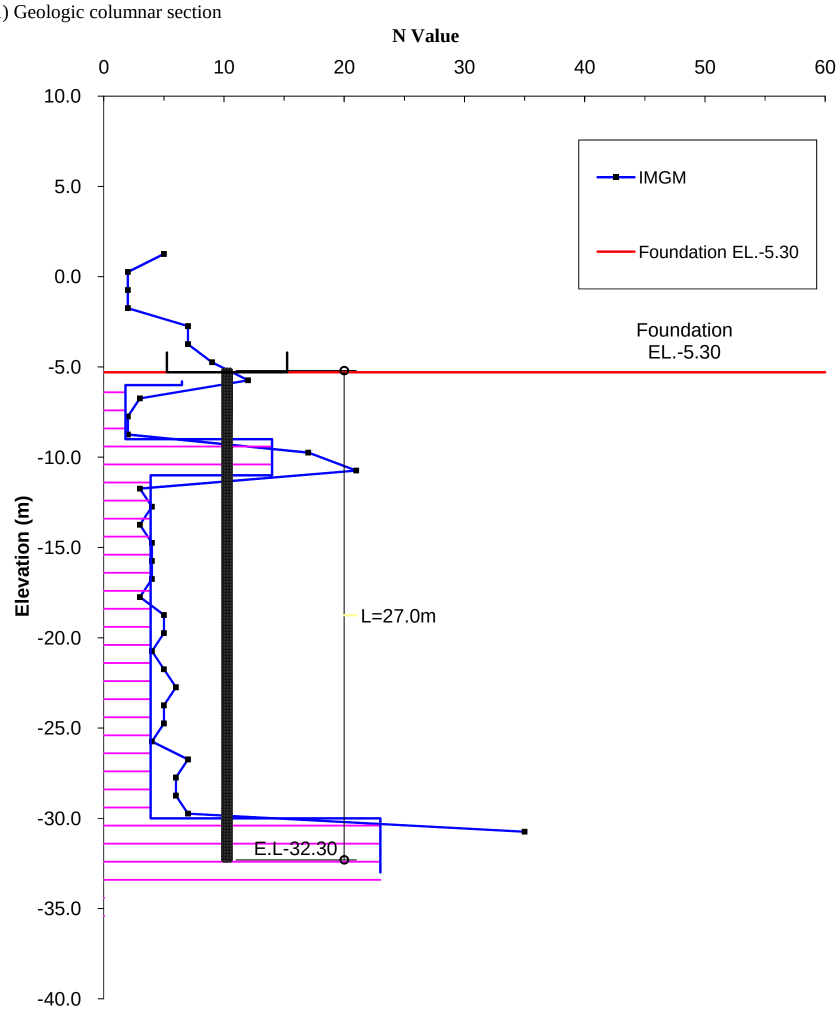
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ300	(mm)
	Thickness	60	(mm)

b) Allowable bearing capacity (Ra)

Data :	River bed (EL.)		-4.30	
	Footing Top Level (EL.)		-5.30	
	L	(m)	27.0	(length of pile)
	D	(m)	0.30	(width of pile)
	n		3	(safety factor: normal condition)
	n		2	(safety factor: seismic condition)
	Ap	(m2)	0.0707	(area of pile top effective in bearing)
	U	(m)	0.942	(peripheral length of pile)
	l	(m)	26.9	(embedded pile length)

2. Pile Arrangement of Longitudinal Direction

(1) Geologic columnar section



Result of Standard Penetration Test

3. Allowable bearing capacity

(1) Ultimate end Bearing Capacity

1) Compensation of N-value

N-value of the pile end ground for use in destaining.

N= 35 (N-value of the pile end)

2) Estimation of ultimate end bearing capacity

-For piles other than open tip steel pile

$$\frac{qd}{N} = (40^* \frac{Df}{D} + 100) = (40^* \frac{1.50}{0.30} + 100) = 300$$

where: D= 0.30 m

Df=5xD= 1.50 m

qd = 300 x 35 = 10500 kN/m^2

3) Estimation of the maximum skin friction power

The friction resistance contribution (Fs) was calculated as follows:

Fs (kN/m^2)
2.0

Table of friction resistance (Fs)

Cons. method / Foundation soil	Precast
Sandy soil	2*N (≤ 100)
Cohesif soil	C (≤ 150)

N value	Layer thick(m)	Sandy 2*N	Cohesif	Fs (kN/m^2)	U (m)	U*Fs (kN)
---	1.0		11.0	11.00		
---	1.0		11.0	11.00		
---	1.0		11.0	11.00		
14.0	1.0	28.0		28.00	0.942	26.39
14.0	1.0	28.0		28.00	0.942	26.39
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
23.0	1.0		100.0	100.00	0.942	94.25
23.0	1.0		100.0	100.00	0.942	94.25
23.0	1.0		100.0	100.00	0.942	94.25
	27.00				Total	335.52

Under normal condition ,Under flood condition

Ra = (qd.Ap + UlFs) / n

= (10500 x 0.0707 + 335.52) / 3 = 359.24 kN/nos

Pile size: ϕ 400(Upstream)

1. Design Data

(1) Allowable capacity of pile

a) Condition of Pile

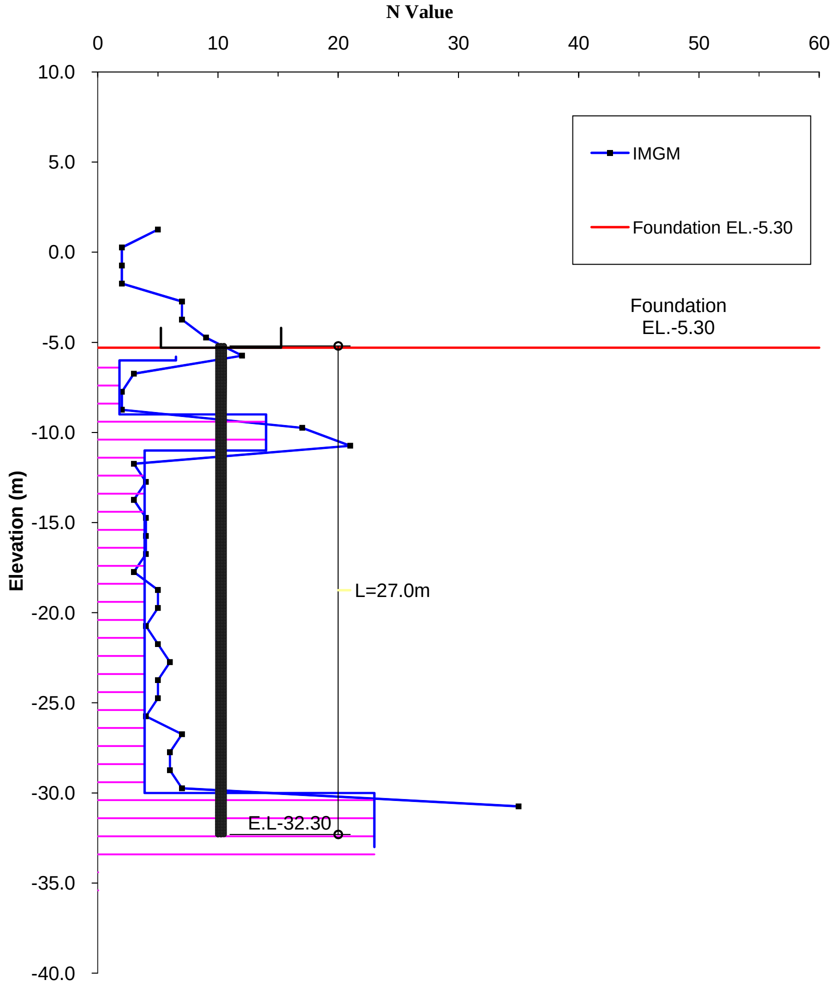
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ400	(mm)
	Thickness	75	(mm)

b) Allowable bearing capacity (Ra)

Data :	River bed (EL.)		-4.30	
	Footing Top Level (EL.)		-5.30	
	L	(m)	27.0	(length of pile)
	D	(m)	0.40	(width of pile)
	n		3	(safety factor: normal condition)
	n		2	(safety factor: seismic condition)
	Ap	(m2)	0.1257	(area of pile top effective in bearing)
	U	(m)	1.257	(peripheral length of pile)
	l	(m)	26.9	(embedded pile length)

2. Pile Arrangement of Longitudinal Direction

(1) Geologic columnar section



Result of Standard Penetration Test

3. Allowable bearing capacity

(1) Ultimate end Bearing Capacity

1) Compensation of N-value

N-value of the pile end ground for use in destaining.

N= 35 (N-value of the pile end)

2) Estimation of ultimate end bearing capacity

-For piles other than open tip steel pile

$$\frac{qd}{N} = (40^* \frac{Df}{D} + 100) = (40^* \frac{2.00}{0.40} + 100) = 300$$

where: D= 0.40 m

Df=5xD= 2.00 m

qd = 300 x 35 = 10500 kN/m^2

3) Estimation of the maximum skin friction power

The friction resistance contribution (Fs) was calculated as follows:

Fs (kN/m^2) 2.0

Table of friction resistance (Fs)

Cons. method / Foundation soil	Precast
Sandy soil	2*N (≤ 100)
Cohesif soil	C (≤ 150)

N value	Layer thick(m)	Sandy 2*N	Cohesif	Fs (kN/m^2)	U (m)	U*Fs (kN)
---	1.0		11.0	11.00		
---	1.0		11.0	11.00		
---	1.0		11.0	11.00		
14.0	1.0	28.0		28.00	1.257	35.19
14.0	1.0	28.0		28.00	1.257	35.19
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
---	1.0		24.0	24.00		
23.0	1.0		100.0	100.00	1.257	125.66
23.0	1.0		100.0	100.00	1.257	125.66
23.0	1.0		100.0	100.00	1.257	125.66
	27.00				Total	447.36

Under normal condition ,Under flood condition

Ra = (qd.Ap + UlFs) / n

= (10500 x 0.1257 + 447.36) / 3 = 588.94 kN/nos

4.5 Backwater Dyke

4.5.1 General

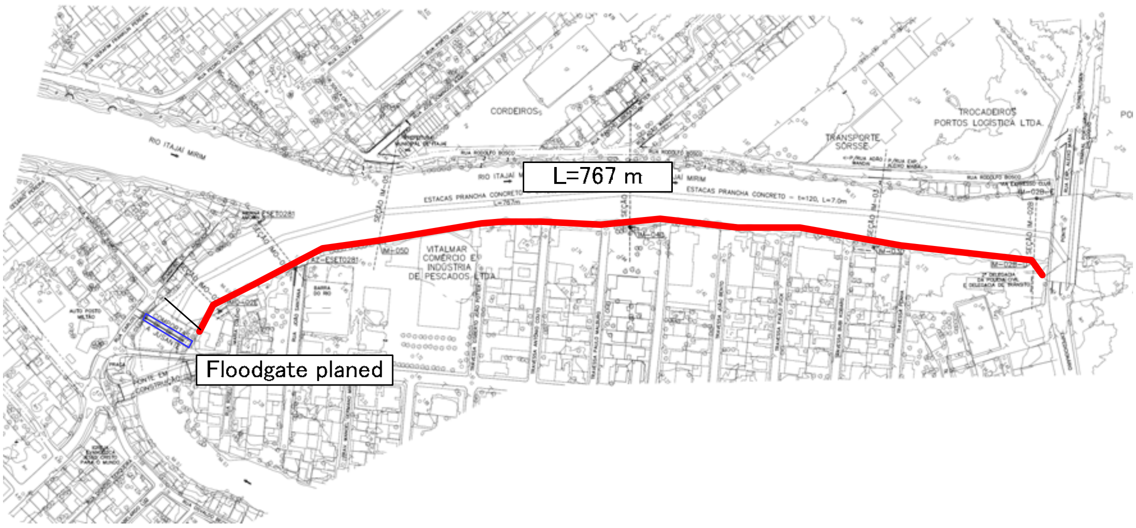
Around 1.0 km long river stretch along the Itajai Mirim River between the confluence with the Itajai River and the downstream floodgate is subject to backwater effect of the Itajai River. The river cross section survey along the Itajai Mirim River has revealed the following findings (see Figure 4.1.6):

i) The existing road (the Rodolfob Bosco Avenue) on the left bank has sufficient elevation to the design water level of 10-year flood and functions as a dyke.

ii) In the stretch of around 0.5-1.0 km from the confluence of Itajai River, height of the right bank is lower than the water level of 10-year flood.

iii) There are several residences immediately riverside on the right bank, where residents have been adapting to the backwater effects due to floods from the Itajai mainstream and tidal level fluctuation by providing brick walls on the riverside and stilt residences with raised floor.

Although the backwater dyke was proposed to provide embankment on both of the banks in the master plan due to limited availability of river section data, concrete sheet pilie was conceived as the alternative to backwater dyke on the right bank (see Figures 4.3.5 and 4.3.6 below) with the main focus on minimizing social issues such as relocation of residences. With provision of backwater dyke by means of sheet pile, no relocation of residences is required.



Source: JICA Survey team

Figure 4.5.1 Objective Stretch of Backwater Dyke at Downstream Floodgate

(2) Geology condition

The geology condition of this area is considered as same condition as the geological survey at downstream gate. As it mentioned on Supporting Report C, the geology condition is shown below.

Table 4.5.1 Geology Condition

Depth (m)	Type	Symbol	N	c (kN/m^2)	φ (degree)	γ (kN/m^3)
1.5 ～ -0.8	Clay	Q2aj-are3	5.1	0	29	15
0.8 ～ -8.1	Clay	Q2aj-are2	1.7	11	0	17
-8.1 ～ -10.7	Clay	Q2aj-are1	6.2	0	29	15
-10.7 ～ -16.8	Clay	Q2am-are	2.7	17	0	18

(3) The water level at downstream

This area is the back water area. Thus the water level at the river confluence runs up to the design area.

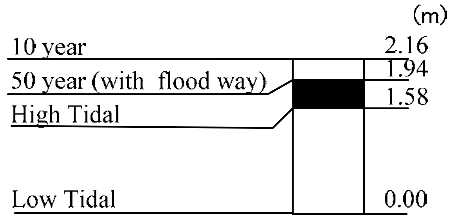


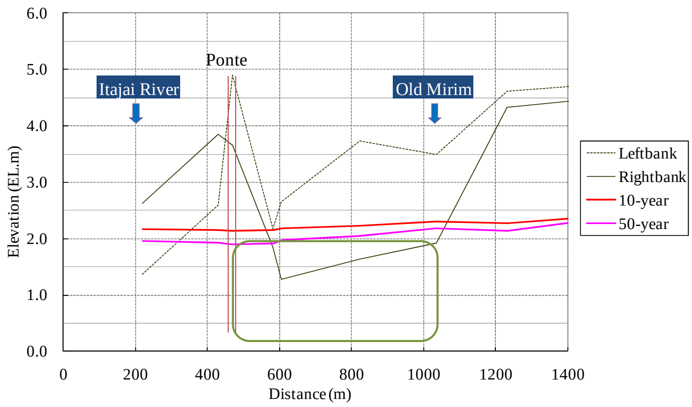
Figure 4.5.2 Water Lever Condition at Downstream

(4) Elevation of ground

At the right side, some part from the bridge over the Old Mirim River is lower and elevation of ground is EL. 1.5 m. The elevation of ground at the left side is high enough not to inundate as the road. Also this road is located along the river. Thus the elevation of road is considered as the elevation of ground.

(5) Flow capacity

As shown in the figure below, the water level of 10-year flood is higher than that of 50-year flood due to consideration of flood way with 50 - year calculation. And the part of sections is low flow capacity.



Source: JICA Survey Team

Figure 4.5.3 Water Level of longitudinal Profile

4.5.2 Layout plan

The area which is supposed to be inundated due to the low flow capacity is required the dyke.

(1) Alignment plan

At the right side, the low elevation area is set at the dyke. The interval from the bridge to the new floodgate is about 800 m.

At the left side, the downstream side from the bridge is being land formed and those areas are out of this project. The upstream side from the bridge has enough elevation, so it does not require the dyke.

(2) Vertical plan

The elevation of the dyke is set at the elevation of the river confluence of the Itajai River and the Mirim River as the standard elevation EL. 2.6 m by the cross section survey. Compared with the existing foundation level, the maximum difference is 1.3 m. The design elevation of ground

(EL. 2.6 m) is 20 cm higher than the calculated water level. Thus 20 cm is considered the freeboard.

At the left side, some parts of the left side are lower than EL. 2.6 m. Those parts required only 10 cm heightening and the freeboard is about 30 cm. Therefore, the left side is left existing.

4.5.3 Type of Structure

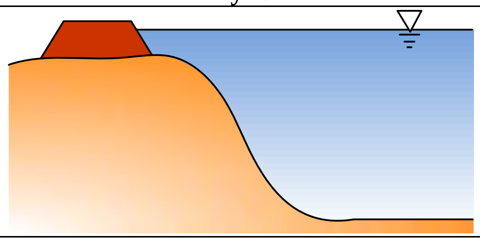
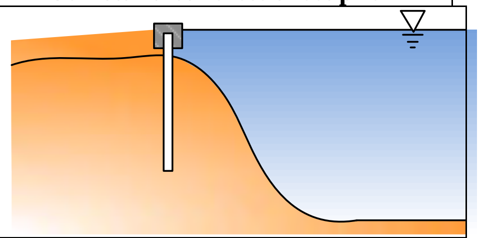
(1) Design condition

The Mirim River is a entrenched channel. Thus the overflow from the river is not a big issue. And the velocity is less than 2.0 m/s and the river flow is comparatively stable, so that only the elevation heightening is required without the bankprotection.

(2) Type of structure

Taking the river conditions into consideration, the two types of structure are adapted: dyke and self-stand sheet pile. Ccomparing of the two type with merit and demerit, the self-stand sheet pile is selected in terms of minimizing social issues.

Table 4.5.2 Comparing Type of Structure

	Dyke	**Self-stand concrete sheet pile**
Model		
Description	Banking the embankment in land area.	Put the sheet pile along the river in water area/land area. The opposite side of river is filled with the earthwork.
Advantage	･Workability is good. ･Maintenance/ re-habilitation is easy	･No necessary to move the houses. ･No necessary the temporary coffering
Dis-advantage	･Need the relocation. ･Need to compensate houses.	･Necessary to put countermeasure to stand pile. ･The maintenance/ re-habilitation needs cost to whole parts. ･The landscape is poor.
Assessment	Poor (impact is very high to residence)	Good

4.5.4 Desgin Strucutre

The length of sheet pile to stand by itself was calculated and detailed in Appendices 1. The length of that is 3.3 m because of poor geology condition. In those areas, the flow is not high to pur the material which is not to move. As showing in the figure below, the influence area of that .

The counterweight is designed to set up at the right figure.

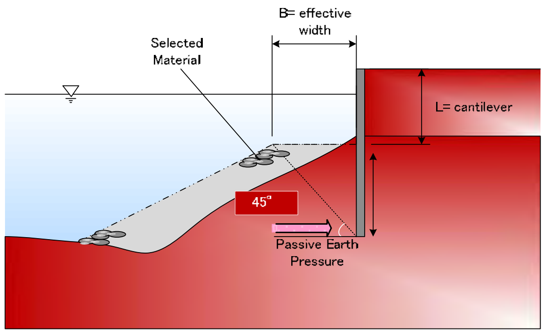
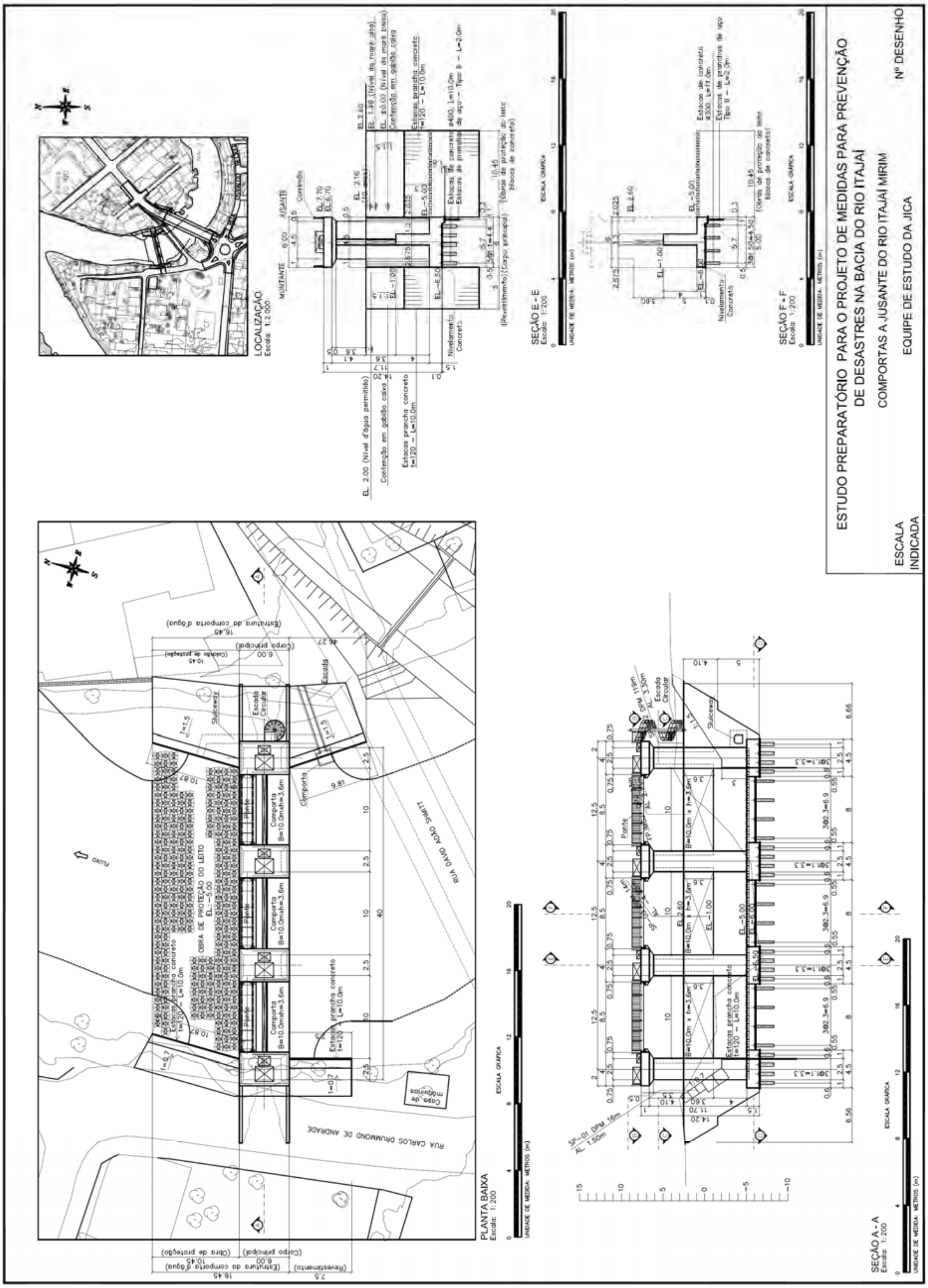


Figure 4.5.4 Water Level of longitudinal Profile



Figur 4.5.5 Downstream Floodgate in Itajai Mirim (1)

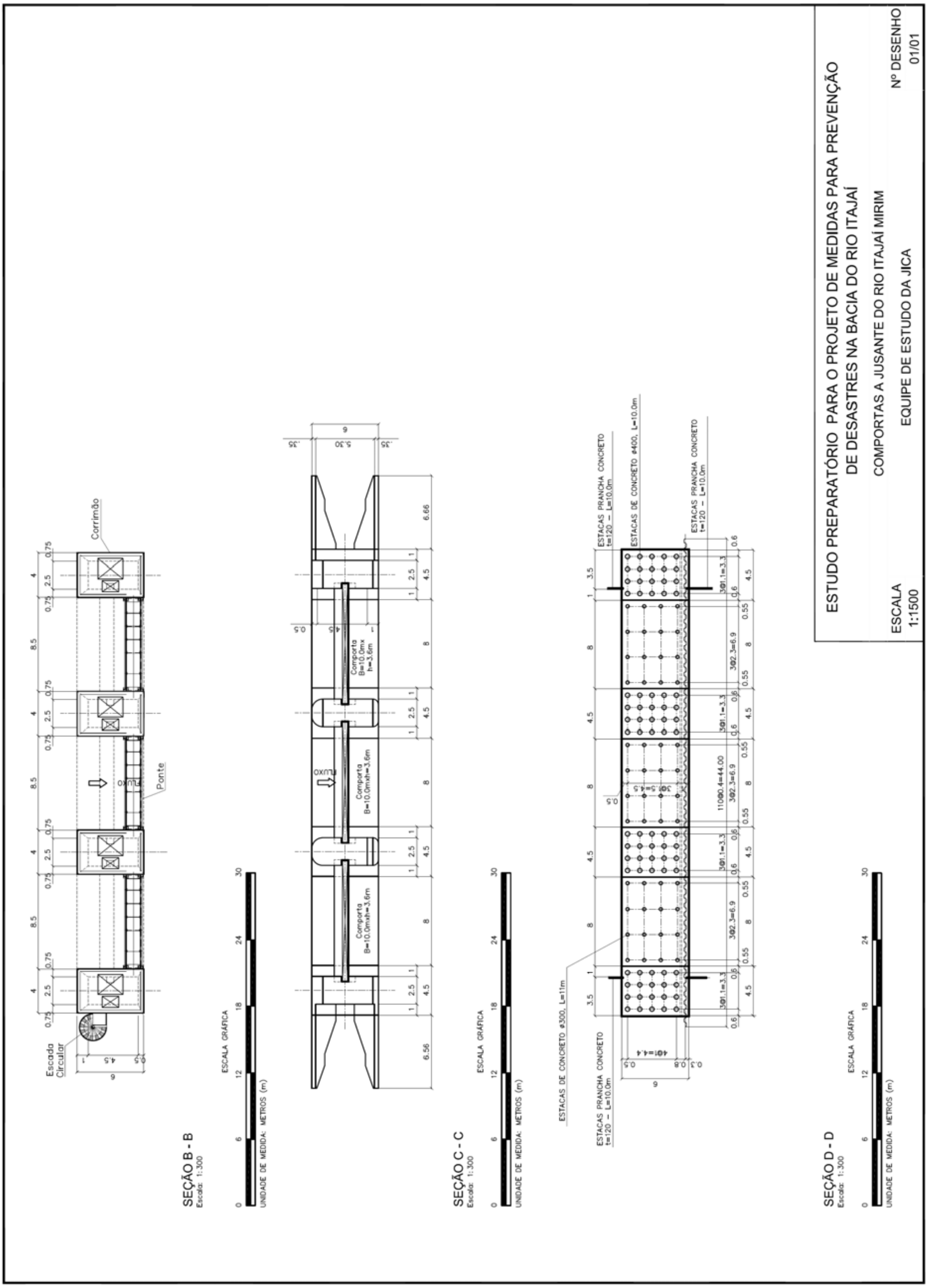


Figure 4.5.6 Downstream Floodgate in Itajai Mirim (2)

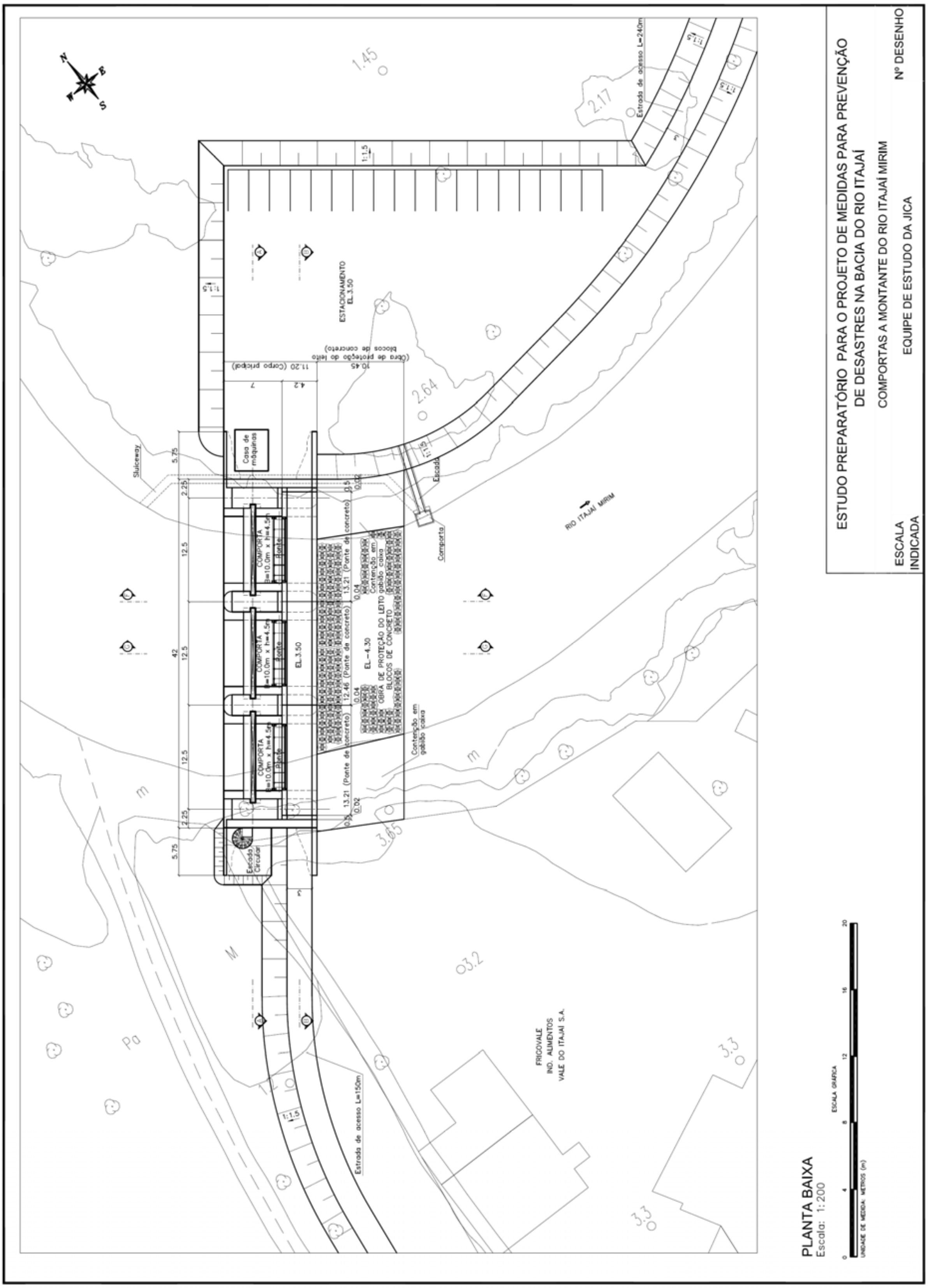


Figure 4.5.7 Upstream Floodgate in Itajai Mirim (1)

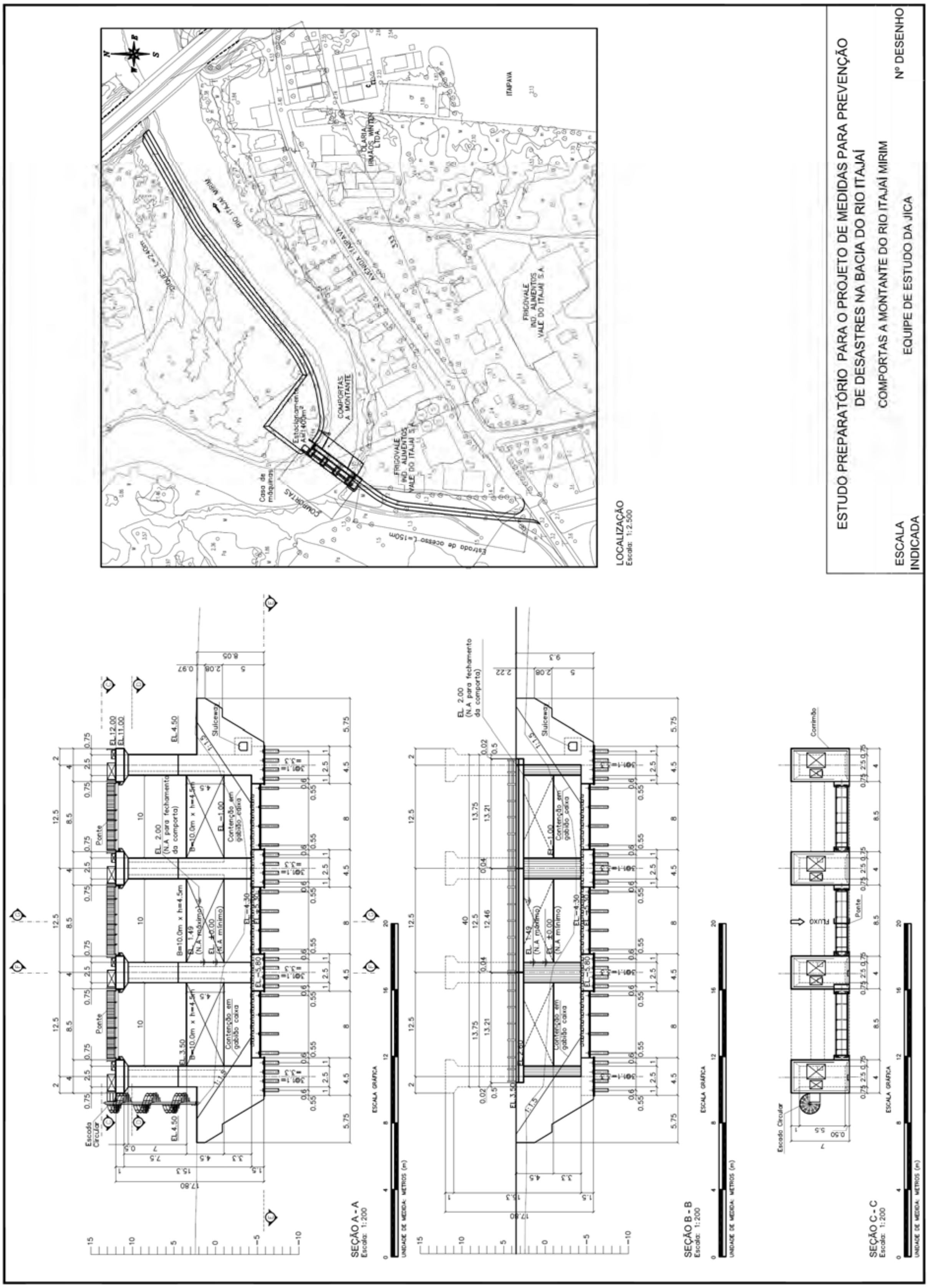


Figure 4.5.8 Upstream Floodgate in Itajai Mirim (2)

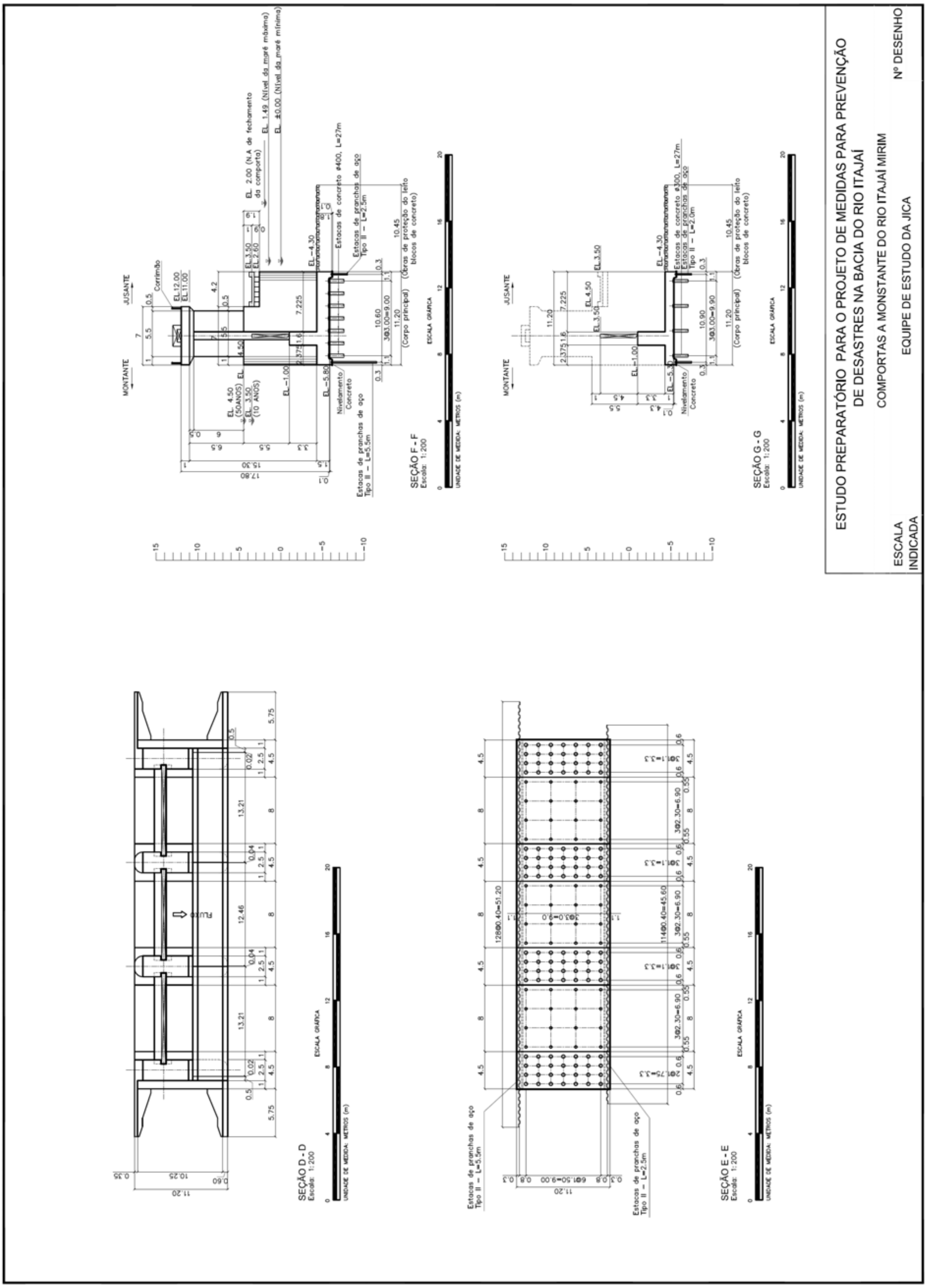


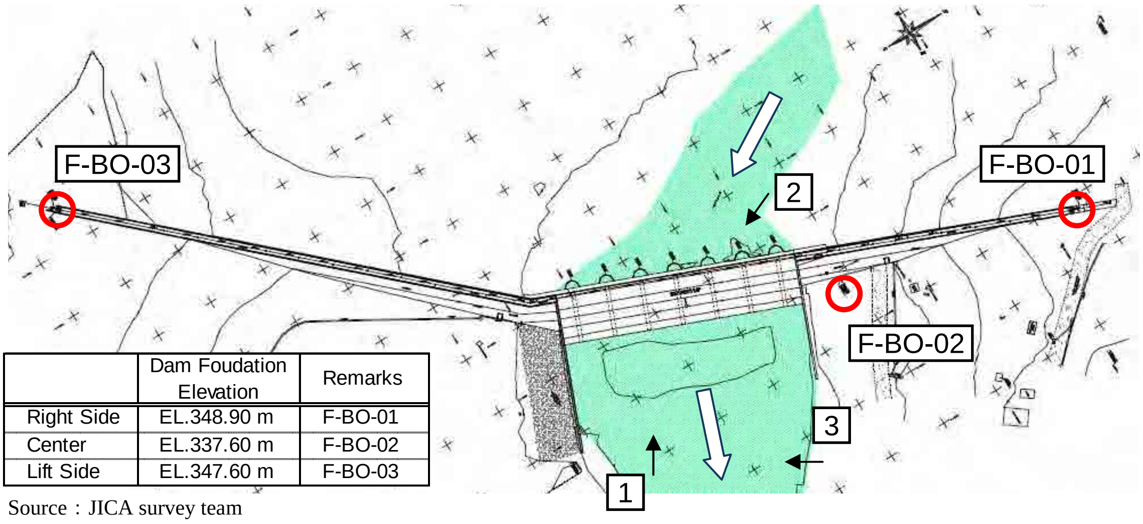
Figure 4.5.9 Upstream Floodgate in Itajai Mirim (3)

CHAPTER 5 HEIGHTENING OF DAMS

5.1 Feasibility study of Oeste dam

5.1.1 Field Investigation

The topographical survey was carried out to confirm major structural dimension of the dams, which was basically required for feasibility design for dam heightening. In addition, geological survey was carried out to estimate the foundation profile of the dams. Drillings were carried out at three (3) locations at the Oeste dam.



	Dam Foudation Elevation	Remarks
Right Side	EL.348.90 m	F-BO-01
Center	EL.337.60 m	F-BO-02
Lift Side	EL.347.60 m	F-BO-03

Source : JICA survey team

Figure 5.1.1 Location Map

Source : JICA survey team

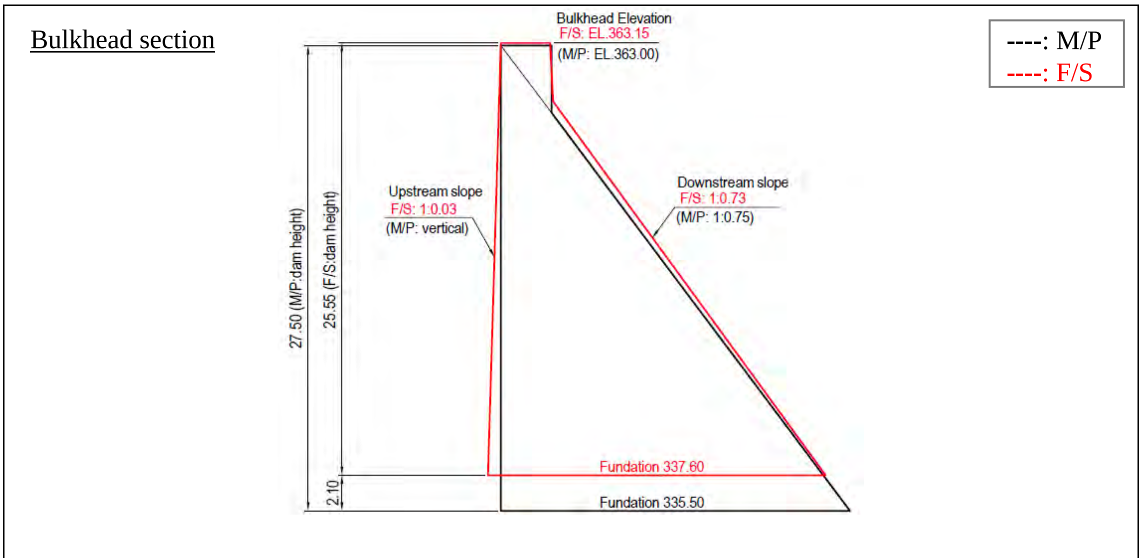
Photo Dam Site of Oeste Dam

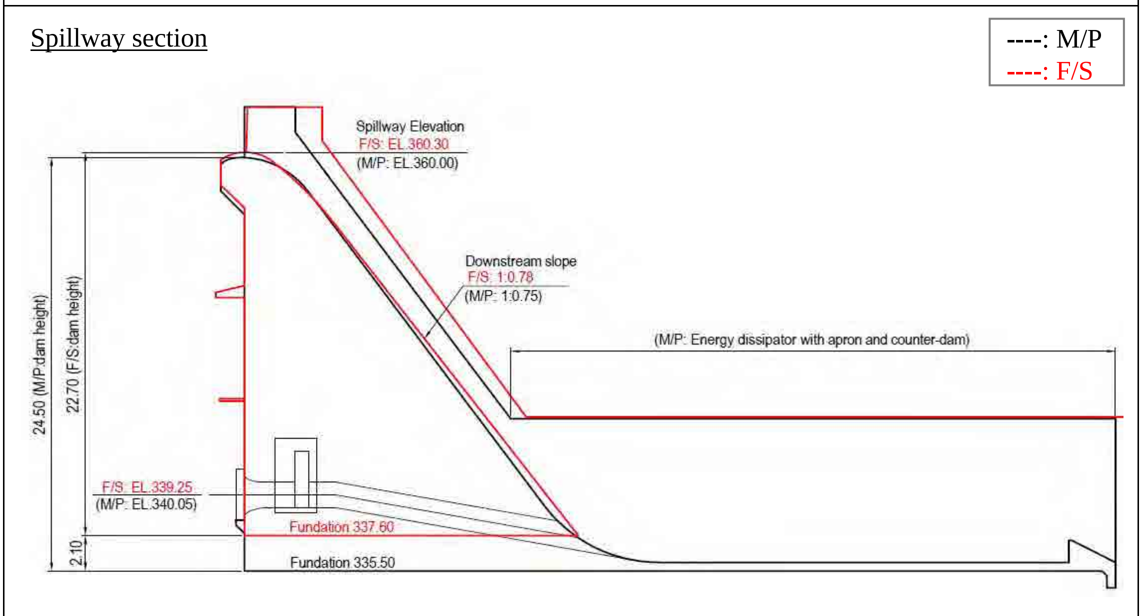
(1) Topology

The result of the survey, the shape of the dam is shown in Figure 5.1.2 below. The main difference between the survey result at feasibility study phase and the dimension at master plan phase is summarized below. In the master plan phase, those dimensions of structure were determined based on the assumption by the old drawing which was hardly to read and field observation.

Table 5.1.1 Outstanding Features

	The surveyed at Feasibility Study Phase	Referred at the Master Plan phase	Difference
Non-overflow Elevation (Spillway Elevation)	363.15 (360.30)	363.00 (360.00)	+0.15 (+0.30)
Foundation Elevation	337.60	335.50	+2.10
Upstream Slope	1:0.73 (1:0.78)	1:0.75 (1:0.75)	−0:0.02 (+0:0.03)
Downstream Slope (Spillway Section)	1:0.03 (---)	--- (---)	+1:0.03 (---)
Energy dissipator	---	Energy dissipator with apron and counter-dam	No Energy dissipator





Source: JICA Survey Team

Figure 5.1.2 Typical Section

(2) Geological condition

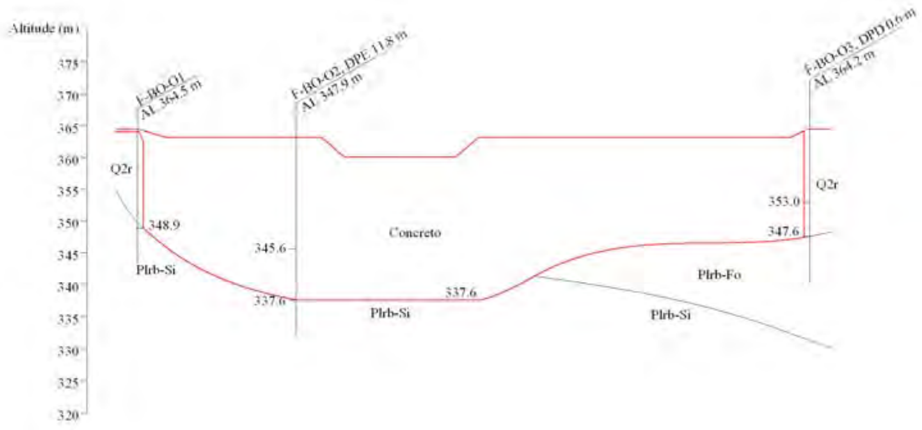
The geological property of the foundation of the Oeste dam is detailed at Annex C. The table on the right is about the geological condition.

Table 5.1.2 Geological Condition

Unconfined Compressive Strength (MN/m^2)	30
Internal Fiction Angle (deg)	38
Shear Strength (MN/m^2)	1

Source : JICA Survey team

The height of the foundation of the Oeste dam is estimated as follows though the foundation was considered to be flat: 335.50 m in the master plan phase.



Source : JICA Survey team

Figure 5.1.3 Foundation Level

5.1.2 Basin Design Concept

(1) Criteria

The following design criteria and standards were applied. The feasibility study design was carried out mainly based on the first Brazilian standard, supported by other standards.

i) CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

ii) River and Sabo Facilities prepared by Ministry of construction of Japan/1997.

iii) Design of Small Dams by A Water Resources Technical Publication, USA/1987

(2) Load Condition

According to the criteria, the stability of the dam is calculated by the following four(4) loading conditions:

Table 5.1.3 Load condition

Load condition	Remarks
CCN:Condicao de Carrengamento Normarl Normal	Normal water
CCE:Condicao de Carregamento Excepcional Excepcional	Maximum flood water
CCL:Condicao de Carregamento Limite Limite	Flood water + Seismic
CCC:Condicao de Carregamento de Construção Constracut	Construction (no-water)

Source:CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

(3) Stability condition

Safety factors for stability analysis vary according to the loading conditions as follows.

Table 5.1.4 Safety factor of load conditions

Load condition		CCN	CCE	CCL	CCC
FSF (Lift)		1.3	1.1	1.1	1.2
FST (Overturning)		3.0	2.0	1.5	1.3
FSD (Sliding)	c	3.0	1.5	1.3	2.0
	φ	1.5	1.1	1.1	1.3
σ_t (Bearing capacity)		3.0	2.0	1.5	1.3

FSF = Fator de seguranca a flutuacao, FSD = Fator de seguranca ao deslizamento
FST= Fator de seguranca ao tombamento

Source: CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

1) Stability calculation formula

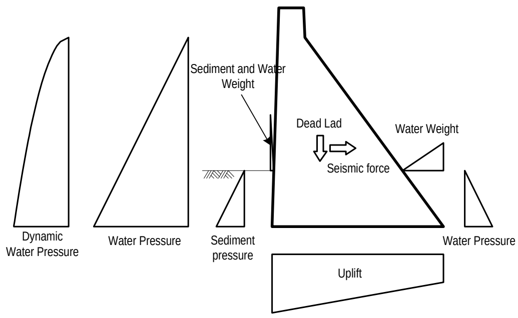
The four (4) safety calculations are these equations as the follows.

Lifting	$FSF = \frac{\Sigma V}{\Sigma U}$	Sliding	$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} \geq 1.0$
Overturning	$FST = \frac{\Sigma M_e}{\Sigma M_t}$	Bearing capacity	$e = \frac{L}{2} - \frac{M_e - M_t}{\Sigma V}$ $q_{(u,d)} = \frac{\Sigma V}{L} \cdot \left(1 \pm \frac{6 \cdot e}{L}\right)$

Source: CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

2) Combination of design load

For the stability calculation, each load is considered as the table below.



Source: JICA Survey Team

Figure 5.1.4 Load Diagram

Table 5.1.5 Combination of Loads for Stability Analysis

Load	**CCN**	**CCE**	**CCL**	**CCC**
1) Dead weight	Yes	Yes	Yes	Yes
2) Water weight	Yes	Yes	Yes	—
3) Dynamic pressure by earthquake	—	—	Yes	—
4) Seismic force	—	—	Yes	—
5) Water pressure	Yes	Yes	Yes	—
6) Uplift pressure	Yes	Yes	Yes	—
7) Sediment weight	Yes	Yes	Yes	—
8) Sediment pressure	Yes	Yes	Yes	—

Source：JICA survey team

5) Design Parameters

Dead Weight/Water Weight

Dead weight and water weight are estimated by unit weight. Generally the selected material is estimated, but because of the lack of the information about the material, the calculation of stability is made by using as the following general figure.

Table 5.1.6 Unit Weight

Item	Unit weight (kN/m^3)
Mass Concrete	23.5
Water	10.0
Soil (underwater weight)	8.5 (=17.5-9.0)

Source: CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

Dynamic Water Pressure

Dynamic water pressure acting on the structure is based on the formula in the below. Westergaard formula will be used.

$$p_d = \frac{7}{8} \cdot W_o \cdot K_d \cdot \sqrt{H \cdot h} (kN/m^2)$$

$$p_d = \int \frac{7}{8} \cdot W_o \cdot K_d \cdot \sqrt{H \cdot h} \cdot dh = \frac{7}{12} \cdot W_o \cdot K_d \cdot \sqrt{H} \cdot h^{\frac{3}{2}} (kN/m)$$ Notes:

$$y_d = 0.4 \cdot h(m)$$

P_d : Dynamicwater pressure (kN)
W_0 : unit water weight (kN/m^3)
K_h : Seismic factor
H : Depth of the water reservoir at base point (m)
h : Depth of the water reservoir at any point (m)
y_d : Working point height (m)

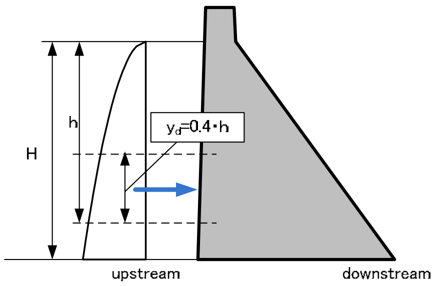


Figure 5.1.5 Diagram of Dynamic Water Pressure

Seismic factor

Seismic force is calculated based on the formula below.

$Fh = 0.05 \cdot P$ (Horizontal)

$Fv = 0.03 \cdot P$ (Vertical)

Inertial force acting on the structure is calculated based on the coefficient in the Table 5.1.7.

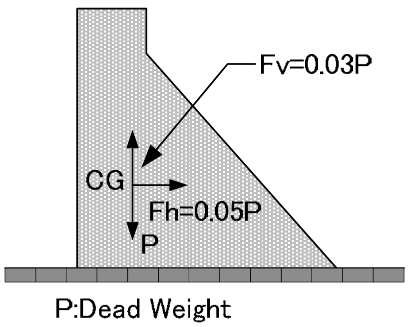


Figure 5.1.6 Diagram of Seismic Factor

Table 5.1.7 Seismic factor

	Modulus	Remarks
Horizontal direction	Fh = 0.05	
Vertical direction	Fv = - 0.03	Up

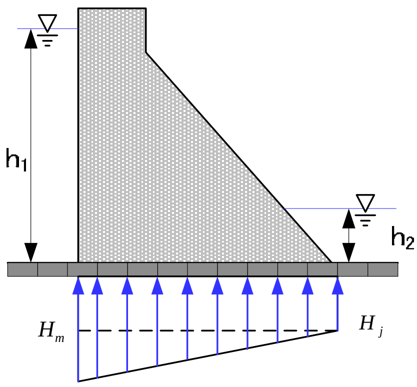


Figure 5.1.7 Diagram of Seismic Factor

Water Pressure

Water pressure is based on the formula below.

$$P = W_0 \cdot h \; Y_w = \frac{1}{3} \cdot h$$

Where
P:Waterpressure (kN/m^2)
W_0:water unit weight
h:water level
Y_w: point of application

Uplift

Uplift is based on the formula below.

$$H_m = h_2 + \frac{1}{3} \cdot (h_1 - h_2), \;\; H_j = h_2$$

Sediment pressure coefficient

Sediment pressure is determined by using the Rankine formula below.

$$Ka = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2\left(45 - \frac{\phi}{2}\right) = \tan^2\left(45 - \frac{25}{2}\right) \fallingdotseq 0.4$$

$$Pe = \frac{1}{2} \cdot Ka \cdot \gamma \cdot h^2 \; (kN/m), \; ye = \frac{h}{3} \; (m)$$

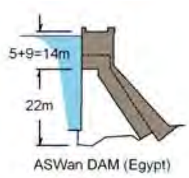
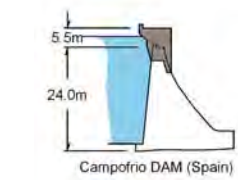
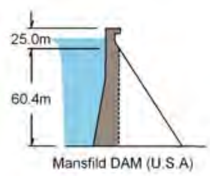
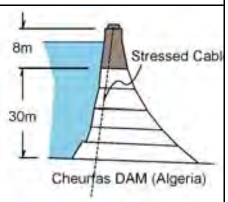
Notes
Internal friction angle: 25 deg (Soft clay)
Height of sediment deposited: EL.338.50 (Inlet of Conduit)

(2) Re consideration of dam heightening method

1) Type of Dam heightening

The Oeste dam is proposed to be the heightening by 2.0 m. The type of the Oeste dam is the concrete gravity which have more experience of heighten without difficulties.
The heightening method of concrete gravity dam is shown in the table below. The two typical methods are the covering with concrete and the attachment with the anchor cable.

Table 5.1.8 Heightening Method of Concrete Gravity Dam

	Covering Method			**Anchor - Method**
	Covering of New Dam	Raising of Dam Crest	Thickening of Upstream Dam Body	Anchoring
Schematic Profile				
Explanation	Placing new concrete on the downstream face of existing dam and forming unified dam body of the new and old concretes.	Placing new concrete on the dam crest and forming unified dam body of the new and old concrete.	Placing new concrete on the upstream face of the existing dam and forming unified body of the new and old concretes.	Placing new concrete on the dam crest and connecting to the upstream dam foundation by stress cable.
Assess	It is effective work to increase the dead	Without enlarging the dead weight itself, it is	Where the connection the new concrete and	The durability of the cable and workability is

	weight and become more stability. Even the height of heightening is applied to this method. This is standard work.	the effective method. It is not selected if the heightening part is very height.	existing concrete is under water level and become the weak point. The experience cases are not high.	complicated. The experience cases are not high.

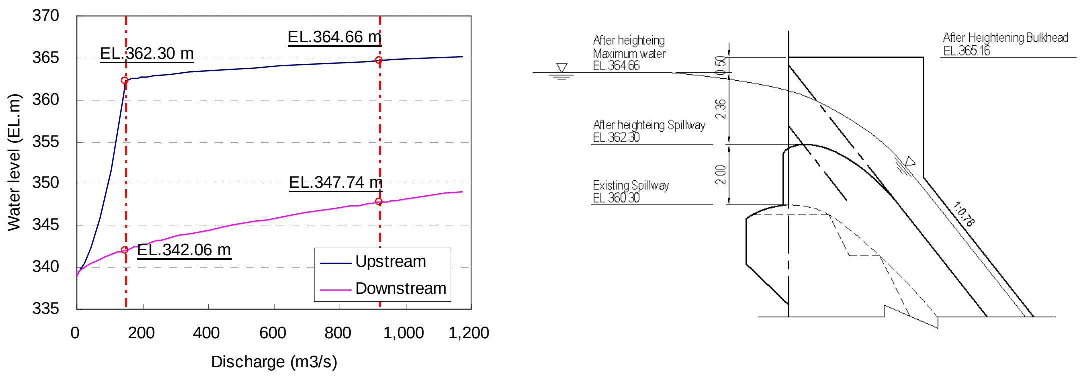
Source：JICA survey team

2) Select the method

The Oeste dam is just 2.0 m and it is relatively short. "Raising of Dam Crest" in table 5.1.9 is the selected method for the small quantity and simple work.
"Covering of New Dam" is the selected method for the spillway since the constant width is required. The slope of the new concrete at downstream side is more gradual than existing dam.

(3) Elevation of Non-overflow section

The height of Non-overflow section requires the height which is design water level and freeboard 0.5 m as criteria. As mentioned in Table 5.1.9, the design discharge of the Oeste dam is 920 m^3/s. As the calculation of hydraulic equation for the circular channel, the water level is EL. 347.16 m at the design discharge. Thus the height of Non-overflow is EL. 347.16 m (EL. 346.66 m + 0.50 m)



Source：JICA survey team

Figure 5.1.8 Water Level of Upstream and Downstream

Table 5.1.9 Water Level of Upstream and Downstream

Upstream water level	Downstream water level	Discharge Q (m3/s)	Δh (m)	Velocity V (m/s)	Conduit Q1 (m3/s)	Spillway Q2 (m3/s)	Total Σ Q (m3/s)
339.00	339.00	0.0	0.00	0.00	0.00	0.0	0.0
339.55	339.50	7.0	0.05	0.56	6.99	0.0	7.0
340.46	340.00	22.3	0.46	1.80	22.28	0.0	22.3
342.31	340.50	44.0	1.81	3.56	44.01	0.0	44.0
345.77	341.00	71.5	4.77	5.78	71.47	0.0	71.5
351.66	341.50	104.3	10.16	8.43	104.26	0.0	104.3
360.88	342.00	142.1	18.88	11.49	142.15	0.0	142.1
362.30	342.06	147.2	20.24	11.90	147.19	0.0	147.2
362.45	342.20	159.3	20.25	11.90	147.22	12.1	159.3
362.51	342.30	167.8	20.21	11.89	147.08	20.8	167.8
362.56	342.40	176.4	20.16	11.88	146.92	29.5	176.4
362.61	342.50	185.0	20.11	11.86	146.74	38.2	185.0
362.67	342.60	194.5	20.07	11.85	146.56	47.9	194.5
362.71	342.70	204.0	20.01	11.83	146.37	57.7	204.0
362.76	342.80	213.6	19.96	11.82	146.17	67.4	213.6
362.84	343.00	232.6	19.84	11.78	145.75	86.9	232.6
363.05	343.50	285.0	19.55	11.69	144.66	140.4	285.0
363.24	344.00	342.2	19.24	11.60	143.53	198.6	342.2
363.43	344.50	404.0	18.93	11.51	142.37	261.6	404.0
363.62	345.00	470.5	18.62	11.41	141.19	329.3	470.5
363.81	345.50	541.7	18.31	11.32	140.00	401.7	541.7
364.00	346.00	617.7	18.00	11.22	138.80	478.9	617.7
364.19	346.50	698.3	17.69	11.12	137.59	560.7	698.3
364.38	347.00	783.7	17.38	11.03	136.38	647.4	783.7
364.57	347.50	874.0	17.07	10.93	135.16	738.8	874.0
364.66	347.74	920.0	16.92	10.88	134.57	785.4	920.0
364.76	348.00	969.0	16.76	10.83	133.94	835.1	969.0
364.95	348.50	1068.9	16.45	10.73	132.71	936.2	1068.9
365.15	349.00	1173.7	16.15	10.63	131.49	1042.2	1173.7

Source: JICA survey team

Design water level was calculation by using the formula below.

Downstream water level is uniform flow calculation.

$$Q = A \cdot V$$

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

Notes:

$Q: Discharge\ (m^3/s)$

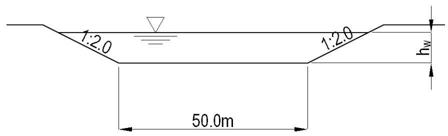
$V: Velocity\ (m/s)$

$A: Flow\ area\ (m^2)$

$n: Roughness\ modules\ (=0.032)$

$R: Hydraulic\ radius\ (m)$

$I: Riverbed\ slope\ (=1/5,000)$



Upstream water level is closed conduit flow

$$Q = A \cdot V, V = \frac{Q}{A}, A = \frac{D^2 \cdot \pi}{4}$$

$$h_{in,out} = (1+0.5) \cdot \frac{V^2}{2 \cdot g}$$

$$h_f = \frac{8 \cdot g \cdot n^2}{(D/4)^{1/3}} \cdot \frac{L}{D} \cdot \frac{V^2}{2 \cdot g}$$

$$H = h_{in,out} + h_f$$

Notes:

$Q: Discharge\ (m^3/s)$

$V: Velocity\ (m/s)$

$A: Flow\ area\ (m^2)$

$n: Roughness\ modules\ (=0.015)$

$H: All\ head\ loss\ (= h_{in,out} + h_f)$

$h_{in,out}:\ Inlet (=0.5), Outlet\ loss\ (=1.0)$

$h_f: Friction\ head\ (m)$

2) Hydraulic Design

The following table is shown as water condition for calculation of stability.

Table 5.1.10 Design Water Level

Load condition		Upstream water level	Downstream Water level	Remarks
CCN		340.79	340.09	Q=28 m^3/s (Normal water level)
CCE	Existing	362.65	347.74	Q=920 m^3/s (Maximum flood water level)
	After heightening	364.66		
CCL	Existing	360.30	341.95	Q=139 m^3/s (Flood water level (Spillway top))
	After heightening	362.30	342.06	Q=147 m^3/s (Flood water level (Spillway top))
CCC		---	---	

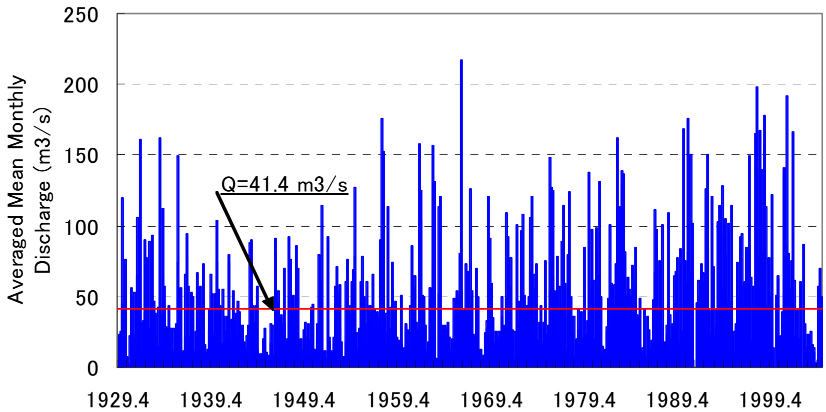
Source：JICA survey team

3) Normal water discharge

The normal discharge at the Oeste dam is calculated by the following steps; (1) The average monthly discharge at Taio city for 75 years, (2) The calculation of the discharge per unit of catchment area, and (3) Conversion to the proper catchment area. The normal discharge is 28.0 m^3/s at the Osete dam site.

$$Q_{normal} = \frac{C.A.(Oeste)}{C.A.(Taio)} \times Q_{A.M.at.Taio} = \frac{1042}{1575} \times 41.4 = 27.7 \approx 28.0(m^3/s)$$

C.A.(Oeste): Catchment Area at Oeste 1,042 km^2
C.A.(Taio): Catchment Area at Taio city 1,575 km^2
Average mean monthly discharge of Taio Q=41.4 m^3/s



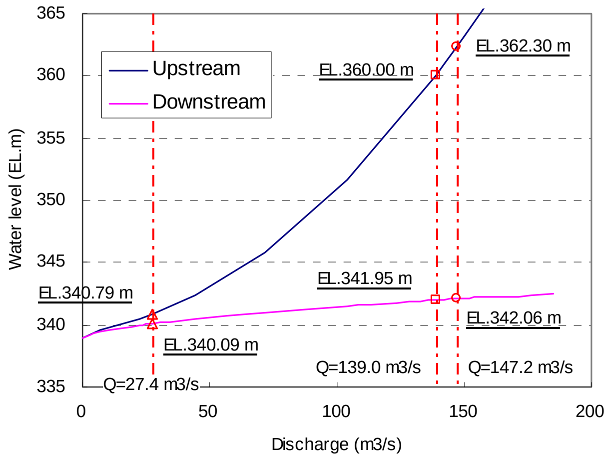
Source：JICA survey team

Figure 5.1.9 Averaged Monthly Discharge (for 75 years, at Taio City)

4) Design water level

The following values and table are shown as water design condition for calculation of stability.

Since the water level at downstream is high enough to influence the outlet discharge, the discharge flow is calculated as the closed conduit flow. The table below is summarized on each water conditions.



Upstream water level	Downstream water level	Δh (m)	Velocity V (m/s)	Conduit Q (m3/s)
339.00	339.00	0.00	0.00	0.00
339.55	339.50	0.05	0.56	6.99
340.46	340.00	0.46	1.80	22.28
340.79	340.09	0.70	2.21	27.40
342.31	340.50	1.81	3.56	44.01
345.77	341.00	4.77	5.78	71.47
351.66	341.50	10.16	8.43	104.26
360.00	341.95	18.05	11.24	138.99
360.88	342.00	18.88	11.49	142.15
362.30	342.06	20.24	11.90	147.19
363.21	342.10	21.11	12.15	150.32
365.73	342.20	23.53	12.83	158.69
368.43	342.30	26.13	13.52	167.26
371.34	342.40	28.94	14.23	176.02
374.46	342.50	31.96	14.95	184.97

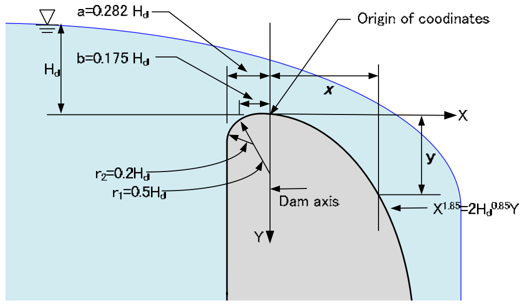
Source : JICA survey team

Figure 5.1.10 Water Level of Upstream and Downstream

5.1.3 Structure Design

(1) Overflow Section after Heightening

The shape of the crest spillway is basically a sharpness-crested due to the current sharp. The dimensions of each part are designed by the following figure with the parameter h_d: the head on the spillway.



Source : JICA survey team (Based on XXX)

Figure 5.1.11 Standard Dimensions and Flow Parameter

$$x^{1.85} = 2H_d^{\ 0.85} \cdot y \quad (y = \frac{x^{1.85}}{2H_d^{\ 0.85}})$$

$$x = 1.096 \cdot H_d \cdot y^{1.176} \quad \text{(End of curve)}$$

$$x = 1.096 \cdot H_d \cdot \left(\frac{1}{0.78}\right)^{1.176} = 3.464$$

determined 3.46 (m)

$$a = 0.282 \cdot H_d = 0.67$$

determined 0.70 (m)

$$b = 0.175 \cdot H_d = 0.41$$

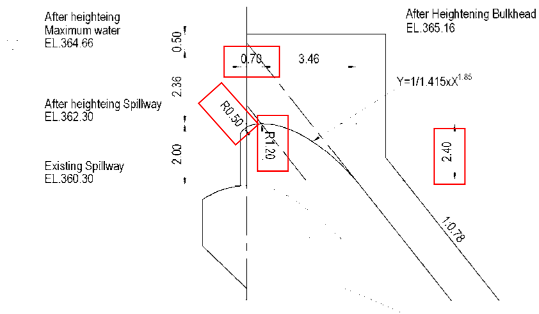
determined 0.45 (m)

$$r_1 = 0.5 \cdot H_d = 1.18$$

determined 1.20 (m)

$$r_2 = 0.2 \cdot H_d = 0.47$$

determined 0.50 (m)



Source : JICA survey team

Figure 5.1.12 Determinate Dimensions of Spillway Section

(2) Energy dissipater

No energy dissipater is provided at the Oeste dam. The energy dissipater is generally installed at the outlet of spillway to dissipate large energy of the overflowed water. Heightening of the spillway might cause larger energy since the overflow head becomes higher. From the hydraulic viewpoint, it was proposed to install the dissipater. The proposed dissipater is the submerged bucket type.

<u>Design Discharge</u>

The design scale of the dissipater is 100-year return period. The discharge of the return period at this site is 690 m^3/s as shown in the table below.

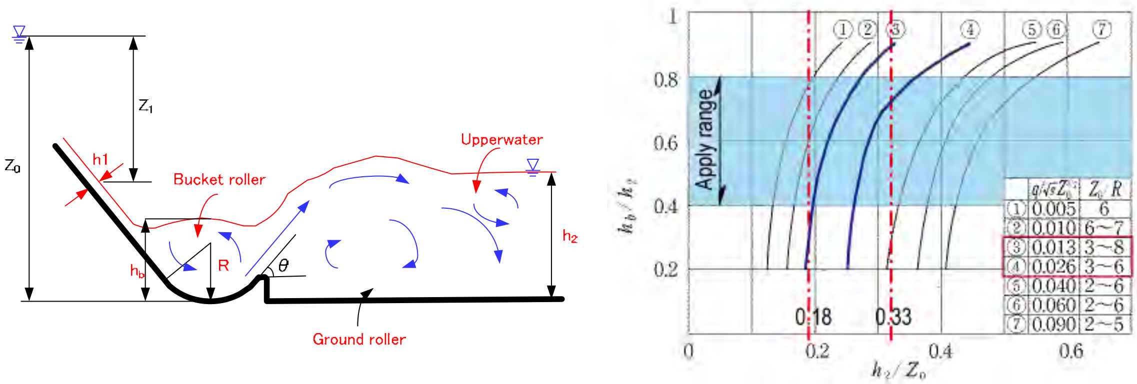
Table 5.1.11 Discharge of 100-year Oeste dam

	Taió catchment area =	1570.13 km2	
Barragem Oeste catchment area =		1042.00 km2	
Füller equation :	Qti=Qt(1+2,66/(A**0,3))		
T(years)	Vazões Máximas (m³/s)		Exponencial 2 Parâmetros
	Taió	Barragem Oeste	
	Daily Mean	Daily mean (Qt)	Instantaneous peak (Füller) Qti
5	436	289	385
10	504	334	445
25	590	392	521
50	654	434	577
100	717	476	633
500	864	573	763
1,000	927	615	818

Source : JICA survey team

<u>Analysis Result of Bucket Type Energy Dissipater</u>

The radius of the bucket carve is designed by the following the parameter and graph. The value of h_2/z_0 is between 0.18 and 0.33, so the radius of the bucket is 7 m for the coverage.



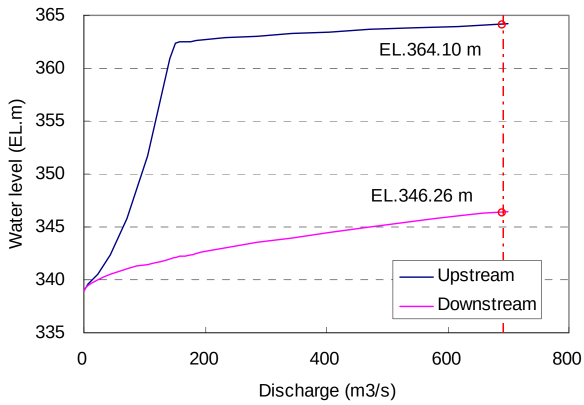
Source : JICA survey team

Figure 5.1.13 Design Chart and Bucket Type Energy Dissipator

Table 5.1.12 Analysis Result of Bucket type energy dissipater

h_2/z_0	z_0/R	z_0	h_2	V (m/s)	Q_1 (m^3/s) Conduit	Q_2 (m^3/s) Spillway	ΣQ (m^3/s) Total	q ($m^3/s/m$)
0.18	3.53	24.70	4.46	11.90	147	0	147	1.5
0.19	3.55	24.70	4.57	11.90	147	11	159	1.6
0.19	3.56	24.70	4.66	11.89	147	20	167	1.7
0.19	3.57	24.70	4.75	11.88	147	29	176	1.8
0.20	3.57	24.70	4.84	11.86	147	38	185	1.8
0.21	3.61	24.70	5.28	11.78	146	87	233	2.3
0.23	3.64	24.70	5.73	11.69	145	140	285	2.9
0.25	3.66	24.70	6.16	11.60	144	199	342	3.4
0.27	3.69	24.70	6.60	11.51	142	262	404	4.0
0.28	3.72	24.70	7.02	11.41	141	329	471	4.7
0.30	3.74	24.70	7.44	11.32	140	402	542	5.4
0.32	3.77	24.70	7.86	11.22	139	479	618	6.2
0.33	3.79	24.70	8.08	13.61	168	522	690	6.9

Source：JICA survey team



Source：JICA survey team

Figure 5.1.14 Upstream and Downstream of Water Level

<u>The Height of Division wall</u>

$$h_v = 0.82 \cdot H_{\max} = 0.82 \times 2.36 = 1.935$$

$$h_w w = \frac{n \cdot h_v}{\sqrt{1+n^2}} = \frac{0.78 \cdot 1.935}{\sqrt{1+0.78^2}} = 1.190$$

determinate 1.20 m

$$H_{\max} = 364.66 - 362.30 = 2.36$$

n:Downstream slop (=0.78)

h_v:Vertical height

h_w:Division wall height

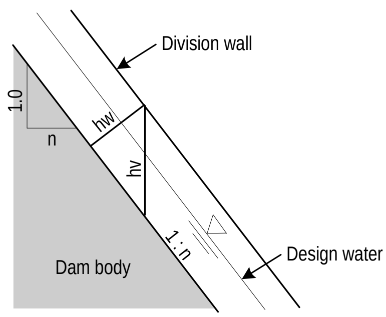


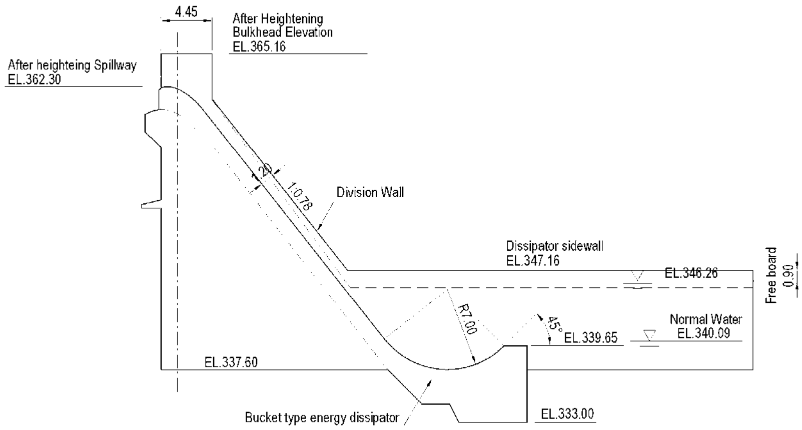
Figure 5.1.15 Diagram of Division Wall

The Height of Dissipator sidewall

The following empirical expression provides values that have proved satisfactory for most basins:

$$Freeboard = 0.1 \cdot (\frac{v^2}{2 \cdot g} + d) = 0.1 \cdot [0.10 + (346.26 - 337.60)] = 0.876, \quad \text{determinate 0.90 m}$$

V=1.43 m/s (Q=690 m^3/s), d:water depth



Side wall elevation EL.347.16 m = 346.26 + 0.90

Source：JICA survey team

Figure 5.1.16 Determinating Height of Bucket Type Energy Dissipater

5.1.4 Stability analysis

(1) Summary

At the current status, the safety against sliding/overturning is satisfied. However, in the case of heightening by 2.0 m, the spillway sections is required for the countermeasure.

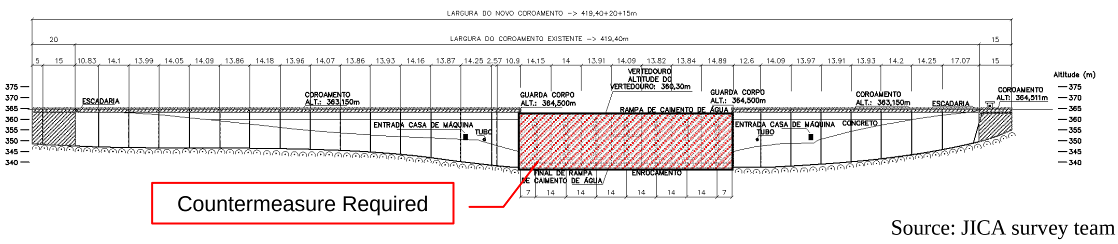
Table 5.1.13 Stability analysis results

	Non-overflow section	**Spillway section**
Existing	Satisfy	Satisfy
Heightening case	Satisfy	Countermeasure required

Source: Jica survey team

Countermeasure Spillway Section

The facing concrete at the downstream slope as a countermeasure was proposed in the section of whole spillway. The downstream slope is set at 1:0.78.



Source: JICA survey team

Figure 5.1.17 Countermeasure Required in Spillway Section

(2) Existing

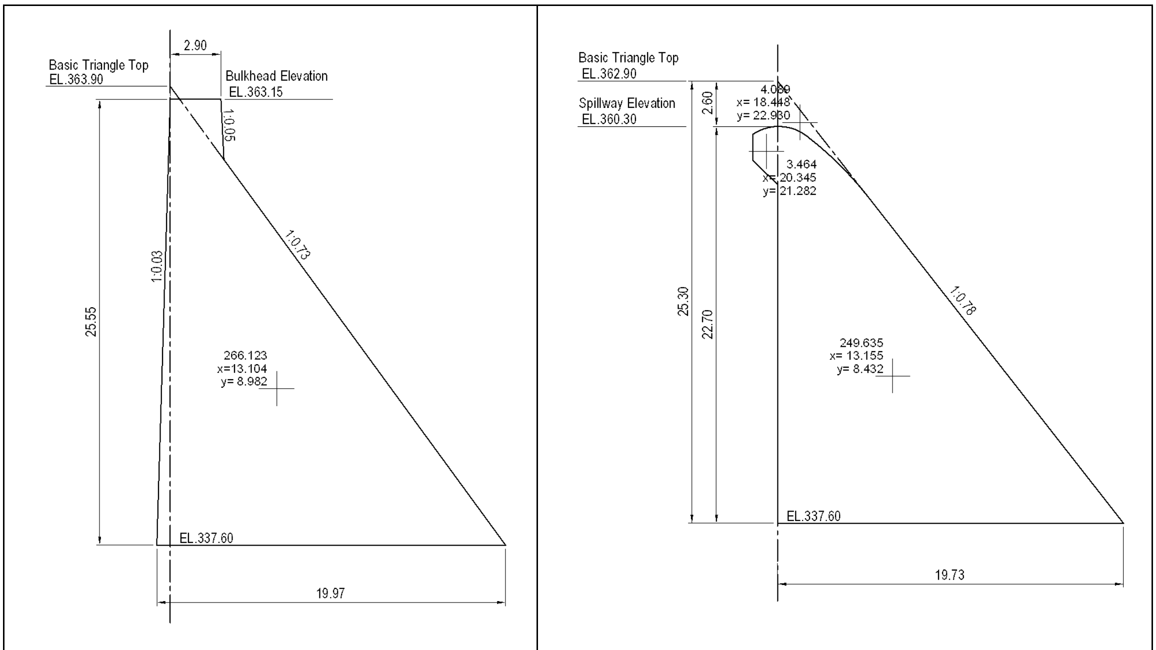
1) Design condition

Design condition of the Oeste dam stability analysis is considered as shown in the table below.

Table 5.1.14 Design condition of Existing

		Bulkhead section	Spillway section
Elevation of Top of Dam	EL.m	363.150	-----
Basic triangle Top Elevation	EL.m	363.900	362.900
Upstream Slope	1:n	0.030	-----
Downstream Slope	1:n	0.730	0.780
Upper surface of the downstream slope	1:n	0.030	-----
Dam base elevation	EL.m	337.600	337.600
Crest width of non-overflow section	m	2.900	-----
Reservoir sediment level	EL.m	338.500	←
Reservoir water level [CCN]	EL.m	340.790	←
[CCE]	EL.m	362.650	←
[CCL]	EL.m	360.300	←
Downstream water level [CCN]	EL.m	340.090	←
[CCE]	EL.m	347.740	←
[CCL]	EL.m	341.950	←
Unit weight of concrete dams	kN/m^3	23.5	←
Weight of sediment in the water	kN/m^3	8.5	←
Unit weight of water	kN/m^3	10.0	←
Seismic Coefficient: Horizontal (kh)	---	0.050	←
Seismic Coefficient: Vertical (kv)	---	0.030	←
Coefficient of earth pressure (Rankine coefficient of earth pressure)	---	0.40	←
Uplift pressure coefficient	---	1/3	←
Shear strength of foundation	kN/m^2	1,000.0	←
Friction angle of foundation	deg	38.00	←
Internal friction coefficient	---	0.78	←

Source：JICA survey team



Non-overflow section | Spillway section

Source：JICA survey team

Figure 5.1.18 Typical Section of Existing

2) Results

Both Non-overflow and Spillway are satisfied in terms of Stability.

The bearing capacity is satisfied since the allowable compressive stress intensity of foundation rock, σa=10M/m^2(=30M/3.0) is more than 0.58 M/m^2.

Non-overflow Section

Table 5.1.15 Analysis Result of Non-overflow Section

	FSF	FST	FSD ≧ 1.0
[CCN]	12.06 > 1.30	2665.24 > 1.50	453.81 ≧ 1.0
[CCE]	2.67 > 1.10	2.37 > 1.20	6.21 ≧ 1.0
[CCL]	4.20 > 1.10	2.73 > 1.10	6.35 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	577.22 ≤ 30M/3.0=10M	-0.73 ≥ -200
[CCE]	82.22 ≤ 30M/2.0=15M	338.51 ≥ -200
[CCL]	139.04 ≤ 30M/1.5=20M	334.87 ≥ -200
[CCC]	606.60 ≤ 30M/1.3=23M	19.63 ≥ -200

Source：JICA survey team

Spillway Section

Table 5.1.16 Analysis Result of Spillway Section

	FSF	FST	FSD ≧ 1.0
[CCN]	11.42 > 1.30	2500.05 > 1.50	440.55 ≧ 1.0
[CCE]	2.51 > 1.10	2.12 > 1.20	6.03 ≧ 1.0
[CCL]	3.93 > 1.10	2.52 > 1.10	6.23 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	564.36 ≤ 30M/3.0=10M	-20.98 ≥ -200
[CCE]	41.96 ≤ 30M/2.0=15M	339.25 ≥ -200
[CCL]	108.48 ≤ 30M/1.5=20M	326.10 ≥ -200
[CCC]	594.23 ≤ 30M/1.3=23M	-1.17 ≥ -200

Source：JICA survey team

Note:
Allowable compressive stress intensity of rock

$$\sigma_{\max} = \frac{\sigma_k}{\sigma_t} = \frac{30MN/m^2}{3.0 \sim 1.3}$$

Allowable tensile stress intensity of concrete

$$\sigma_{\min} = -\frac{\sigma_{ck}}{80} = -\frac{16}{80} = -0.2N/mm^2 = -200N/m^2$$

(3) Heightening Case

1) Design Condition

The condition of heightening is shown in the table below.

Table 5.1.17 Design Condition of Heightening Oeste Dam Case

		Bulkhead section	Spillway section
Elevation of Top of Dam	EL.m	365.160	-----
Basic triangle Top Elevation	EL.m	363.900	364.900
Upstream Slope	1:n	0.030	-----
Downstream Slope	1:n	0.730	0.780
Upper surface of the downstream slope	1:n	-----	-----
Dam base elevation	EL.m	337.600	337.600
Crest width of non-overflow section	m	2.900	-----
Reservoir sediment level	EL.m	338.500	←
Reservoir water level [CCN]	EL.m	340.790	←
[CCE]	EL.m	364.660	←
[CCL]	EL.m	362.300	←
Downstream water level [CCN]	EL.m	340.090	←
[CCE]	EL.m	347.740	←
[CCL]	EL.m	342.060	←
Unit weight of concrete dams	kN/m^3	23.5	←
Weight of sediment in the water	kN/m^3	8.5	←
Unit weight of water	kN/m^3	10.0	←
Seismic Coefficient: Horizontal (kh)	---	0.050	←
Seismic Coefficient: Vertical (kv)	---	0.030	←
Coefficient of earth pressure (Rankine coefficient of earth pressure)	---	0.40	←
Uplift pressure coefficient	---	1/3	←
Downstream cover thickness	m	-----	1.83
Concrete mat elevation (Top point)	EL.m	342.500	-----
Concrete mat length (Base point)	m	1.000	-----
Shear strength of foundation	kN/m^2	1,000.0	←
Friction angle of foundation	deg	38.00	←
Internal friction coefficient	---	0.78	←

Source：JICA survey team

2) Results

<u>Stability Analysis of Non-overflow section</u>

All conditions of all stability is satisfied. The bearing capacity requirement is satisfied sufficiently (σ_a=10MN/m^2 σmax=0.61M N/m^2, 1 M=10^6).

Table 5.1.18 Analysis Result of Heightening (Oeste Dam)

	FSF	FST	FSD ≧ 1.0
[CCN]	12.31 > 1.30	2762.40 > 1.50	457.06 ≧ 1.0
[CCE]	2.66 > 1.10	2.09 > 1.20	5.19 ≧ 1.0
[CCL]	4.06 > 1.10	2.26 > 1.10	5.44 ≧ 1.0
[CCC]	∞ > 1.20	∞ > 1.30	∞ ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	605.91 ≤ 30M/3.0=10M	-16.08 ≥ -200
[CCE]	83.05 ≤ 30M/2.0=15M	345.86 ≥ -200
[CCL]	82.22 ≤ 30M/1.5=20M	397.92 ≥ -200
[CCC]	628.91 ≤ 30M/1.3=23M	10.63 ≥ -200

Source：JICA survey team

Stability Analysis of Spillway section

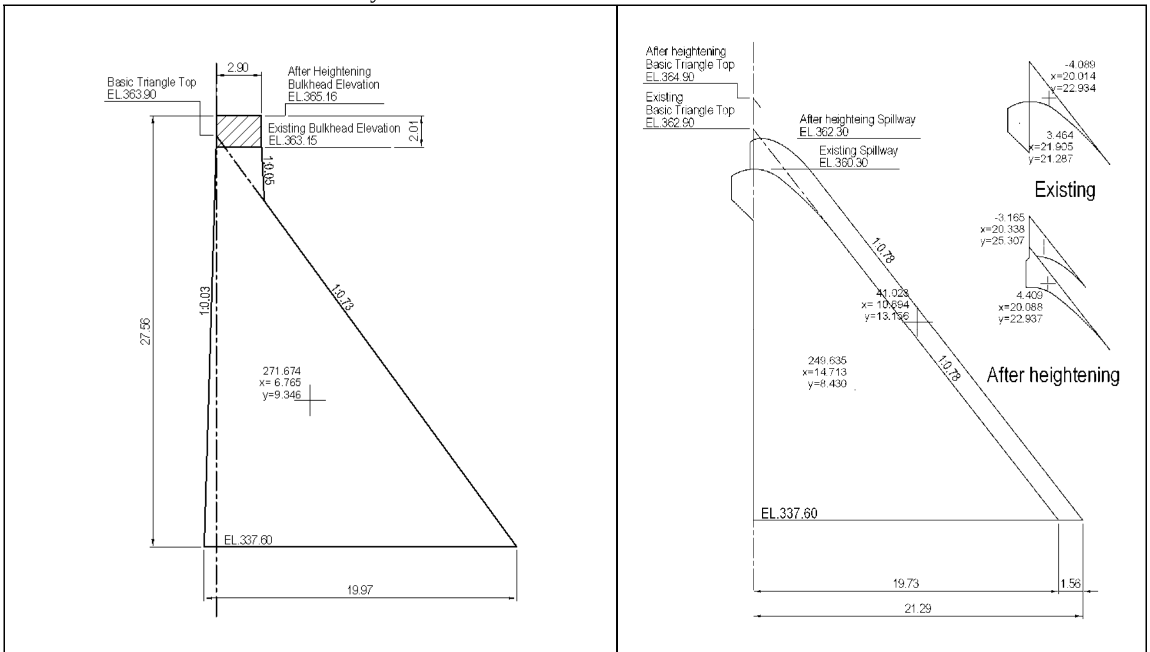
All conditions of stability are satisfied under the condition that the downstream sloop is 1:0.78.The bearing capacity requirement is satisfied sufficiently (σ_a=10 M N/m^2 σmax=0.62 M N/m^2, 1 M=10^6).

Table 5.1.19 Analysis Result of With Countermeasure

	FSF	FST	FSD ≧ 1.0
[CCN]	11.70 > 1.30	650.32 > 1.50	94.71 ≧ 1.0
[CCE]	2.63 > 1.10	1.67 > 1.20	4.16 ≧ 1.0
[CCL]	3.95 > 1.10	2.16 > 1.10	5.42 ≧ 1.0
[CCC]	59.31 > 1.20	1668.43 > 1.30	287.17 ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	615.46 ≤ 30M/3.0=10M	17.79 ≥ -200
[CCE]	7.63 ≤ 30M/2.0=15M	456.91 ≥ -200
[CCL]	111.00 ≤ 30M/1.5=20M	403.88 ≥ -200
[CCC]	583.51 ≤ 30M/1.3=23M	-0.44 ≥ -200

Source：JICA survey team



Determinatimg Non-overflow Section	Determinatimg Spillway Section

Source：JICA survey team

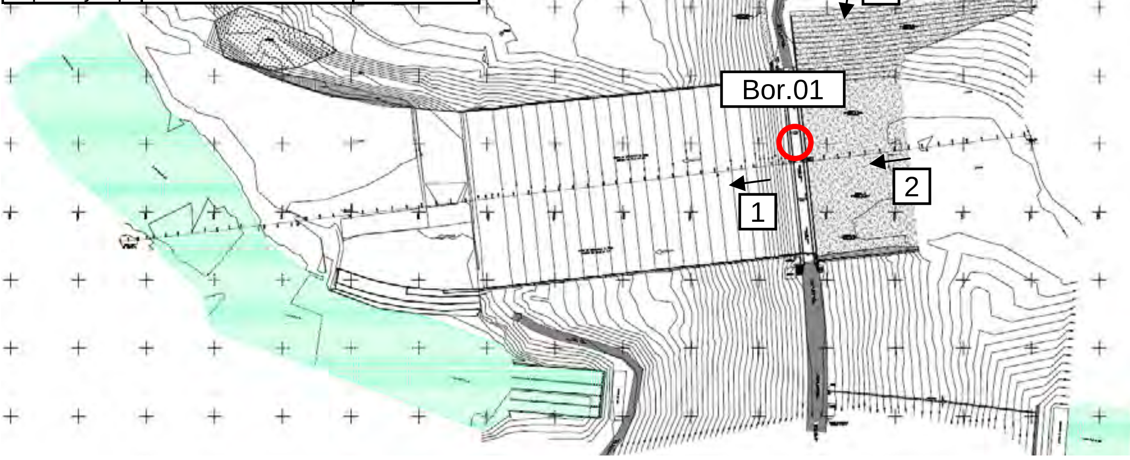
Figure 5.1.19 Determinating Heighten Spillway Section

5.2 Feasibility Study of Sul Dam

5.2.1 Field Investigation

The topographical survey was carried out to confirm the major structural dimension of the dams, which was basically required for feasibility design for dam heightening. In addition, geological survey was carried out to estimate the foundation profile of the dams. Drillings were carried out at one (1) location at Sul dam.

	Spillway foudation elevation	Remarks
Spillway top	EL.383.80 m	



Source：JICA survey team

Figure 5.2.1 Location Map

Source：JICA survey team

Photo Dam Site of Sul Dam

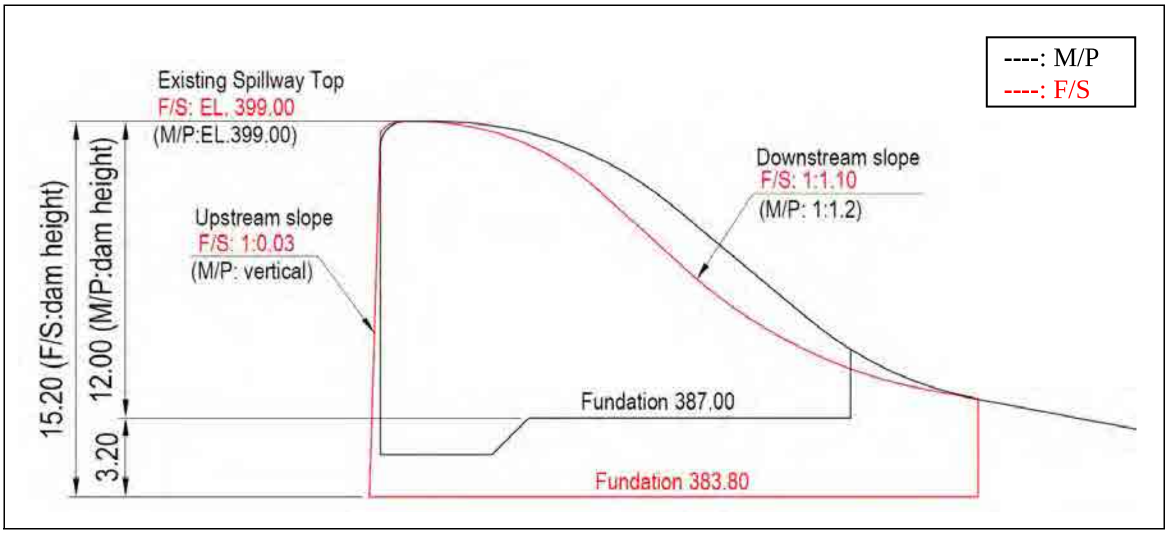
(1) Topography

The result of the survey, the shape of the dam is shown in Figure 5.2.2. The main difference between the survey result at Feasibility study phase and the figure at Master plan phase is summarized below. At the master plan phase, those dimensions of structure were determined based on the assumption by the old figure which was hardly to read.

Table 5.2.1 Outstanding Features

Item	The survey result at Feasibility Study	Used at Master plan phase	Difference
Spillway Elevation	399.00	399.00	±0.00
Bridge Top	410.15	410.00	+ 0.15
Foundation Elevation	383.80	387.00	-3.20
Upstream Slope	1:0.03	---	---
Downstream Slope	1:1.10	1:1.2	−0:0.2

Source：JICA survey team



Source：JICA survey team

Figure 5.2.2 Comparison to Figures at Each Phase

(2) Geological Condition

The geological property of the foundation of the Sul dam is detailed in Annex C. The table on the right is the geological condition.

Table 5.2.2 Geological Condition

Unconfined Compressive Strength (MN/m^2)	30
Internal Fiction Angle (deg)	38
Shear Strength (MN/m^2)	1

Source : JICA Survey team

5.2.2 Basic Condition

(1) Standards

As well as the Oeste dam, the same criteria manual is applied to the Sul dam.

(2) Hydraulic design

1) Spillway overflows capacity

The discharge of overflow is estimated by using the formula below.

$$Q_{overflow} = C \times B \times (H_{overflow})^{1.5}$$

where

C: a coefficient of discharge (=2.07), B:width of the spillway, $H_{overflow}$:the head on the spillway

The discharge of conduit is estimated as the below formula.

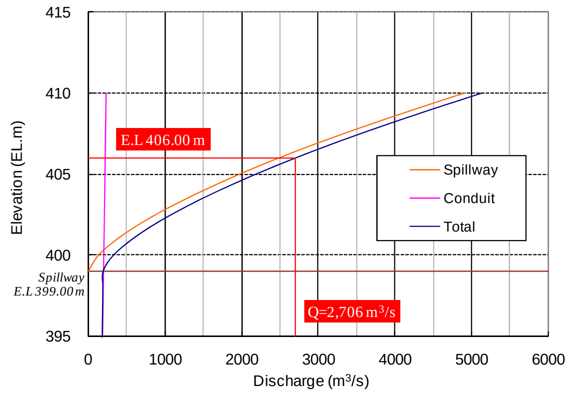
$$Q_{conduit} = C_1 \times N \times C_2 \times (2 \times g \times H_{conduit})^{0.5}$$

where

C_1,C_2: a coefficient of discharge (C1=0.89, C2=1.7663), N:Number of gates

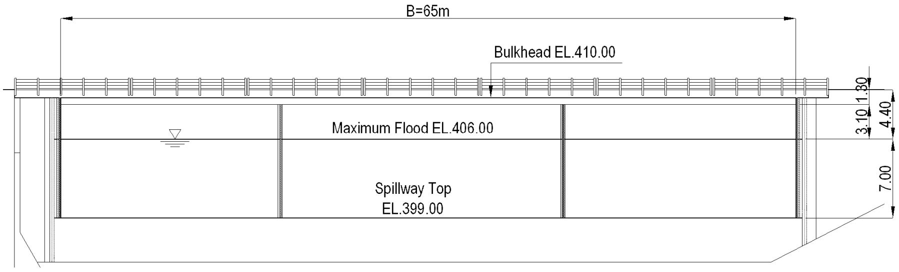
$H_{conduit}$:the head on the conduit

As showing in the graph on the right, the discharge from condit at 1000-year return period is 2,706 m^3/s and the head of overflow is 7.0 m.



Source：JICA survey team

Figure 5.2.3 H-Q Curve



Source：JICA survey team

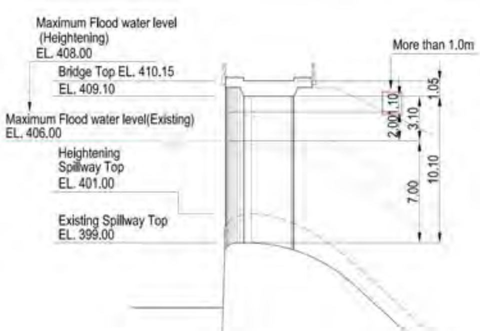
Figure 5.2.4 Front View of Sul Dam Spillway

2) Water level Relationship

The relationship between bridge beam and maximum flood water level is described as below. After the spillway is heightened by 2.0 m, there would be more than 1.0 m space (see the red square in the Figure 5.2.5 below).

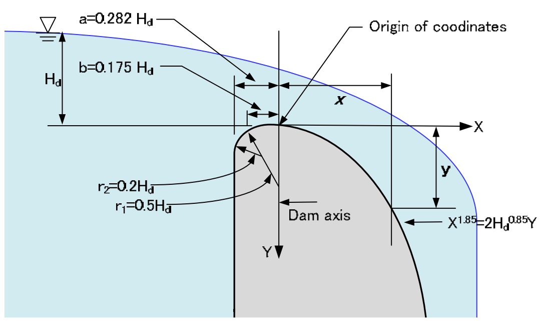
(3) Structure design of Heightening Overflow Section

The shape of the crest spillway is basically a sharp-crested as it is the present sharp. The dimensions of each part are designed as the following figure with the parameter h_d: the head on the spillway.



Source：JICA survey team

Figure 5.2.5 Water Level Relationship



Source：JICA survey team

Figure 5.2.6 Standard Dimensions and Flow Parameter

$$x^{1.85} = 2H_d^{\ 0.85} \cdot y \quad \left(y = \frac{x^{1.85}}{2H_d^{\ 0.85}} \right)$$

$$x = 1.096 \cdot H_d \cdot y^{1.176} \quad \text{(End of curve)}$$

$$x = 1.096 \cdot H_d \cdot \left(\frac{1}{1.10} \right)^{1.176} = 6.859$$

determined 6.86 (m)

$$a = 0.282 \cdot H_d = 1.97$$

determined 2.00 (m)

$$b = 0.175 \cdot H_d = 1.23$$

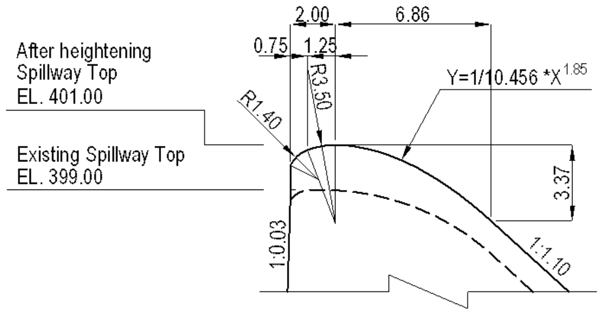
determined 1.25 (m)

$$r_1 = 0.5 \cdot H_d = 3.50$$

determined 3.50 (m)

$$r_2 = 0.2 \cdot H_d = 1.40$$

determined 1.40 (m)



Source：JICA survey team

Figure 5.2.7 Determinating dimensions of overflow spillway

5.2.3 Stability analysis of dam spillway

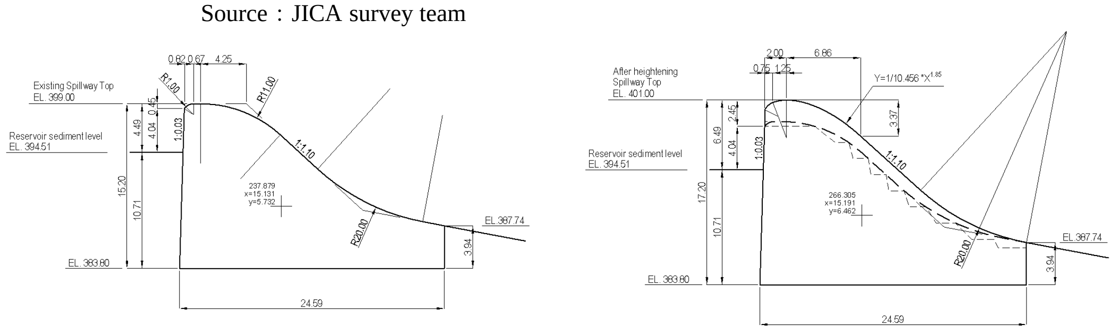
(1) Design condition

Design condition of Dam Spillway stability analysis is considered as shown in the Table 5.2.3 below.

Table 5.2.3 Design Condition of Existing

		Existing	After heightening
Elevation of Top of Dam	EL.m	399.000	401.000
Upstream Slope	1:n	0.030	←
Downstream Slope	1:n	1.100	←
Dam base elevation	EL.m	383.800	383.800
Reservoir sediment level	EL.m	394.510	←
Reservoir water level [CCN]	EL.m	383.800	←
[CCE]	EL.m	406.000	408.000
[CCL]	EL.m	399.000	401.000
Unit weight of concrete dams	kN/m^3	23.5	←
Weight of sediment in the water	kN/m^3	8.5	←
Unit weight of water	kN/m^3	10.0	←
Seismic Coefficient: Horizontal (kh)	---	0.050	←
Seismic Coefficient: Vertical (kv)	---	0.030	←
Coefficient of earth pressure (Rankine coefficient of earth pressure)	---	0.40	←
Uplift pressure coefficient	---	1/3	←
Shear strength of foundation	kN/m^2	1,000.0	←
Friction angle of foundation	deg	38.00	←
Internal friction coefficient	---	0.78	←

Source：JICA survey team



Existing **Heightening**

Source：JICA survey team

Figure 5.2.8 Typical Section of Existing

(2) Results

1) Existing dam

The stability condition is satisfied.

The bearing capacity requirement is satisfied since the allowable compressive stress intensity (10 MN/m^2) is more than σ_{max} (370 kN/m^2).

Table 5.2.4 Result of the calculation

	FSF	FST	FSD ≧ 1.0
[CCN]	∞ > 1.30	59.02 > 1.50	27.66 ≧ 1.0
[CCE]	6.14 > 1.10	4.76 > 1.20	8.17 ≧ 1.0
[CCL]	8.71 > 1.10	8.39 > 1.10	13.15 ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	370.45 ≤ 30M/3.0=10M	84.22 ≥ -200
[CCE]	165.44 ≤ 30M/2.0=15M	215.23 ≥ -200
[CCL]	237.47 ≤ 30M/1.5=20M	152.89 ≥ -200

Source：JICA survey team

Note:

Allowable compressive stress intensity of rock

$$\sigma_{\max} = \frac{\sigma_k}{\sigma_t} = \frac{30MN/m^2}{3.0 \sim 1.3}$$

Allowable tensile stress intensity of concrete

$$\sigma_{\min} = -\frac{\sigma_{ck}}{80} = -\frac{16}{80} = -0.2N/mm^2 = -200N/m^2$$

2) Heightening Dam

The stability condition is satisfied.

The bearing capacity requirement is satisfied since the allowable compressive stress intensity (10 MN/m^2) is more than σ_{max} (420 kN/m^2).

Table 5.2.5 Result of the calculation

	FSF	FST	FSD ≧ 1.0
[CCN]	∞ > 1.30	66.34 > 1.50	28.52 ≧ 1.0
[CCE]	6.31 > 1.10	4.04 > 1.20	6.99 ≧ 1.0
[CCL]	8.61 > 1.10	6.84 > 1.10	10.96 ≧ 1.0

	Upstream (kN/m^2)	Downstream (kN/m^2)
[CCN]	420.18 ≤ 30M/3.0=10M	88.82 ≥ -200
[CCE]	159.92 ≤ 30M/2.0=15M	268.41 ≥ -200
[CCL]	247.00 ≤ 30M/1.5=20M	189.40 ≥ -200

Source：JICA survey team

5.2.4 Stability Analysis of Rock-fill Section

The design of the spillway heightening does not impact the dam body since the highest water level does not change. This section is consists of the seepage and sliding failure because the original design report is not available.

(1) Basic Condition

1) Property of Dam

Since there is no available data about the physical parameter and drawings, the physical parameters are defined as general values (see the table below) and the dimension were traced the old drawings which were difficult to read.

Table 5.2.6 Property of Material for Calculation

	Material	**κ (cm/s)**	**e**	**t (g/cm^3)**	**W_n (%)**	**s (kN/m^3)**	**φ (deg)**	**C (kN/m^2)**
1	Core	5.0 E-5	0.48	1.8	10.0	19	---	80
2	Filter	5.0 E-2	0.37	1.9	5.0	20	30	---
3	Transit (Random)	5.0 E-4	0.48	1.8	5.0	19	25	---
4	Rock	Free drain	0.25	2.0	2.0	21	37	---
5	Foundation (Rock)	1.0 E-7	0.20	2.2	2.0	23	38	1000

κ :Hydraulic conductivity
e :void ratio
t :wet density
W_n :Natural water content
s :saturated density
φ :Internal friction angle
c :Cohesion

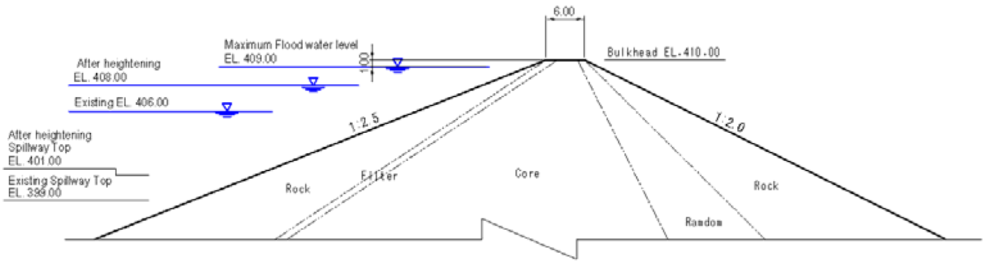
2) Water Level Condition

The most critical water condition for seepage flow is the head water level coming to "the bulkhead elevation – 1.0 m". Therefore the analysis is carried out with this water level (See the below table)

Table 5.2.7 Design water level

	Water level (El.m)	Remarks
Existing	406.00	1/10,000 year probability flood
Heightening	408.00	1/10,000 year probability flood
Design Criteria of Brazil	409.00	Non-overflow Elevation - 1.0m

Source : JICA survey team



Source : JICA survey team

Figure 5.2.9 Design Water Level

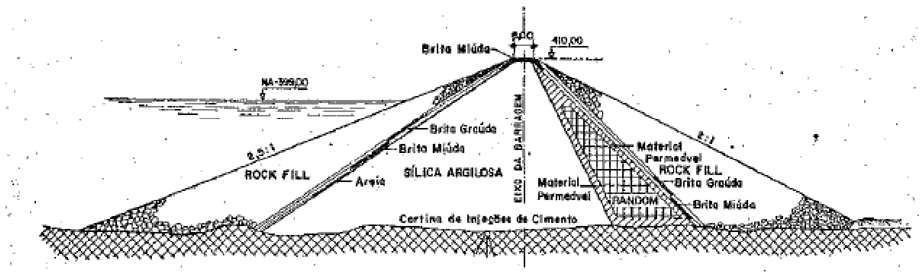


Figure 5.2.10 Traced Old Drawing

3) Analysis method for seepage flow

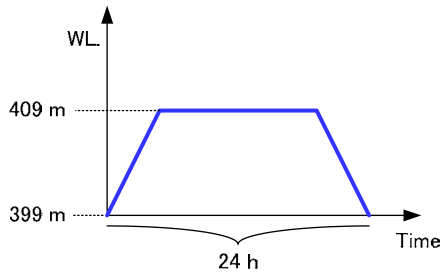
Two-dimensional

The finite element method

Unsteady – flow

upstream 399 - 409 – 399 m (as right figure)

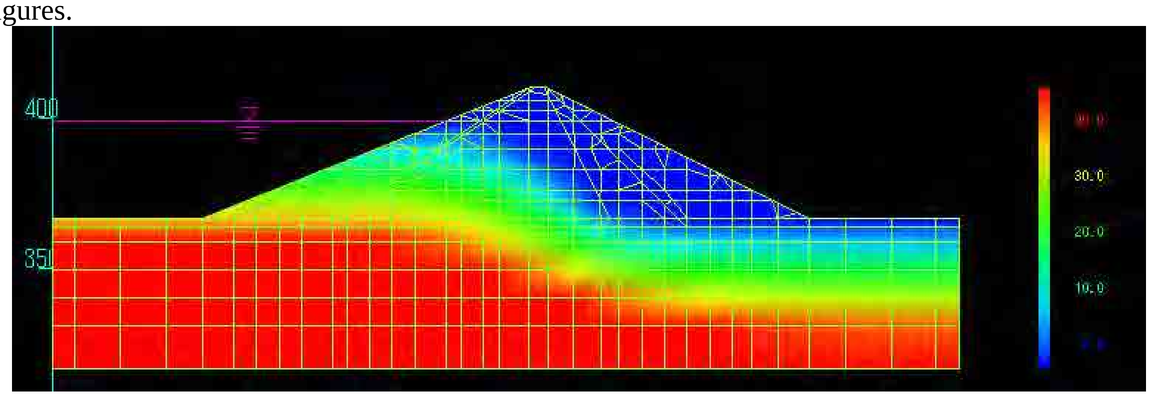
downstream 366.5 m (ground level)



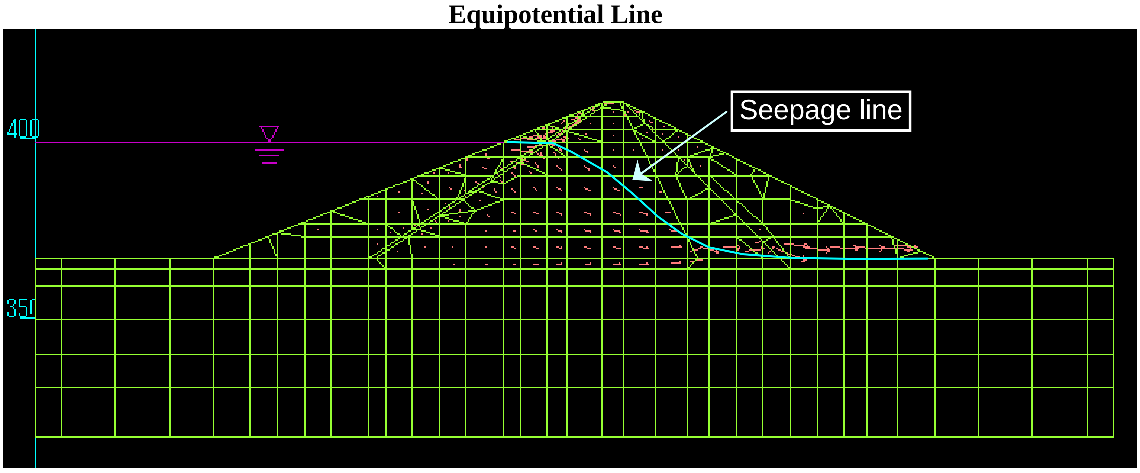
(3) Seepage flow analysis

1) Calculation Result

The estimated equipotential line and flow vector of seepage are illustrated as in the following figures.



Equipotential Line



Flow Vector

Source JICA Survey team

Figure 5.2.11 Isobaric and Velocity Chart

Table 5.2.8 Seepage velocity at each zoom

	Velocity (cm/s)	Hydraulic Gradient(x)	Hydraulic Gradient(y)
Core	2.08 E-04	4.68 E+00	7.73 E-01
Filter	7.58 E-03	6.60 E-01	1.44 E-01
Transit	1.99 E-04	1.84 E+01	6.55 E-00
Rock	1.35 E-02	2.63 E-02	1.81 E-04

Source JICA Survey team

Table 5.2.9 Critical Velocity of Justin formula

Grain Diameter (mm)	Critical Velocity of Ground Water (cm/s)	Remarks
0.01	1.02	Cray
0.03 - 0.05	1.77 - 2.29	Silt
0.08	2.89	Very fine sand
0.10	3.23	Fine sand
0.30 - 0.50	5.60 - 7.23	Medium sand
0.80 - 5.00	9.14 - 22.86	Gravel

Source: JICA Survey team (based on Handbook of soil mechanics and foundation engineering (1983)

2) Assessment of Safety

The safety against piping is examined. If the seepage force (γw x i) exceeds the effective weight of the particle, the particle will be lifted upward.

The hydraulic gradient which makes the effective stress zero is called a critical hydraulic gradient. The maximum hydraulic gradient which is estimated from seepage analysis should not be more than the critical hydraulic gradient.

With respect to piping occurring in dam body, soil particles would be easily eroded at the toe of slope because seepage flow velocity and hydraulic gradient are largest there. In order to check such a seepage failure, the safety at the toe of the core part was studied for reference. The dam safety where the surface of pervious foundation in downstream side is covered by cohesive soil is checked by the following equation:

$$\frac{G}{W} = \frac{(\rho_E \cdot H)}{(\rho_W \cdot P)} > 1.0$$

where,

G = weight of covering layer (kNf/m^3)
W = uplift pressure acting to the bottom of the covering layer (kNf/m^3)
ρ_E = density of covering layer (kN/m^3)
H = height of covering layer (m)
ρ_W = density of water (kN/m^3)
P = pressure head at the bottom of covering layer (m)

The following values are estimated by the equation above:

ρ_E = 19.0 (kN/m^3) as saturated density of the core
H = 84.0 (m) as the bottom width
ρ_W = 10.0 (kN/m^3)
P = 45.50 (m) as the water depth for Maximum Flood water level of EL. 409.00 m (=409.00-363.50)
($P = P_w/\rho g = \rho g h/\rho g = h$)

$$\frac{G}{W} = \frac{19 \text{x} 84.0}{10 \text{x} 45.5} = 3.51 > 1.0$$

The result indicates that the estimated G/W is larger than 1.0. Thus, the piping of dike and foundation is assessed to be less likely to occurre.

In general, no matter high dams, the impervious cores having widths of 30 % to 50 % of water head usually perform satisfactory. The Sul dam is wide enough to be considered since the rate of the width and water head is 185%.

$$\frac{\text{Bottom width}}{\text{Waterhead}} = \frac{84.0}{45.5} = 185\ \% \ (>30 - 50\ \%)$$

(4) Calculation Stability Analysis of Main dam

1) Required Safety Factor

The required safety factor against slope failure is 1.3 as shown the table below.

Table 5.2.10 Safety Factor of Circular Slip

	Safety faoctor	Remarks
Construction	$1.3^{(a)}$	Upstream and downstream slopes
Unsteady-state	$1.1 \sim 1.3^{(b)}$	
Steady-state	1.5	Downstream Slope
Seismic	1.0	Upstream and downstream slopes

Notes:
(a) Fs=1.4 in case the height of dam is over 15 m
(b) if more frequency, Fs=1.3
Source: CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003
Equation for Safe Factor

The equation used for safety factor calculation is as follows:

$$SF = \frac{\sum\left\{cl + \left(N - U - N_e\right)\tan\varphi\right\}}{\sum\left(T + T_e\right)}$$

SF:	Safety factor
N:	Vertical component of load on slip surface of each slice (dead weight: W + hydrostatic pressure: E)
T:	Tangent component of load on slip surface of each slice (dead weight: W + hydrostatic pressure: E)
U:	Pore pressure on slip surface of each slice
N_e:	Vertical component of sesmic inertia force on slip surface of each slice:
T_e:	Tangent component of seismic inertia force on slip surface of each slice
Φ:	Internal frictional angle on slip surface of each slice
c:	Cohesion on slip surface of each slice
l:	Length of slip surface of each slice

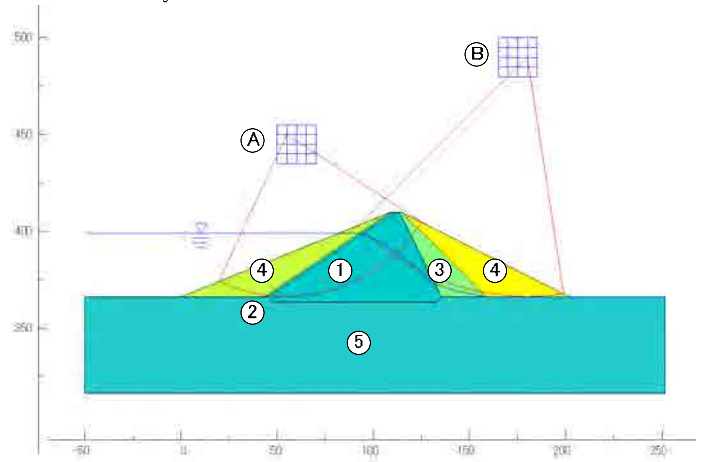
2) Result and Assessment

The result of stability analysis is summarized as in the following table and figure. The minimum safety factors for both cases satisfy the required safety factor for both normal and seismic conditions. The result indicates that the Sul dam can keep the stability in terms of sliding failure.

Table 5.2.11 Result of Circle Slip

Circle	Central coordinates		Radius	Safety .
	X (m)	Y (m)	(m)	Factor
A (upstream)	55.0	450.0	83.5	1.396
B (downstream)	180.0	490.0	123.5	1.439

Source:JICA Study Team



Source:JICA Study Team

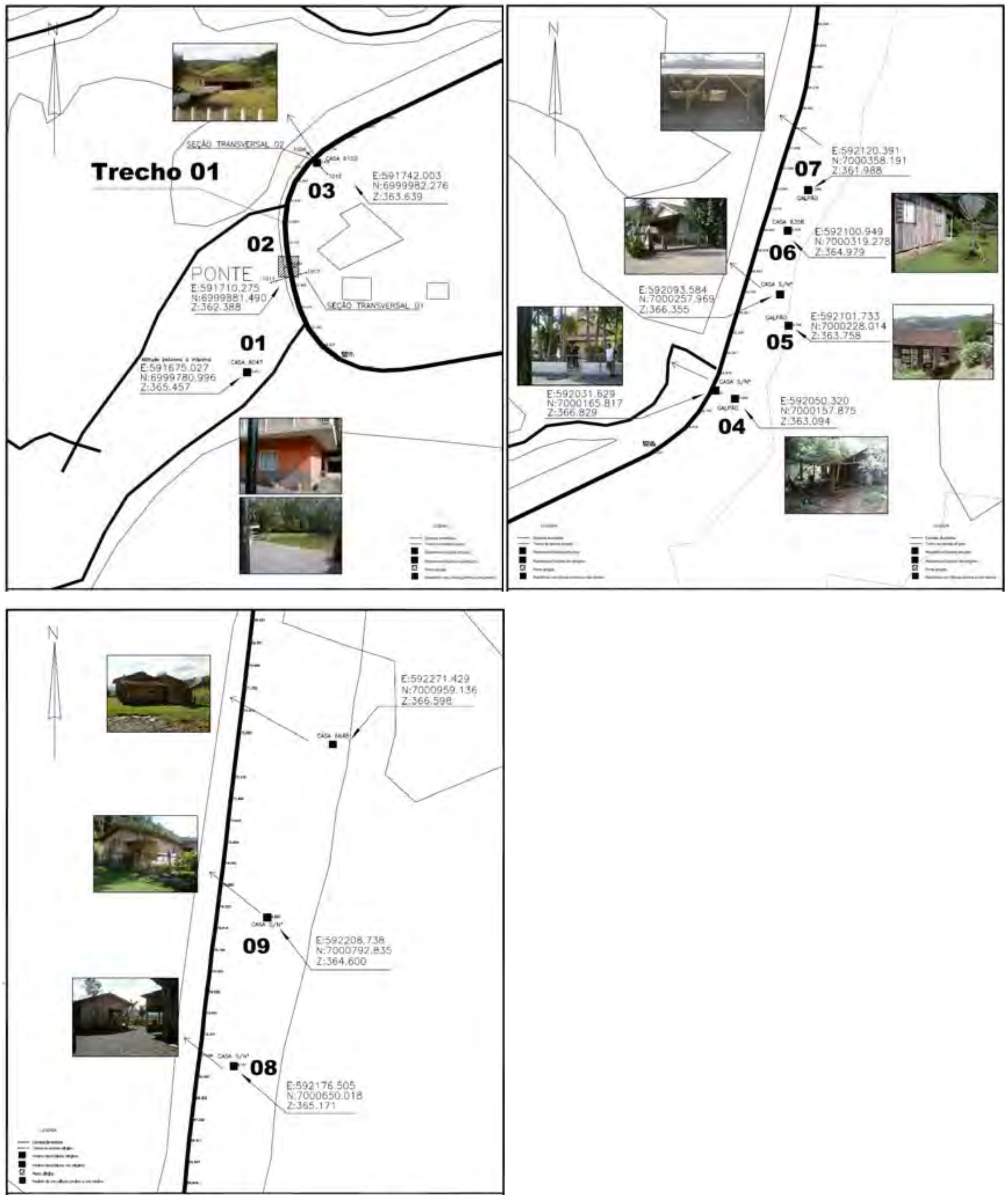
Figure 5.2.12 Result of Slip Circle

5.3 Additional facility

Due to the heightening of the Oeste dam by 2.0 m, the elevation of the part of the houses and road is less than the PMF water level (which is the Probable maximum flood). Thus the land acquisition requires the area whose elevation is less than the new dam crest (EL. 365.16 m) due to the current condition that the area of land acquisition is that of the height of the dam crest (EL. 363.0 m).

(1) Condition of the reservoir area of the Oeste dam

The figure below is shown the result of the field observation. There are four houses and three coops which is influential.



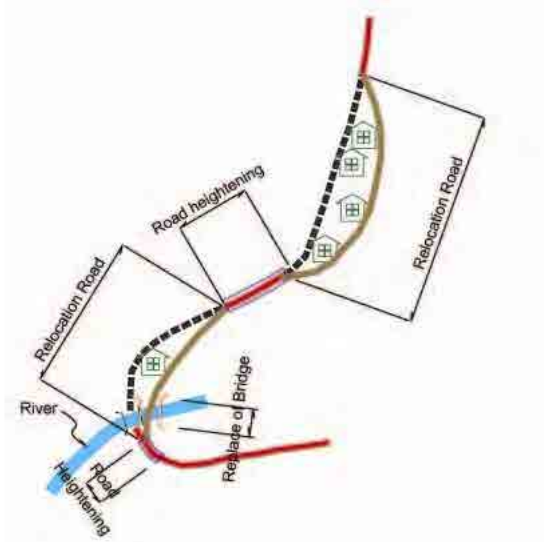
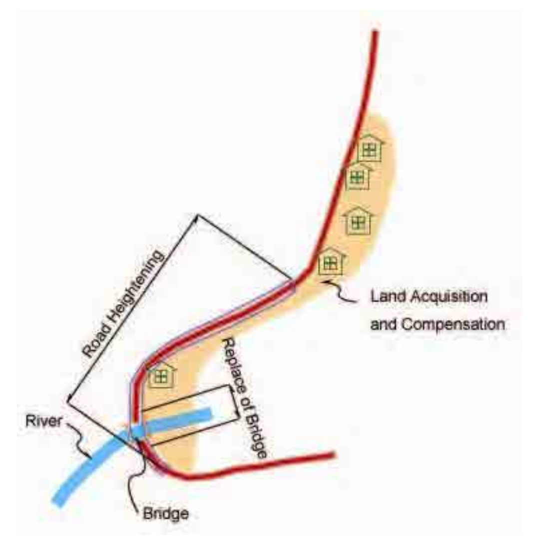
Source: JICA survey team

Figure 5.3.1 Result of Survey Study (Oeste Dam)

(2) Countermeasure

It is proposed that the countermeasure against the inundation houses is 2 ways.

Table 5.3.1 Comparison of Countermeasure Against Inundation

	Alternative measure-1: with road relocation	**Alternative measure-2: with resettlement**
Chart		
General description	・Some sections of roads and bridges shall be rerouted/relocated to protect existing buildings from being inundated by heightening the dam. Hence, the height of the rerouted roads and relocated bridges shall be higher than that of the heightened dam crest.	・The buildings located in the potential inundation areas shall be relocated. ・Some sections of the roads and bridges, whose heights are lower than that of the heightened dam crest, shall be relocated
Merit	・ No resettlement of the communities	・Less cost due to decrease of volume of construction works
Demerit	・Increase of construction cost due to road relocation education of inundation area due to installation of the road	
Project cost	R$ 4,797,000 (100%)	R$ 2,819,000 (58.8%)

Source: JICA survey team

Table 5.3.2 Implementation Cost for Countermeasure

(R$)

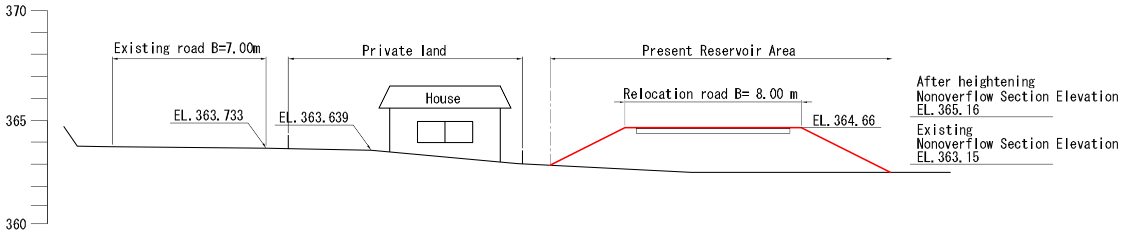
			Alternative of Road relocation		Alternative of Compensation		Remarks
	unit	unit cost	quantity	amount	quantity	amount	
Replace of Bridge	m2	3,000	160	480,000	80	240,000	
Relocation Road	m	1,570	1,500	2,355,000	500	785,000	
Other works	%	30	---	851,000	---	308,000	Main works *30%
[1] Sub total (Construction cost)				**3,686,000**		**1,333,000**	
Land acquisition	m2	1.388	670,000	930,000	670,000	930,000	All target areas
Permanent Crops	LS	36,000	1	36,000	1	36,000	
Compensation	LS	326,000	---	---	1	326,000	7 Buildings(=4+3)
Price contingency for area delineation	%	15	---	145,000	---	194,000	
[2] Sub total (Land, Compensation)				**1,111,000**		**1,486,000**	
Total [1]+[2]				**4,797,000**		**2,819,000**	

Source: JICA survey team

The proposed measure of the relocation road is that the new road is constructed in reservoir. Thus the reservoir loses the water storage volume about 90,000 m^3. This figure equals that the design water storage level requires 1 cm higher than proposed. However with the heightening by 1 cm, the comparison of countermeasure is not considered as the following reasons; the shape of the dam is not changed, and the construction volume is only 10.3 m^3 of concrete.

(3) Selected Countermeasure

The relocation road is selected in view point of no resettlement. As the image is the figure below, the elevation of relocation road is higher than the possible raising water level so that the existing houses is not required to inundate.



Source: JICA survey team

Figure 5.3.2 Typical Section of Relocation Road

(4) Under Construction Bridge in Sul dam Receiver

There is a construction bridge in reservoir whose elevation is about EL.405.0 m. The impact of heightening of Spillways is only that the frequency of inundation is higher. But if the bridge was inundated with flood, the period of that time is short. Therefore the re-construction of bridge is not selected project for Feasibility Study.

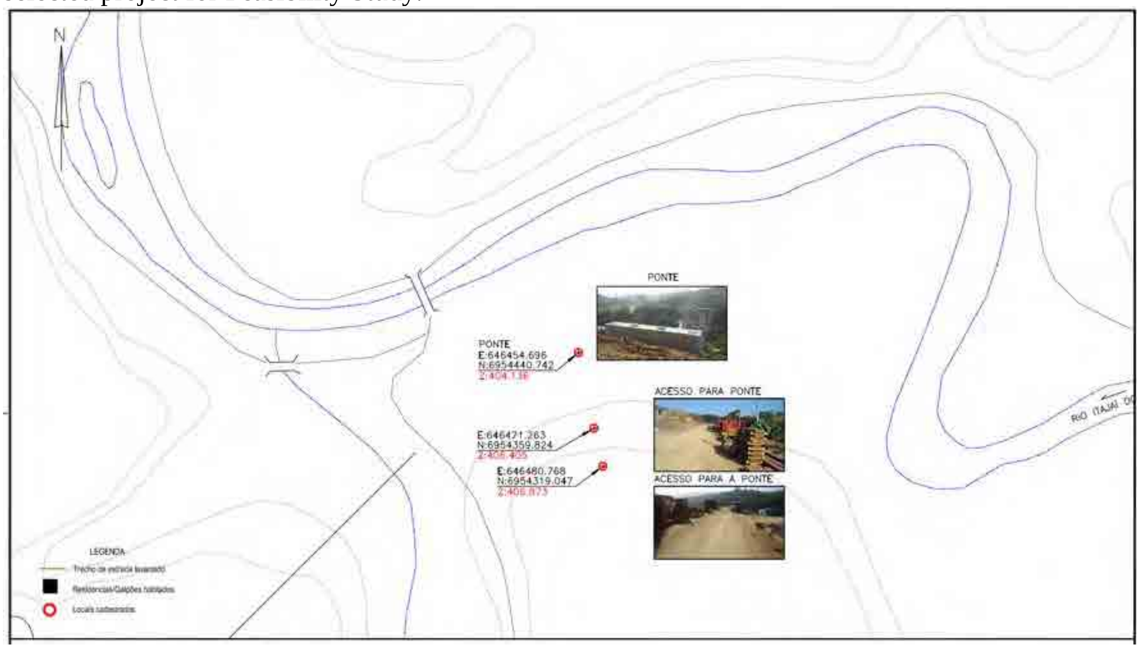


Figure 5.3.3 Survey Result on Sul Dam

5.4 Recommendation

The standard design which is stable under general design conditions was applied to the design of related structures. However, the detailed computation for structural analysis is not made in this phase of the study. In the future, the following recommendation will be studied .

(1) Oeste dam

- It is not insufficient to understand the geological structure. In this phase study, the foundation level is judged by three frilling points. The countermeasure required depends on the height of foundation. Thus the height should be surveyed more detailed.
- The elevation of the foundation is determined based on the assumption by three drilling points. The countermeasure required depends on the height of foundation. Thus the height should be surveyed more detail.
- The physical properties of foundation and dam body themselves are supposed to the general value. The physical, geotechnical rock test should be done on laboratory with the site material.
- The stability of dam body should be tested by FEM analysis in terms of the safety against crack, because the connection between the old concrete and the new concrete might become the weak point.

(2) Sul dam

- It is not insufficient to understand the geological structure. In this phase study, the geological information of spillway is surveyed. There is no geological information for dam body.
- It is not insufficient to understand the geological structure since the drilling survey is carried out at only one point in the whole area.
- The part of fill is not surveyed, so each sections should be surveyed about the shape and the physical properties and reanalyze the stability.

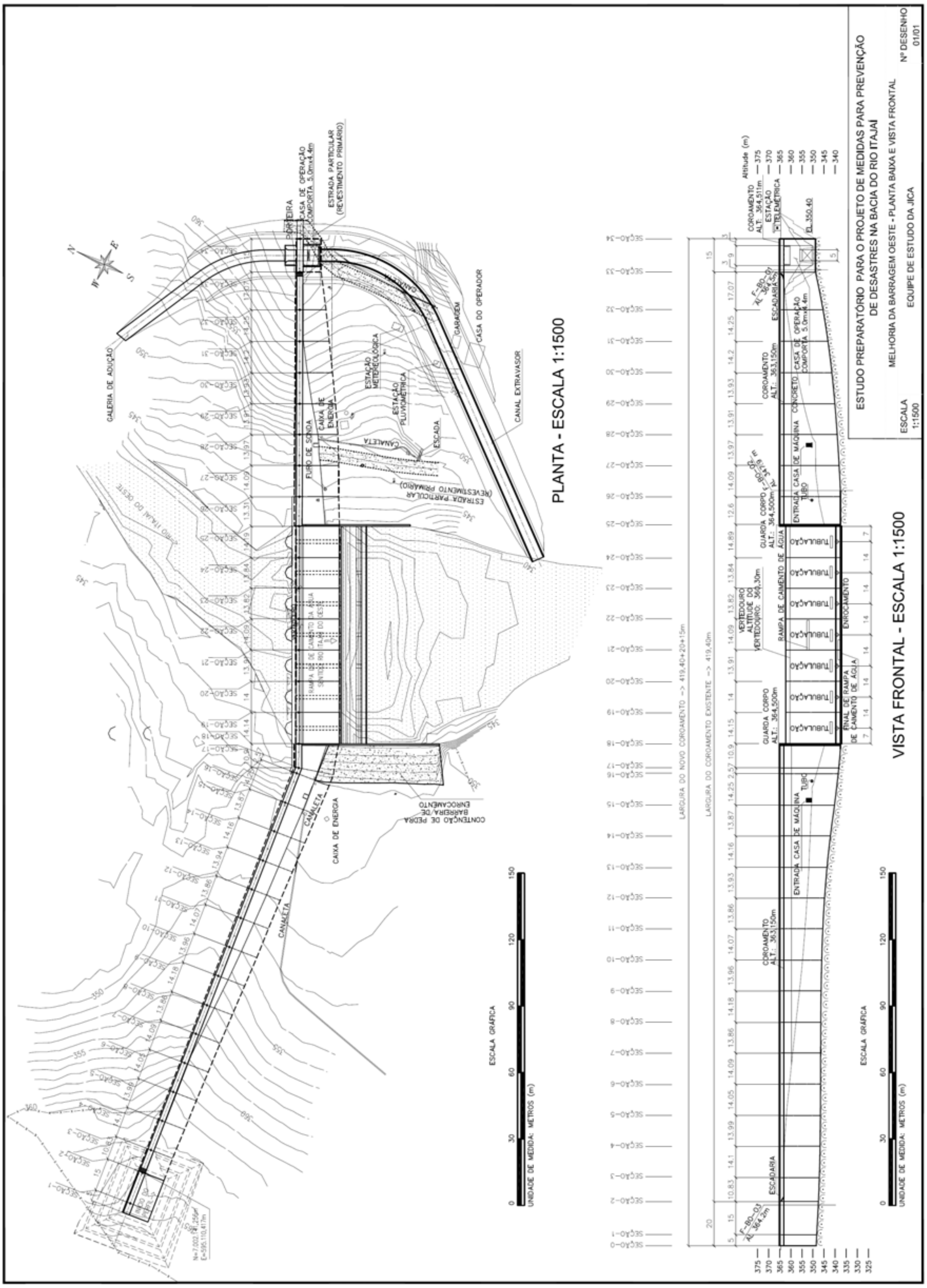


Figure 5.3.4 Heightening Oeste Dam (1)

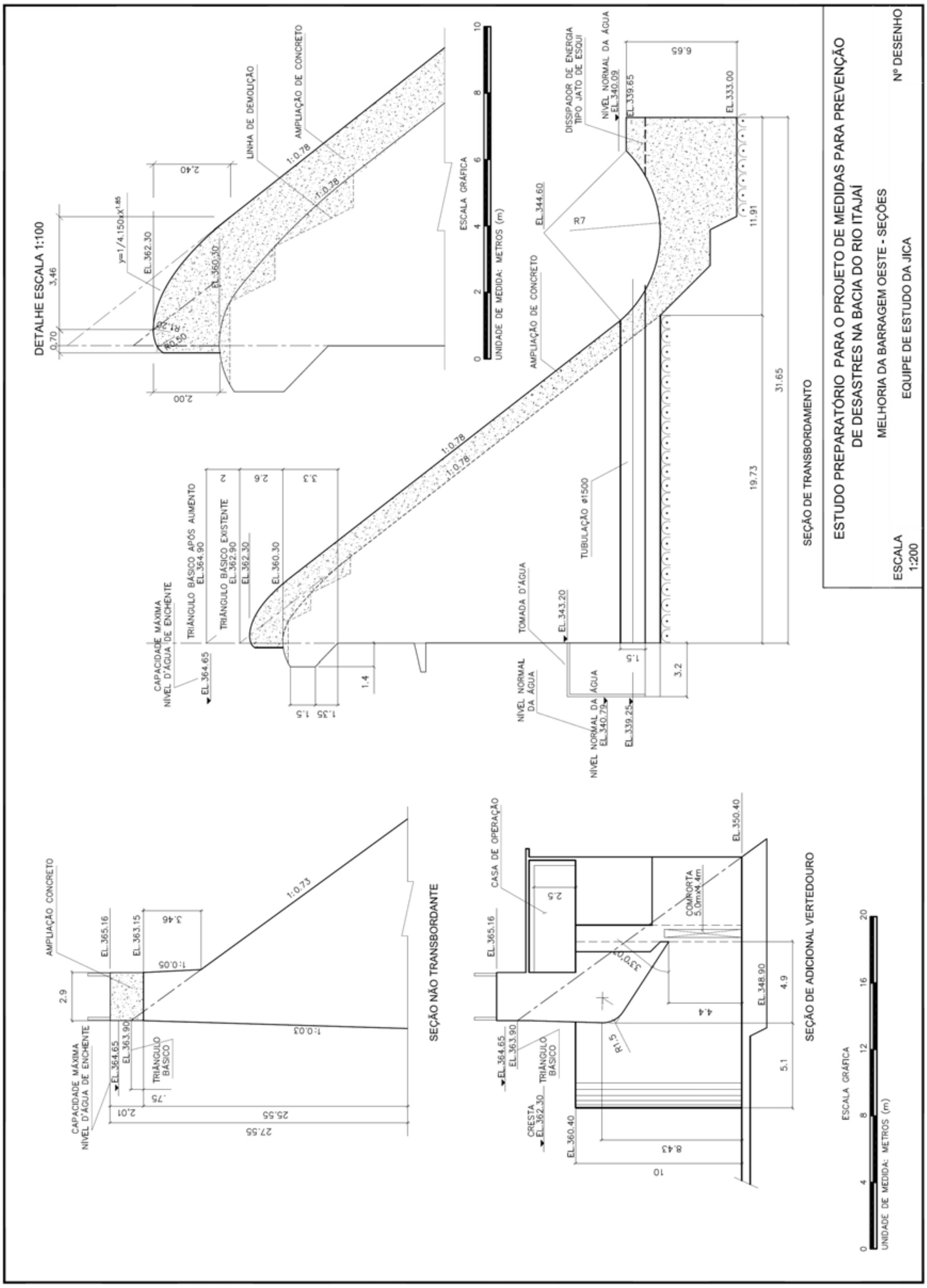


Figure 5.3.5 Heightening Oeste Dam (2)

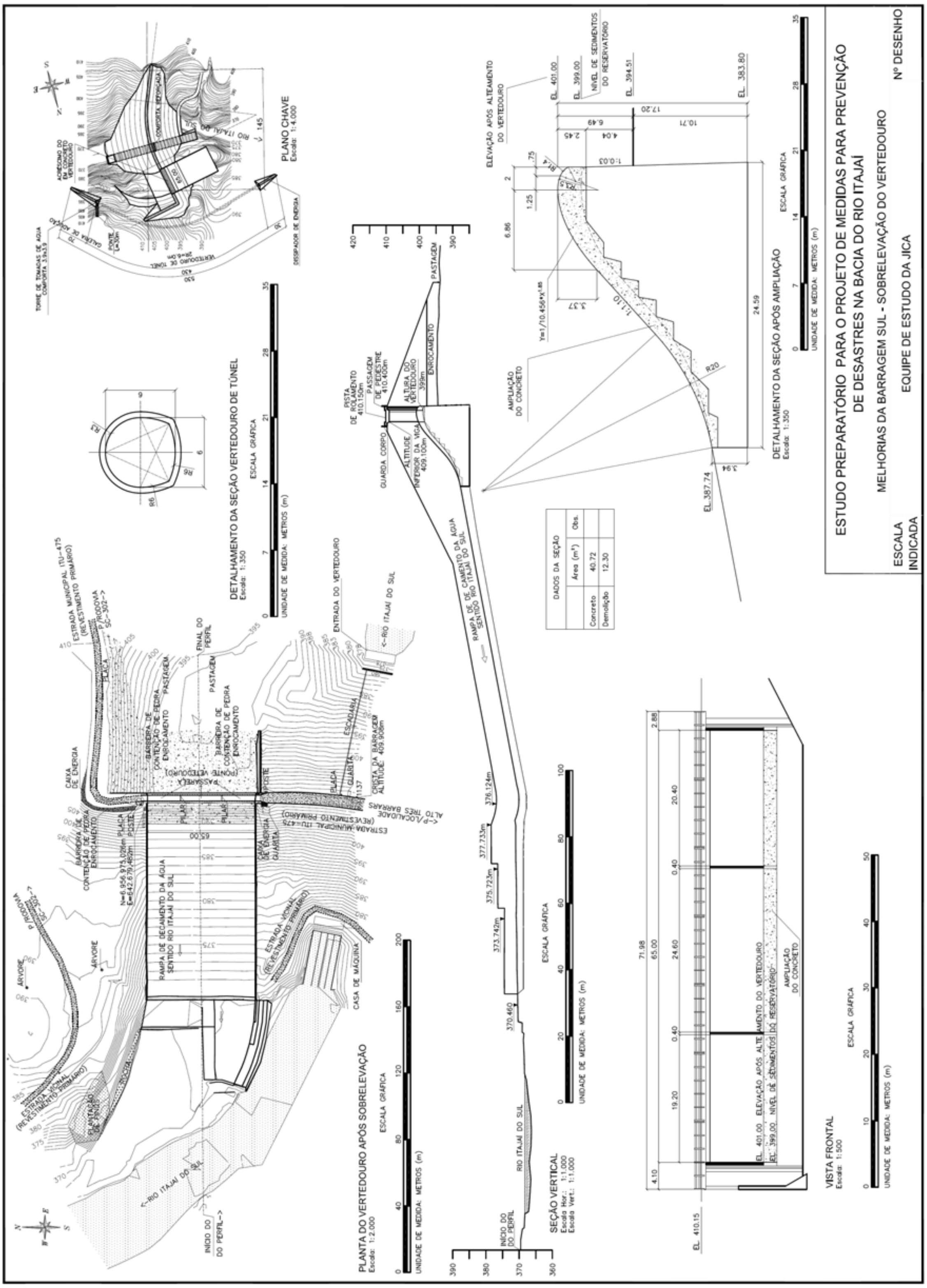


Figure 5.3.6 Heightening Sul Dam

CHAPTER 6 EXAMINATION FEASIBILITY DESIGN OF STEEL STRUCTURES

6.1 Introduction

The purposes of examination are to:

i) Assess the necessity of replacement of new gates due to heightening of two dams, and

ii) Make a feasibility design of the proposed flood gates on the Itajai Mirim River.

Table 6.1.1 presents the objective steel structures for examination.

Table 6.1.1 Objective Steel Structures

Facilities	Location	Steel Structure	Quantity	Size
Control Gate (Dam Heightening)	Oeste Dam	Slide gate Conduit pipe	7 sets	φ1500mm
	Sul Dam	Slide gate Conduit pipe	5 sets	φ1500mm
Flood Gate	Upstream of Itajai Mirim river	Fixed wheel gate	4 sets	W12.5m×H4.5m
	Downstream of Itajai Mirim river	Fixed wheel gate	4 sets	W12.5m×H3.6m

Source: JICA Survey Team

The contents of examination are enumerated in the table below.

Table 6.1.2 Contents of examinations feasibility design

Location	Steel Structure	Contents of examinations feasibility design
Oeste Dam	Slide gate Conduit pipe	(1) Site investigations (2) Assessment of the necessity of replacement (3) Repairing items and methods (4) Cost estimate
Sul Dam	Slide gate Conduit pipe	
Upstream of Itajai Mirim river	Fixed wheel gate	(1) Selection of gate type (2) Selection of corrosion protection measure (3) Estimation of design loads (4) Cost estimate
Downstream of Itajai Mirim river	Fixed wheel gate	

Source: JICA Survey Team

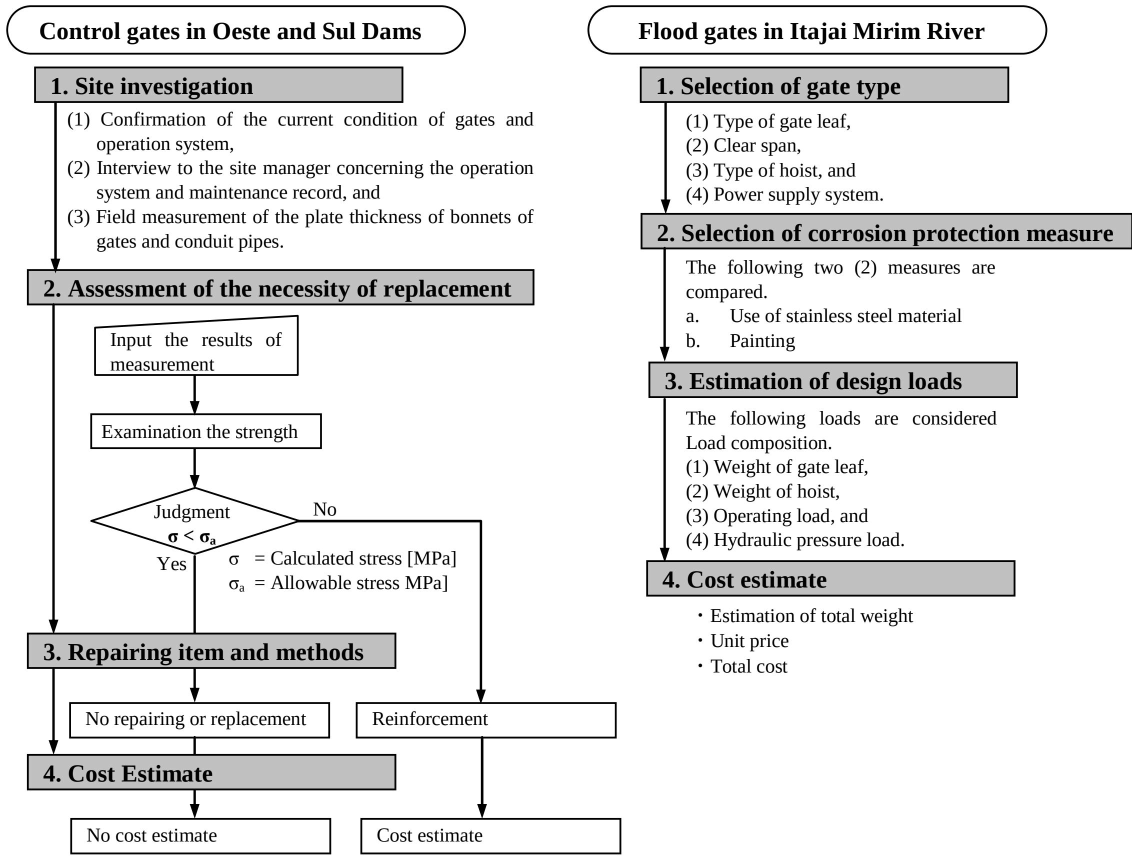
The work flow under examination is shown below.

6.2 Control Gates

6.2.1 Design Conditions

(1) Design data of gates

The design conditions of the control gates are summarized as Table 6.2.1.



Source: JICA Survey Team

Figure 6.1.1 Work Flow of Examination

The result of the examination is described be in after.

Table 6.2.1 Design Conditions of Control Gates

Particulars	Control gate in Oeste Dam	Control gate in Sul Dam
Type	Steel made slide gate	Steel made slide gate
Quantity	7 sets	5 sets
Diameter	1500mm	1500mm
Max. water level	EL.364.65m	EL.408.00m
Flood water level	EL.362.30m	EL.401.00m
Normal water level	EL.340.79m	EL.387.00m
Gate center elevation	EL.339.25m	EL.368.00m
Foundation rock elevation	EL.337.60m	EL.357.50m
Material of gate	A36 (ASTM)	A36 (ASTM)
Sealing system	Metal seal at both sides of gate leaf	Metal seal at both sides of gate leaf
Operating device	Hydraulic cylinder	Hydraulic cylinder
Size of cylinder	Inside diameter of cylinder:160mm Outside diameter of rod:90mm Stroke:1570mm	Inside diameter of cylinder:200mm Outside diameter of rod:100mm Stroke:1570mm
Oil pressure	Normal (rating) pressure: 21MPa Max. pressure: 35MPa	Normal (rating) pressure: 16MPa Max. pressure: 20MPa
Operation system	Local	Local
Constructed year	1978	1969
Repaired year	—	2007
Repaired items	—	Hydraulic unit & Operating panel
Manufacturer	HISA*	HISA*

Remarks; HISA: Hidráulica Industrial S.A. Ind.

Source: JICA Survey Team

(2) Water levels

Flood operation water levels will be raised by 2.0 m after heightening as follows:

Table 6.2.2 Operation Water Levels

Water Level	Oeste dam (Gravity Type)		Sul dam (Earth fill Type)	
	Before Heightening	After Heightening	Before Heightening	After Heightening
Max water level	EL.362.65m	EL.364.65m	EL.408.00m	EL.408.00m
Flood water level	EL.360.30m	EL.362.30m	EL.399.00m	EL.401.00m
Normal water level	EL.340.79m	EL.340.79m	EL.387.00m	EL.387.00m

Source: Survey results under thr JICA Survey Team

6.2.2 Site Investigations

The site investigation was carried out for the following items:

- Confirmation of the current condition of gates and operation system,
- Interview to the site manager concerning the operation system and maintenance record, and
- Field measurement of the plate thickness of bonnets of gates and conduit pipes.

(1) Condition of gates

The current condition of the gates is clarified as shown in the following Table 6.2.3.

Table 6.2.3 Current condition of Gates

Check item	Oeste Dam	Sul Dam
Water leakage	・Water leakage was observed at the flange of all gates. ・Water leakage was observed at the expansion joints of all gates.  No.1 slide gate	・Water leakage was observed at the flanges and expansion joints of all gates.  No.4 expansion joint
Oil leakage	・No oil leakage was observed from the hydraulic unit and cylinder.  Hydraulic unit	・No oil leakage was observed from the hydraulic unit and cylinder.  Cylinder of No.5 slide gate
Dirt	・Dirt caused by water leakage was observed at all gates.	・No dirt was observed for all gates because the pits were covered with the leakage water.

Check item	Oeste Dam	Sul Dam
	 Pit of No.5 slide gate	 Leakage water in pit (No.2 gate)
Damage	・ No damage was observed at the gates.	・ No damage was observed at the gates.

Source: JICA Survey Team

Large water leakage was found at the both dam gates. It might be due to that the water leakage was caused by dismantling the bonnet flanges at the time of overhaul in 1983. A lot of sand has been accumulated in the pits. Although the accumulated sand and leakage water might not affect to the gate operation directly, drain pumps might be effected to cause trouble.

Drain pumps in Oeste Dam (left) and Sul dam (right)

Source: JICA Survey Team

(1) Operation system

The current condition of gate operation system is summarized below.

Table 6.2.4 Operation System of Gates

Check Item	Oeste Dam	Sul Dam
Operation staff	・ One operator is stationed in day-time. ・ No data on the night operation shift	・ One operator is stationed in day-time. ・ The residents in the vicinity of the dam reported the abnormal operation to the operation staff in night time.
Opening range of gate	・ 0% and 100%	・ 0%, 33%, 66% and 100%
Operation system	・ Local	・ Local
Emergency generator	・ No emergency generator is installed.	・ Emergency generator is installed. 
Emergency power	・ When the motor is out of service, the stand-by engine can supply the power.	・ When the motor is out of service, the stand-by engine can supply the power.

Check Item	Oeste Dam	Sul Dam
		

Source: JICA Survey Team

(2) Maintenance records

The maintenance records of the gates are as shown in the following Table.

Table 6.2.5 Maintenance Records of Gates

Check item	Oeste Dam	Sul Dam
Repainting	・ No repainting has not been made so far.	・ No repainting has been made so far.
Overhaul	・ Overhaul has been carried out in the past, but the date is unknown. ・ After removing the gate leaf, the openings are covered by the bulkhead plates.  **Bulkhead plates**	・ Overhauled was carried out in 1983. ・ The overhaul procedure is as follows: 1) Installation of chain block on a ceiling hook 2) Removal of cylinder 3) Removal of bonnet 4) Removal of gate leaf ・ The overhaul is carried out in the dry season and it took about 1 week for a unit. ・ After removing the gate leaf, the opening is covered by the bulkhead plate.  **Bulkhead plate**
Replacement	・ No record	・ The operating panels and hydraulic units were replaced with new ones in 2007.

Source: JICA Survey Team

(3) Measurement of plate thickness of bonnets of gates and conduit pipes

1) General

Since no design calculations on the gates and conduit pipes are available at the present, the plate thicknesses of the bonnets of gates and conduit pipes are unknown. Accordingly, the measurement for thickness thereof was carried out so as to confirm the strength of bonnets of gates and conduit pipes. The ultrasonic thickness gauge, was used for the measurement of plate thicknesses thereof.

2) Measuring items

The gate was constructed by the same structure each other and are manufactured at the same time. Further, the operation and maintenance thereof are also the same conditions each other. The measurement of plate thickness of bonnets of gates and conduit pipes was therefore carried out for the following gates.

a. No.2 gate in the Oeste dam

b. No.1 gate in the Sul dam

3) Measuring locations

The plate thickness can be measured from the outside thereof by measurement instrument. The thickness of gate leaf can not be measured since the gate leaf is stored in the bonnet.

a. Gates

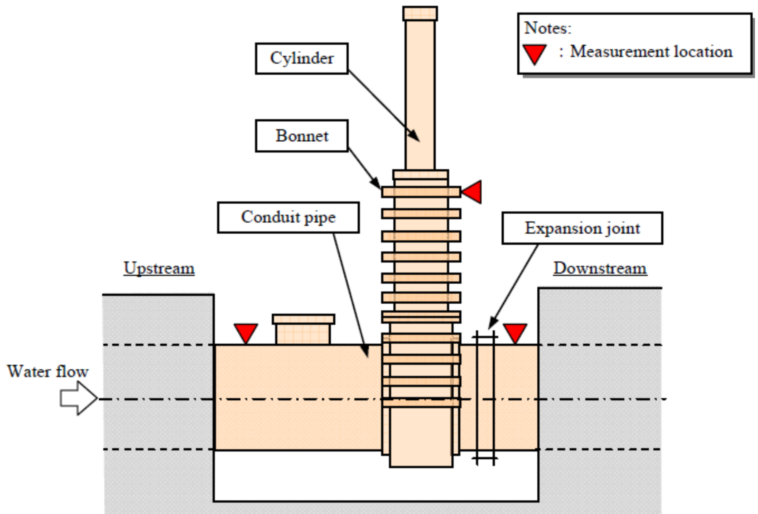
a-1 Thickness of stiffener girder (Bonnet)

a-2 Bonnet outline dimensions

b. Conduit pipes

b-1 Thickness of conduit pipe

The location of measurement is illustrated below.



Source: JICA Survey Team

Figure 6.2.1 Control Gate and Conduit Pipe

4) Measuring instrument

a. Ultrasonic thickness gauge (manufactured by JFE-Advantech in Japan)

b. Tape measure and vernier caliper

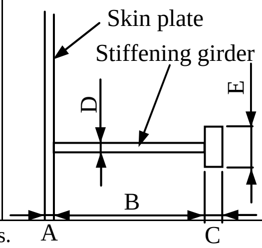
Source: JICA Survey Team

Figure 6.2.2 Ultrasonic Thickness Gauge

5) Results of measurement

The results of measurement are summarized below.

Table 6.2.6 Results of Measurement

Item	Oeste Dam	Sul Dam	Remarks
Plate thickness of conduit pipe	Upstream: 5.93mm Downstream: 6.51mm	Upstream: 9.17mm Downstream: 8.66mm	–
Plate thickness of stiffener girder	A: 12.50mm (12.7mm) B: 100.00mm (100.0mm) C: 20.00mm (20.0mm) D: 12.80mm (12.7mm) E: 65.0mm (65.0mm)	A: 12.58mm (12.7mm) B: 122.00mm (123.0mm) C: 26.00mm (25.4mm) D: 16.20mm (16.0mm) E: 100.00mm (100.0mm)	

Notes: 1. Figureures in parentheses are the estimated design values derived from the drawings.

2. The detailed measurement results are attached in the Appendix 1.

Source: JICA Survey Team

6.2.3 Assessment of the necessity replacement

(1) Applied standards

The applied standards designing the existing gates are unknown since the design calculations thereof were lost due to the flood in 1983. Therefore, the standard of ABNT NBR 8883:2008 in Brazil to the gate design was applied. Therefore, the strength of the existing gates and conduit pipe were analyzed using the said standard. It is confirmed through the interview the gate manufacturer that the main material of the gate and conduit pipe is based on the A36 of ASTM standard.

(2) Allowable stresses

According to the ABNT NBR 8883, the allowable stresses to material are stipulated in the table below:

Table 6.2.7 Allowable Stresses

Yield point [MPa] (basic design strength)	Loading Condition*2)	Coefficient*3)	Allowable Stresses [MPa] *4)
250*1)	CCN: Normal water level only	0.50	125.0
	CCE1:Normal water level + Dynamic water pressure during earthquake	0.90	225.0
	CCE2: Flood water level only	0.63	157.5
	CCL: Flood water level+ Dynamic water pressure during earthquake	0.80	200.0

Notes: *1) ASTM A36/A36M-08 [TABLE3 Tensile Requirements]

*2) CRITÉRIOS DE PROJETO CIVIL DE USINAS HIDRELÉTRICAS Outubro/2003

*3) ABNT NBR 8883: 2008, [Tabela 6-Coeficientes “S” definidores de tensôes admissives]

*4) Allowable stress = [Yield point]×[Coefficient]

Source: ABNT NBR 8883 in Brazil

(3) Result of calculation

1) Bonnet of control gate

The strength of bonnet of control gate was calculated and detailed in Appendices 2 and 3. The strength of bonnet was calculated under the maximum converted load in case of CCN, CCE1, CCE2 and CCL. The converted load of each case is calculated that the actual load divides by the coefficient. The maximum converted load occurred at the flood water level (CCE2) from the relation between actual load and the coefficient. Accordingly, strength calculation is made for the CCE2.

Table 6.2.8 Relation between Actual Load and Coefficient

Design to Water Level		Coefficient	Load [kN]	
			Actual load	Converted load
Oeste	CCN	0.50	39.00	78.00
	CCE1	0.90	41.25	45.83
	CCE2	0.63	399.55	634.21 (Max.)
	CCL	0.90	417.65	464.06
Sul	CCN	0.50	329.35	658.70
	CCE1	0.90	347.31	385.90
	CCE2	0.63	572.03	907.99 (Max.)
	CCL	0.90	600.76	667.52

Notes: CCN: Normal water level only

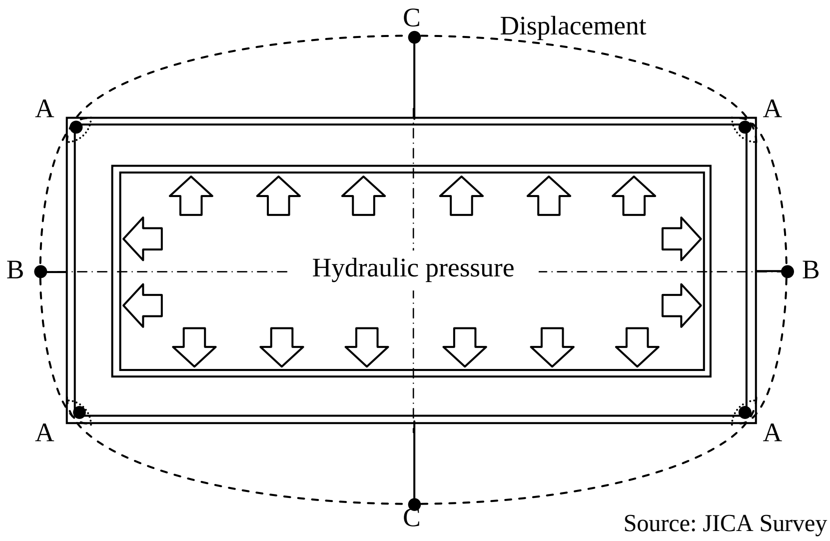
CCE1: Normal water level + Dynamic water pressure during earthquake

CCE2: Flood water level only

CCL: Flood water level+ Dynamic water pressure during earthquake

Source: JICA Survey Team

The strength calculation of stiffener girder is calculated for the following points A, B and C as illustrated below.



Source: JICA Survey

Figure 6.2.3 Location of Strength Calculation (Sectional View)

The stiffener girder has enough strength at the present since the calculated stresses are less than the allowable stresses as summarized in the table below.

Table 6.2.9 Result of Calculation (Stiffener girder)

Dam	Location	Stress	σ (Calculated stress) [MPa]		σ a(Allowable stress) [MPa]	Judgment σ < σ a
			After	Before		
Oeste	Point of A	Bending stress (Inside)	79.2	72.3	157.5	OK
		Bending stress (Outside)	61.6	56.3	157.5	
		Shear stress	41.7	38.0	90.9	
	Point of B	Bending stress (Inside)	111.4	101.7	157.5	OK
		Bending stress (Outside)	58.2	53.2	157.5	
		Shear stress	9.8	9.0	90.9	
	Point of C	Bending stress (Inside)	37.8	34.5	157.5	OK
		Bending stress (Outside)	77.3	70.6	157.5	
		Shear stress	41.7	38.0	90.9	
Sul	Point of A	Bending stress (Inside)	79.2	74.4	157.5	OK
		Bending stress (Outside)	40.7	38.2	157.5	
		Shear stress	39.6	37.2	90.9	
	Point of B	Bending stress (Inside)	105.5	99.1	157.5	OK
		Bending stress (Outside)	38.7	36.3	157.5	
		Shear stress	9.6	.9.0	90.9	
	Point of C	Bending stress (Inside)	36.9	34.6	157.5	OK
		Bending stress (Outside)	57.8	54.3	157.5	
		Shear stress	39.6	37.2	90.9	

Notes: After: After heightening, Before: Before heightening

Source: JICA Survey Team

2) Operating force

The operating force is calculated as shown in Appendices 2 and 3. The summary of calculation is given below. In conclusion, the cylinders have enough capacity for gates operation.

Table 6.2.10 Result of Calculation (Operating force)

Dam	Pulling force of cylinder[kN]				Pushing force of cylinder[kN]			
	Opening load		Operating force	Judgment	Closing load		Operating force	Judgment
	After	Before			After	Before		
Oeste	200.0	180.0	259.8	OK	170.0	150.0	228.0	OK
Sul	310.0	310.0	339.3	OK	260.0	260.0	271.4	OK

Notes: After: After heightening, Before: Before heightening

Source: JICA Survey Team

3) Conduit pipe

As shown in Table 6.2.8 above, the maximum converted load is also acted at CCE2. Accordingly, the strength calculation is also made for CCE2. The strength of the conduit pipe is calculated as shown in Appendices 4 and 5. In conclusion, the conduit pipes have enough strength at the present, since the calculated stresses are less than the allowable stresses.

Table 6.2.11 Result of Calculation (Conduit pipe)

Dam	Position	CASE of Calculation	σ (Calculated stress) [MPa]		σ a (Allowable stress) [MPa]	Judgment σ < σ a
			After	Before		
Oeste	Upstream	CCE2: Flood scale water level only	28.6	26.1	157.5	OK
	Downstream		26.1	23.8	157.5	OK
Sul	Upstream		26.5	24.9	157.5	OK
	Downstream		28.0	26.3	157.5	OK

Notes: After: After heightening, Before: Before heightening

Source: JICA Survey Team

6.2.4 Repairing Items and Methods

(1) Conduit pipes

At moment, repairing or replacement of the conduit pipes is not required since the pipes have enough strength even if the water level is raised up by 2.0m. However, there is a possibility of increasing the leakage water due to rising of water level. Though the leakage water does not affect the strength of the conduit pipe for the time being and can be drained by the drain pump easily, the water leakage shall be stopped with the replacement of packing and seal rubber, etc., as one of the maintenance work.

(2) Control gates

The repairing or replacement of control gates is also not required by the same reason of conduit pipes. The hydraulic cylinders have the ample operating forces even if the operation water level is raised up by 2.0m. The water leakage from the gates shall also be stopped as one of the maintenance work by the dam office.

6.2.5 Cost estimate

No cost estimate is required since any repairing or replacement work is not required substantially for the conduit pipes and control gates.

6.3 Flood Gates

6.3.1 Design Conditions

The design conditions of the flood gates are summarized as follows:

Table 6.3.1 Design Conditions

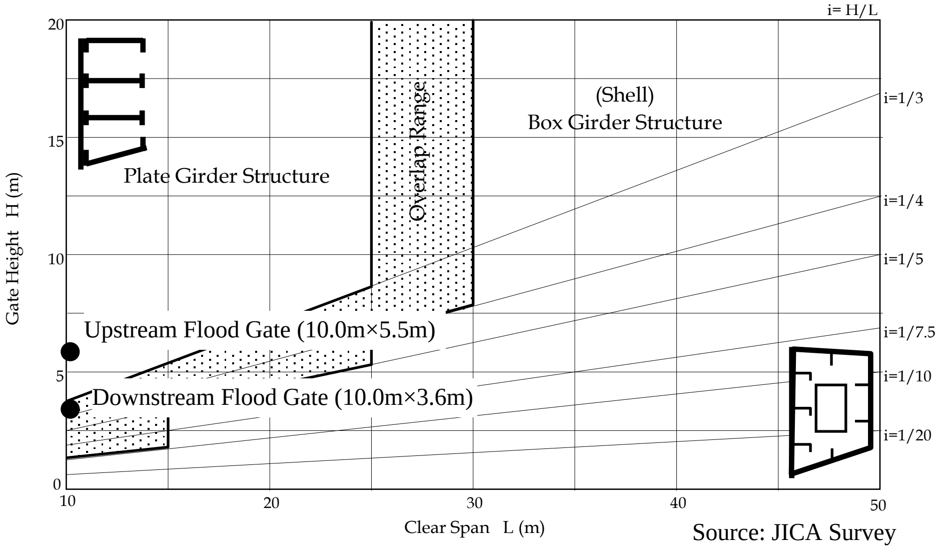
Particulars	Upstream Flood Gate	Downstream Flood Gate
Type of gate	Fixed wheel gate	Fixed wheel gate
Quantity	4 sets	4 sets
Clear span	10.0m	10.0m
Gate height	5.5m	3.6m
Sill elevation	EL.-1.00m	EL.-1.00m
Type of hoist	Wire rope winch hoist	Wire rope winch hoist

Source: JICA Survey Team

6.3.2 Selection of gate type

(1) Type of gate leaf

The fixed wheel gate is proposed because of its plate girder structure or box (shell) girder structure. The relationships between gate span and gate height as shown in the Figureure (Relation of Gate Dimensions and Structure) below:



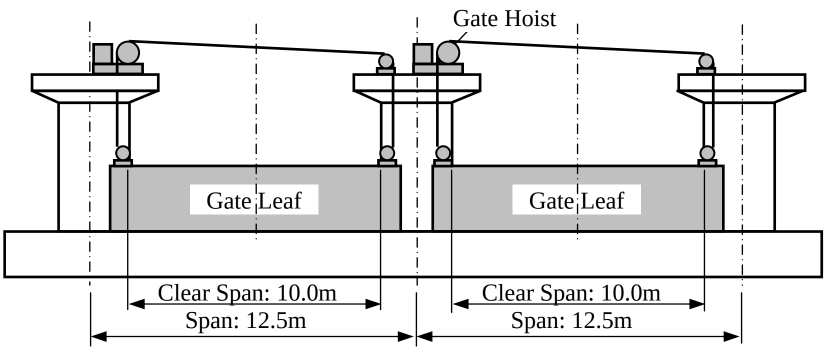
Source: JICA Survey

Figure 6.3.1 Relation of Gate Dimensions and Structure

The plate girder structure type is widely used for up to 30 m span gate because of simple and durable construction and easy maintenance. The box girder structure type is used for the gate in case the ratio of gate height and clear span (i) is less than one-fifth (1/5) and clear span is more than 20 m from the construction point of view. Since the ratio (i) of downstream flood gate is 1/2.78, both of the gates can be fabricated by the plate girder structure type. Therefore, it is decided from the fabrication and maintenance points of view that the plate girder type is used for the flood gates. The plate girder type has been generally manufactured in Brazil and the box girder type is not used in Brazil according to information of the gate manufacturer (HISA). Accordingly, the type of gate leaf for flood gates is the plate girder structure type.

(2) Clear span

The "Clear span" and "Span" are different each other. The "Span" is the distance between centers of the gatepost, and the "Clear span" is width of the waterway as shown in the Figureure below.



Source: JICA Survey Team

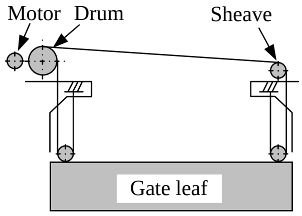
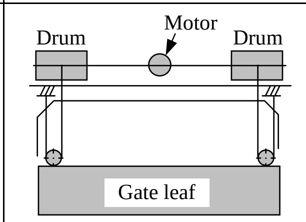
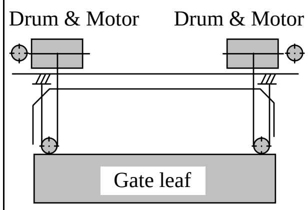
Figure 6.3.2 Clear Span and Span of Gate

(3) Type of hoist

The flood gates are operated by the stationary wire rope winch hoist. There are three types in the stationary wire rope winch hoist, that is, 1M-1D (1 motor-1 drum), 1M-2D and 2M-2D. 2M-2D is not applied to the hoist of flood gates as shown in the table below. The 1M-2D is a slightly expensive compared with 1M-1D because there are many component parts more than 1M-1D.

Accordingly, the 1 motor – 1 drum wire rope winch hoist was selected for operation of the flood gates in due consideration of the applicable span, simple construction, reliable operation and convenience of maintenance.

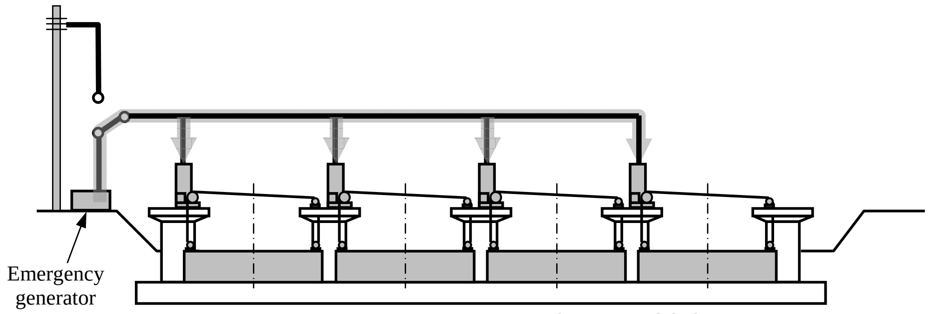
Table 6.3.2 Type of Hoist

Type	**1M-1D**	**1M-2D**	**2M-2D**
Applied clear span	10m ～ 30m	5m ～ 15m	20m ～
Layout			
	Main machine is arranged on the one gatepost and only a rope terminal and a fixed sheave are arranged on the other side. Each one set of motor and drum are provided.	Drums on both gateposts are connected with the shaft. Main machine is arranged at the center of hoist or on the one gatepost.	Main machine and the drum are arranged on both gateposts. The lifting speed shall be electrically synchronized. This hoist is applied to wide span gate.

Source: JICA Survey Team

(4) Power supply system

It is necessary to provide the stand-by (emergency) generator for the power supply of the gate operation when the permanent electricity is cut off.



Source: JICA Survey Team

Figure 6.3.3 Power Supply System

6.3.3 Selection of corrosion protection measure

The flood gates will be constructed in the tidal area of lower Itajai River. Though the flood gates will be kept at the fully opened position under the dry condition, the gate leaf will be rusted by seawater. The corrosion protection is absolutely necessary to the gate leaf. For this purpose, following two (2) measures are conceivable.

Use of stainless steel material

Painting

The unit price of a stainless steel material is very expensive compared with the mild steel as listed in the Table 8.3.3 and the stainless steel has not been used for the gate structure in Brazil so far. Accordingly, the flood gate is to be fabricated by the mild steel and the painting shall be applied on the gate leaf as the corrosion protection.

Table 6.3.3 Unit Price of Steel Material

Material	Mild steel A36 (ASTM) (equal to SS400 of JIS)	Stainless steel S30400 (ASTM) (equal to SUS304 of JIS)
In Brazil	R$ 2.5/kg	R$ 15.0/kg
In Japan	R$ 2.3/kg	R$ 9.5/kg

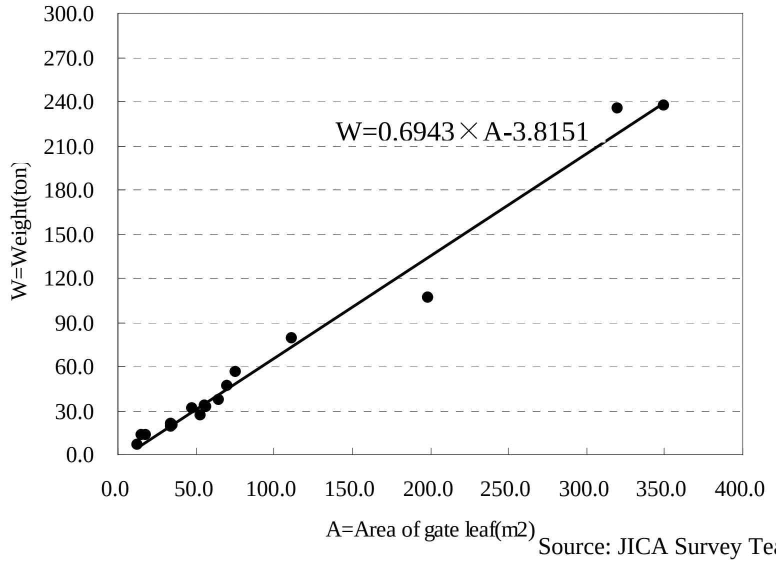
Notes: The unit price in Brazil depends on the HISA hearing survey (May, 2011).

Source: JICA Survey Team

6.3.4 Estimation of design loads

(1) Weight of gate leaf

The gate weight is in proportion to the gate leaf area. The relation between the gate weight and gate leaf area in Japan is as shown below:



Source: JICA Survey Team

Figure 6.3.4 Relation between Gate Weight and Gate Leaf Area

The gate weight is calculated by the following formula:

$W= 0.6943 \times A+3.8151$

Where, W: Weight of gate leaf (ton)

A: Area of Gate Leaf (m2)

The weights of both gate leaves are as listed in the table below.

Table 6.3.4 Weight of Gate Leaves

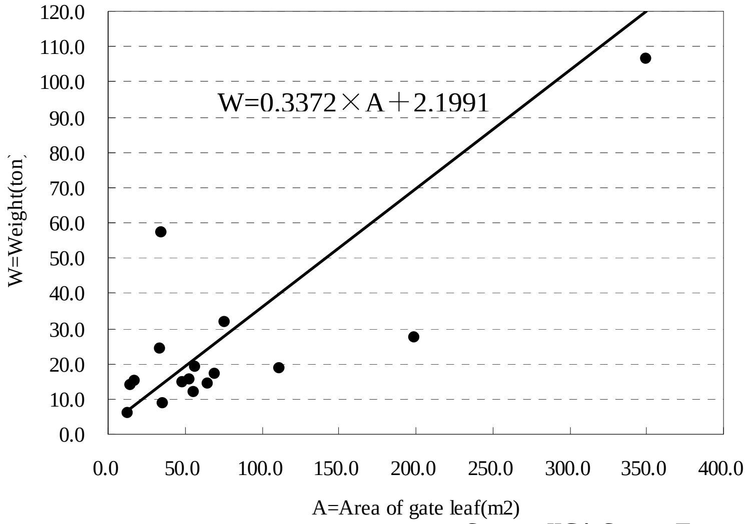
Gate	Clear span (m)	Gate height (m)*	Area (m2)	Weight (ton)	Weight (kN)
Upstream flood gate	10.0	5.5	55.0	42.0	412.1
Downstream flood gate	10.0	3.6	36.0	28.8	282.6

Notes; Gate height is for the 50-year flood.

Source: JICA Survey Team

(2) Weight of hoist

The weight of wire rope winch hoist is also in proportion to the gate leaf area. The relation between the hoist weight and gate leaf area in Japan is as shown in the Figureure below:



Source: JICA Survey Team

Figure 6.3.5 Relation between Hoist Weight and Gate Leaf Area

The hoist weight is calculated by the following formula:

$W= (0.3372 \times A+2.1991) \times 1.10$

Where, W: Weight of hoist (ton)

A: Area of Gate Leaf (m2)

The weights of both hoists are listed in the table below: The weight of operation panel is expected by 10 %.

Table 6.3.5 Weight of Hoists

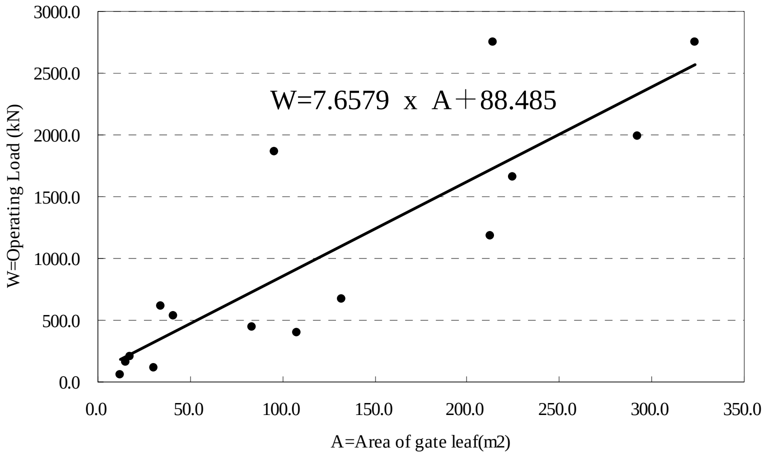
Gate	Clear span (m)	Gate Height (m)*	Area (m2)	Weight (ton)	Weight (kN)
Upstream flood gate	10.0	5.5	55.0	22.8	223.7
Downstream flood gate	10.0	3.6	36.0	15.8	155.0

Notes; Gate height is for the 50-year flood.

Source: JICA Survey Team

(3) Operating load

The relation between the operating load and gate leaf area in Japan is as shown in the Figureure below:



Source: JICA Survey Team

Figure 6.3.6 Relation between Operating Load and Gate Leaf Area

The operating load is calculated by the following formula:

W= 7.6579 x A ＋ 88.485

Where, W: Operating load (kN)

A: Area of Gate Leaf (m2)

The operating loads of both gates are listed in the table below:

Table 6.3.6 Operating Loads

Gate	Clear span (m)	Gate Height (m)*	Area (m2)	Operating load (kN)
Upstream flood gate	10.0	5.5	55.0	509.7
Downstream flood gate	10.0	3.6	36.0	364.2

Notes; Gate height is for the 50-year flood

Source: JICA Survey Team

(4) Hydraulic pressure load

The hydraulic pressure load (WG4) is calculated by the following formula.

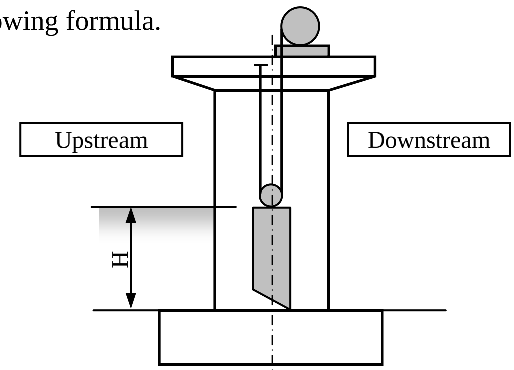
$$WG4 = \frac{1}{2} \times (H^2 \times w_0) \times B$$

Where, WG4: Hydraulic pressure load (kN)

H: Design head (m)

W0: Specific gravity of water (kN/m3)

B: Sealing span (m)



Side View

Source: JICA Survey Team

The hydraulic pressure loads “WG4” are listed in the table below:

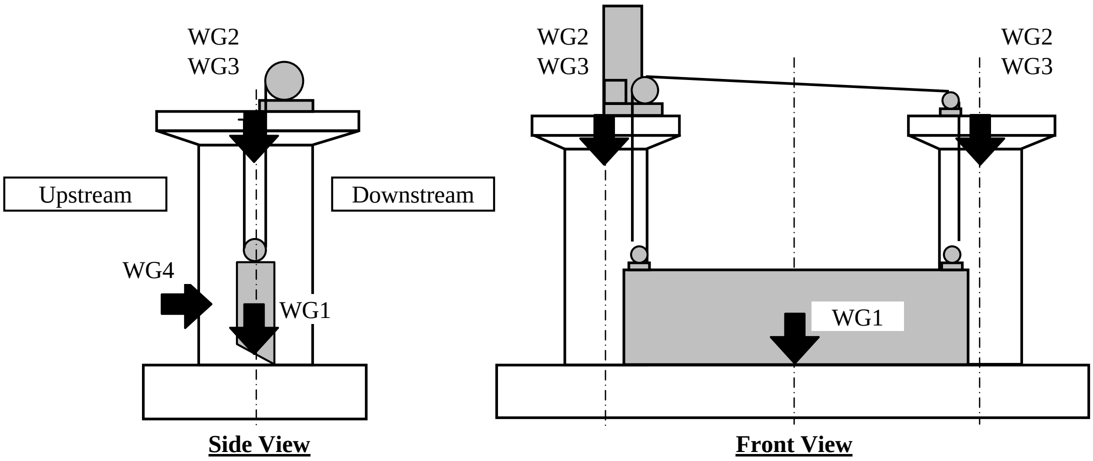
Table 6.3.7 Hydraulic Pressure Load

Gate	H(m)*	B(m)	W0(kN/m3)	WG4(kN)
Upstream flood gate	5.5	12.5	10.101	1909.7
Downstream flood gate	3.6	12.5	10.101	818.2

Notes; Gate height is for the 50-year flood
Source: JICA Survey Team

(5) Design loads

The design loads illustrated below are listed in the Table 8.3.8. The loads of “WG2” and “WG3” act on the gatepost in one side, i.e., 2 x (“WG2”+“WG3”) act on a gatepost.



Source: JICA Survey Team

Figure 6.3.7 Design Loads

Table 6.3.8 Design Loads

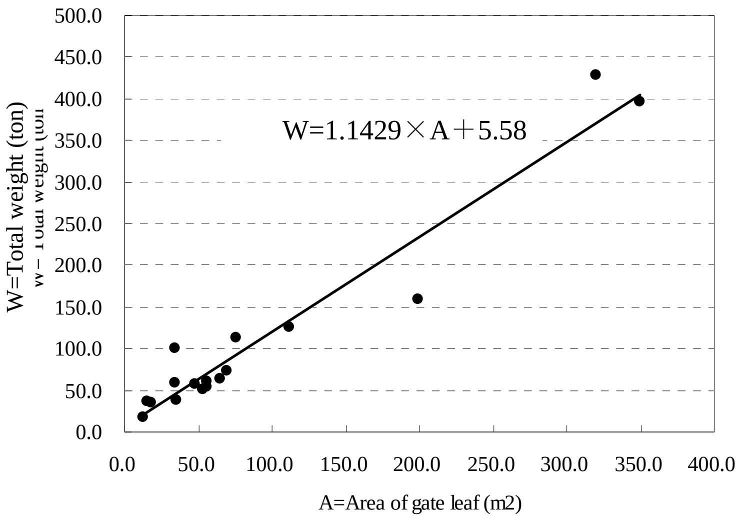
Gate	**WG1 (kN)**	**WG2 (kN)**	**WG3 (kN)**	**WG4 (kN)**
Upstream flood gate	412.1	111.9	254.9	1909.7
Downstream flood gate	282.6	77.5	182.1	818.2

Notes; WG1: Weight of Gate Leaf, WG2: Weight of Hoist, WG3: Operating Load, WG4: Hydraulic Pressure Load

Source: JICA Survey Team

6.3.5 Cost Estimate

The cost of gates is estimated from the total weight and the unit price. The cost of gate contains the costs of the design, manufacturing, installation, and inspection. The total weight of gate was estimated from the relationship between the weight and its area of various gates in Japan Figure 6.3.8 below.



Source: JICA Survey Team

Figure 6.3.8 Relation between Total Weight of Gate and Gate Leaf Area

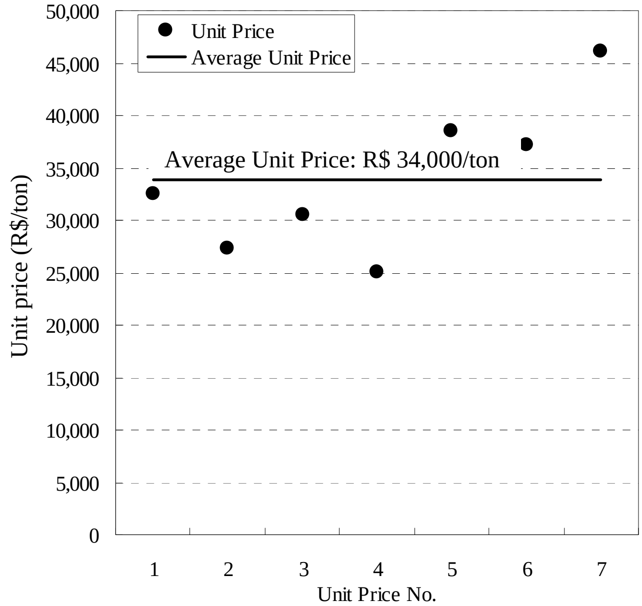
The total weight of gate is estimated by the following formula:

$W = 1.1429 \times A + 5.58$

where, W: Total weight of gate (ton)

A: Area of gate leaf (m2)

The unit price of gate is estimated based on the actual bid prices of manufactures in Brazil. Figure 6.3.9 shows the comparison of bid prices. The unit price for cost estimate under this feasibility study is determined R$40,800 per ton by adding 20% to the average bid price, considering the unit price widely applied in Japan.



Source: JICA Survey Team

Figure 6.3.9 Unit price results

Table 6.3.9 Cost Estimate of Flood Gates

Gate	Clear Span (m)	Gate Height (m)*	Gate Area (m^2)	Quantity (unit)	Weight (ton)	Unit Price (R$/ton)	Cost (R$)
Upstream flood gate	10.0	4.5	45.0	4	228.1	40,800 (=Ave.34,000×1.20)	9,306,480
Downstream flood gate	10.0	3.6	36.0	4	186.9		7,625,520

Notes; Gate height is for the 10-year flood.
Source: JICA Survey Team

CHAPTER 7 CONSTRUCTION PLAN AND COST ESTIMATES

7.1 Introduction

Construction plan for the selected priority projects was performed to formulate the construction time schedule and to obtain the basic data for the cost estimates.

The main study items are as follows:

(1) Construction Plan

- To formulate the basic conditions for construction plan, workable days and materials.
- To select the standard construction method for major works.
- To formulate the construction time schedule.

(2) Cost Estimates

To review the basic concepts for the cost estimates and unit costs.

- To estimate the financial and economic project costs of the selected priority projects.

7.2 Construction Plan

This chapter is to support, on the construction plan, feasibility study of the main reports.

7.2.1 Outline of Project

(1) Implementation schedule

According to the feasibility study, there are five(5) projects as follows.

- Heightening of the Oeste dam
- Heightening of the Sul dam spillway
- Upstream floodgate in the Mirim River
- Downstream floodgate in the Mirim River
- Mirim Concrete sheet pile revetment

(2) Work quantities

The work quantity of five (5) projects is summarized as shown in the table below.

Table 7.2.1 Summary of Quantities list

Location	Quantities	Remarks
Oeste dam	concrete : 12,500 m^3 excavation sand : 20,000 m^3 excavation rock : 1,650 m^3	
Sul dam spillway	concrete : 2,700 m^3 Demolish : 800 m^3	
Mirim downstream water gate	concrete : 1,300 m^3 excavation sand : 3,600 m^3 precast concrete pile : 130 nos steel sheet pile : 110 sheet gate : 140 t	
Mirim concrete sheet pile revetment	concrete sheet pile : 5,400 m^2 rubble mound : 10,400 m^3	
Mirim upstream water gate	concrete : 2,200 m^3 excavation sand : 4,800 m^3 embankment : 7,400 m^3 precast concrete pile : 160 nos steel sheet pile : 243 sheet tributary switching channel : 1,060 m drainage channel : 2,000 m x3 place gate : 170t	

Source: JICA survey team

7.2.2 Basic condition

(1) Workable day

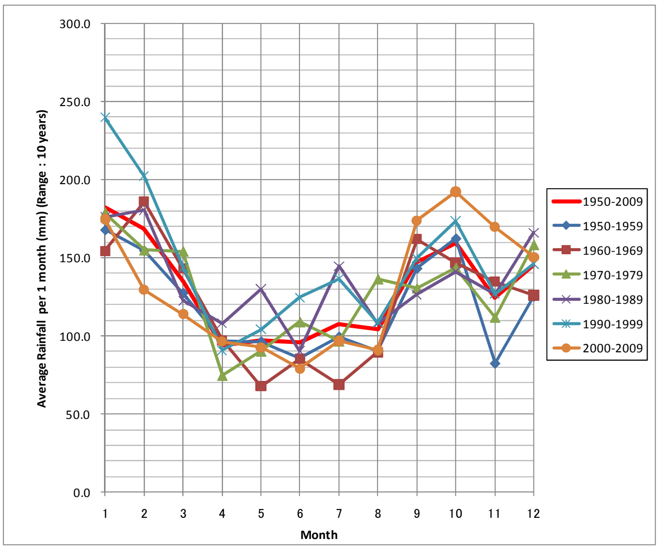
In Brasil, working hours are 44 hours per week and the typical working hours are eight(8) hours. And holidays and weekends are not included. Working days per one month are 20 days calculated by using the equation below, considering 3 days off such as rainy days.

$$d = \frac{\text{44 hours per 1 week}}{\text{8 hours per 1 day}} = \frac{30}{7} \text{ - 3 days (rainy day) = 20 day per 1 month.}$$

The following figure is about the average monthly rainfall of 59 years data. In Santa Catarina state, there is not a clear border between the rainy-season and the dry-season. However according to the following figure, the six(6) months duration from March to September is considered to be the dry-season.

Since the construction of the dam heightening has more risk to encounter floods, the construction must be held during the dry season. Conversely, the construction of floodgates has less risk to encounter floods, and the only obstacle to the construction is the tide. Thus, the construction can be implemented thought a whole year.

- Dry season: May to August (6 months)
- Rainy season: January to March, September to December (6 months)



Source: JICA survey team

Figure 7.2.1 Monthly Average Rainfall

(2) Construction Materials

All materials are available in Brazil.

7.2.3 Standard Construction method

(1) Heightening of the Oeste dam.

The construction of the heightening of the Oestes dam requires to careful consideration to these points.

- Even during the construction duration, in order not to lose the function of flood control of dam, the temporary diversion facility is installed.
- Considering the risk of delay in construction schedule, the height of the installing cofferdam is calculated with consideration of the no overflow water level with the conduit discharge.

1) Temporary diversion facility

The temporary diversion facility is installed so as not to lose the function of flood control.

- Design discharge for the temporary diversion facility

Design discharge equals to the discharge from conduit when the water level equals to the elevation of the dam crest.

The design discharge is estimated by the formula below.

$$Q = 0.667 \times 7 \times 1.7663 \times \sqrt{2 \cdot g \cdot (H - 340.05)}$$
$$= 0.667 \times 7 \times 1.7663 \times \sqrt{2 \cdot g \cdot (360.0 - 340.05)} = 163 m^3 / s$$

where

H : *spillway elevation* ($EL.360.00m$)

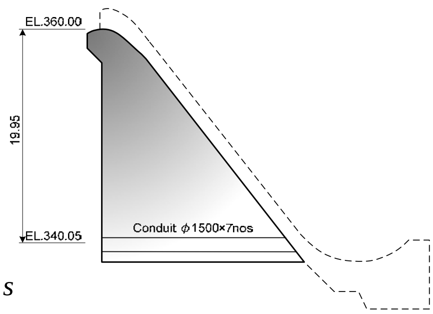
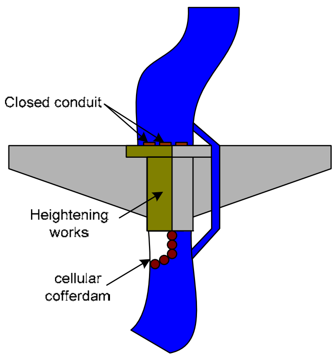


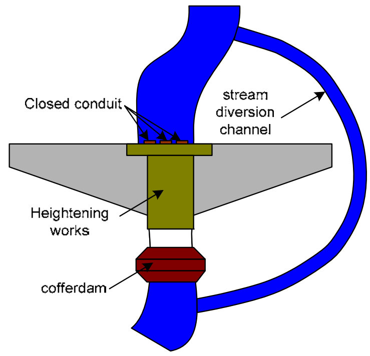
Figure 7.2.2 Image of Calculation of Design Discharge

2) Method of Temporary Diversion Facility

Two methods of the temporary diversion facility are considered: multiple-stage diversion and diversion tunnel. As showing in Table 7.2.2, the tunnel method requires more time and expense. Thus, the multiple-stage diversion method is selected.

Table 7.2.2 Method of Temporary Diversion Facility

<table>
<tr><th></th><th>Multiple-stage Diversion</th><th>Diversion Tunnel</th></tr>
<tr><td>Outline</td><td>

The construction work space is divided two parts alternately.</td><td>

Make the tunnel as much size as the conduit. In construction duration, the tunnel is used as water path.</td></tr>
<tr><td>Dimension</td><td>cellular cofferdam φ8.5,h=8.5
x 3set x 2time
φ6.0,h=6.0
x9 set x 2time
stream diversion channel
B=12mx3m</td><td>horse shaped tunnel
φ6.0m, i=1/200, L=200m</td></tr>
<tr><td>Construction term</td><td>short</td><td>long</td></tr>
<tr><td>Construction cost</td><td>R$2.9×10⁶</td><td>R$7.7x10⁶</td></tr>
<tr><td>Adjudication</td><td>good</td><td>---</td></tr>
</table>

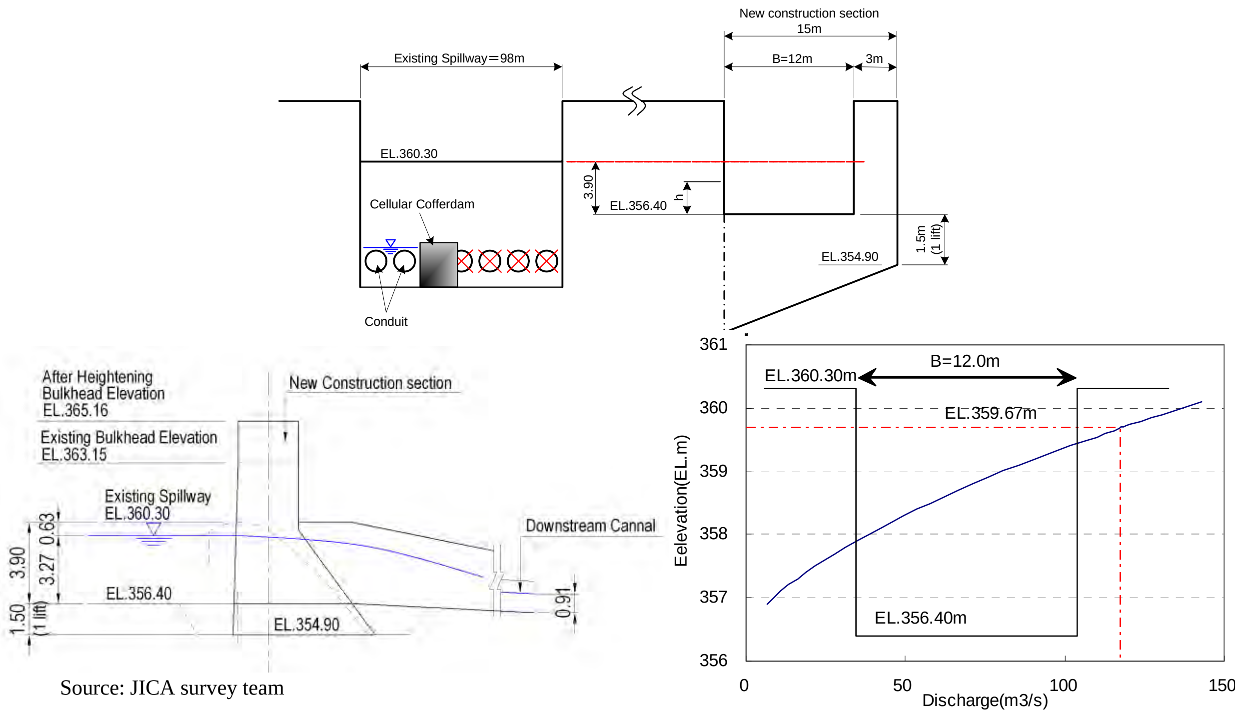
Source: JICA survey team

3) Scale of Cutting Area of Dam Body

The construction with multiple-stage diversion method disables the original function of conduit discharge, so that the alternative facility requires to compensate the discharge. As shown in the figure below, Two portions are excavated in the wing part of the dam body in order to dlow discharge when the water level is under the crest of the dam body. There are at least two(2) conduits when the multiple-stage diversion is applied. Thus the wing part covers the discharge = 117 m^3/s.

$$Q = 163 - \frac{163}{7} \times 2 = 116.4 \Rightarrow 117 m^3 / s$$

The scale of excavation is 12.0 m wide and 3.27 m high based on calculation with the formula of Rectangular-weir. The bottom of the excavation is EL.356.4 m, which is 1.5 m higher than the height of the dam and is shorter than the overflow depth.



Source: JICA survey team

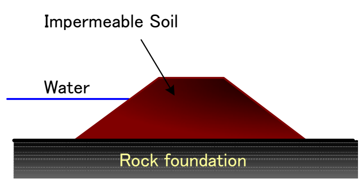
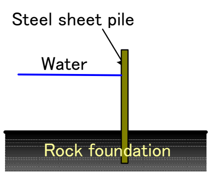
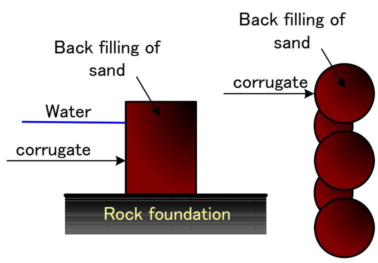
Figure 7.2.3 Scale of Excavation of Wing of Dam Body

4) Type of Cofferdam

Due to following reasons, the cellular dam is proposed as a type of cofferdam at the downstream in the Oeste dam. The Table 7.2.2 shows as the image of the type of cofferdam.

- Normal water lever is almost 5.0 m and the typical cofferdam (earth type) is big earth work and also the deteriorate flow capacity
- The flow velocity from the spillway is high. Thus the cofferdam is required to be a hard structure.
- The foundation is bedrock so it is difficult to place the sheet pile.

Table 7.2.3 Type of Cofferdam

<table>
<tr><th></th><th>Earth Type</th><th>Steel Sheet Pile Type</th><th>Cellular Cofferdam Type</th></tr>
<tr><td>Figure</td><td>
</td><td>
</td><td>
</td></tr>
</table>

Source: JICA survey team

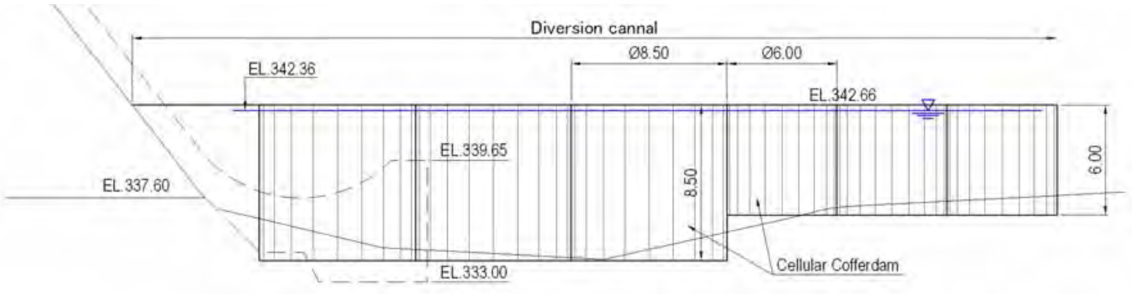
The following figure shows an example of the cellular cofferdam under construction.

Source: MLIT tsugaru dam construction work office

Figure 7.2.4 Example of Construction Cellular Cofferdam

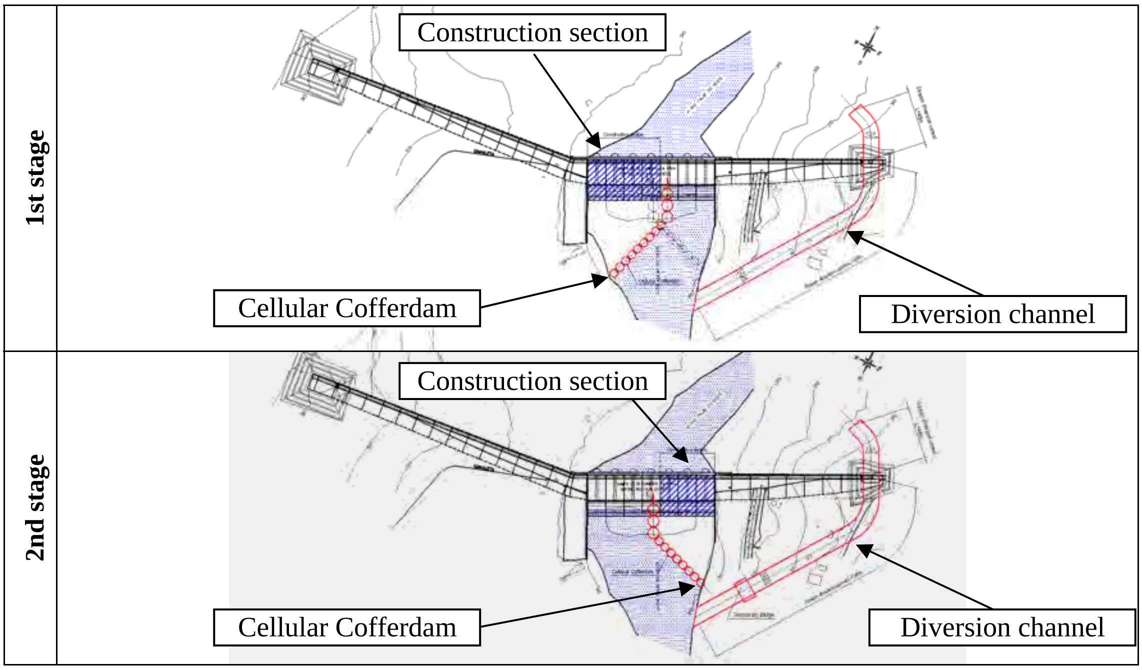
5) Design of cofferdam

The water level at the design discharge 163 m^3/s is EL. 343.36 m based on calculation. Considering 30 cm as freeboard, the top elevation of cellular cofferdam is EL. 343.66 m. The scale of cellular cofferdam are ϕ 8.5x8.5-3nos and ϕ 6.0x6.0-9nos. The figure below shows the layout and the section.



Source: JICA survey team

Figure 7.2.5 Typical Section of Cellular Cofferdam

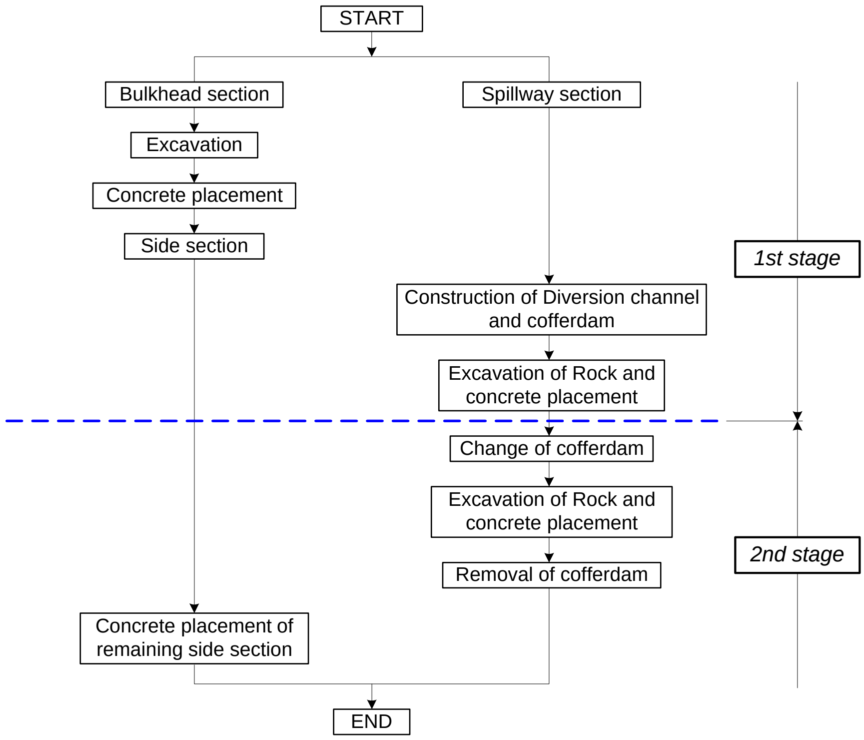


Source: JICA survey team

Figure 7.2.6 General Plan of Multiple-stage Diversion Method

2) Procedure and area of Construction

The Procedure is as follows.



Source: JICA survey team

Figure 7.2.7 Heightening of the Oeste dam Construction Flow

Figure 7.2.8 below illustrated as the area of countermeasure.

- Non-overflow section and spillway are heightened by 2.0 m

- Spillyway is designed as widening.

- The wing part is designed to extend 15 m and 20 m

Source: JICA survey team

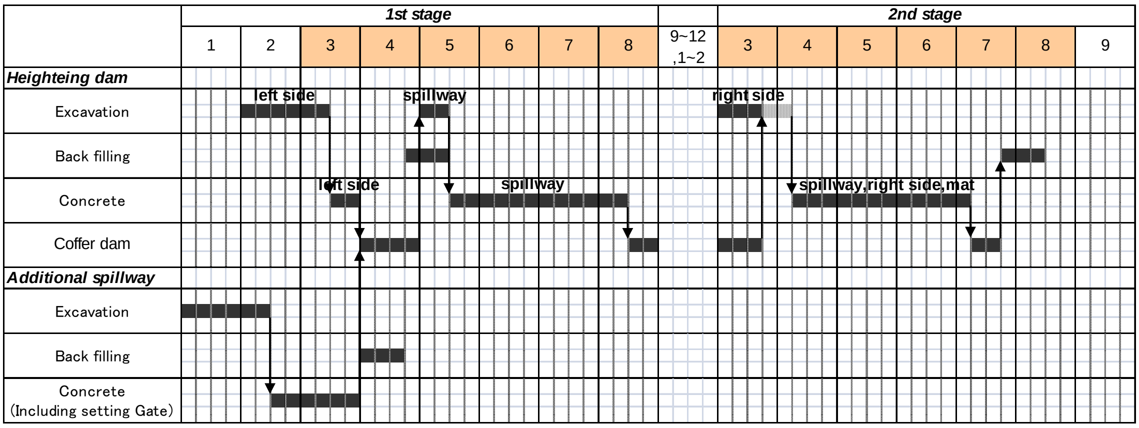
Figure 7.2.8 Scope of Construction Work

3) Construction schedule

The approximate schedule of the Oeste dam construction is as follows.

1st stage : 7 months (including rainy season 1month)

2nd stage : 6 months



Source: JICA survey team

Figure 7.2.9 Construction Schedule

Table 7.2.4 Operation Capability

		unit	[1] quantity	[2] capacity		[3]workable days	[4] month [3]/20	Remarks
[Left side]								
excavation	soil	m3	13,300	220	×2 set	30.2	1.5	backhoe-0.8m3
	rock	m3	825	63	×2 set	6.5	0.3	excavator(breaker)
backfilling		m3	5,200	410	×1 set	12.7	0.6	bulldozer
concrete	bulkhead	m3	4lift	---	---	---	0.5	interval is 5days
	spillway		18lift	---	---	---	3.0	interval is 5days
[Right side]								
excavation	soil	m3	6,700	220	×2 set	15.2	0.8	backhoe-0.8m3
	rock	m3	825	63	×2 set	6.5	0.3	excavator(breaker)
backfilling		m3	10,000	410	×2 set	12.2	0.6	bulldozer
concrete	bulkhead(right)	m3	12lift	---	---	---	2.0	interval is 5days
	bulkhead(left)	m3	6lift	---	---	---	1.0	interval is 5days
	spillway		18lift	---	---	---	3.0	interval is 5days

Source: JICA survey team

(2) Heightening of the Sul dam spillway

Due to following reasons, the construction of the Sul dam does not require the temporary diversion facility.

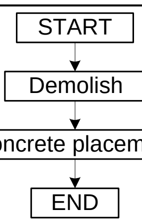
- Compared with the Oeste dam, the capacity of conduit discharge is not changed.
- The construction term is short and the only concrete material is need to be done. Thus there is little risk of flood.

1) Procedure and Area of Construction

The procedure is shown as follows.

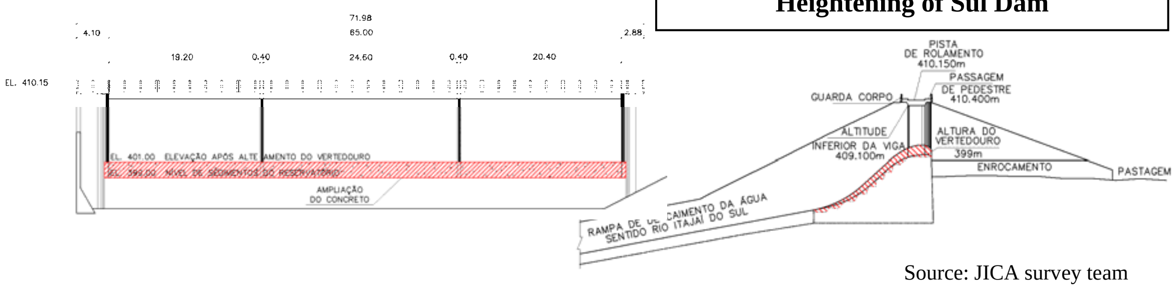
The reinforcing area is shown below.

- Spillway Section: Heightening by 2.0 m and widening to downstream.



Source: JICA survey team

Figure 7.2.10 Construction Flow of Heightening of Sul Dam



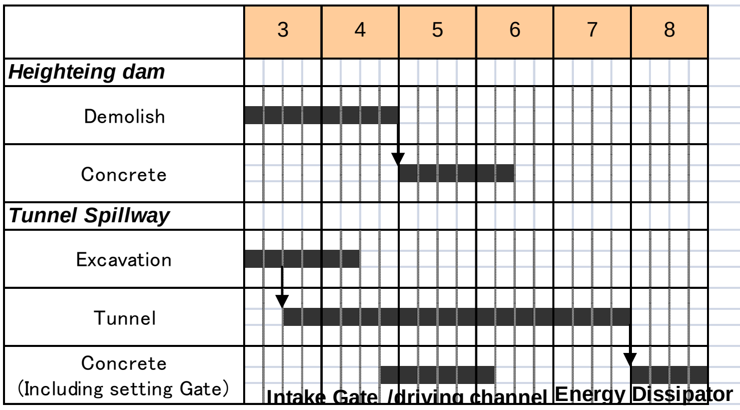
Source: JICA survey team

Figure 7.2.11 Scope of construction work

3) Construction schedule

The approximate schedule of the Sul dam construction is shown below.

1st stage : 3.5 month



Source: JICA survey team

Figure 7.2.12 Construction schedule

Table 7.2.5 operation capability

	unit	[1] quantity	[2] capacity	[3]workable days	[4] month [3]/20	Remarks
demolish	m3	800	4 ×5 set	40.0	2.0	concret breaker
concrete	m3	9lift	--- ---	---	1.4	interval is 5days

Source: JICA survey team

(3) Downstream Mirim Gate and Concrete Sheet pile revetment

1) Water level

The floodgate at the Mirim River is normally to get affected by tides. The water level in 10-year probable flood at this site is summarized as below.

- High tide water level : EL. 1.49 m
- Low tide water level : EL. 0.00 m
- Water level in 10-year probable flood : EL. 2.16 m

2) Setting of coffer dam

The construction of floodgate at downstream and concrete sheet pile revetment starts after the construction of upstream floodgate.

The flood capacity of the Old Mirim River is relatively small -- 50 m^3/s. Thus with or without cofferdam, it is likely to be inundated. The cofferdam of the floodgate at downstream closes at all sections. Thus the construction is implemented all the time.

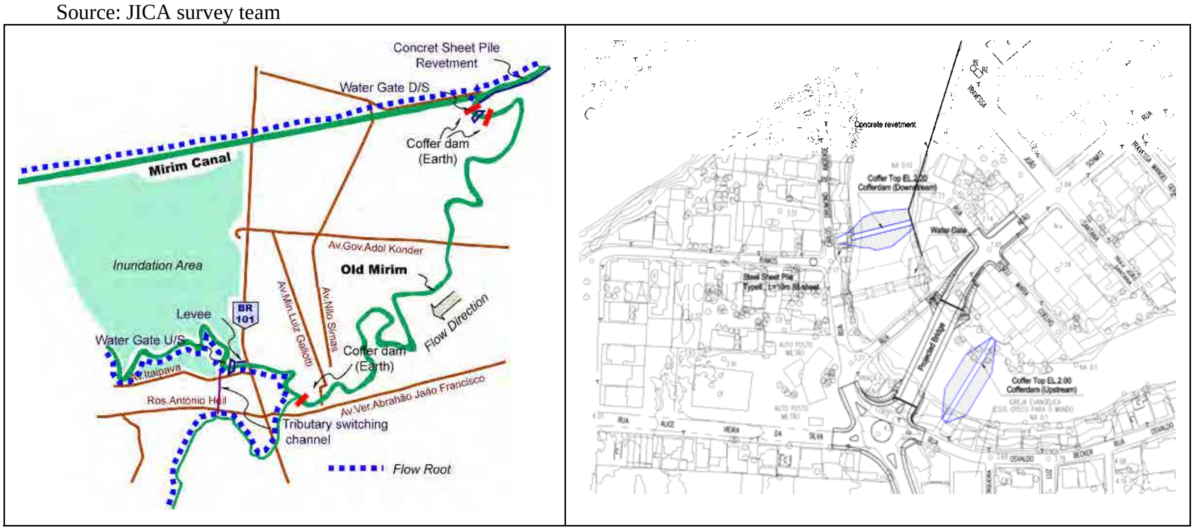
- The elevation of cofferdam height is set at the water level, which is less than 10-year flood at the Itajai River.
- The drainage of the runoff of original catchment area is turned to the upstream gate side.
- The tributary of the Old Mirim River is turned to the upstream gate side through a tunnel.

The height of cofferdam is summarized as below.

Table 7.2.6 Height of Cofferdam

	Top Elevation of Cofferdam	Remarks
Downstream of Floodgate	EL.2.20	Itajai River 10-year flood
Upstream of Floodgate	EL.2.00	Minimum Ground Elevation of Old Mirim zone
Diversion of Tributary River	EL.2.00	Minimum Ground Elevation of Old Mirim zone

Source: JICA survey team

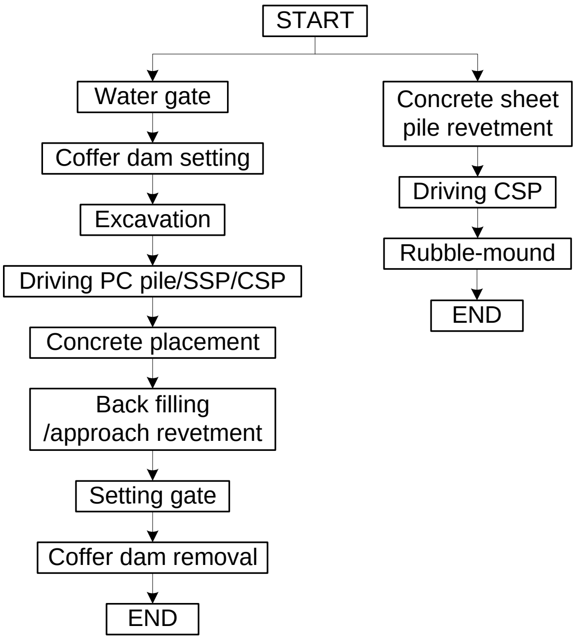


Source: JICA survey team

Figure 7.2.13 Location of Cofferdam

3) Procedure and Area of Construction

The next figure show the procedure of construction



Note: PC pile: precast concrete pile
SSP : steel sheet pile
CSP : concrete sheet pile

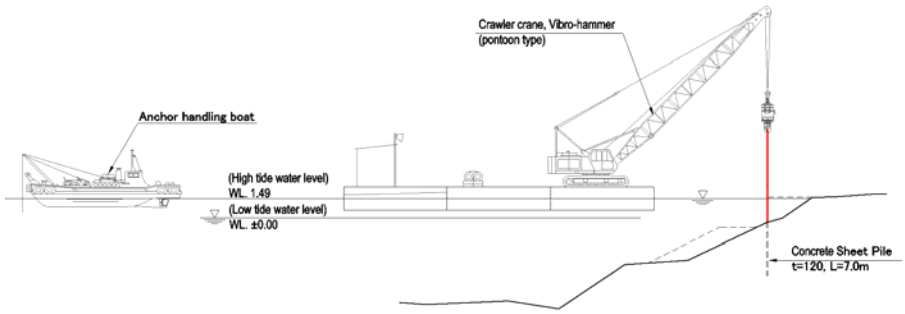
Source: JICA survey team

Figure 7.2.14 Construction Flow of Downstream Floodgate

4) Method of Construction

The construction of floodgate is implemented during the dry season. The construction of concrete sheet pile revetment is implemented on pontoons from the river side since the construction site is near the residential area.

The image of the construction is illustrated in the figure below.

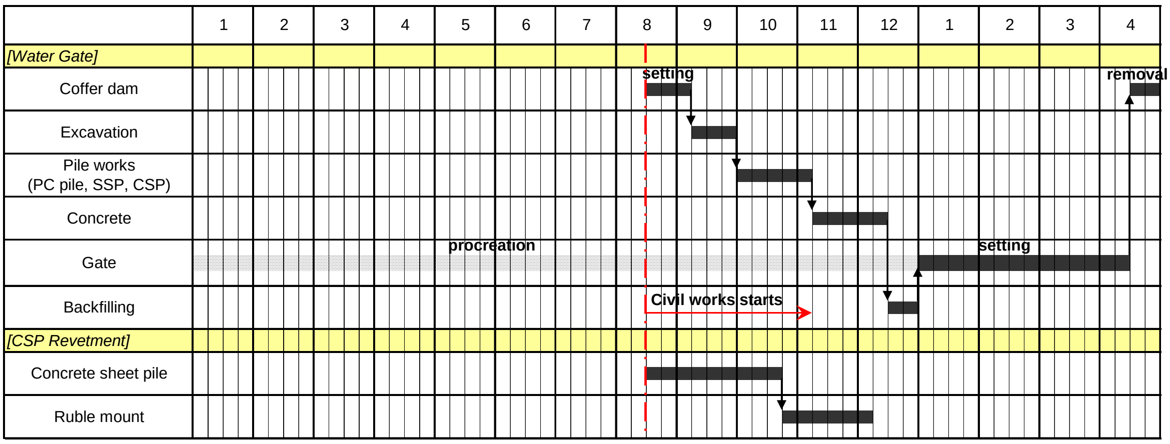


Source: JICA survey team

Figure 7.2.15 Working diagram (driving of concrete sheet pile on pontoon)

5) Construction Schedule

The schedule of construction of the floodgate including the making gate is 16 months. The approximate schedule is shown below.



Source: JICA survey team

Figure 7.2.16 Construction Schedule

Table 7.2.7 Operation Capability

		unit	[1] quantity	[2] capacity		[3]workable days	[4] month [3]/20	Remarks
[Water Gate]								
coffer dam	setting	m3	6,100	220	×2 set	13.9	0.7	backhoe-0.8m3
	removal	m3	6,100	260	×2 set	11.7	0.6	clasmshell-0.8m3
excavation	soil	m3	3,600	220	×1 set	16.4	0.8	backhoe-0.8m3
PC pile	ϕ300,400	nos	130	6.1	×1 set	21.3	1.1	driving
SSP	type2,L=2m	sheet	110	56	×1 set	2.0	0.1	driving
CSP	L=10m	sheet	80	29	×1 set	2.8	0.1	driving
backfilling		m3	650	61	×1 set	10.7	0.5	tamping machine
concrete		m3	8lift	---	---	---	1.2	interval is 5days
gate, setting		---	---	---	---	---	4.0	
gate, procreation		---	---	---	---	---	12.0	
[CSP Revetment]								
CSP	L=7m	m3	1,500	35	×1 set	42.9	2.1	driving
Rubble mount		m3	2,800	76	×1 set	36.8	1.8	backhoe-0.8m3

Source: JICA survey team

(4) Upstream Mirim floodgate and levee

1) Water level

The upstream floodgate is easy to get affected by tides. The tide condition is as below.

- High tide water level : EL. 1.49
- Low tide water level : EL. ±0.00

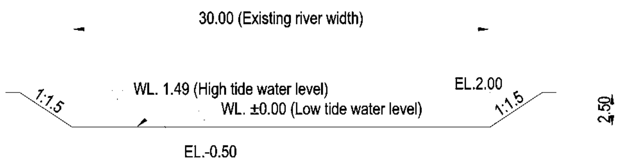
2) Setting of diversion cannel / coffer dam

The upstream floodgate is equipped with a diversion channel and all section closed. Thus the construction is implemented thought a whole time. The design size of diversion channel and cofferdam is summarized in the table below.

Table 7.2.8 Diversion Channel and Cofferdam Scale

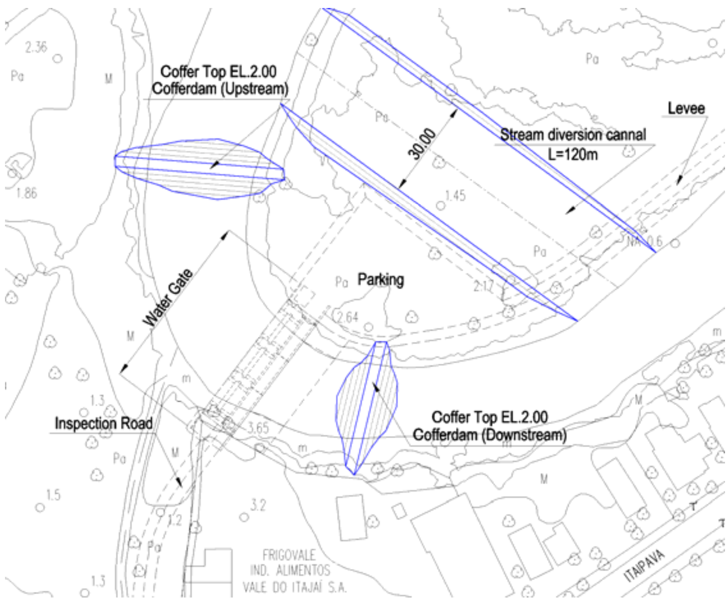
Diversion Channel		**Remarks**
Bottom Elevation	EL.-0.5 m	Low tide water level -0.50
Top Elevation	EL.2.0 m	Present ground elevation
Diversion Channel Width	30.0m	Present river width
Cofferdam		**Remarks**
Elevation of Top	EL.2.0 m	Minimum ground elevation of surrounding land

Source: JICA survey team



Source: JICA survey team

Figure 7.2.17 Section of Diversion Channel

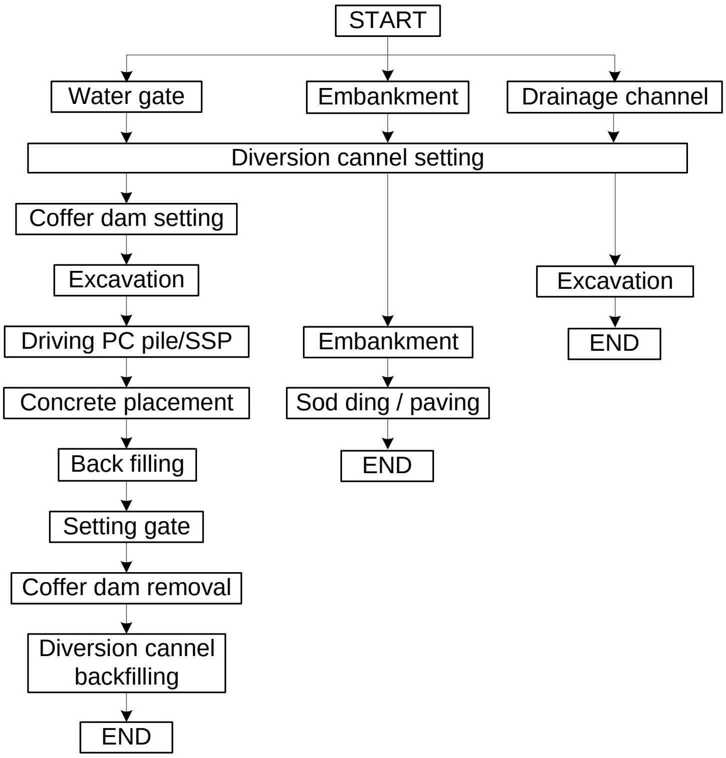


Source: JICA survey team

Figure 7.2.18 Diversion Channel and Cofferdam Location

3) Procedure and Area of Construction

The next figure shows the procedure of construction



Note: PC pile: precast concrete pile
SSP : steel sheet pile

Source: JICA survey team

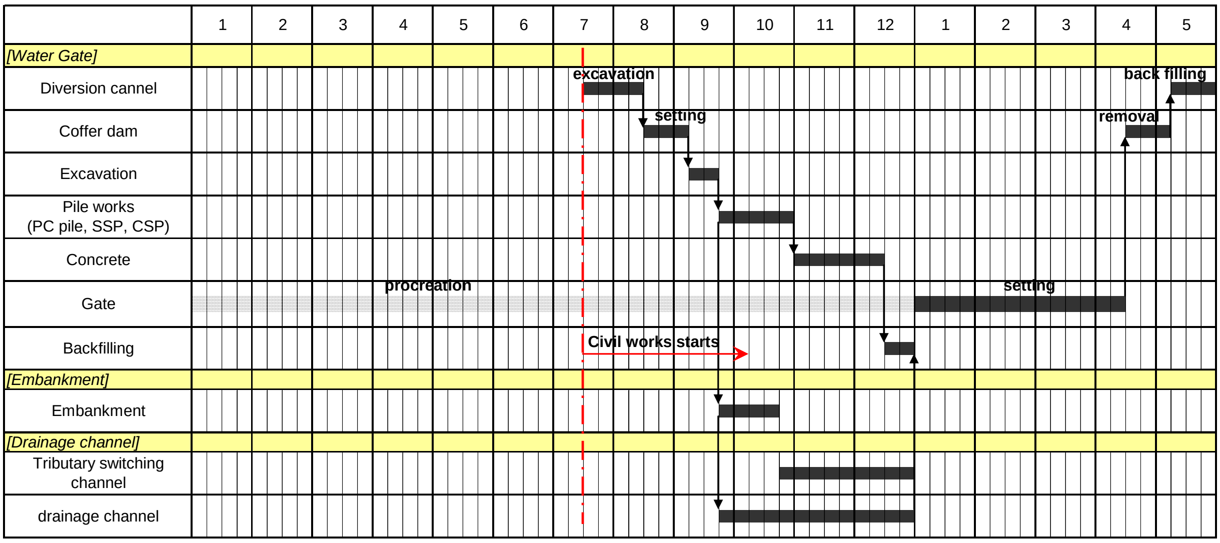
Figure 7.2.19 Construction Flow of Upstream Floodgate

4) The method of construction

The construction of the floodgate is carried out at dry condition.

5) Construction schedule

The schedule of floodgate including the making gate is 17 months. The approximate schedule is shown below.



Source: JICA survey team

Figure 7.2.20 Construction Schedule

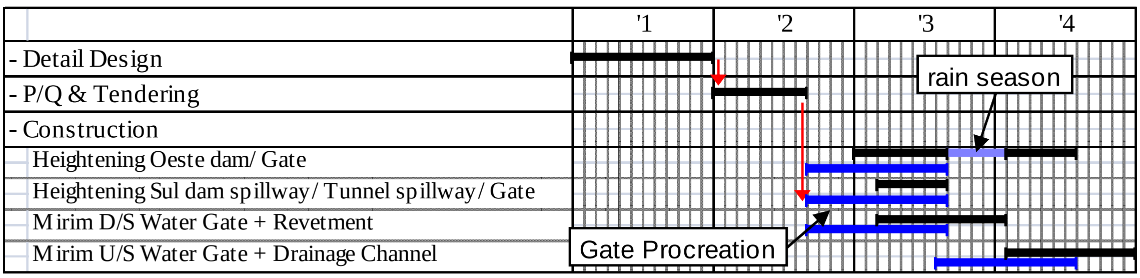
Table 7.2.9 Operation Capability

		unit	[1] quantity	[2] capacity		[3]workable days	[4] month [3]/20	Remarks
[Left side]								
excavation	soil	m3	13,300	220	×2 set	30.2	1.5	backhoe-0.8m3
	rock	m3	825	63	×2 set	6.5	0.3	excavator(breaker)
backfilling		m3	5,200	410	×1 set	12.7	0.6	bulldozer
concrete	bulkhead	-	4lift	---	---	---	0.5	interval is 5days
	spillway	-	18lift	---	---	---	3.0	interval is 5days
[Right side]								
excavation	soil	m3	6,700	220	×2 set	15.2	0.8	backhoe-0.8m3
	rock	m3	825	63	×2 set	6.5	0.3	excavator(breaker)
backfilling		m3	10,000	410	×2 set	12.2	0.6	bulldozer
concrete	bulkhead	-	12lift	---	---	---	2.0	interval is 5days
	spillway	-	18lift	---	---	---	3.0	interval is 5days
[Additional Spillway]								
excavation	soil	m3	39,000	220	×6 set	29.5	1.5	backhoe-0.8m3
backfilling		m3	10,000	410	×2 set	12.2	0.6	bulldozer
concrete		-	---	---	---	---	1.5	interval is 5days

Source: JICA survey team

7.2.4 Project schedule

The project schedule of construction is shown in the figure below. The project duration is 4 year.



Source: JICA survey team

Figure 7.2.21 Project Schedule

7.3 Cost Estimates

7.3.1 Conditions for Cost Estimates

(1) Price level

1) Price level

Price level is set in April 2011.

2) Exchange rate

The following shows exchange rates used for the cost estimates.(4/2011)

i) US$ 1.0 = Y 84.48

ii) US$ 1.0 = R$ 0.617

(Y1.0 = R$52.12)

Where US$: U.S dollar;

Y: Japanese yen; and

R$: Brazil Real

3) Currency of cost estimate

Cost is estimated in Brazil Real.

(2) Cost Component

1) Project cost

The following shows project cost components.

i) Construction cost

ii) Land acquisition and compensation

iii) Government administration cost

iv) Engineering service cost

v) Physical contingency

vi) Price contingency

Note: Tax is included in each cost estimate.

2) Construction cost

Construction cost is estimated under the agreement on the following parts.

i) Cost for major works :to multiply the work quantities by their unit cost,

ii) Cost for other works :30% of the major works, and

iii) Cost for temporary works :to multiply the work quantities by their unit cost, and 20 % (depending on the accuracy of quantification) of the temporary works.

3) Government administration cost

Government administration cost is estimated as below.

Government administration= (Construction cost + Land acquisition and compensation) x 3%

4) Engineering service cost

Engineering service cost is estimated at below.

- Engineering service= Construction cost x 15%~20%

  (Detailed design=5~10%, supervision=10%)

- Dam renewal is estimated 15% to 20% depending on the situation.

- Other works is estimated 15%.

5) Physical contingency

Physical contingency is estimated at 10% of the total construction cost including the administration and engineering service cost, the land acquisition, and compensation, respectively.

6) Price contingency

Price contingency is estimated at 5% of the total construction cost including the administration and engineering service cost, the land acquisition, compensation, and physical contingency respectively.

7.3.2 Work Quantities

(1) Heightening of Dam

Major work quantities of heightening of dam are summarized as shown in Table 7.3.1 below.

Table 7.3.1 Summary of Heightening of Dam Quantities

(unit:R$)

		Unit	Oeste dam	Sul dam spillway
			Quantity	Quantity
Earth Works				
Excavation (Sand)	(DMT up to 5km)	m3	59,000	4,400
Excavation (Rock)	(DMT up to 5km)	m3	1,650	500
Back Filling, Selected Materials	(DMT up to 5km)	m3	25,000	---
Embankment, Selected Materials	(DMT up to 5km)	m3	---	---
Concrete Works				
Concrete (including Batcher plant,Scaffold, etc)	fck=16Mpa	m3	12,500	---
Concrete (including Form, Scaffold, etc)	fck=25Mpa	m3	3,500	4,050
Reinforcement - deformed bar		t	140	70
Demolishing of Existing Concrete Structure	(DMT up to 5km)	m3	250	800
Consolidation Grout		m	380	---
Substructure Work				
Driving and Furnishing Steel Sheet Pile Type II	L=2.0m	sheet	---	---
Driving and Furnishing Steel Sheet Pile Type II	L=2.5m	sheet	---	---
Driving and Furnishing Steel Sheet Pile Type II	L=5.5m	sheet	---	---
Driving and Furnishing Precast Pc Pile	f400,L=10.0m	nos	---	---
Driving and Furnishing Precast Pc Pile	f300,L=11.0m	nos	---	---
Driving and Furnishing Precast Pc Pile	f400,L=27.0m	nos	---	---
Driving and Furnishing Precast Pc Pile	f300,L=27.0m	nos	---	---
Concrete Block (Production, Installation cost)	w=0.5t/m2	m2	---	---
Revetment Works				
Driving and Furnishing Concrete Sheet Pile	T=120,B=500	m2	---	---
(Including head cover)				
Driving and Furnishing Concrete Sheet Pile on the Water	T=120,B=500	m2	---	---
(Including head cover)				

Gabion Box (including geotextile)		m3	---	---
Sodding		m2	---	---
Rubble-mound		m3	---	---
Drainage Channel Works				
Tributary switching channel (Earth type)		m	---	---
Tributary switching channel (Box culvert type)		m	---	---
Drainage channel		m	---	---
Tunnel Works				
Horse Shaped Tunnnel (2R Type)	2R=5m	m	---	430
Road Works				
Macadam Pavement (Crushed Stones(10-40))	T=100	m2	---	---
Super Structure (Including handrail, paving, etc)		m2	---	---
General Road(including paving)	width=8m,h=3m	m	1,500	---
Road Bridge (Including Substructure, ancillary works)		m2	160	---
Other Works				
Main works * 30%				
Temporary Work				
Cofferdam (Eexcavation Common / Dredging As Temporary Works)		m3	---	
Driving Steel Sheet Pile Type II	L=10.0m	sheet		
Cellular Cofferdam	f8.5, h8.5	set	3	
	f6.0, h6.0	set	9	
Cellular Cofferdam (Only move)	f8.5, h8.5	set	3	
	f6.0, h6.0	set	8	
Stream Diversion Channel (B=30.0*h=2.5)		m		
Temporary main works * 20%				
(dewatering, site cleaning, etc)				
Civil Works Total				
Water gate		t	29	22

Source: JICA survey team

(2) Water Gate and Revetment

Major work quantities are summarized as shown in Table 7.3.2 below.

Table 7.3.2 Summary of Water Gate and Revetment Quantities

		Unit	Water Gate U/S	Water Gate D/S	Revetment
Civil Works					
Earth Works					
Excavation (Sand)	(DMT up to 5km)	m3	4,800	3,600	---
Excavation (Rock)	(DMT up to 5km)	m3	---	---	---
Back Filling, Selected Materials	(DMT up to 5km)	m3	1,600	650	2,800
Embankment, Selected Materials	(DMT up to 5km)	m3	7,400	---	---
Concrete Works					
Concrete (including Batcher plant,Scaffold, etc)	fck=16Mpa	m3	---	---	---
Concrete (including Form, Scaffold, etc)	fck=25Mpa	m3	2,150	1,300	---
Reinforceing bar		t	170	100	---
Substructure Work					
Driving and Furnishing Steel Sheet Pile Type II	L=2.0m	sheet	---	110	---
Driving and Furnishing Steel Sheet Pile Type II	L=2.5m	sheet	115	---	---
Driving and Furnishing Steel Sheet Pile Type II	L=5.5m	sheet	128	---	---
Driving and Furnishing Precast Concrete Pile	ϕ400,L=10.0m	nos	---	80	---
Driving and Furnishing Precast Concrete Pile	ϕ300,L=11.0m	nos	---	50	---
Driving and Furnishing Precast Concrete Pile	ϕ400,L=27.0m	nos	112	---	---
Driving and Furnishing Precast Concrete Pile	ϕ300,L=27.0m	nos	48	---	---
Concrete Block (Production, Installation cost)	w=0.5t/m2	m2	320	370	---
Revetment Works					
Driving and Furnishing Concrete Sheet Pile (Inc. head cover)		m2	---	400	---
Driving and Furnishing Concrete Sheet Pile on the Water (Inc. head cover)		m2	---	---	5,400
Gabion Box (including geotextile)		m3	---	140	---
Sodding		m2	3,000	200	---
Rubble-mound		m3	---	---	10,400
Drainage Channel Works			---	---	---
Tributary switching channel (Earth type)		m	1,000	---	---
Tributary switching channel (Box culvert type)		m	60	---	---
Drainage channel		m	6,000	---	---
Road Works			---	---	---
Macadam Pavement (Crushed Stones(10-40))	T=100	m2	300	---	---
Super Structure (Including handrail, paving, etc)		m2	165	---	---
Temporary Work			---	---	---
Cofferdam (Eexcavation Common / Dredging As Temporary Works)		m3	5,000	6,100	---
Driving Steel Sheet Pile Type II	L=10.0m	sheet	220	280	---
Stream Diversion Channel (B=30.0*h=2.5)		m	120	---	---
Metal works			---	---	---
Water gate		t	170	140	---

Source: JICA survey team

(3) Land acquisition and compensation

Land acquisition and compensation quantities are summarized as shown in Table 7.3.3.

- The heightening Oeste dam requires land acquisition and compensation. That area is 670,000 m^2.
- The Mirim upstream floodgate requires roads and levees area.

Table 7.3.3 Summary of land acquisition and compensation Quantities

Location	Land Acquisition (m^2)	Compensation
Heightening of Oeste dam	670,000	----
Heightening of Sul dam	----	----
Mirim Upstream floodgate	6,300	----
Mirim Downstream floodgate	----	----

Source: JICA survey team

7.3.3 Unit Cost Analysis

(1) Reference to Economic Analysis

Project cost and each of work rates is classified as four(4) resources and elements. Those unit costs are included overhead, profit, and taxes

1) Labor,
2) Materials,
3) Equipment, and
4) Overhead and profit.

The proportion of the resources is classified as two(2) types,

1) Civil works;
2) Metal works;

(2) Construction

Projects cost and work rates are set for major work items, such as excavation (m^3), filling (m^3), concrete (m^3), reinforcing bar (ton), steel/concrete sheet pile (m, m^2) and steel gates (ton). Construction unit price is referred through DNIT (National Department of Transport Infrastructure) and PINI (Construction price research firm)

As illustrated in Table 7.3.4, with the aim of calculating the costs for the purpose of the feasibility Study, the unit costs of 38 types of works were determined. All unit costs were based on the rate of April, 2011. The finally unit cost applied for the cost estimate are summarized as follows.

Table 7.3.4 Summary of Unit Cost for Cost Estimate

No.	Work Item	Unit	(R$)
EARTH WORKS			
A1	Excavation (Sand, DMT up to 5km)	m^3	15
A2	Excavation (Rock, DMT up to 5km)	m^3	100
A3	Back Filling, Selected Materials (DMT up to 5km)	m^3	40
A4	Embankment, Selected Materials (DMT up to 5km)	m^3	15
CONCRETE WORKS			
B1	Concrete (including Batcher plant, Scaffold, etc) fck=16Mpa	m^3	730
B2	Concrete (including Form, Scaffold, etc) fck=25Mpa	m^3	600
B3	Reinforcement - deformed bar	t	7,500
B4	Demolishing of Existing Concrete Structure (DMT up to 5km)	m^3	540
B5	Consolidation Grout	m	1,250
SUBSTRUCTURE WORKS			
C1	Driving and Furnishing Steel Sheet Pile Type II, L=2.0m	sheet	1,100
C2	Driving and Furnishing Steel Sheet Pile Type II, L=2.5m	sheet	1,400
C3	Driving and Furnishing Steel Sheet Pile Type II, L=5.5m	sheet	3,000
C4	Driving and Furnishing Precast Concrete Pile ϕ400,L=10.0m	nos	2,000
C5	Driving and Furnishing Precast Concrete Pile ϕ300,L=11.0m	nos	1,640
C6	Driving and Furnishing Precast Concrete Pile ϕ400,L=27.0m	nos	5,500
C7	Driving and Furnishing Precast Concrete Pile ϕ300,L=27.0m	nos	4,000
C8	Concrete Block (Production, Installation cost w=0.5t/m2)	m2	300
REVETMENT WORKS			
D1	Driving and Furnishing Concrete Sheet Pile (Including head cover), T=120,B=500	m^2	360
D2	Driving and Furnishing Concrete Sheet Pile (Including head cover, on the water),	m^2	440

	T=120,B=500 (Including head cover)		
D3	Gabion Box (including geotextile)	m^3	290
D4	Sodding	m^2	2
D5	Rubble-mound	m^3	80
DRAINAGE CHANNEL WORKS			
E1	Tributary switching channel (Earth type)	m	260
E2	Tributary switching channel (Box culvert type)	m	16,000
E3	Drainage channel	m	250
ROAD WORKS			
F1	Macadam Pavement (Crushed Stones(10-40), T=100)	m^2	20
F2	Super Structure (Including handrail, paving, etc)	m^2	1,400
F3	General Road (Including paving)	m^2	1,570
F4	Road Bridge (Including Substructure, ancillary works)	m^2	3,000
METAL WORKS			
G1	Water gate	t	40,800
TEMPORARY WORKS			
H1	Cofferdam (Excavation Common / Dredging As Temporary Works)	m^3	50
H2	Driving Steel Sheet Pile Type II(Material recycle), L=10.0m	sheet	660
H3	Cellular Cofferdam, , ϕ8.5, h8.5	set	113,000
H4	Cellular Cofferdam, ϕ6.0, h6.0	set	43,000
H5	Cellular Cofferdam (Move only) , ϕ8.5, h8.5	set	56,500
H6	Cellular Cofferdam (Move only) , ϕ6.0, h6.0	set	21,500
H7	Stream Diversion Channel (Concrete cannel B=12.0*h=3.0)	m	6,000
H8	Stream Diversion Channel (B=30.0*h=2.5)	m	600
Tunnel Works			
G1	House shoe Tunnel (2R 6.0 m)	m	35000

Source: JICA survey team

(3) Land acquisition and compensation

Land acquisition costs are estimated as below. The compensation cost is detailed at Annex F.

Land acquisition Average=1.4 R$/$m^2$ (Range:0.43～2.0 R$/$m^2$)

7.3.4 Direct Construction Cost

The summary of direct construction cost is estimated based on the work quantities and unit costs as shown in Table 7.3.5. And Table 7.3.6 shows the breakdown of summary of direct construction cost.

Table 7.3.5 Summary of Direct Construction Cost

(unite : R $)

	Oeste dam	Sul dam	Floodgate (U/S)	Floodgate (D/S)	Revetment
Earth Works	1,073,000	---	247,000	80,000	112,000
Concrete Works	10,260,000	2,127,000	2,565,000	1,530,000	---
Substructure Work	---	---	1,449,000	474,000	---
Revetment Works	---	---	6,000	185,000	3,208,000
Drainage Channel Works	---	---	2,720,000	---	---
Road Works	2,835,000	---	237,000	---	---
Other Works	4,250,000	638,000	2,167,000	681,000	996,000
Temporary Work	2,939,000	277,000	1,497,000	584,000	432,000

	Oeste dam	Sul dam	Floodgate (U/S)	Floodgate (D/S)	Revetment
Civil Works Total	21,357,000	3,042,000	10,888,000	3,534,000	4,748,000
Metalworks Total	---	---	6,936,000	5,712,000	---
Total	21,357,000	3,042,000	17,824,000	9,246,000	4,748,000

Source: JICA survey team

7.3.5 Land Acquisition and Compensation Cost

The summary of land acquisition and compensation costs estimated based on the quantities and unit costs is shown in the table below.

Table 7.3.7 Summary of Land acquisition and Compensation Cost

(**unite :** R$)

Location	Land acquisition unit cost=R$1.75*		Compensation unit=R$1,100/house		Total
	Area (m^2)	Amount	House	Amount	
Heightening of Oeste dam	670,000	966,000	----	0	966,000
Heightening of Sul dam Spillway	----		----	0	----
Mirim Upstream Gate	6,300	9,000	----	0	9,000
Mirim Downstream Gate	----		----	0	----
Total		975,000		0	975,000

- Note : Land acquisition place is rural zone

Source: JICA survey team

Table 7.3.6 Summary of Direct Construction Cost (details)

(unit:R$)

Item	Specification	Unit	Oeste dam		Sul dam spillway		Water Gate U/S		Water Gate D/S		Revetment		Remarks
			Quantity	Amount	Quantity	Amount	Quantity	Amount	Quantity	Amount	Quantity	Amount	
Earth Works													
Excavation (Sand)	(DMT up to 5km)	m3	59,000	885,000	4,400	66,000	4,800	72,000	3,600	54,000	---	---	
Excavation (Rock)	(DMT up to 5km)	m3	1,650	165,000	500	50,000	---	---	---	---	---	---	
Back Filling, Selected Materials	(DMT up to 5km)	m3	25,000	1,000,000	---	---	1,600	64,000	650	26,000	2,800	112,000	
Embankment, Selected Materials	(DMT up to 5km)	m3	---	---	---	---	7,400	111,000	---	---	---	---	
Concrete Works													
Concrete (including Batcher plant,Scaffold, etc)	fck=16Mpa	m3	12,500	9,125,000	---	---	---	---	---	---	---	---	
Concrete (including Form, Scaffold, etc)	fck=25Mpa	m3	3,500	2,100,000	4,050	2,430,000	2,150	1,290,000	1,300	780,000	---	---	
Reinforcement - deformed bar		t	140	1,050,000	70	525,000	170	1,275,000	100	750,000	---	---	
Demolishing of Existing Concrete Structure	(DMT up to 5km)	m3	250	135,000	800	432,000	---	---	---	---	---	---	
Consolidation Grout		m	380	475,000	---	---	---	---	---	---	---	---	
Substructure Work													
Driving and Furnishing Steel Sheet Pile Type II	L=2.0m	sheet	---	---	---	---	---	---	110	121,000	---	---	
Driving and Furnishing Steel Sheet Pile Type II	L=2.5m	sheet	---	---	---	---	115	161,000	---	---	---	---	
Driving and Furnishing Steel Sheet Pile Type II	L=5.5m	sheet	---	---	---	---	128	384,000	---	---	---	---	
Driving and Furnishing Precast Pc Pile	ϕ400,L=10.0m	nos	---	---	---	---	---	---	80	160,000	---	---	
Driving and Furnishing Precast Pc Pile	ϕ300,L=11.0m	nos	---	---	---	---	---	---	50	82,000	---	---	
Driving and Furnishing Precast Pc Pile	ϕ400,L=27.0m	nos	---	---	---	---	112	616,000	---	---	---	---	
Driving and Furnishing Precast Pc Pile	ϕ300,L=27.0m	nos	---	---	---	---	48	192,000	---	---	---	---	
Concrete Block (Production, Installation cost)	w=0.5t/m2	m2	---	---	---	---	320	96,000	370	111,000	---	---	
Revetment Works													
Driving and Furnishing Concrete Sheet Pile	T=120,B=500	m2	---	---	---	---	---	---	400	144,000	---	---	
(Including head cover)													
Driving and Furnishing Concrete Sheet Pile on the Water	T=120,B=500	m2	---	---	---	---	---	---	---	---	5,400	2,376,000	
(Including head cover)													
Gabion Box (including geotextile)		m3	---	---	---	---	---	---	140	40,600	---	---	
Sodding		m2	---	---	---	---	3,000	6,000	200	400	---	---	
Rubble-mound		m3	---	---	---	---	---	---	---	---	10,400	832,000	
Drainage Channel Works													
Tributary switching channel (Earth type)		m	---	---	---	---	1,000	260,000	---	---	---	---	
Tributary switching channel (Box culvert type)		m	---	---	---	---	60	960,000	---	---	---	---	
Drainage channel		m	---	---	---	---	6,000	1,500,000	---	---	---	---	
Tunnel Works													
Horse Shaped Tunnnel (2R Type)	2R=5m	m	---	---	430	15,050,000	---	---	---	---	---	---	
Road Works													
Macadam Pavement (Crushed Stones(10-40))	T=100	m2	---	---	---	---	300	6,000	---	---	---	---	
Super Structure (Including handrail, paving, etc)		m2	---	---	---	---	165	231,000	---	---	---	---	
General Road(including paving)	width=8m,h=3m	m	1,500	2,355,000	---	---	---	---	---	---	---	---	
Road Bridge (Including Substructure, ancillary works)		m2	160	480,000	---	---	---	---	---	---	---	---	
Other Works													
Main works * 30%				5,331,000		1,051,000		2,167,000		681,000		996,000	
Temporary Work				1,617,000		1,960,000		939,000		---		432,000	(Minimam 10%)
Cofferdam (Eexcavation Common / Dredging As Temporary Works)		m3	---	---		---	5,000	250,000	6,100	305,000		---	
Driving Steel Sheet Pile Type II	L=10.0m	sheet		---		---	220	143,000	280	182,000		---	
Cellular Cofferdam	ϕ8.5, h8.5	set	3	339,000		---		---		---		---	
	ϕ6.0, h6.0	set	9	387,000		---		---		---		---	
Cellular Cofferdam (Only move)	ϕ8.5, h8.5	set	3	171,000		---		---		---		---	
	ϕ6.0, h6.0	set	8	172,000		---		---		---		---	
Stream Diversion Channel (B=30.0*h=2.5)		m				---	120	72,000		---		---	
Temporary main works * 20%				214,000		---		93,000		97,000		---	
(dewatering, site cleaning, etc)													
Civil Works Total				26,001,000		21,564,000		10,888,000		3,534,000		4,748,000	
Water gate		t	29	1,183,000	22	898,000	170	6,936,000	140	5,712,000			
Metal works Total				1,183,000		898,000		6,936,000		5,712,000		---	
Total				27,184,000		22,462,000		17,824,000		9,246,000		4,748,000	