Supporting Report (D)

Flood Forecasting and Warning System

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

DRAFT FINAL REPORT

VOLUME I : SUPPORTING REPORT
ANNEX D: STRENGHTENING OF EXISTING FFWS

Table of Contents

Page

Tables

Figures

CHAPTER 1 INTRODUCTION

1.1 Background

An Integrated Plan for Prevention and Mitigation of Natural Disaster Damage Risks in the Itajai River Basin (Plano Integrado de Prevencao e Mitigacao de Desastres Naturais na Basia Hidrografica do Rio Itajai) was formed by the S.C. state government in September 2009. The plan lists up 6 programs with 77 projects, which were proposed by Technological and Scientific Group (GTC). In the plan, one of the highest priority programs focuses to "Strengthen the monitoring and warning system" for natural disasters in the Itajai River Basin.

This study particularly focuses on the prevention and mitigation of flood disasters. This involves three stages (1) preparation for disaster prevention before flood events; (2) disaster measures at floods; (3) restoration and recovery after floods.

Within these stages, a series of activities: observation, data management (data transmission and monitoring), weather and flood forecasting, warning, evacuation and flood prevention is defined as "Flood Forecasting and Warning System (FFWS)" in this report.

1.2 Main objectives

This supporting report, Annex D reviews the present condition of the prevention and mitigation activities for flood disasters, especially FFWS, in the Itajai River Basin. In addition, this report aims to propose a master plan for strengthening FFWS.

The following points shall be indicated in this report.

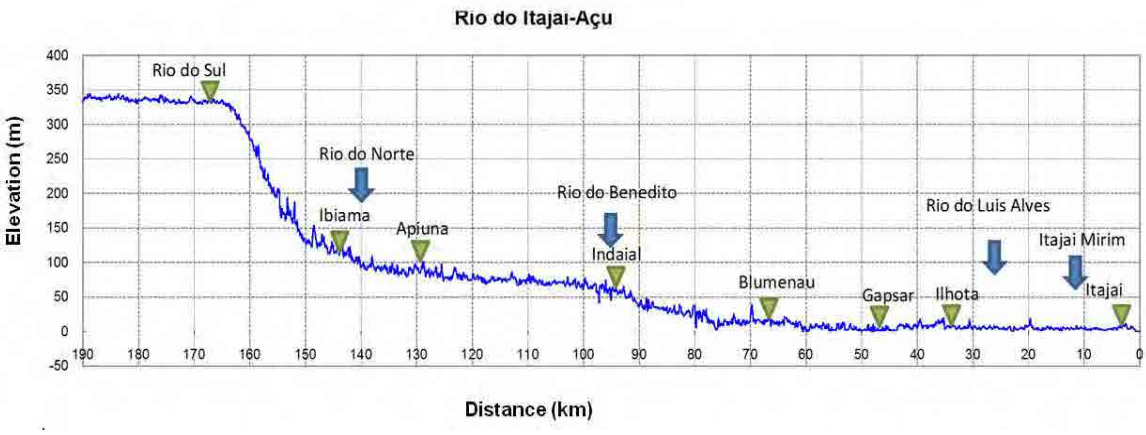
1) Improvement of observation and monitoring system by installing new gauging stations and monitoring centre.
2) Evaluation of the existing forecasting methods and suggestion for future requirements.
3) Improvement of warning system using GPRS telecommunication system.
4) Establishment of new institutional organization and river management system including flood prevention and mitigation measures.
5) Estimating the cost for proposed FFWS.

CHAPTER 2 EXISTING CONDITIONS

2.1 Features of Itajai River

The Itajai River Basin ranges from Rio do Sul city in northern area of Santa Catalina state down to the Itajai Estuary; it is across three major municipalities, Rio do Sul, Blumenau and Itajai. The catchment area is 15,500 km^2 and the water source mainly comes from the mountainous region of Serra Geral.

The length of the Itajai River is approximately 250 km and it flows into the Sul and the Oeste River and they meet at Rio do Sul city. There are two dams in upstream, the Sul dam (catchment area of 1,273 km2) and the Oeste dam (catchment area of 1,042 km^2). The Norte River (catchment area of 2,318 km^2) joins into the Itajai River at Apiuna city which is located in 130 km from the Itajai estuary. The Cedros River flows into the Benedito River at Timbo city and it meets the Itajai River at Indaial city. The river slope is 1/500 and the torrent flow continues up to Blumenau city which is located 70 km from the estuary. However, the downstream of Blumenau is almost flat with 1/3,500 slope including Gaspar and Ilhota city. Furthermore, the Luis Alves River (catchment area of 580 km$^{2)}$ flows into the Itajai River at 37.3 km upstream of Itajai city and the Mirim River (catchment area of 1,207 km^2) which flows down from Bursque city meets the main river of Itajai River at Itajai city. The longitudinal profile of the Itajai River is shown in Figure 2.1.1.



Source: JICA Study Team

Figure 2.1.1 Itajaí River Declivity Course

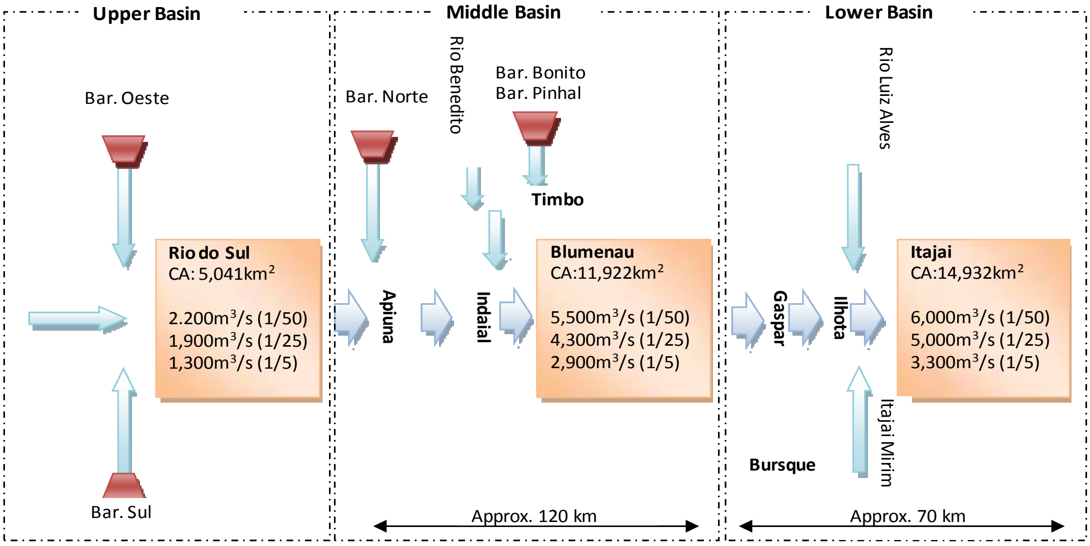
The river basin is classified into upper-basin, middle-basin and lower-basin according to its characteristics (Table 2.1.1). The upper-basin has Sul dam and Oesta dam for flood control purpose, the middle-basin has Norte dam also for flood control, and Bonito and Pinhal dams for power generation. The river merges into the Benedito River and then confluence with the Itajai River at Indial city.

Table 2.1.1 Characteristics of the Itajai River Basin

Itajai River	From Estuary	Slope	Basin Area	Remark
Upper Basin	190 km	1/1,200	5,041km^2	The area is mountain area more than 300m altitude, the slope around Rio do Sul is flat level and the flow is slow velocity.
Middle Basin	70 km	1/500	11,922km^2	From Rio do Sulto Blumenau, the slope is steep and the flow confluence with the Norte and Benedito river.
Lower Basin	0 km	1/3,500	14,932km^2	Between Blumenau and Itajai estuary, the slope is almost all flat, and the flow is slowly.

Source: JICA Study Team

According to the characteristics of the Itajai River Basin, the distribution of flood discharge in 5 years, 25 years and 50 years probability is estimated as shown in Figure 2.1.2.



Source: JICA Study Team

Figure 2.1.2 Distribution of Flood Discharge for the Itajai River Basin

2.2 Existing Integrated Plan for Prevention and Mitigation of Natural Disaster Damage Risks

The existing Integrated Plan for Prevention and Mitigation of Natural Disaster Damage Risks in Itajai River Basin (Plano Integrado de Prevencao e Mitigacao de Desastres Naturais na Basia Hidrografica do Rio Itajai) (PPRD-Itajai) was formed by the S.C. state government in September 2009. The plan lists up 6 programs with 77 projects which were proposed by Technological and Scientific Group (GTC) (Table 2.2.1).

Table 2.2.1 Integrated Plan for Prevention and Mitigation of Natural Disaster Damage Risks in Itajai River Basin

Program	Detailed Measure	Priority
No.1	**Institutional development for the preparation for emergencies and disasters**	
	1a) Qualify human resources at basic, intermediary and advanced levels	
	1a1) Qualification of teachers, technicians and community leaders for the integrated support to the civil defense (natural disasters, introduction of risk management and environmental legislation).	**3**
	1a2) Qualification of municipal technicians in risk management.	**1**
	1a3) Creation of a post-graduate course on risk management.	**3**
	1a4) Qualification of municipal technicians in geological, geotechnical and engineering fundaments to interpret hazard maps (of the risk areas) and to subsidize master plans, qualification of planning technicians from municipalities associations, from Itajaí basin municipalities and from state agencies.	**3**
	1a5) Conduction of seminars for the integration of regional, national and international experiences about natural disasters.	**3**
	1a6) Exchange between national and international institutions in the field of risk management.	**3**
	1a7) Definition of cooperation with high education institutions to support the civil defense.	**2**
	1b）Structure of civil defense and other related agencies	
	1b1) Restructuring and/or implementation of civil defense agencies at state and regional level, according to the Law in force.	**3**
	1b2) Restructuring and/or implementation of municipal civil defense coordination – COMdeCs.	**1**
	1b3) Reequipping of institutions responsible for emergencies, members of the state and municipal systems of civil defense, including purchase of equipment, vehicles, among others, to support disasters preparation actions.	**3**
	1b4) Development of municipal plan(s) of civil defense.	**3**
	1b5) Articulation between civil defense plans and the instruments of the sanitation, housing, environment, water resources and urban planning instruments for each municipality.	**3**
	1b6) Strengthening municipal environment agencies and municipal environment councils.	**1**
	1b7) Elaboration of the plan for the issuance of warning.	**3**
	1b8) Preparation of the manual of critical situations procedures.	**3**
	1b9) Periodic simulation and drill of the warning plan.	**3**
	1b10) Strengthening of a local inter-institutional scientific advisory group for the reduction of disaster risks.	**3**
No.2	**Monitoring and warning system**	
	2a) Institutionally strengthen the monitoring and warning system	
	2a1) Implementation of an inter-institutional arrangement to strengthen the Alert system of Itajaí Basin (de INFRA, SDS, civil defense, Universities, Epagri/Ciram, Itajaí Committee/water agency Foundation), and improvesment of the contacts network of the alert system at the Itajaí Basin.	**1**
	2b) Structure of the warning system (equipments, methodologies and supports)	
	2b1) Development of the communication system for the alert and diffusion network of the warning system.	**3**
	2b2) Maintenance and expansion of the hydro-meteorological-oceanographic telemetric network at the Itajaí Basin.	**1**
	2b3) Development of models to monitor and forecast extreme events.	**2**
	2b4) Development of a hillsides monitoring system.	**2**

	2b5) Development of environmental monitoring methodologies including use of satellite (for rainfall/temperature).	**2**
	2b6) Development, implementation and validation of warning systems.	**2**
No.3	**Perception, communication, motivation and mobilization for resiliency and reduction of vulnerability**	
	3a) Diagnosis of natural disasters causes (landslides and inundations) from the point of view of the affected persons (perception).	
	3a1) Diagnosis of natural disasters causes (landslides and inundations) from the point of view of the affected persons (perception).	**2**
	3a2) Production of educational material about: natural disasters management, risk situations, appreciation of human life, first aid and cardiorespiratory reanimation, and technical and legal criteria for the occupation of risk areas.	**2**
	3a3) Socio-educational programs for natural disasters management and related themes.	**2**
	3b) Implement social participation mechanisms	
	3b1) Qualification and mobilization of volunteers for the integrated support to the civil defense.	**3**
	3b2) Development and maintenance of a volunteers network to support civil defense in risks.	**3**
	3b3) Development and maintenance of a bank of volunteer specialists to provide support at risks.	**2**
	3b4) Protection of populations against focal disaster risks.	**3**
	3c) Establish partnerships with utilities providers and private companies aiming at risk reduction	
	3c1) Creation of the certification seal for companies that develop good environmental conservation practices, including earthworks practices.	**2**
	3d) Establish culture and attitudes change in health and communication of epidemiological studies resulting from natural disasters	
	3d1) Studies on the implications of disasters on the human health.	**3**
	3d2) Importance of personal and housing hygiene in preventing epidemics.	**3**
	3d3) Formation of group for psychological assistance during disaster situations.	**3**
No.4	**Evaluation of disasters** risk**s reduction**	
	4a) Develop basic and thematic maps	
	4a1) Aero-survey of S.C. (SDS – 1:10,000 – 2009/2011), with priority for Itajaí basin.	**3**
	4a2) Elaboration of basic mapping (1:10,000 scale for the whole basin, and 1:2,000 scale for urban areas and potential higher risk areas).	**1**
	4a3) Detailed maps on soils, geology, geotechnology, land use and others, per hydrographic basin, and their land use aptitude focused on risk areas.	**2**
	4a4) Maps of available social equipments – such as temporary shelters – and of relevant public and private services in natural disasters situation including local roads, natural springs and alternatives of drinking water.	**2**
	4b) Develop integrated system of information on disasters	

	4b1) implementation of an integrated information system in Geographic Information system environment, containing the cartographic base and thematic maps (including geology, geo-technology, soils, rainfall, occurrence of disasters, levels of rivers in respective sections with elevation-discharge curves, among others) based on the existing information in different institutions, such as ANA, ANEEL, SDS, CEOPS, EPAGRI/CIRAM, CPRM, IBGE, Universities, Prefectures, water agency Foundation of Itajaí Valley among others, considering the need of data conversion.	**1**
	4c) Register and evaluate disasters risk	
	4c1) Analysis of meteorological systems, development and evaluation of intense rainfall models, and temporal and trend analysis of rainfall in Itajaí Basin.	**2**
	4c2) development of methodologies for the identification and evaluation of risk, for different types of disasters that occur in Itajaí Basin.	**3**
	4c3) Creating hazard maps of risk/multiple threats areas for developing a natural disasters registration system.	**1**
	4c4) Elaboration of the geo-statistics and probability analysis of the occurrence of different types of threats in the region, and identification of higher risk potential regions.	**2**
	4c5) Development of the Atlas of Natural Threats at Itajaí Basin.	**3**
	4d) Evaluate the drainage network	
	4d1) Inventory and registration of interventions in water courses, and evaluation of activities developed in the drainage network: study of influence (positive and negative impacts) of non-structural changes executed in the basin, and of structural measures executed in the drainage network.	**2**
No.5	**Reduction of disaster** risks	
	5.1）Subprogram of land use and occupation management – non-structural measures.	
	5.1a) Subsidize the development of municipal urban development legislation.	
	5.1a1）Development of municipal legislation aiming at the restriction of urban areas impermeabilization and/or at the incentive to the reservation of rainwater, opposing impacts resulting from impermeabilization.	**2**
	5.1a2) Development of state mechanism to update municipal legislation concerning urban land allotment, taking into consideration risk areas and their specificities.	**2**
	5.1a3) Revision, adjustment and update of municipal master plans including risk reduction of municipal civil defense plans.	**1**
	5.1a4) Development and approval of a Bill (of law) for the regulation and inspection of earthwork, sand extraction and rolling stone extraction activities.	**1**
	5.1b) Implement land use and occupation inspection	
	5.1b1) Development and implementation of municipal integrated systems of inspection, monitoring and evaluation of land use and occupation at the Itajaí Basin.	**2**
	5.1c) Establish a housing policy to avoid risk **areas occupation**	
	5.1c1) Development of alternative housing programs for low and no income population that live in risk area.	**2**
	5.1c2) Development of a housing registry at state level to control the beneficiaries of such programs.	**3**
	5.1d) Improve and expand the forest coverage	

	5.1d1) Establishment of ecological economic zones as areas intended for the implementation of commercial forests.	**3**
	5.1d2) Developments and implementation of municipal plans of forest coverage maintenance and enrichment, and of expansion of vegetal coverage in the urban area.	**2**
	5.1d3) Re-capitations and maintenance of Permanent Preservation Areas.	**2**
	5.1d4) Incentives to the implementation of legal reservations.	**3**
	5.1d5) Studies for the adoption of payment for environmental services.	**2**
	5.1d6) Studies on the restoration in areas affected by landslides.	**3**
	5.1d7) Analysis of successful step of reforestation for the containment of landslides.	**3**
	5.1e) Adjust the land use in rural areas	
	5.1e1) planning agricultural properties according to the aptitude of the soil and to legal restrictions.	**2**
	5.1e2) Implementation soil management practices that respect their natural aptitude, as well as measures of utilization, retention and infiltration of rainwater in agriculture management, in order to support the storage of water, as well as to stimulate the increase of forest coverage.	**2**
	5.1f) Provide a proper destination to solid waste and debris	
	5.2）Sub-program of adequate management of water courses	
	5.2a) Keep water courses in their original configuration and revitalize changed water courses	
	5.2a1) Elaboration of criteria and a guiding manual for the management of water courses.	**2**
	5.2a2) Projects of rivers revitalization.	**3**
	5.2b) Multiple use of existing hydraulic structures	
	5.2b1) Execution of inventory of existing hydraulic structures (dams, rice pads, lakes, tanks, etc.), including the verification of the compliance with technical and legal criteria in their construction.	**1**
	5.2b2) Evaluation of effects of the existing hydraulic structures in absorbing flood waves, and study on the optimization of such system.	**2**
	5.2b3) Modeling of the hydrological, hydraulic and sedimentological behavior of the drainage network, based on an updated diagnosis of the hydraulic-sedimentological situation, for evaluation of interventions with structural measures.	**2**
	5.2b4) Feasibility studies on water retentions and storages (at microbasin scale) through the implementation of pilot-projects.	**1**
	5.2c) Manage urban drainage waters	
	5.2c1) Development of drainage plans (municipal), considering the utilization, retention and infiltration of water.	**2**
	5.2c2) Adjustment and maintenance of existing drainage systems, according to such plans.	**2**
	5.2c3) Implementation of new rainwater drainage systems.	**2**
No.6	**Recuperation of areas affected by disasters**	
	6a) Identify affected areas	

	6a1) Mapping areas and quantity of families affected, and classification of areas per type of intervention: with removal and without removal of occupation.	**1**
	6b) Environmentally recuperate occupied areas, in conjunction with civil works (totally keeping the current occupation)	
	6b1) Elaboration of intervention project(s), with definition of interventions to be executed (structural and non-structural).	**2**
	6b2) Execution of the above elaborated projects, followed by future monitoring and inspection of the area.	**3**
	6c) Environmentally recuperate occupied areas, in conjunction with civil works (but totally or partially removing the current occupation)	
	6c1) Elaboration of intervention project(s), with definition of interventions to be executed (structural and non-structural), plus: • Quantification of families to be removed; • Determination of the approximate cost for the implementation of the removal measure, and • Definition of area for the production of regularized land lots, with housing units and available infrastructure.	**3**
	6c2) Execution of the intervention project(s) above elaborated, including: • Awareness building and negotiation with families; • Reallocation of families; • Psycho-social follow-up of reallocated families; and • Environmental recuperation, destination of use for the recuperated area, monitoring and inspection.	**3**
	6c3) Creation of conservation units in risk areas and high risk areas, where occupation is not allowed or recommendable.	**3**

Source: Plano Integrado de Prevencao e Mitigacao de Desastres Naturais na Basia Hidrografica do Rio Itajai

The programs and measures are listed under the "Master Plan for Water Resources in the Itajai River Basin" (Plano Director de Recursoso Hidricos da Bacai do Itaja) that shall be implemented by the Itajai Committee (Comite do Itajai) in the future. The detailed measures for each program indicate guide lines for the prevention and mitigation of natural disaster damage risks according to the GTC.

Of those 77 projects proposed by GTC, the high priority projects regarding FFWS are described following.

i) Program No.1: Institutional development for the preparation for emergencies and disasters

The following projects shall be prioritized to implement and the institutional organization for flood prevention shall be established a.s.a.p. including the river management organization.

- 1a2) Qualification of municipal technicians in risk management
- 1b1) Restructuring and/or implementing civil defense agencies at state and regional level, according to the law in force.
- 1b7) Elaboration of a plan for issuance of warning.

ii) Program No.2: Monitoring and warning system

The following projects shall be prioritized to implement and the master plan of the proposed FFWS shall include the following projects.

- 2a1) Implementation of an inter-institutional arrangement to strengthen the warning system of the Itajaí Basin (de INFRA, SDS, civil defense, Universities, EPAGRI/CIRAM, Itajaí Committee/water agency foundation), and improvement of the contacts network of the warning system at the Itajaí Basin.
- 2b2) Maintenance and expansion of the hydro-meteorological-oceanographic telemetric network at the Itajaí Basin.

iii) <u>Program No.4: Evaluation of disasters risks reduction</u>

The following projects shall be prioritized to implement. Currently, the measure 4a2 has been in progress, as implemented by S.C. State and the basic topography map is being prepared based on the aerophoto. The measures 4b1 and 4c3 shall be coordinated with the UN after establishing a structured organization for disasters as well as after forming a management system for flood prevention.

- 4a2) Elaboration of basic mapping (1:10,000 scale for the whole basin, and 1:2,000 scale for urban areas and potential higher risk areas). The Itajai River Basin particularly in urban cities adjacent to the river courses have started preparing for basic hazardous mapping (1:10,000 scale for the whole basin, and 1:2,000 scale for urban areas and potential higher risk areas) in accordance with the urban master plan. However, the hazardous map only covers the urban areas.
- 4b1) Development of an integrated information system using geographic information system; containing the cartographic base and thematic maps (including geology, geo-technology, soils, rainfall, occurrence of disasters, levels of rivers in respective sections with elevation-discharge curves, among others) based on the existing information in different institutions, such as ANA, ANEEL, SDS, CEOPS, EPAGRI/CIRAM, CPRM, IBGE, Universities, Prefectures, water agency Foundation of Itajaí Valley among others, considering the need of data conversion.
- 4c3) Creating hazard maps of risk/multiple threats areas for developing a natural disasters registration system.

iv) <u>Program No.5: Reduction of disaster risks</u>

The following projects shall be prioritized to implement with consideration of the land development plan for each city.

- 5.1a3) Revision, adjustment and update of the existing municipal master plans, including the risk reduction of municipal civil defense plans.
- 5.1a4) Development and approval of a Bill (of law) for the regulation and inspection of activities such as earthwork, sand extraction and rolling stone extraction.
- 5.2a1) Elaboration of criteria and a manual for the management of water courses.
- 5.2b4) Feasibility studies on water retentions and storages (at microbasin scale) through the implementation of pilot-projects.

v) <u>Program No.6: Recuperation of areas affected by disasters</u>

The following project shall be prioritized to implement together with preparation of the land development plan as well as hazard maps development.

- 6a1) Mapping areas and quantity of families affected, and classification of areas per type of intervention: with removal and without removal of occupation.

2.3 Existing Institutional Framework

The present activities in relation to flood prevention and mitigation in the Itajai River Basin are shared among the various institutions depending on the scale of a disaster. The related institutions and their responsibilities are shown in Table 2.3.1.

Table 2.3.1 Related Institutions in FFWS

Organization		Activity
ANA	National Water Authority	ANA is responsible for policy execution and implementation of the national water resources in Brazil. ANA sets up hydraulic gauging stations all over Brazil and make the rainfall and river water level data available to the related institutions. In the Itajai River Basin, ANA owns 43 rainfall gauging stations and 23 river water level gauging stations. The past records from these gauging stations have been stored in a database and available to be downloaded on their website. However, most available data is daily record only. ANA is a national institution and so that currently their data is not sent at real-time to state institutions such as CIRAM.
EPAGRI/ CIRAM	S.C. state Agricultural Research/ Hydro-meteorology Information Centre	The information centre belonging to the Depart of Agriculture sends the meteorological data to the related organizations through the Internet. The S.C. state has precipitation stations at 41 locations and the river water level stations at 15 locations. The date is transmitted to the CIRAM Information centre by Tele-meteor or Satellite system. The CIRAM carries out simulation of weather forecast by using the data and other on-line information. The weather forecast is reported on the TV, Radio and WEB sites on the Internet. In a case of any emergencies, the CIRAM directly sends the forecast to the related organizations in the state.
FURB CEOPS	Information system control centre in Blumenauz University	The control centre of flood Information system in Blumenau University, the flood forecast in Blumenau is executed by the precipitation and river water level of SDS separate from ANA/CIRAM. At flooding, the CONDEC with Mayor will be organized to issue a flood warning.
UNIVALI	ITAJAI University	The university is a technical adviser for the Itajai municipality. Prof. Carvalho is a member of the Itajai River Basin Committee as well as the counterpart. The specialty of Professor is ocean and coastal engineering and in charge of the disaster prevention for flooding and reinforcement of monitoring for the Itajai River.
UNIFEBE	Brusque University	The university is a technical adviser for Brusque municipality and plans flood control dams in upstream of the Mirim River.
AMAVI	Association in upstream municipality of the Itajai River.	The municipal associations in upstream of the Itajai River report information of the river to the related organizations in order to manage the administration in upstream.
AMMVI	Association in midstream municipality of the Itajai River	The municipal associations in midstream of the Itajai River report information of the river to the related organizations in order to manage the administration in midstream.
AMFRI	Association in downstream municipality of the Itajai River	The municipal associations in downstream of the Itajai River report information of the river to the related organizations in order to manage the administration in downstream.
SEDEC	Civil defense	The civil defense for the national disaster is activity for the emergency disaster,

	for national level	and carried out the Policy and management concerning the civil defense in the all Brazil.
SDC	Civil defense for state level	The civil defense for S.C. state is sent out at medium scale disaster which is not managed by municipalities. The S.C. state consist 293 political regions and 36 rural administrations. However, civil defenses are only set up in 7 municipalities in the S.C. state, and only 3 municipalities in the Itajai River Basin. The SIEDEC carries out capacity building and training for disaster prevention for the civil defenses. The SIEDEC is also in charge of planning state policy for disaster prevention.
COMDEC	Commission of civil defense for municipality	The civil defense for municipalities is sent out at small scale disaster to guide the refuges. The CONDEC usually patrols, and trains/educates the related organizations for disaster prevention. At a disaster, civil defense has to report the damage situation to the CONDEC which is formed by the Mayor.
CD Municipal Council	Council for flood prevention in municipality	The council is consisted the chairman of the Mayor and the vice chairman of the civil defense, and instruct to the civil defense in order to issue the warning and smoothly evacuate. In case of the large scale disaster, the CONDEC will request the support to state government.
GRAC	Prevention of flood in municipality	The GRAC is formed with fire fighters, polices, city officers and volunteers at a disaster. Their activity is guidance of refuge at disaster and restoration after disaster.
SDS	Economic development sustains	The SDS is the economic development sustains in the S.C. state. The SDS executes the policy and operation regarding water resources and is responsible for water use planning related to economic development. Their main mission is to permit and approve the hydropower dam and irrigation facilities, and to manage water permission. Recently, the SDS is developing a meteorological model using the radar of Urubisi's Air Force other than CIRAM and now planning to develop a flood warning system using hydrology model (America, Texas model); this is to reduce flood damage in order to avoid negative impacts on economy.
DEINFRA	Department of infrastructure for S.C. state	DEINFRA is the department of infrastructure in S.C. State. Within the Itajai River Basin, they are in charge of operation and maintenance of infrastructures in S.C. State including flood control dams..
CELESC	S.C. State electronic company	CELESC is responsible for the electricity utilities in the S.C. state. They are in charge of operation and maintenance of electronic supply including power generation dams.
CASAN	S.C. State water company	CASAN is responsible for the water utilities in the S.C. state; in charge of water supply, sewage collection and maintenance.
TELECOM	Brazil Telecom	TELECOM is a major Brazilian telecommunications company.
OBRAS	Municipal public works	OBRAS is the department of public works for municipalities.
ABNT/ NBR	Brazilian National Standards Organization	ABNT is an non-profit organization that is responsible for standards and codes for technological development in Brazil.

Source: JICA Study Team

According to the prioritized measures in the integrated plan, PPRD-Itajai (refer section 2.2), the current situation of flood disaster prevention and mitigation activities in the Itajai River Basin is analyzed in the following checklist (Table 2.3.2/ Table 2.3.3/ Table 2.3.4). The checklist covers the concerned activities in three different stages: (1) preparation for disaster prevention before flood events; (2) disaster measures during floods; (3) restoration and recovery after floods. (It shall be noted that this checklist only covers three major cities, Rio do Sul in upper-basin, Blumenau in the middle-basin and lItajai (Ihota and Gaspar) in the lower basin; and so that, further detailed studies shall be required for the rest of cities in the basin.

The present issues of the Itajai River Basin are as followings.

- Currently, multiple institutions are involved in stage (1) and (2), but there is no clear institutional organization among the concerned activities.
- Universities such as UNIASSEVI, FURB and UNIVALI work as a technical adviser on river management. However, there is no institution in charge of the integrated river management for the entire Itajai River Basin.
- As a result, the conditions of the river features as well as river structures/facilities are not understood particularly at the preparation stage (1). Therefore, hazardous areas where there may be high possibility to cause serious damages by coming floods are not maintained appropriately.
- On the other hand, presently each city in Itajai River basin owns manuals for flood prevention and evacuation activities mainly for during and after flood. However, prevention activities during floods are not composed in the existing manuals and not really implemented.
- Furthermore, manuals for some activities are not standardized and/or approved officially; they are still in progress of establishment.
- in the Itajai River Basin, particularly the urban cities adjacent to the river have started preparing for basic hazardous mapping (1:10,000 scale for the whole basin, and 1:2,000 scale for urban areas and potential higher risk areas) in accordance with the urban master plan. However, the hazardous map only covers the urban areas. Furthermore, another existing issue is that the actual condition of land use is not understood well and resulting in existence of some residents in the flood hazardous areas.

Table 2.3.2 Checklist for Preparation Activities for Disaster Prevention before Flood (in case of the major cities) in S.C. State

			Rio du Sul (Upper Stream)		Blumenau (Middle Stream)		Itajai /lhota/Gaspar (Lower Stream)	Remark
1.1	**Provision of Regulation reg. Inundation Area**							Master plan for only urban area, information of flood damaged area relied on past record
	Land Use Plan	-	Master Plan (2006) urban area only	-	Master Plan (2006) urban area only	-	Master Plan (2006) urban area only	
	Development Regulation	-		-		-		
	Building Standard	○	National Standard (ABNT/NBR)	○	National Standard (ABNT/NBR)	○	National Standard (ABNT/NBR)	
	Others							
	Information of Disaster Forecasting							
	Information of Disaster Area	○	Past Inundated Area	○	Past Inundated/Land Sliding Area	○	Past Inundated Area	
	Flood Hazardous Map	-		○	Urban Area Only	-		
	Hazard Map	-		-		-		
1.2	**Status of Dam Management**							
	Gate Management Manual	○	DEINFRA	○	DEINFRA	-	None	
	Management of Warning System and Siren	○	Oeste/Sul Dam	○	Norte Dam/Cedros Rv.	-	None	
	Understanding of River Feature							Condition of existing river is not understood because data for river condition before floods is not available due to absence of river management unit.
	Discharge and Water Level	-	UNIASSEVI supports monitoring for rainfall and water level.	○	Technical Support FURB/CEOPS	-	UNIVALI manages water level of estuary.	
	Discharged Sedimentation	-		-		-		
	River fluctuation	-		-		-		
	Estuary Condition	-		-		○		
	Information of River Bank							
	Overtopping Area	-	Municipal/OBRAS (28Municipal)	-	Municipal/OBRAS (14Municipal)	-	Municipal/OBRAS (5Municipal)	
	Leakage Area	-		-		-		
	Scouring Area	-		-		-		
	Cracking Area	-		-		-		
	Collapseing Area	-		-		-		
	Management of River Facilities							
	Bridge	-	Municipal/OBRAS	-	Municipal/OBRAS	-	Municipal/OBRAS	
	Sluiceway	-		-		-		
	River Structure	-		-		-		
1.3	**Announcement of Flood Warning**							Evacuation warning and announc
	Obtaining of Meteorological Information	○	Discharge converted by rainfall of Oeste Dam and Sul Dam (Municipal)	○	Forecasting water level at Blumenau from water level of Apiuna and Timbo by FURB/CEOPS	○	Rainfall of CIRAM and water level at site	
	Alert by Precipitation	-		○		-		
	Obtaining of Water Level Information	○		○		-		
	Alert by Water Level	○		○		-		
1.4	**Method for Transmission of information**							Aged, handicapped people and children may delay to evacuate due to no patrol in the night brown out.
	Mass Media such as TV, Radio	○	TV, Radio, Internet	○	TV, Radio, Internet	○	TV, Radio, Internet	
	Web site of Internet	○		○		○		
	Warning Tower and Bulletin Board	-		-		-		
	Patrol by Civil Defense	○		○		○		
	Sirens	-		-		-		
1.5	**Flood Prevention activities for Evacuation**							Evacuation only implemented for measures of flood prevention
	Evacuation Plan	○	Completed by CD Municipal Council	○	Completed by CD Municipal Council	-	Gaspar: Completed Ilhote: until December Itajai: until December	
	Communication Network	○		○		-		
	Evacuation Method	○		○		-		
	Management of Material and Heavy Machinery for flood prevension							In case of lacking equipments, SC state will support.
	Sheet, Pipe, Sandbag	○	Municipal/OBRAS	○	Municipal/OBRAS	○	Municipal/OBRAS	
	Machinery	○		○		○		
	Transportation vehicle	○		○		○		
	Boat	-		-		○		
	Management of miscellaneous goods at evacuation center							In case of lacking miscellaneous goods, SC state will support.
	Water	○	CASAN	○	CASAN	○	CASAN	
	Foods	○	Municipal	○	Municipal	○	Municipal	
	Evacuation center	○	Municipal	○	Municipal	○	Municipal	
1.6	**Ordinary Inspection and Patrol**							Inspection before floods
	Seasonal Patrol	○	Civil Defense	○	Civil Defense	○	Civil Defense	
1.7	**Training and Capacity Building**							Implementing by Civil Defense of SC state before floods
	Training for Flood Prevention	○	Civil Defense	○	Civil Defense	○	Civil Defense	
	Training for Evacuation	○	Civil Defense	○	Civil Defense	○	Civil Defense	
	Capacity Building	○	State's Civil Defense	○	State's Civil Defense	○	State's Civil Defense	

Source: JICA Study Team

Table 2.3.3 Checklist for Disaster Measures during Flood (in case of the major cities) in S.C. State

			Rio du Sul		Blumenau		Itajai	Remark
2.1	**Recognizing of Flood Situation**							Flood forecasting of Rio do Sul by rainfall of Oeste Dam and Sul Dam basins、flood forecasting of Blumenau by Apiuna and Timbo
	Meteorological Information	○	CIRAM/Site	○	CIRAM/Site	○	CIRAM/Site	
	Water Level Information	-	None	○	FURB/CEOPS	-	None	
	Understanding of Site condition	○	Civil Defense	○	Civil Defense	○	Civil Defense	
	Flood Forecasting	○	Municipal Engineer	○	FURB/CEOPS	-	Pom Model	
2.2	**Procedure of Warning Announcement**							Residents to evacuate by announcement by CD Municipal Council
	Warning Announcement	○	Municipal/CD Municipal Council	○	Municipal/CD Municipal Council	○	Municipal/CD Municipal Council	
	Communication Organization	○	Municipal/CD Municipal Council	○	Municipal/CD Municipal Council	○	Municipal/CD Municipal Council	
	Evacuation Conduct	○	Civil Defense	○	Civil Defense	○	Civil Defense	
	Traffic Control	○	OBRAS	○	OBRAS	○	OBRAS	
2.3	**Coordination with related Organization**	○	Municipal/CD Municipal Council	○	Municipal/CD Municipal Council	○	Municipal/CD Municipal Council	Manual applied
2.4	**Procurement of Material and others**	○	OBRAS	○	OBRAS	○	OBRAS	Manual applied
2.5	**Flood Prevention activities**							Evacuation is more important for measures of disaster prevention.
	Overtopping Prevention	-	No concrete measures regarding flood prevention activities by CD Municipal Council	-	No concrete measures regarding flood prevention activities by CD Municipal Council	-	No concrete measures regarding flood prevention activities by CD Municipal Council	
	Scoring Prevention	-		-		-		
	Leakage Prevention	-		-		-		
	Cracking Prevention	-		-		-		
	Collapse Prevention	-		-		-		
2.6	**Inspection and Patrol**							Manual applied
	Inspection of Flood Situation	○	Civil Defense	○	Civil Defense	○	Civil Defense	
	Inspection of Life Line	○	Concerned Institution	○	Concerned Institution	○	Concerned Institution	
	checking of Evacuation Situation	○	Civil Defense	○	Civil Defense	○	Civil Defense	

Source: JICA Study Team

Table 2.3.4 Checklist for Restoration & Recovery Activities after Flood (in case of the major cities) in S.C. State

			Rio du Sul		Blumenau		Itajai	Remark
3.1	**Measures for Dewatering and Drainage**							work of municipal but it is delayed. Problem is drainage of inner basin in downstream.
	Restoration of Disaster Area	○		○		○		
	Temporary Drainage System	○	Municipal/OBRAS	○	Municipal/OBRAS	○	Municipal/OBRAS	
	Dewatering of Inland Drainage	○		○		○		
3.2	**Restoration of Life Line**							Municipal is delayed the work due to finance.
	Electric Power	○	CELESC	○	CELESC	○	CELESC	
	Water Supply	○	CASAN	○	CASAN	○	CASAN	
	Telecommunication	○	TELECOM	○	TELECOM	○	TELECOM	
	Restoration of Transportation						Gaspar:Gas Pipeline	
	Road	○	Municipal/DEINFRA	○	Municipal/DEINFRA	○	Municipal/DEINFRA	
	Landslide	○	DEINFRA	○	DEINFRA	○	DEINFRA	
3.3	Removal of Obstacle							Municipal is delayed the work due to finance.
	Obstacle	○		○		○		
	Cleaning by Dustbin Lorry	○	Municipal/OBRAS	○	Municipal/OBRAS	○	Municipal/OBRAS	
	Hygiene and Sanitation	○		○		○		
3.4	**Report of Damage**							Manual applied
	Deaths and Missing Person	○		○		○		
	Inundated House and Refuge Number	○	Municipal/civil defense but civil defense of state/federal government when big disaster	○	Municipal/civil defense but civil defense of state/federal government when big disaster	○	Municipal/civil defense but civil defense of state/federal government when big disaster	
	Livestock and Farm Produce	○		○		○		
	Damage Situation	○		○		○		
	Others	○		○		○		

Source: JICA Study Team

From the above checklists (Table 2.3.2 – 2.3.4), the detail analysis especially about FFWS is discussed in the following. The flow diagram, Figure 2.3.1 shows the existing institutional organization and responsible activities particularly regarding to FFWS in SC State. Currently, FURB/CEOPS is charged FFWS activities for the Itajai River Basin committed by SDS. Although FURB/CEOPS is in charge of flood forecasting for the entire river basin, it is not operated well due to the following reasons.

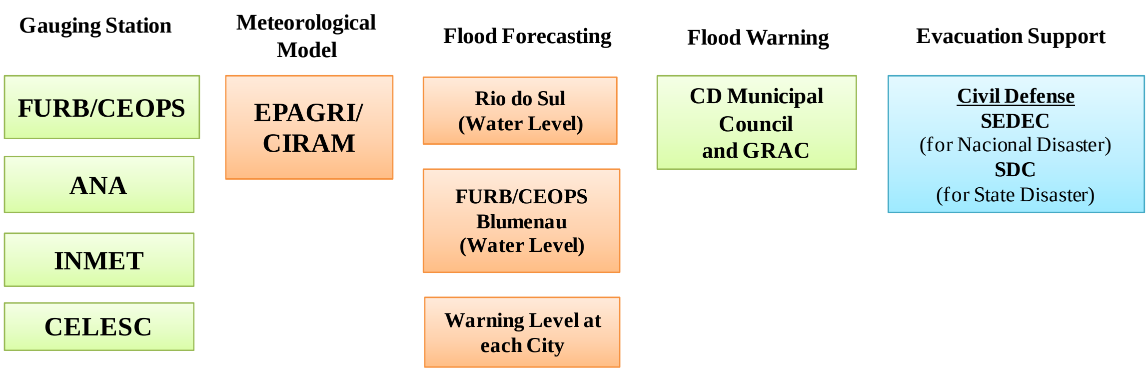
i. CIRAM which is the meteorological department of the SC State Gevornment collects meteorological and hydrological data that is observed at 38 gauging stations by various institutions; however, no data has been transmitted from CIRAM to FURB/CEOPS.

ii. However, currently, FURB/CEOPS only conducts flood forecasting for Blumenau, where FURB/ CEOPS is located and not for other cities. For flood forecasting, FURB/CEOPS utilized data from the 14 hydraulic gauging stations (rainfall and river water levels gauging station as one set) operated by themselve.

iii. The forecasted results are only informed to Defensa Civil in Blumenau. This is to say, the existing FFWS activities are not systematically planned and conducted among the related institutions throughout the entire Itajai River Basin.For flood forecasting,

iv. For the present flood forecasting for Blumenau city, FURB/CEOPS utilized the water level data only from three stations in Blumenau, Apiuna and Timbo; the data from the rest of 11 stations are currently not utilized for forecasting.

v. On the other hand, Defensa Civil in Rio do Sul city tries to conduct flood forecasting; however, the present forecasting is not appropriate for practical use. One of the reasons is that DEINFRA, the operator of the Oeste dam and Sul dams that locate at upstream areas of Rio do Sul has not recorded and informed the outflow discharges from the dams to the downstream rivers.



Source: JICA Study Team

Figure 2.3.1 Present Institutional Organization in SC State for FFWS

2.4 Existing Metrological and River Water Level Observation

Currently in Itajai River Basin, the hydraulic conditions (rainfall and river water level) are observed by multiple institutions for various purposes; such as FURB/ CEOPS for flood control, ANA for river/water resource management, EPAGRI for agriculture and CELESC for hydraulic power generation (refer Figure 2.3.1). The main issue is that data observation, equipments maintenance and data management has not been integrated and managed.

The numbers of related hydraulic gauging stations in the Itajai River Basin is listed in Table 2.4.1.

Table 2.4.1 The Existing Gauging Stations in Itajai River Basin.

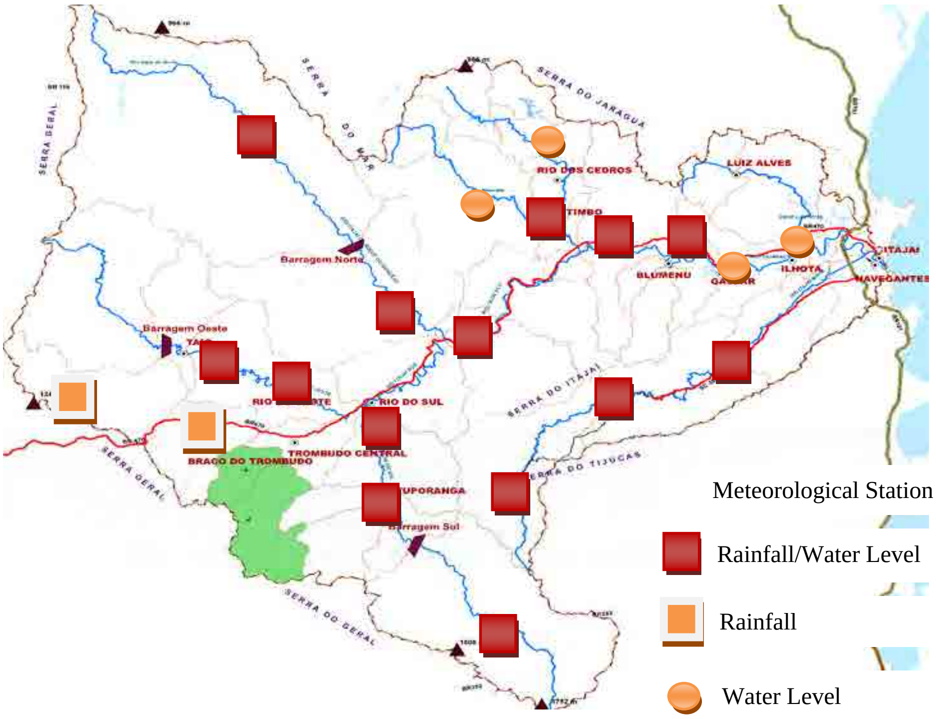
Gauging Types	FURB/CEOPS	ANA
Rainfall gauging stations	16	43
River water level gauging stations	14	23
TOTAL Number of Locations	**16**	**66**

Source: EPAGRI/CIRAM

Moreover, there are more gauging stations run by different institutions other than listed in Table 2.4.1.

- Itajai city: owns 9 rainfall gauging stations and 8 river water level gauging stations.
- CELESC: owns some hydraulic gauging stations around their dams and power plants.
- INMET and other universities that locate in the major cities: owns some hydraulic gauging stations.

FURB/ CEOPS has established hydraulic gauging stations at 16 locations throughout the Itajai River Basin (Figure 2.4.1) with an order from SDS in 1985. These stations were set up for the purpose of FFWS for the entire basin. Of these locations, 14 locations are rainfall and river water level gauging stations with automatic data transmission systems (using telemetry or satellite) (refer Table 2.4.2).



Source: JICA Study Team

Figure 2.4.1. Location of FURB/CEOPS Gauging Stations

Table 2.4.2 List of FURB/CEOPS Gauging Stations

Location	Gauging Type	Transmission System
Mirim Doce	Rainfall	Satellite
Taio	Rainfall/ Water Level	Telemetry
Rio Oeste	Rainfall/ Water Level	Telemetry
Puuso Redondo	Rainfall	Telemetry
Saltinho	Rainfall/ Water Level	Telemetry
Ituporanga	Rainfall/ Water Level	Telemetry
Rio do Sul	Rainfall/ Water Level	Telemetry
Barra do Prata	Rainfall/ Water Level	Satellite
Ibirama	Rainfall/ Water Level	Telemetry
Apiuna	Rainfall/ Water Level	Telemetry
Timbo	Rainfall/ Water Level	Telemetry
Indaial	Rainfall/ Water Level	Telemetry
Blumenau	Rainfall/ Water Level	Telemetry
Salseiro	Rainfall/ Water Level	Telemetry
Botuvera	Rainfall/ Water Level	Satellite
Brusque	Rainfall/ Water Level	Telemetry
14 Rainfall/Water Level & 2 Rainfall		

Source: JICA Survey Team

The observation activities by FURB/CEOPS are not functioning well due to the following problems.

- The existing 14 gauging stations and where the existing warning levels are set (refer Table 2.5.1) are not corresponding in the locations. For an instance, there is no gauging station along Luis Alves River which is one of the major tributary rivers.
- The related equipments of the 14 hydraulic gauging stations are not maintained enough due to lack of financial source. As a result, the related equipments such as automatic data logger and transmission systems are not functioning well.
- Furthermore, many water level gauges (particularly hydraulic pressure type) are affected by riverbed erosion and sedimentation while, many rainfall gauges are blocked up with leaves.
- As considering the present condition listed above, FURB/CEOPS employ some residents, who live nearby the stations, to observe and record water levels by watching scales. However, data is not recorded appropriately in this way because some have discontinued recording from delinquency in administration payment.

Therefore, the data is not appropriate for practical use. The current situations of the existing 14 hydraulic gauging stations (rainfall and water level) are summarized in Table 2.4.3.

Table 2.4.3 Situation of Existing 14 Gauging Stations under FURB/CEOPS

Existing Stations		Gauging Type	Transmission System	Situation of Stations
1	Taio	Rainfall/ Water Level	Telemetry	Transmission system needs to be improved.
2	Rio Oeste		Telemetry	Monitoring equipment does not function, GSM is defective, and there is not person in charge for maintenance.
3	Saltinho		Telemetry	Monitoring equipment does not function, GSM is defective, and there is not person in charge for maintenance.
4	Ituporahga		Telemetry	Transmission system needs to be improved.
5	Rio do Sul		Telemetry	Operated
6	Barra do Prata		Satellite	Monitoring is not executed on time due to the failure of monitoring equipment and absence of persons in charge for maintenance.
7	Ibirama		Telemetry	Transmission system needs to be improved.
8	Apiuna		Telemetry	Thought residents around the station observed water level because of the failure of the sensor and telemetry, they now stop due to delinquency of administration payment from CEOPS.
9	Timbo		Telemetry	Operated
10	Indaial		Telemetry	Monitoring equipment does not function, GSM is defective, and there is not person in charge for maintenance.
11	Blumenau		Telemetry	Operated
12	Salseiro		Telemetry	GSM transmission system is defective.
13	Botuvera		Satellite	GSM transmission system is defective.
14	Brusque		Telemetry	Water level gauging sensor does not function due to sedimentation

Source: JICA Survey Team

2.5 Existing Forecasting and Warning System

2.5.1 Meteorological Forecasting by CIRAM

CIRAM (the S.C. state's Meteorological/Hydrological Environment Center) collects all the data that is observed by multiple institutions. CIRAM connects to the Doppler Radar (SIMEPAR) that is now installed in Northern Parana area and Meso-data (1,200 m, 5,000 m and 12,000 m of SONDE) that is set in the Florianopolis Airport through the Internet (as shown in Figure 2.5.1.).

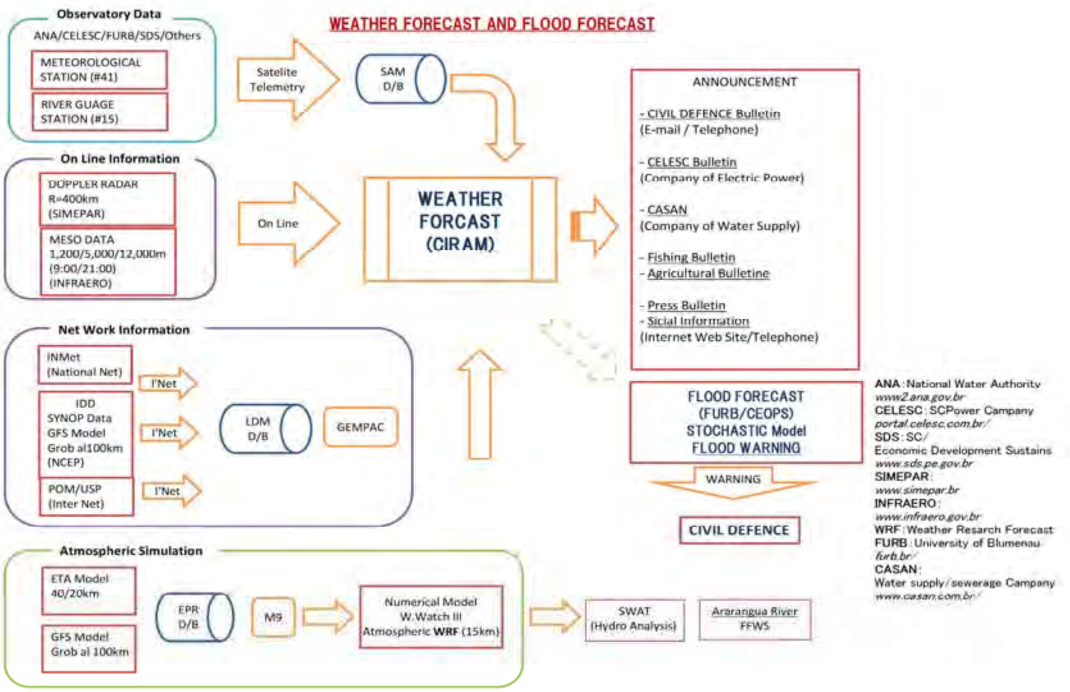
Besides these observatory data, the database (INMet, IDD and NCEp) called GEMPA and the meteorological simulation database called M9 (ETA40/20 km model and GPS 100 km model) are secured by cooperating with oversea metrological networks.

Based on WRF (15 km) program, weather forecasting is conducted by CIRAM every 1 hour, 3 hours, 6 hours and 12 hours using a model, which is established by ETA (Brazil) and WRF (America).

The weather forecasting information is announced to the S.C. state's citizen through TV, Radio and the Internet. In an emergency, information is reported to the civil defense of S.C. state government, CELESC (S.C. Electric Corporation), Fishing and Agricultural Union, at the same time, FURB/CEOPS that are located in Blumenau.

However, the current situation is that the collected data as well as the result of weather forecasting are not well organized as one database. Therefore, the information is not shared among the related institutions and so that they are not reflected to the practical FFWS.

CIRAM is now planning to upgrade the metrological forecasting by adopting satellite data from INPE. It is expected that this shall improve the precision of the forecasting.



Source: EPAGRI/CIRAM

Figure 2.5.1 Weather Forecasting by EPGRI/CIRAM

2.5.2 Flood Forecasting and Warning Activities

The existing FFWS is not systematically planned and conducted throughout the entire Itajai River Basin. The flood forecasting is executed only in Blumenau and Rio do Sul cities. In Rio do Sul city, the City Defesa Civil has a flood forecasting method using rainfalls at the Oeste dam and Sul dam but this is not utilized for practical use. This is because the existing method does not reflect the DEINFRA's dam operation and outflow discharges from the two dams and the result is not reliable. Therefore, presently flood forecasting based on the observed data is only conducted in Blumenau city.

Other 16 cities including Itajai city have "warning levels" as guideline (based on river water level) for warning announcement (Table 2.5.1). These warning levels are established based on the past flood water level. When the river water level starts rising, the civil defenses of each city patrols and reports the river condition to CD Municipal Council immediately.

However, some of the cities have no water level gauging stations or the lack of water level gauging stations in the upstream of the cities. For example, two cities (Agu Clera and Gurabiruba) that locate along the tributary river of Brusque, and three cities in mountain region (Salete, Mirim Doce and Pouso Redondo) where have been suffering from flood damage as an influence of recent land development.

In these small cities, evacuation activities are not conducted appropriately. One of the reasons is lack of hydrological data as mentioned above and this causes delay in warning announcement particularly at night. Furthermore, in these mountain region, "flash floods" are also a serious concern, however the rainfall in the area are not monitored currently. Disorganization in forecasting, warning and evacuation activities is also one of the issues in these areas.

Table 2.5.1 Warning Standards Based on River Water Level

Itajai River Basin (each city)	Elevation EL+m	Catchment Area (km2)	Normal level (m)	Standby level (m)	Warning level (m)	Emergency level (m)
Taio	360	1,575	4.0m	6.0m	6.5m	over 7.5m
Rio do Oeste	-	-	4.0m	6.0m	9.0m	over 9.0m
Trombudo	350	248	3.0m	4.0m	7.5m	over 7.5m
Ituporanga	370	1,670	2.0m	3.0m	4.0m	over 4.0m
Vidal Ramos	-	-	3.0m	4.0m	6.0m	over 5.0m
Rio do Sul	350	5,100	4.0m	5.0m	6.5m	over 6.5m
Ibirama	151	3,314	2.0m	3.0m	4.5m	over 4.5m
Apiuna	93	9,241	3.0m	6.0m	8.5m	over 8.5m
Benedito Novo	90	692	1.5m	2.5m	3.5m	over 3.5m
Rio dos Cedros	80	510	1.5m	2.5m	3.5m	over 3.5m
Timbo	73	1,342	2.0m	4.0m	6.0m	over 6.0m
Indaial	60	11,151	3.0m	4.0m	5.5m	over 5.5m
Blumenau	12	11,803	4.0m	6.0m	8.5m	over 8.5m
Gaspar	11	12,141	4.0m	6.0m	8.5m	over 8.5m
Ilhota	-	12,357	6.0m	8.0m	10.5m	over 10.5m
Itajai	-	15,221				

Source: FURB/CEOPS

2.6 Existing Evacuation and Flood Prevention Activities

In Itajai River Basin, each city owns evacuation manuals that were developed based on the present flood experiences. The manuals basically include institutional organization and instructions for evacuation activities at flooding but not guidelines for detailed flood prevention activities. For example, even if river embankment or revetment is corrupted, no urgent measures (such as using sand bags to stop water) are currently implemented.

The approved official evacuation manuals only exist in Rio do Sul and Blumenau cities. These manuals regulate (1) institutional organization for each activity; (2) potentially higher risk areas; (3) refuge places for each regional block (Refer Table 2.6.1) and; (4) instructions for smooth evacuation activities. On the other hand, 13 other cities are currently developing their evacuation manuals to be approved following the two former cities.

The numbers of refuges for each regional block in Rio do Sul, Blumenau and Itajai is shown in Table 2.6.1.

Table 2.6.1 Numbers of Refuges Sites and Regional Blocks fore Rio do Sul, Blumenau and Itajai

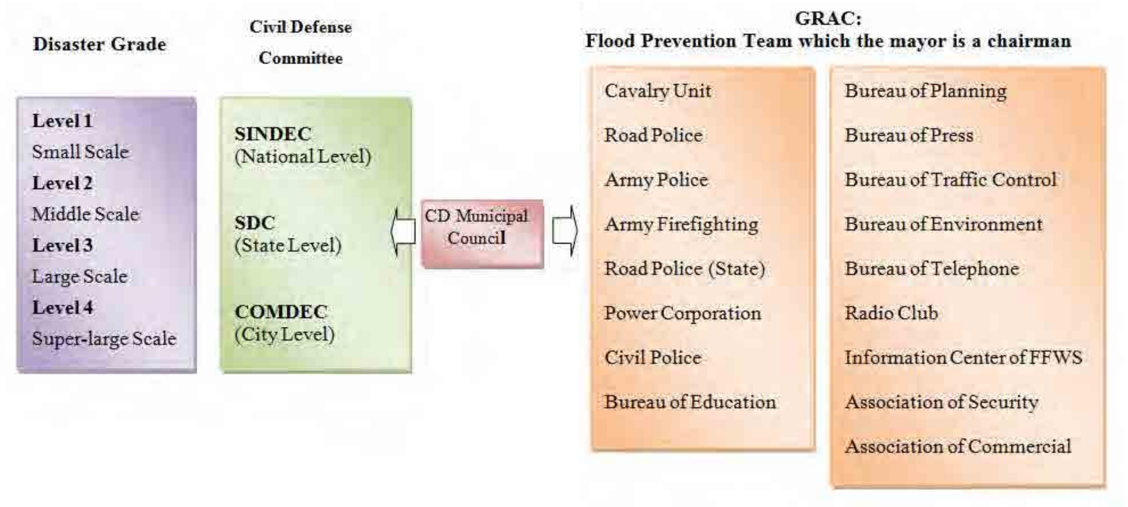
	Rio do Sul	Blumenau	Itajai
Regional Blocks	7	5	In progress
Refuge Sites	55	57	62

Source: JICA Study Team

According to those evacuation manuals, CD municipal council is formed by the city civil defense with the city mayor as a chairman. The city civil defense patrols and reports the river condition (including river water levels) to the council. Then, the council is responsible for flood warning announcement according to the warning level (as listed in Table 2.5.1).

In addition, GRAC, the flood prevention team is also formed by the institutions listed in Figure 2.6.1 (in a case of Blumenau city for example). GRAC is in charge of supporting and securing safe evacuation activities; however, the present situation is that they are like a communication coordinator among the related intuitions.

When a large or medium scale flood occurred, the CD municipal council reports the situation to the S.C. state's civil defense (SDC) and the nation civil defense (SINDEC) to requests their supports.



Source: JICA Study Team

Figure 2.6.1 Present Institutional Organization for Evacuation and Flood Prevention Activities in S.C. State

As an example, the evacuation manual of Blumenau city prescribes a list of institutions that are involved in GRAC (Table2.6.2).

Table 2.6.2 GRAC organization Member (in case of Blumenau)

No.	Related Organization and Party	
1	Batalhao de Infantaria	Cavalry Unit
2	Policia Rodoviria Federal	National Road Police
3	Batalhao de Policia Militar	Army Police
4	Batalhao de Bombeiro Militar	Army Firefighting
5	Policia Rodoviria Estadual	State Road Police
6	Centrais Eletricas de Santa Catarina (CELESC)	State Power Corporation
7	Delegacia Regional de Policia Civil/Bl	Municipality Civil Police
8	Gerencia Regional de Educacao (GERED)	Bureau of Education
9	Secretaria Municipal de Obras (SEMOB)	Municipality Public Works
10	Secretaria de Servicos Urbanos (SESUR)	Bureau of Urban Service
11	Secretaria Municipal de Saude (SEMUS)	Bureau of Sanitation
12	Secretaria Municipal de Assistemcia Social, da Crianca e do Adolescente (SEMASCRI)	Bureau of Welfare/Child Protection
13	Secretaria Municipal de Administracao (SEdeAD)	Bureau of Administration
14	Secretaria Municipal de planejamento (SEPLAN)	Bureau of Planning
15	Secretaria Municipal de Comunicacao (SECOM)	Bureau of Press
16	Secretaria Municipal de Educacao (SEMED)	Bureau of Education
17	Servico Autonomo Municipal de Transito e Transportes de Blumenau (SETERB)	Bureau of Traffic Control
18	Fundacao Municipal de Meio Ambiente (FAEMA)	Bureau of Environment
19	Oi Telecomunicacoes	Bureau of Telephone
20	Clube de Radioamador	Radio Club
21	Centro de Operacao do Sistema de Alerta (CEOPS/FURB)	Flood Information Centre
22	Associacao dos Profissionais de Seguranca de Blumenau e Regiao (APSEBRE)	Association of Security
23	Camara de Diretores Lojistas (CDL)	Association of Commercial Store
24	Associacao Comercial e Industrial de Blumenau (ACIB)	Chamber of Commerce and Industry
25	Jeep Clube e Moto Clube	Jeep/Auto Motor Club

Source: Evacuation plan of flooding in Blumenau

CHAPTER 3 MASTER PLAN FOR STRENGTHEN EXISTING FFWS

3.1 Proposal for Flood Prevention and Mitigation

From the analysis of the existing situations in the Itajai River basin, the following Table 3.5.1 outlines the proposed flood prevention and mitigation measures in accordance with the programs and projects that are highlighted in the Integrated Plan for Prevention and Mitigation of Natural Disaster Damage Risk (PPRD-Itajai). The measures are classified into (1) flood plain regulation, (2) disaster forecasting, (3) measures during disaster, (4) river management and (4) restoration and recovery activities.

Table 3.1.1 Proposed Flood Prevention and Mitigation Measures

Flood Prevention and Mitigation Activities	Details of Proposed Methods	Actual Measures	The Concerned Programs in PPRD-Itajai
1. Flood plain regulation	• Regulation of land use plan • Building standard and codes • Regulation of development • Re-development plan • Town planning against flood disasters • Specification for building	• Reorganization of administrative institutions • Enforcement of the laws • Development of penalty regulations	Program 5 • Land use and occupation management • Urban development legislation Program 3 • Perception, communication motivation and mobilization for resiliency and reduction of vulnerability
2. Disaster forecasting	• Strengthening of observation system (gauging stations) • Strengthening of monitoring, flood forecasting and warning system • Development of risk map/hazard map	• Database management • Evaluation of risk	Program 4 • Evacuation of disasters risk reduction
3. Disaster measure	• Flood prevention • Evacuation • Disaster recovery	• Related organization • Evacuation manual • Evaluation of risk • Reconstruction of lifeline	Program 2 • Monitoring, alert and Alarm Program 6 • Recuperation of areas affected by disasters Program 1 • Structure of civil defense and other related agency
4. River management	• Re-organization of river management • River improvement plan • Inventory list/book • Information center for disaster	• Administration	Program 1 • Structure of civil defense and other related agency Program 4 • Evacuation of disasters risk reduction
5. Restoration and recovery activity	• Disaster insurance • Zoning of flood area • Assessment of flood potential	• Administration • Insurance Company	Program 3 • Perception, communication motivation and mobilization for resiliency and reduction of vulnerability

Source: JICA Study Team

Currently, preparation activities for disaster prevention before flood event are hardly implemented. This is mainly because that there is no distinct administrator of river management. It is important to organize institutional framework for river management including FFWS (refer section 3.2). To improve the institutional organization, the state government must be responsible of integrated river management for the entire Itajai River Basin.

The measures that shall be implemented by the appropriate government agencies (S.C. state government/ the administrator of river management) are listed as followings.

- The conditions of both river features and river structures/facilities should be understood, managed and maintained by the river manager. The river inventory shall also be established by the river manager to manage river water use. In addition to this, a flood information center shall be established within the administrative institution in order to strengthen FFWS.
- The potential flood areas (zones) and expected flood damages should be assessed and determined. The concerned flood hazardous areas must be reflected to the urban master plan and land use regulations of each city.
- The flood plain regulations for flood hazardous area should be developed in accordance with the master plan of land use. (The land use master plan shall be reflected the social conditions: such as living condition, industrial component and population density and the natural conditions such as hydraulics and geology.)
- A disaster insurance proponent shall be determined. In addition, the residents must be relocated from the flood hazardous area according to the land use plan and flood plain regulations.
- Official/approved manuals for flood prevention and mitigation activities for before, during and after flood disaster should be established for each city in the basin.
- The manuals also shall include hazard maps: 1:10,000 scale for the whole basin, and 1:2,000 scale for urban areas and potential higher risk areas.

3.2 Proposal for Institutional Organization for Strengthening Existing FFWS

The appropriate institutional organization must be properly constructed so that the proposed FFWS exercise effectively. Presently the FFWS conducted by FURB/CEOPS which is delegated by SDS is not functional enough and it has no good communication between FURB/CEOPS and other organizations of the SC State.

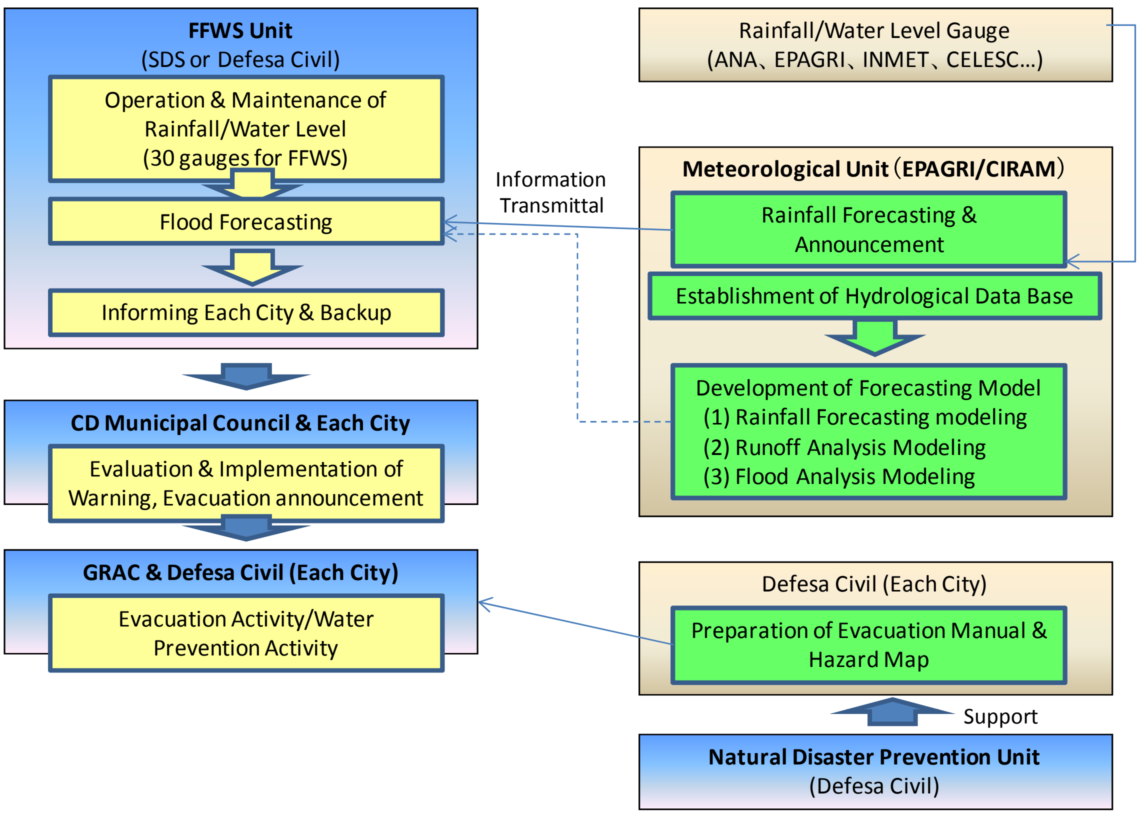
To implement improvement of the institutional organization for FFWS all over the Itajai River basin, the state government must have a responsibility for the matter as shown in Figure 3.21, but not just by local university unit.

This study proposes the improvement of 14 existing gauging stations and additional 16 gauging stations, so that all cities in the basin are able to have the flood forecasting and warning system or warning system. All of 30 gauging stations must be managed and operated by the state government unit responsible for FFWS (SDS/Defesa Civil).

i) The state government unit in charge for FFWS shall improve the flood forecasting method and waning level not only Blumenau but also other cities in the basin. On the other hand, EPAGRI/CIRAM of meteorological unit of the state must establish new system which transmits all hydrological and meteorological data observed by different organizations. CIRAM must manage data and develop data base system for all data comes from different organizations including aforementioned 30 gauging stations. These data must be utilized for future development of FFWS such as runoff analysis, inundation analysis and rainfall forecasting.

ii) The result for flood forecasting should be transmitted to all of CD Municipal Counsel in the basin and the flood warning and evacuation announcement shall be implemented and evaluated by each city in consideration of warning level. Therefore the propriety of warning level shall be verified.

iii) GRAC shall conduct evacuation activities based on the evacuation announcement. Defesa Civil of the state government unit for prevention and mitigation of natural disaster should support to make the evacuation manual and hazard map for some city which has not prepared yet.



Source: JICA Study Team

Figure 3.2.1 Proposed Institutional framework for FFWS

3.3 Proposal for Target City for Flood Warning

(1) Proposal for Establishment of Warning Level

As described in Section-2.4/Chapter-2, presently 16 cities have established the warning level. However the warning level was established 25 years ago and the situations of the river channel and its riparian areas would have been changed. The existing warning level for those cities should be reviewed based on study of discharge capacity in urban area and H-Q curve at the gauging station.

At present, there is no warning level for Brusque City which is the third important city in the Itajai River Basin. In addition to those cities, Mirim Dose, Salete, Pouso Redondo located in mountainous area in upstream of Rio do Sul and Agua Clera, Gurabiruba in mountainous area along tributaries of Brusque City are endangered by sudden floods caused by local downpour as an influence of recent land development. Therefore these 6 cities have to establish the warning level newly. Hydrological gauging stations are also needed together with a siren, which is to announce evacuation warning.

The target cities for proposed FFWS including existing are shown in Table-3.3.1.

(2) Cities for Flood Forecasting

The flood forecasting for the Itajai River Basin should be conducted by an integrated system with rainfall forecasting, run-off analysis and flood forecasting in the future. Therefore, presently Defesa Civil, EPAGRI/CIRAM and SDS are planning for establishing the model.

On the other hand, Rio do Sul, Blumenau and Itajai City, which are the major cities of upper, middle and lower area of Itajai River respectively, had serious damages by floods before. These cities need a flood warning system by flood forecasting using the water level correlation formula as a provisional solution until establishment of the aforesaid model.

3.4 Proposal for Improvement/Additional Rainfall and Water Level Gauging Station

(1) Improvement for Existing Rainfall and Water Level Gauging Station

Existing 14 gauging stations described in Table-2.4.3 in Section 2.4 are foundation and are located at the most important sites for the FFWS. Therefore, improvement of equipments and updating transmission system for these stations should be prioritized for the future observation of rainfall and water level. In discussion with SDS, this improvement and updating is supposed to be implemented by the finance of the state government lead by SDS. Improvement of these existing stations must be implemented certainly.

(2) Proposal for New Gauging Stations for Rainfall/ Water Level

i) New Gauging Stations for Target Cities for Proposed FFWS

New gauging stations shall be installed for cities, which have no rainfall/water level gauging stations in spite of the target for setting warning level. Those cities are 11 in total such as Ilhota, Gaspar, Benedito Novo, Rio dos Cedros, Agura Clera, Burabiruba, Vidal Ramos, Trombudo Central, Pouso Redondo, Salete and Mirim Doce.

ii) New Gauging Station for Flood Forecasting and Early Warning and Evacuation

The informations of outflow discharge from Oeste and Sul Dams for flood control located in the upstream river of Rio do Sul are necessary for the flood forecasting at Rio do Sul. Presently DEINFRA as the dam administrator has no data of outflow discharge from dams. DEINFRA should establish a system which is able to monitor the outflow discharge from dams in future. However in the present moment, new gauging stations should be installed at downstream reaches of the dams to monitor the outflow discharge. These new stations were strongly requested by Rio do Sul City. In addition to this purpose, these new stations are useful for earlier warning to Taio City and Ituporanga City.

On the other hand, existing stations of Blumenau and Brusque to be improved and new gauging stations at Gasper and Ilhota will be utilized for the flood forecasting at Itajai city. Presently municipal Defesa Civil has 9 automated rainfall stations and 8 automated water level stations inside Itajai City. It is proposed to integrate these data with the state observation system.

CELESC as the hydropower dam administrator should be responsible for data management of Rio Bonito and Pinhal dam operation, but it is difficult to establish a system immediately. Therefore new gauging stations should be installed at immediate downstream of the dams for monitoring discharge from the dams. These new stations are also useful for earlier warning to Rio dos Cedros and Timbo. Timbo City strongly requested to install CCTV cameras for visual inspection of floods together with discharge volume from dams.

As a result of above, 4 new gauging stations at immediate downstream of Oeste, Sul, Rio Bonito and Pinhal Dams are proposed to be installed with 2 CCTV careras.

iii) New Gauging Station for Establishment of Flood Forecasting Model in Future

Presently there is a rainfall station of ANA along Luiz Alves River which is a main tributary of Itajai Lower River but hourly data is not available as mentioned in Chapter-4. New gauging station is proposed to install at Luiz Alves City for establishment of flood forecasting model in future which Defesa Civil, CIRAM, and SDS is planning now. Details of the existing and proposed new gauging stations are shown in Table-3.4.1 and their locations are shown in Figure-3.4.1.

iv) Proposed New CCTV Cameras to Monitor River Situation

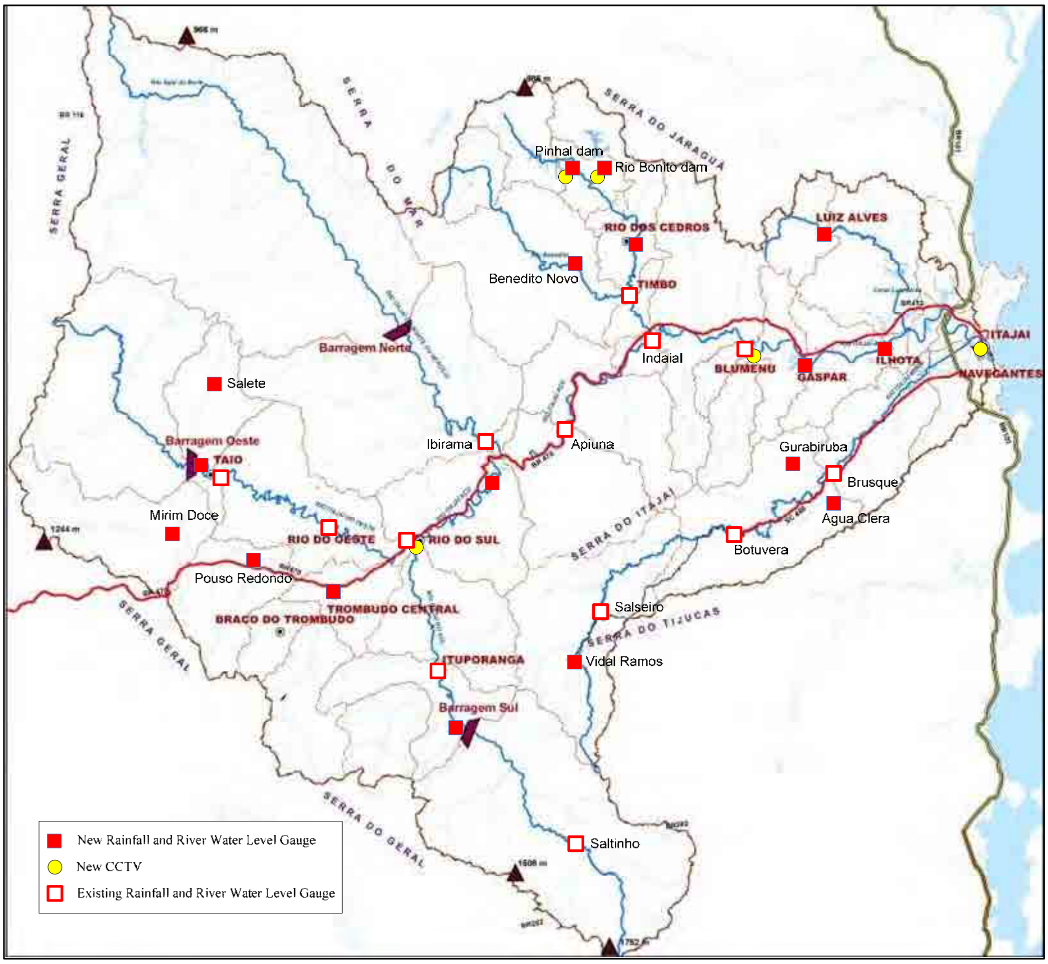
Presently, Defesa Civil and SDS have no regional offices in the Itajai River basin. As the administrator of river management and disaster prevention, they have to implement it far from Florianopolis. Therefore, installation of CCTV at Rio do Sul, Blumenau and Itajai City is proposed to monitor river situation from distance.

Table-3.4.1 Target City for FFWS and Gauging Stations

No.	Name of River	Name of Sta.	Target City for FFWS: Existing warning water level	Target City for FFWS: Target Location for Warning	Target City for FFWS: Forecasting & Warning	Gauging Station: Existing Gauging Station (FURB/CEOPS) (Require for replacement/up-grade): Equipment		Gauging Station: Existing: Present Condition	Gauging Station: Proposed Gauging Station: Equipment		Gauging Station: Proposed: Remarks
1	Rio Itajai	Rio do Sul	○	○	Reviewing existing warning level/Establishing flood forecasting formula	○	R/W	Operational	○	CCTV	Monitoring river from Florianopolis
2	Rio Itajai	Bumenau	○	○		○	R/W	Operational	○	CCTV	Monitoring river from Florianopolis
3	Rio Itajai	Itajai	○	○		City government already installed 8 water level gauges			○	CCTV	Monitoring river from Florianopolis
4	Rio Itajai do Sul	Ituporanga	○	○	Reviewing existing warning water level	○	R/W	Up-grade of transmission system			
5	Rio Itajai do Sul	Vidal Ramos	○	○					○	R/W	Warning for Vidal Ramos
6	Rio Itajai do Oeste	Taio	○	○		○	R/W	Up-grade of transmission system			
7	Rio Itajai do Oeste	Rio Oeste	○	○		○	R/W	Replacement of Equipments			
8	Rio Trombudo	Trombudo	○	○					○	R/W	Warning for Trombudo
9	Rio Itajai do Norte	Ibirama	○	○		○	R/W	Up-grade of transmission system			
10	Rio dos Cedros	Rio dos Cedros	○	○					○	R/W	Warning for Rio dos Cedros
11	Rio dos Cedros	Timbo	○	○		○	R/W	Operational			
12	Rio Benedito	Benedito Novo	○	○					○	R/W	Warning for Benedito Novo
13	Rio Itajai	Apiuna	○	○		○	R/W	Replacement of Equipments			
14	Rio Itajai	Indaial	○	○		○	R/W	Replacement of Equipments			
15	Rio Itajai	Gasper	○	○					○	R/W	Warning for Gasper
16	Rio Itajai	Ilhota	○	○					○	R/W	Warning for Ilhota
17	Rio Itajai Mirim	Brusque	-	○	Establishing warning water level	○	R/W	Replacement of Equipments			
18	Rio Itajai do Oeste	Mirim Doce	-	○					○	R/W	Warning for Mirim Doce
19	Rio Itajai do Oeste	Salete	-	○					○	R/W	Warning for Salete
20	Rio Itajai do Oeste	Pouso Redondo	-	○					○	R/W	Warning for Pouso Redondo/Rio do Sul due to flood by housing development
21	Rio Itajai Mirim	Agua Clera	-	○					○	R/W	Warning for Agua Clera
22	Rio Itajai Mirim	Gurabiruba	-	○					○	R/W	Warning for Gurabiruba
23	Rio Itajai do Sul	Saltinho	-	-	-	○	R/W	Replacement of Equipments			
24	Rio Itajai do Sul	Sul Dam	-	-					○	R/W	Flood forecasting at Rio do Sul/ Monitoring discharge from dam to protect Ituporanga
25	Rio Itajai do Oeste	Oeste Dam	-	-					○	R/W	Flood forecasting at Rio do Sul/ Monitoring discharge from dam to protect Taio
26	Rio Itajai do Norte	Barra da Prata	-	-		○	R/W	Replacement of Equipments			
27	Rio dos Cedros	Pinhal Dam	-	-					○	R/W, CCTV	Monitoring discharge from dam to protect Timbo
28	Rio dos Cedros	Rio Bonito Dam	-	-					○	R/W, CCTV	Monitoring discharge from dam to protect Timbo
29	Rio Luiz Alves	Luiz Alves	-	-					○	R/W	To obtain hydrological data of Luiz Alves river basin for flood forecasting model
30	Rio Itajai Mirim	Salseiro	-	-		○	R/W	Replacement of Equipments			
31	Rio Itajai Mirim	Botuvera	-	-		○	R/W	Replacement of Equipments			
TOTAL			16	22		14			19	16 R/W & 5 CCTV	

R/W: Rainfall and Water level gauge as one set

Source: JICA Study Team



Source: JICA Study Team

Figure 3.4.1 Location Map for Proposed Gauging Station and CCTV

3.5 Proposal for Strengthen Systems for FFWS

The following 8 steps shall be applied to maintain and upgrade the related systems for strengthening FFWS.

(1). Improvement of existing gauging stations (changing observation equipments)

(2). Installing of additional gauging stations (rainfall and water level gauge)

(3). Improvement of network system (internet) including additional monitoring stations

(4). Extending server and establishing database of the central station (Florianopolis)

(5). Analyzing and improvement of flood forecasting (review of water level correlation formula)

(6). Installation of monitoring system at Monitoring Station (Rio do Sul, Itajai City)

(7). Installation of monitoring system at Central Station (Florianopolis City)

(8). Improvement of flood warning system and evacuation announcement

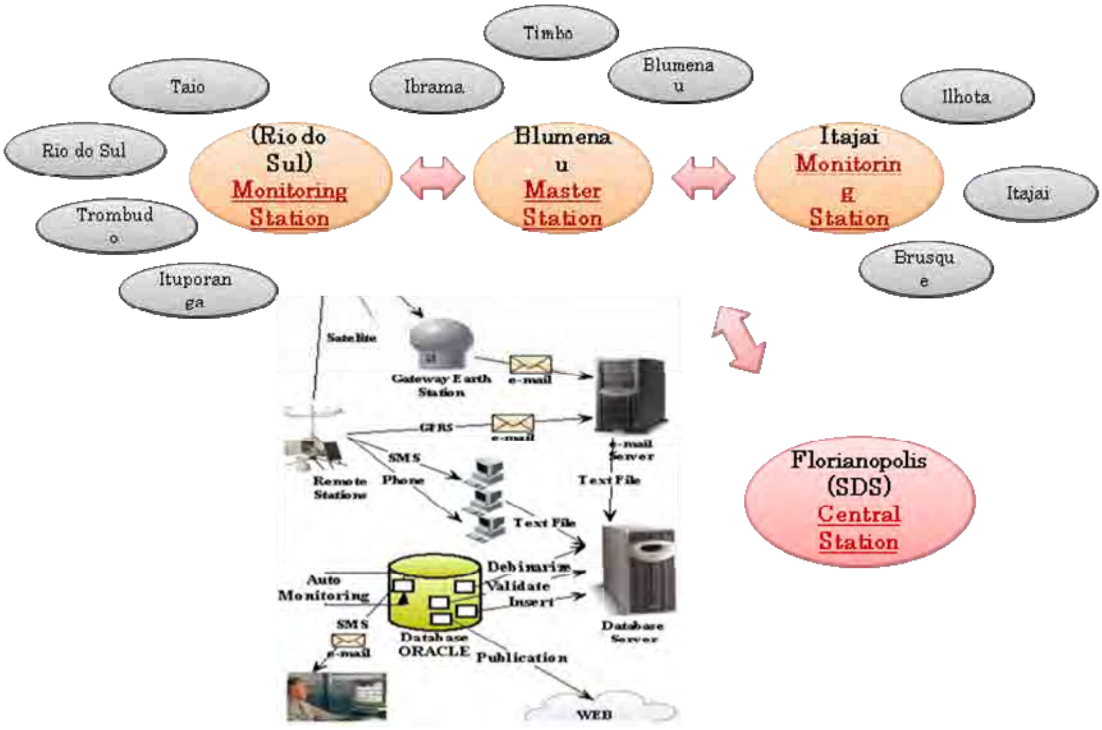
3. 5.1 Data Observation, Transmission and Monitoring System

The data of rainfall and water level from the proposed 30 gauging stations is transmitted to the server of Florianopolis' centre station through by Email of GPRS and saved to a data base. The saved data will be transmitted to the monitoring stations of Defesa Civil at Rio do Sul, Blumenau and Itajai City through the internet. The pictures of Itajai River taken in every minute by CCTV at Rio do Sul, Blumenau and Itajai City are transmitted to the Florianopolis' centre station through the internet and monitored. The schematic diagram for the proposed data network system is shown in Figure 3.5.1 and the further details of each system are as followings.

- The data transmission system shall be changed to package switching type, which transfers data from the Link Radio to the cellular-phone. This enables to send the observation data directly to the river engineer by E-mail every 10 minutes. This system should secure electric energy source with the solar panel. There is no needs of any auxiliary equipment like repeaters at the transmission points; only one stem enables to install the rainfall, river water level gauge, solar panel, battery, data register (data logger) and a telephone center.
- Furthermore, the digital system changes from GSM (Group Special Mobile) 2G (Mobile Telephone system of Second Generation) to GPRS (General Packet) 2.5G which this enables to strengthen a stable communication and efficiency of the broadband use and to increase data communication. Install or modify GPRS and/or GSM transmission system and more reliable data shall be sent through emails to the monitoring stations at real time.
- Regarding recovery of the surveillance camera for monitoring, nowadays it is possible to acquire a highly precise and light CCTV (Closed Circuit TV) at low prices. Therefore, in Rio do Sul, Blumenau and Itajaí, surveillance cameras will be installed to capture the flood

situation in real time.

- The data sent through E-mail by GPRS and/or GSM and stored into a server at the central station of Florianopolis. The stored data is then forwarded to the monitoring stations in Rio do Sul, Blumenau and Itajai through the Internet. The pictures taken every one minute at CCTV in Rio do Sul, Blumenau and Itajai city can also be sent to each city's civil defense for a real time view through the master station internet.



Source: JICA Study Team

Figure 3.5.1 Data Observation, Transmission and Monitoring System for FFWS

3. 5.2 Proposal for Warning System and Data Management System

To improve warning system, a monitoring station should be established at Municipal Defensa Civils. one must collect data from each gauging station in order to build a database and flood forecast. In addition, surveillance equipment will be placed at monitoring stations in the Defensa Civil offices in Rio do Sul, Blumenau and Itajaí and this enables to monitor using the Internet. In Florianopolis, the facilities for monitoring will be installed as central station and the floods conditions can be captured by the river engineer

The flood warning must be announced immediately to the residents in the target cities by radio, TV, the Internet and an electronic board, which shall be set up within the cities. In hazardous areas, the patrol by the civil defense and real-time monitoring of the flood situation by CCTV enable to inform CD Municipal Council to announce earlier evacuation warning. In addition to this, sirens shall be set up in the cities and used together with the other systems; this is to urge residents who live in hazardous areas to evacuate immediately.

CHAPTER 4 FEASIBILITY STUDY FOR STRENGTHEN EXISTING FFWS

4.1 Verification of Existing Flood Forecasting

Currently in the Itajai River Bain, flood forecasting under FFWS is only conducted in Blumenau (by FURB) and Rio do Sul (by City Defesa Civil), while the rest of cities don't conduct any flood forecasting. Even in Blumenau and Rio do Sul, the flood forecasting is not utilized for warning and evacuation activities systematically. Especially, the forecasting for Rio do Sul is still in "Trial and Error" stage.

It is recommended to establish FFWS at least for three major cities, (1) Rio do Sul, (2) Blumenau and (3) Itajai city. In this section, the existing flood forecasting formula for the two cities, Rio do Sul and Blumenau shall be verified and the recommendations for improvement of the system shall be proposed. For Itajai city which is one of the most important cities along the Itajai River, the flood forecasting method shall be proposed in this section.

For other cities and towns, it is important to maintain that the present warning system using of the warning level and these cities shall be adopted using existing gauging stations and additional gauging stations which are proposed under the study.

4.1.1 Verification of Existing Flood Forecasting Formula for Rio do Sul

The present formula for forecasting flood water level at Rio do Sul was developed by Defesa Civil in Rio do Sul City. The formula was developed by the rainfall correlation method using the rainfall at Rio do Sul (C.A.:5,042 km2), Oeste Dam (C.A.: 1,042 km2, 81km upstream of Rio do Sul) and Sul Dam (C.A.: 1,273km2, 43km upstream of Rio do Sul). The formula was developed in a trial stage therefore presently the flood/ evacuation warning shall not be implemented using the forecasted water level computed by this formula.

i) The daily average discharge shall be computed by the daily average water level using H – Q Curve at Rio do sul. The water level at Rio do Sul is ocularly observed at 7am and 5 pm daily and the observation of flood water level is conducted hourly. The daily average water level means the average of 2 times of water level for ordinary days and maximum 24 times of water level for during the flood.

$$Q = 44.7757\ (H - 0.235)\ 1.48789 \quad \cdots\cdots\cdots\cdots(1)$$

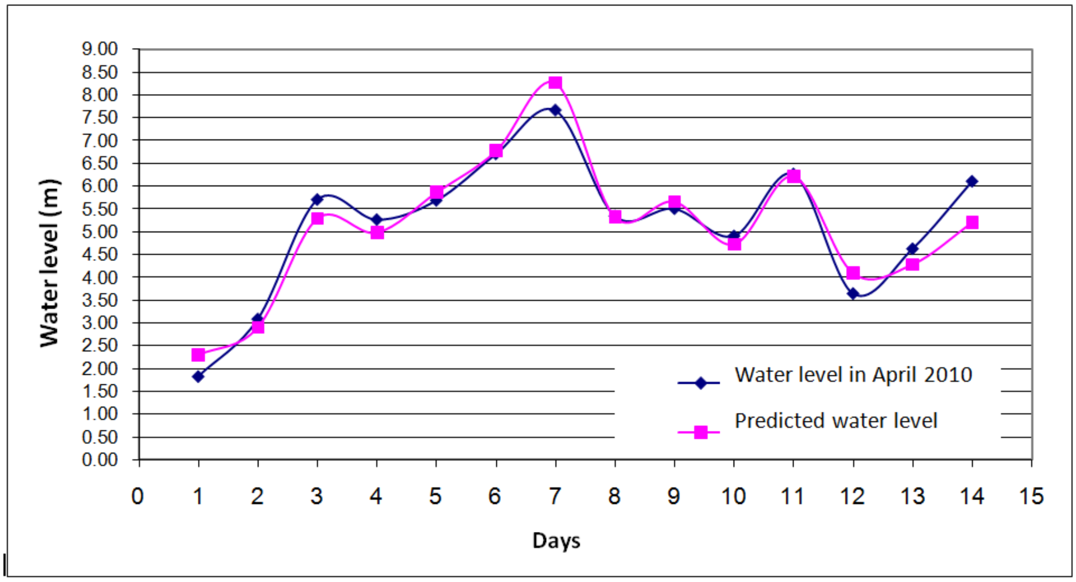
ii) The fluctuation discharge (ΔQ) shall be computed by daily rainfall data at aforementioned 3 locations using the following formula. The fluctuation discharge (ΔQ) means the different discharge volume between that day and next day.

$$\Delta Q = 6.07 + 1.66 * (\text{Rainfall at Rio do Sul}) + 2.51 * (\text{Rainfall at Sul Dam})$$
$$+ 0.45 * (\text{Rainfall at Oeste Dam}) \quad \cdots\cdots\cdots\cdots(2)$$

iii) The daily average discharge for next day (Q2) shall be computed to add the fluctuation discharge (ΔQ) to the daily average discharge for that day.

iv) The daily average water level for the next day shall be computed using aforementioned H-Q Curve formula (1).

The forecasted daily average flood water level at Rio do sul was computed by using above method regarding the flood on April 2010 and the comparison between the actual daily average flood water level and forecasted daily average flood water level is shown in Figure 4.1.1. The forecasted daily average water level is 30 cm in difference from the actual and it is quite accurate.



Source: Rio do Sul, Defesa Civil

Figure 4.1.1 Comparison between Actual Daily Average Water Level and Forecasted Daily Average Water Level at Rio do Sul

However:

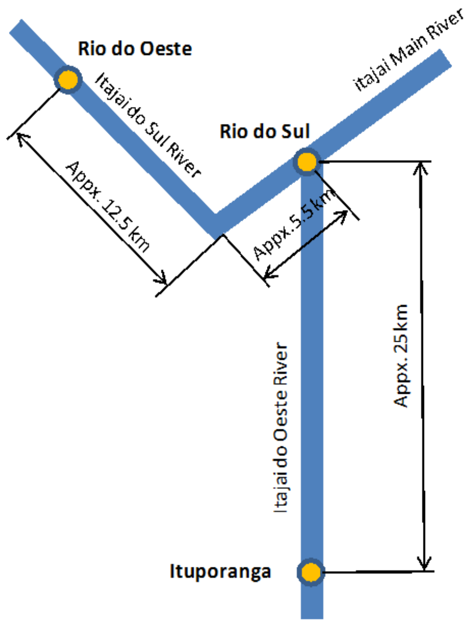
- The forecasting method is only applicable for daily average but not allowed to compute hourly base. Therefore the forecasting method is not appropriate to utilize for the warning system. It can be used only for the warning announcement for the next day.
- The definition of the daily average water level is not clear for the ordinary day and the flood time. In case of the flood time, the daily average water level could not be computed until the end of day (12pm) therefore the water level for the next day could not be forecasted until midnight. In this case the warning for the next day maybe difficult due to preparation of the warning after midnight.

Due to those above mentioned matters, the following comments are well considered for the future.

- In future, the correlation method should be applied for the flood forecasting at Rio do Sul as an ordinary method and it must be hourly forecasting.
- However Rio do Sul has 2 flood control dams in upstream. In case of low coefficient of correlation among 2 stations in downstream of the dams and Rio do Sul, the unsteady flow-runoff analysis would be recommended including the mechanism of control gates at dams.

- In case of the modeling of the flow-runoff analysis, in addition to the existing rainfall gauging stations, the system for transmitting data of dam discharge including the gate control status to the city/ provincial Defensa Civil by the DEINFRA as the dam operator must be set up and implemented essentially.

Herein the flood forecasting at Rio do Sul using the water level correlation method is tried and shown as followings. The forecasting flood water level at Rio do Sul is computed based on the correlation of the water level (ARIMAX Model Method) at Rio do Sul, Ituporanga and Rio do Oeste. The schematic diagram for gauging stations of Rio do Sul, Ituporanga and Rio do Oeste is shown in Figure 4.1.2. Ituporanga and Rio do Oeste are approximately located in 25 km along Itajai do Oeste River and 18 km along Itajai do Sul River upstream respectively from Rio do Sul City.



Source: JICA Study Team

Figure 4.1.2 Schematic Diagram for Gauging Stations

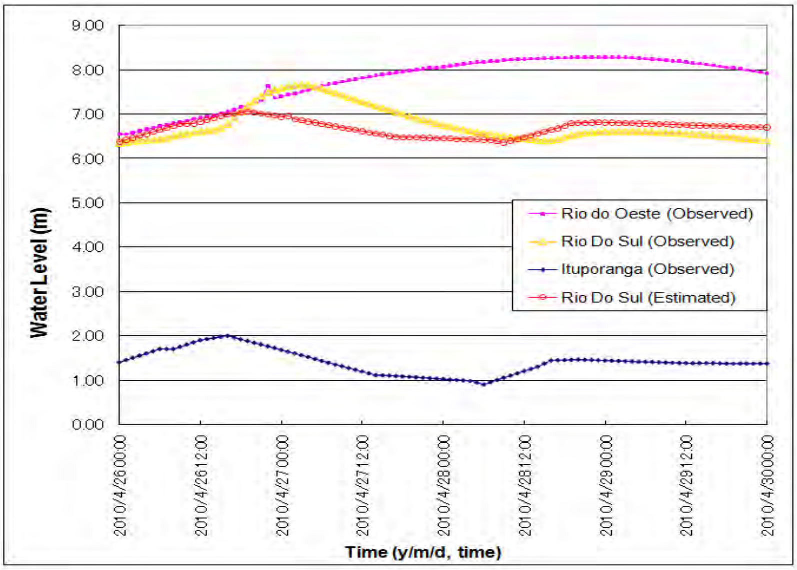
New multiple correlation formula is created using water level at Rio do Sul, Ituporanga and Rio do Oeste and the accuracy of forecasted water level used by the new multiple correlation formula shall be confirmed with the actual flood water level at Rio do Sul.

Table 4.1.1 Summary of Flood Forecasting Formula by Multiple Correlation at Rio do Sul

Method	Gauging Sta. 1	Gauging Sta. 2	Coefficient of Correlation	Forecasting Formula
Multiple Correlation	Using 3 hours before actual water level at Ituporanga considered arrival time	Using 3 hours before actual water level at Rio do Oeste considered arrival time	0.445	Y = 0.7908315 * (Water Level at Ituporanga) + 0.1460886 * (Water Level at Rio do Oeste) + 4.44770646

Source: JICA Study Team

The flood forecasting by the multiple correlation among Rio do Sul, Ituporanga and Rio do Oeste using the flood in April 2010 is as shown in Figure 4.1.3 and the correlation coefficient is R = 0.445 which is quite low.



Source: JICA Study Team

Figure 4.1.3 Comparison between Actual and Water level Forecasted by Multiple Correlation Formula at Rio do Sul;

Here the arrival time shall be computed by using the Kraven formula as shown as the following.

T (hr) = (1/ 3,600) x (L/W)

Where T (hr): Flood arrival time from upstream station to Rio do Sul

L (m): Channel Length from upstream station to Rio Do Sul

W (m/s): Flood velocity (refer to the following table)

I	1/100 <	1/100 ~ 1/200	< 1/200
W	3.5 m/s	3.0 m/s	2.1 m/s

I = H (m) / L (m)

Where, H (m): Difference in Elevation

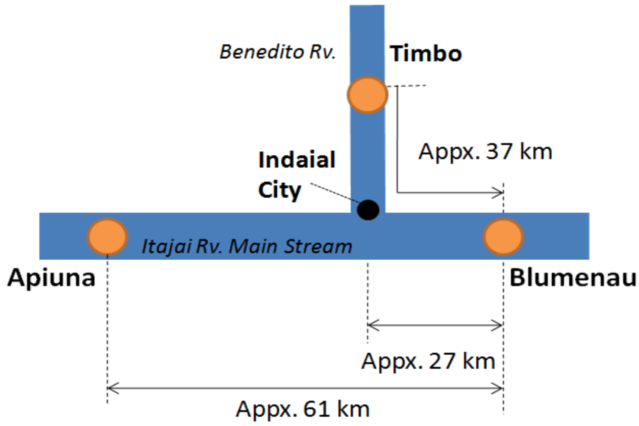
L (m): Channel Length

The river slope for Ituporanga and Rio do Oeste is 1/800 and 1/5,000 respectively therefore the flood velocity for Ituporanga and Rio do Oeste iare are 2.1 m/s. The arrival time shall be computed as follows:

- Ituporanga: T (hr) = (1/3,600) x (25 x 1,000/ 2.1) = 3.3 hours
  → 3.0 hours
- Rio do Oeste: T (hr) = (1/3,600) x (18 x 1,000/ 2.1) = 2.4 hours
  → 3.0 hours

4.1.2 Verification of Existing Flood Forecasting Formula for Blumenau

The forecasting flood water level at Blumenau is computed based on the correlation of the water level (ARIMAX Model Method) at Blumenau, Apiuna and Timbo. The schematic diagram for gauging stations of Blumenau, Apiuna and Timbo is shown in Figure 4.1.4. Apiuna and Timbo are approximately located in 61 km along Itajai River and 37 km along Benedito River upstream respectively from Blumenau.



Source: JICA Study Team

Figure 4.1.4 Schematic Diagram for Gauging Stations

The water level correlation formula for the flood forecasting at Blumenau was developed by the FURB/ CEOPS in 1990's as follows.

$$Y(t)=1.98063 \times Y(t-1) - 0.98506 \times Y(t-2) + 0.009200 \times u1(t-4) - 0.08732 \times u1(t-5) + 0.01806 \times u2(t-4) - 0.01411 \times u2(t-5) + 0.03083 \quad \cdots\cdots (3)$$

where,

Y(t):	Forecasted water level at time of (t) hour at Blumenau station
y(t-1):	Water level at time of (t-1) hour at Blumenau station
y(t-2):	Water level at time of (t-2) hour at Blumenau station
u1(t-4):	Water level at time of (t-4) hour at Apiuna station
u1(t-5):	Water level at time of (t-5) hour at Apiuna station
u2(t-4):	Water level at time of (t-4) hour at Timbo station
u2(t-5):	Water level at time of (t-5) hour at Timbo station

The evacuation manual of SC states says that the evacuation shall be announced 3 hours before the flood in consideration of priority people to be protected such as aged people, handicapped people, children and tourists and so on.

The required water level at Blumenau for calculating of forecasted flood water level using the proposed formula is the 1 hour before and 2 hours before the flood therefore the following steps shall be conducted to compute the forecasted water level after 3 hours from present.

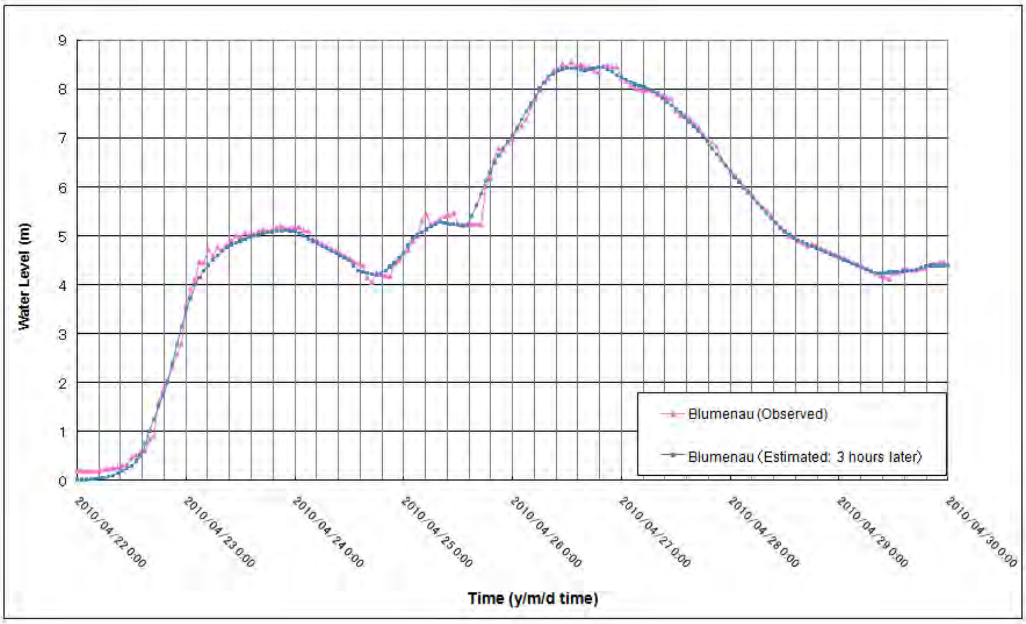
i) To forecast water level after 1 hour at Blumenau using present and 1 hour before water level at Blumenau

ii) To forecast water level after 2 hours at Blumenau using forecasted water level after 1 hour and present water level at Blumenau

iii) To forecast water level after 3 hours at Blumenau using forecasted water level after 1 hour and after 2 hours at Blumenau

The formula was developed approximately 20 years ago. This verification shall be conducted using the flood data in April 2010.

The following Figure 4.1.5 is shown the comparison between the actual flood water level at Blumenau and the forecasted flood water level (after 3 hours at Blumenau) computed by the proposed formula. The forecasted water level is quite similar to the actual one.



Source: JICA Study Team

Figure 4.1.5 Comparison between Actual and Water Level Forecasted by Present Formula at Blumenau

In addition to above verification, new multiple correlation formula is created using water level at Blumenau, Apiuna and Timbo and the accuracy of forecasted water level used by the new multiple correlation formula shall be confirmed with the actual.

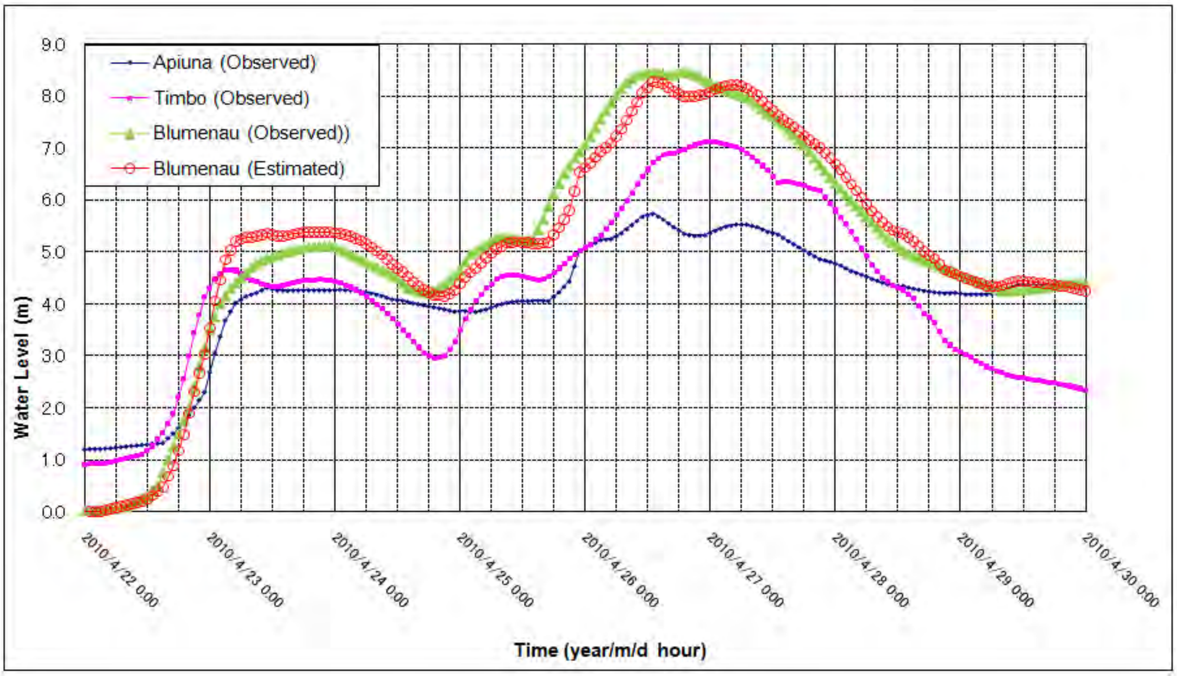
Table 4.1.2 Summary of Forecasting Formula by Multiple Correlation at Blumenau

Method	Gauging Sta. 1	Gauging Sta. 2	Coefficient of Correlation	Forecasting Formula
Multiple Correlation	Using 6 hours before actual water level at Apiuna considered arrival time	Using 3 hours before actual water level at Timbo considered arrival time	0.991	Y = 1.101225 * (Water Level at Apiuna) + 0.56226 * (Water Level at Timbo) - 1.83067

Source: JICA Study Team

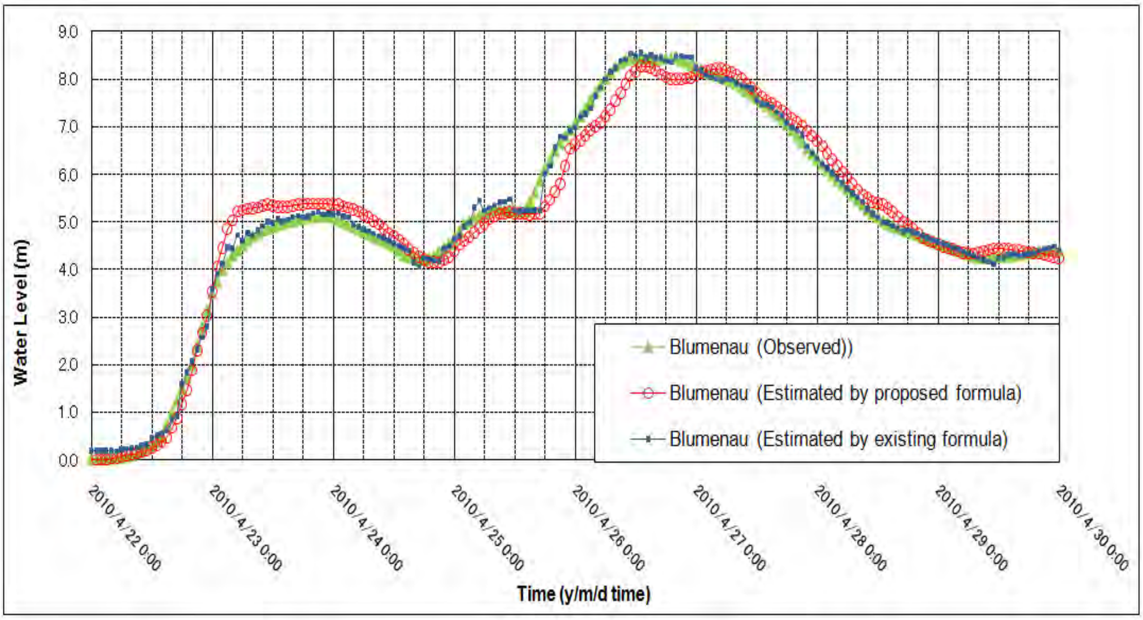
The forecasting formula by the multiple correlation among Blumenau, Apiuna and Timbo using the flood in April 2010 is quite accurate as shown in Figure 4.1.6 and the correlation coefficient is R=0.991 but not as high as the present forecasting formula (see Figure 4.1.7).

Therefore the present forecasting formula at Blumenau is still maintaining accuracy and even now it is applicable to use the present forecasting formula for forecasting flood water level at Blumenau. In future, whenever the flood comes the formula should be checked and updated if necessary.



Source: JICA Study Team

Figure 4.1.6 Comparison between Actual and Water level Forecasted by Multiple Correlation Formula at Blumenau



Source: JICA Study Team

Figure 4.1.7 Comparison among Actual, Present Formula and multiple Correlation Formula

4.1.3 Flood Forecasting System at Itajai City

The Itajai City has very important role for the economy in the Itajai River Basin and the flood forecasting system should be organized as same as Rio do Sul and Blumenau. However the Itajai City is located at the estuary and the city is seriously affected by the tide and influenced by the tributary of Mirim River. Therefore the forecasting of flood water level at Itajai city is very complicated and difficult.

The Defesa Civil at Itajai City has already installed new 8 water level gauges around the city in February 2011 and they are planning to set up new system for the flood warning and evacuation announcement but they has presently hard time to set up the flood forecasting system since the information in upstream such as Blumenau and Brusque is not transmitted to Itajai City.

Other hand, the Defesa Civil, SDS and EPAGRI in SC State are planning to develop the flood forecasting modeling for the flow-runoff analysis and inundation analysis but it needs more time to get accurate forecasting system using sufficient data collected by the proposed gauging stations under this FFWS. However the development of the flood forecasting modeling should be continued. Meantime the flood warning and evacuation announcement should be implemented by the SC State due to the flood water level of upstream such as Blumenau and Brusque.

4.2 Selection of Equipments and Facilities

The Master plan which is considered the additional gauging stations and necessary transmission facilities for flood forecasting and warning system are composed as followings:

(1). Automatic rainfall gauge (tilting-siphon type)

(2). Automatic river water level gauge (radar type)

(3). Data logger (magnetic tape recorder)

(4). Solar panel and battery (save electricity)

(5). Converter to send monitored data (global mobile telecommunication system: GPRS)

(6). Receiving system at the central station at Florianopolis (server and database)

(7). Connection network between gauging stations (Internet/ civil defense at each city)

(8). Connection network for the central station (Internet/Florianopolis)

(9). Real-time flood Information system by the Internet (web site)

Observation equipments were chosen with following considerations and the monitoring components were listed in Table 4.2.1.

(1). Rainfall:

There are tipping-bucket type and tilting-siphon type. The chosen rainfall gauge for the Itajai River is tilting-siphon type with 200mm diameter brass-muzzle with sharp edge on the top, which collects rain water and water get stored in triangle siphon. Once water reach up to 0.5 mm, the siphon tilts.

(2). River water level gauge

There are pneumatic, ultrasonic and radar types. The pneumatic type has difficulty in maintenance as it gets affected by erosion and sedimentation of the river bed. On the other hand, the ultrasonic and radar types are easier in its maintenance and management. Especially the radar type does not affected by temperture and wind as well as low electric consumption and reasonable. Therefore the radar type is applied to the proposed basin.

(3). Data logger

This can record accumulative rainfall and river water level simultaneously at each interval and send the data to the central station every 10 minutes via E-mails by utilizing CDMA2000 1X radio transmission technology.

(4). Data transmission system

Rainfall and river water level data monitored at civil defense of each city get sent through the central station (Florianopolis) to the monitoring stations in Rio do Sul, Blumenau and Itajai.

(5). Flood forecasting and warning system

The monitored data that are sent to the central station and stored in database. Based on information such as river water level of the upstream, flood forecasting is implemented in each city as following.

iv) In Rio do Sul, it is based on information of Sul dam, Oeste dam, tributary rivers in Taio and mountain ranges and rainfall forecasting from whether report.

v) In Blumenau, it is based on river water level of Rio do Sul, Apiuna, Indaial and Timbo.

vi) In Itajai, it is based on river water level of Blumenau, Salseiro and Brusque as well as tidal level from the Itajai bay.

When the flood water level is forecasted and analyzed by the river water level correlation, the earlier warning is sent to each hazardous area through CD Municipal Council and the civil defense and urges immediate and safe evacuation of the residents.

The records of rainfall and river water level from each gauging station are stored in the database at the central station (Florianopolis). The records are also sent to civil defense at each city to be monitored. The following equipment is required for each monitoring center (civil defense of Rio do Sul, Blumenau and Itajai).

(6). Central station (Florianopolis)

Data transmission control Server (DTCS) to receive the monitored records, data Base (DB) server and WEB server are required. In addition, installation of DB, set up of network, system engineer, network engineer and designer for creation of maps are also needed. As equipments, three computers and radar printers and two of 52’ inches-monitors (LDC) are needed. Essential items such as A/C and telephone for DB are installed to the Florianopolis and maintained as reference image of Figure 4.7.1.

(7). Monitoring center (Rio do Sul, Blumenau and Itajai)

Two computers, two 52’ inch monitor (LDC) and one radar printer are needed. The system engineer is required to install these items as well as the network facilities. It is also considered that the central station at Florianopolis also requires the same facilities to monitor the flood situation in real-time.

(8). Rainfall and river water level gauge stations

Aluminum pole (250 mm diameter) with a solar panel as an electronic battery is installed. This pole has a rainfall gauge on top and a water level gauge at bottom which is set at a bridge and they are connected by a cable line to a data logger. The monitored records are sent to the Internet by the GPRS transmission via E-mails every 10 minutes and the records are saved at the central station server.

(9). CCTV

Two CCTV cameras need to be installed at each city (Rio do Sul, Blumenau and Itajai) where can monitor flood situation. CCTV needs to be connected to digital disk recorder. Pictures are taken every one minute and sent through the Internet by GPRS transmitter and then received at the central station server. These pictures are projected on monitoring screens.

Table 4.2.1 Monitoring Components at The Central Station

Monitor	Display	Detailed
Web Site	Relief Map for river	It is a river map at the Itajai Basin. When clicked at an observation point on the map, the detailed monitoring data is confirmed on the display.
	Diagram	The flow of the Itajai River and the location of monitoring stations are showed in diagrams.
Graphic Display	Daily rainfall	The rainfall graph for 24 hours at each monitoring station
	Hourly rainfall	The Hydro Graph for hourly rain fall at each monitoring station
	Hourly river water level	The continuous graph for hourly river water level at each monitoring station
Table Display	Hourly rainfall	Table for hourly rainfall at each monitoring station
	Hourly river water level	Table for hourly river water level at each station
Video Display	CCTV	The monitors at the Master station and Monitor station displays the photos that are transmitted every minute by the set CCTV at Rio do Sul, Blumenau and Itajai.

Source: JICA Study Team

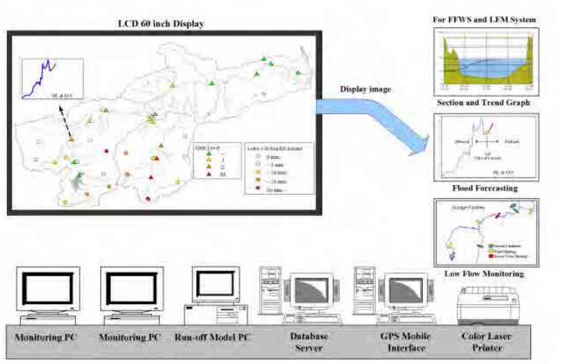


Figure 4.2.1 Monitoring Center and Sample Photo

4.3 Cost Estimate for Proposed Master Plan

The cost estimation for strengthening FFWS is shown as follows.

i) Equipments cost including installation

ii) System development cost including consultant services

The summary of new gauging station is shown in Table 4.3.1 and the specification of equipments is shown in Table 4.3.2 as follows.

Table 4.3.1 Quantity and Location of New Gauging Stations

Station		Rainfall	Water Level	CCTV	Warning Tower	Warning Siren
1	Rio do Sul			1	3	3
2	Blumenaur			1	3	3
3	Itajai			1	3	3
4	Vidal Ramos	1	1			3
5	Trombudo	1	1			3
6	Rio dos Cedros	1	1			3
7	Benedito Novo	1	1			3
8	Gasper	1	1			3
9	Ilhota	1	1			3
10	Mirim Doce	1	1			3
11	Salete	1	1			3
12	Pouso Redondo	1	1			3
13	Agua Clera	1	1			3
14	Gurabiruba	1	1			3
15	Sul Dam	1	1			3
16	Oeste Dam	1	1			3
17	Pinhal Dam	1	1	1		3
18	Rio Bonito Dam	1	1	1		3
19	Luiz Alves	1	1			3
Total		**16**	**16**	**5**	**9**	**57**

Source: JICA Study Team

In addition to above, the Master Station at the Defesa Civil at Florianopolis City shall be established as described in Chapter-3 and the three (3) monitoring Centers shall be established at the Defesa Civil at Itajai City, Blumenau City and Rio do Sul City.

Table 4.3.2 Specification of Equipments

Item	Specification	Nos.
Telemetry rainfall gauge	Radar type OTT/RLS model	16
Telemetry water level gauge	Radar type OTT/RLS model	16
Software	Automatic E-mail system	16
Solar panel/battery	Polycrystalline silicon type 12V	16
Data transmission (GPRS)	For data transmission 2.0Mbyte	16
CCTV	In-line transmission CCD	5
Data transmission (GPRS)	For CCTV 2.0Gbyte	5
Database	Memory capacity for 20 years record	5
Monitoring panel	52 inches display LDC	5
Electric bulletin board (warning tower)	5.0 m width and 3.0 m height Rio do Sul (3), Blumenau (3), Itajai (3)	9
Alarm (siren)	Large disaster warning siren, for external moisture	57

Source: JICA Study Team

The cost estimate for new gauging stations including the Master Station and Monitoring Centers is shown in Table 4.3.3.

Table 4.3.3 Cost Estimate for Proposed Gauging Stations

Item	Nos.	Unit Price (R$)	Amount (R$)
Telemetry rainfall gauge	16	5,300	84,800
Telemetry water level gauge	16	6,800	108,800
Software	16	-	-
Solar panel/battery	16	17,000	272,000
Data transmission (GPRS)	16	20,000	320,000
CCTV	5	26,500	128,000
Data transmission (GPRS)	5	20,000	100,000
Database	5	120,000	600,000
Monitoring panel	5	8,000	40,000
Electric bulletin board (warning tower)	9	30,000	270,000
Alarm (siren)	57	2,000	114,000
Monitor station	3	80,000	240,000
Central station	1	257,000	257,000
Total			**2,534,600**

Note: The cost for development of programs regarding the telemeter system is not included in the above cost.

Source: JICA Study Team

The river register book is needed to prepare for total river management together with the proposed gauging stations above. The river register book must include total 350 cross section survey (1.0 km interval) along Itajai Main River and major tributaries, cross section survey for existing water level gauges and km piles (KP) along the river. The work for river register book shall also include the development of modeling for the flood runoff analysis.

The purposes for the consultant services regarding the system development are as follows.

vii) Tendering for procurement of equipments

viii) Training for concerned organizations

ix) Development for flood forecasting modeling and simulation modeling

x) Establishment for H-Q curves for all water level gauging stations including existing and proposed stations

xi) Study for preparation of operation system for Bonito and Pinhal Dams

xii) Study for preparation of operation system for Oeste and Sul Dams

The cost for the consultant services for system development is shown in Table 4.3.4.

Table 4.3.4 Cost for Consultant Services for System Development

Item	M/M	Unit Price (R$)	Amount (R$)
Project Manager	10	60,000	600,000
Hydrologist	8	50,000	400,000
River Engineer	8	50,000	400,000
Program Engineer	4	50,000	200,000
Telecommunication Engineer	5	50,000	300,000
System Engineer	5	50,000	300,000
Network Engineer	5	50,000	300,000
Database Engineer	5	50,000	300,000
Supporting Staff	10	20,000	200,000
Total	-	-	**3,000,000**

Source: JICA Study Team

The direct cost for the propose FFWS is shown in Table 4.3.5.

Table 4.3.5 Direct Cost for Strengthening Existing FFWS

Item	Amount (R$)
1. Equipment cost	2,534,600
2. River survey for a river inventory book	1,500,000
3. System development	3,000,000
Total	**7,034,600**

Source: JICA Study Team

4.4 Implementation Schedule

In order to strengthen the existing monitoring network, the additional monitoring station will be carried out to set up the rainfall and river water level gauge and to improve the data transmission system. As well as the central station which is located at Florianopolis will be established in the SDS Office in order to monitor in real time the flood in Itajai River. The items of building up for the proposed FFWS are listed in Table 4.4.1.

Table 4.4.1 Implementation Schedule for Proposed FFWS

Item	First year				Second year				Remark
Additional station (Nos.16) for Monitoring									
Setting up CCTV (Nos.5) (Rio do Sul, Blumenau, Itajai)									Including Bonito and Pinhal dams
Commissioning (Data Transmission)									
Flood Forecasting Analysis (Rio do Sul, Blumenau, Itajai)									
Monitoring Station (Rio do Sul, Blumenau, Itajai)									
Build-up of Central Station (Florianopolis)									
Calibration of Monitoring									
Preparation for River Log Book (Cross Section Survey)									
Training of Engineer									

Source: JICA Study Team

CHAPTER 5 RECOMMENDATIONS

5.1 Recommendations for Future

i) The monitoring stations related to the FFWS must be increased for the future in order to apply such as the X band monitoring of Doppler radar and the weather forecasting using by the numerical model of satellite information.

ii) It is important to organize the river management system and one governmental agency should be responsible for the total management of the Itajai River Basin. The river inventory shall also be arranged properly by the river management unit. A flood information center should be also set up inside of the organization for “River Management” in order to raise the awareness of the flood management.

iii) The evacuation manuals have been exists only in Rio do Sul, Blumenau and Itajai cities. In these manual, the high risk areas are listed, however, hazard maps are not developed. In addition, the manual has no guidelines for flood prevention activities such as using of sand bags to stop water. Therefore, the river management unit shall improve the existing evacuation manuals and develop hazard maps for more appropriate evacuation activities. Flood prevention teams shall be responsible for conducting urgent repairing works in order to prevent and mitigate the flood damages.

iv) The river management unit has to consider the river maintenances including preparing the river inventory book for the river improvement and repair at hazardous areas. The hazardous area must be reflected the land use plan of each city and the residents must be relocated from the hazardous area.

v) In order to implement a safe urban planning, the master plan for land use must be prepared in considering of the social conditions such as living condition, industrial component and population density and the natural conditions such as hydraulics and geology.

vi) The residents in hazardous area have vested rights therefore the resettlement from the hazardous area could not be implemented forcibly and it is also difficult to look for resettlement area. Therefore the implementation of the regulation of land development may have difficulty so the compulsory execution may need to ask the residents for resettlement from the hazardous area. Execution of real flood prevention activities is quite difficult since the data of hazardous area is not available due to absence of the river management unit. Civil Defense is implementing the training of flood prevention activities for during and after the floods but not for before the floods.

Supporting Report (E)
Water Storage in Paddy Fields

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

FINAL REPORT

VOLUME III : SUPPORTING REPORT
ANNEX E : WATER STORAGE IN PADDY FIELDS PROJECT

Table of Contents

Page

Tables

Page

Figures

Page

CHAPTER 1 PROJECT CONTEXT OF THE WATER STORAGE IN PADDY FIELD

1.1 Background

(1) Mitigation Function of Water Storage in Paddy Fields

The paddy fields in the basin, accounting an area of 26.295 ha, distribute in all basins, and mainly locate in the higher basin between the Taió to Rio do Sul, Timbó area, and the lower basin among Gaspar to Itajaí. In spite of the paddy fields does not exceed 1, 8% of the total basin, the rice field has an important element for the flood regulation, being distributed in the margins of the river from higher basin to river mouth. The existence of paddy fields shows significant roll in the control / reduction of the inundations in the lower basin.

Through the rain storages in the 80% of the paddy fields (22.000 have) distributed in the basins, with the storage depths from 20 to 50 cm, it hopes to create approximately 2 to 5 thousand m^3 of capacities of storages pr hectare, equivalent to the 44 million to the 110 million m^3 of storage capacities in the Itajaí Basin. It can emphasize that this volume is equivalent to the Oeste and Sul Dams that has exclusive object to control inundation. The project has objective of exploring that capacity of reductions of rains in the paddy fields to soften the impacts of the inundations through the heightening of paddy ridge.

(2) External Factor

However, the potentiality to mitigate or attenuate an inundation does not still explore efficiently. On the other hand, in Brazil, the demands tendency, in all of the productive areas, has if more rigorous control, besides in the area of agricultural production. The main norms regulations implicated in the agricultural sector are:

- Land Tile regulation in accordance with the Forest Code (require existence of the legal reservation (RL) and the permanent reservation areas (APP), besides of riparian forest)
- Regulation of the use of agricultural defensive

In the forest Code, it forces to obtain the "Certificate of Land Title Regulation" for every rural property that requires the normalizations of the APP and of the RL.

This acquisition of the certificate of lands use regularity is forced for the all of the lands in the Brazilian territory; also, the paddy expanded in the basins also is inside of this regulation. For the acquisition of this certificate of land use regularity, the needs exist of updating them title of the soils, respecting the established requirements in the forest code of RL and of APP. They put these activities of environmental regularization becomes the increment of the costs of agricultural productions.

Besides, in the agricultural production sector, the consumers' demands in relation to safety foods have been increasing, requesting the certifications and tractability of the productive chain from seedling to commercialization.

This rigidity tendency in the control and quality demand in the productive chains, bring to increase the complexity in the production and consequently, elevation of the productions cost, until provoking rural exodus, because of the deterioration of the agricultural economy. As well as most of the rice farmers of the basin are of small scale farmers who doesn't have capacity to accompany the demands and of quality. Its tendency takes risks of abandoning rice cultivation, consequent abandonment of the paddy and deteriorating in the decrease of the rain retention capacity in the paddy for the loss of ridge of the farming in the point of basin management.

Like this, with the abandonment of the farming caused by the rural exodus, it brings to degradations of the paddy, consequent the disappearance of the paddy ridge, and the loss of the capacity of reduction of the rains retention in the basin. To maintain and to improve the capacity of attenuation of the rains in the paddy, it is necessary to realize the measures and strategies for the producers to stay in the rural area and to motivate the productions through the measures that favor the productions of rice in the Basin.

In this project, through construction of paddy ridge that contribute in the reductions of the floods, the farmers can modernize the productive system, capable to adjust to the forest codes, taking place the necessary activities, such as topographical survey, recoveries of the riprap forests and introduction of the PIA (integral rice production). In following Fig, it is indicated the interrelations between the paddy and the project of heightening of paddy ridge:

As the measures to produce the safety foods, the Federal Government, through the Ministry of Agriculture and Livestock and Provisioning (MAP), this introducing the system of the Integrated Production of Rice (PIA) to supply at the market the rice with quality assured and good you practice of cultivations of rice, and also to obtain larger joined prices.

In order to introduce PIA practices in the basin, there are following requirements;

- Regenerations of the riprap forests and environmental regularizations.
- Heightening of paddy ridge, water management for control of weeds that make possible the least uses of pesticides.
- Prohibition of burning to mitigate emission of gases CO_2.
- Introduction of the agricultural practice respectful with the environment.

(3) Relationship between rice culture and reduction of floods by water storage in paddy

At present, the rural areas, for the rigid control of environment, are forced to respect the environmental codes that bring to increases of the costs of the productions. These restrictions are the factors of being less economical viabilities of the productions of rice and consequently in the high risks in the reductions of the income of the rural producers that in the long medium period cause the rural exodus or abandon of the paddy.

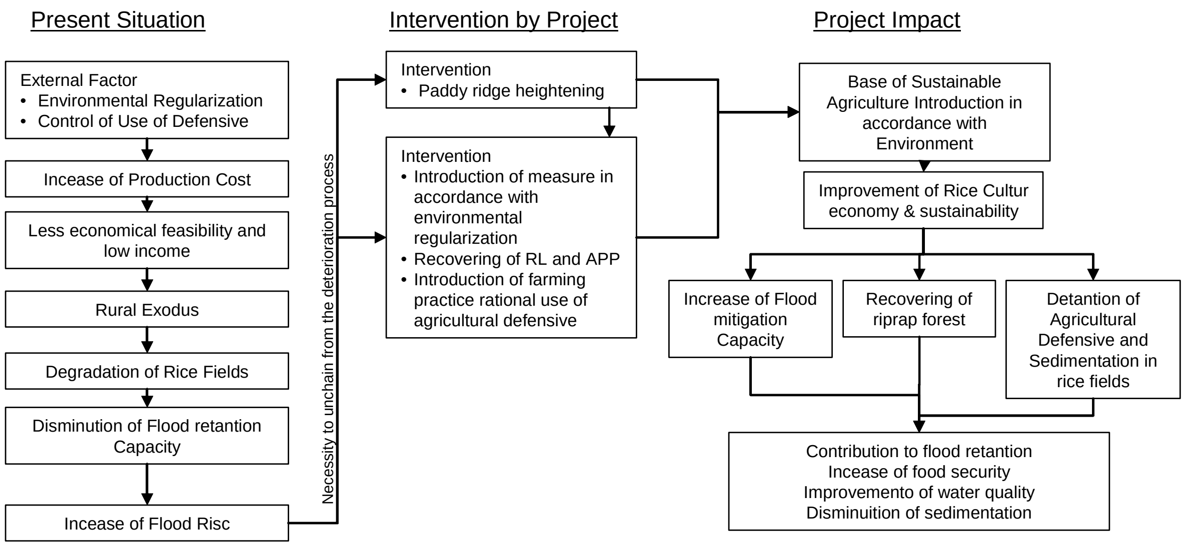
These abandonment of the paddy, result the degradations of the rice fields, especially in the disappearance of the paddy ridge, that drive the decreases of the capacities of reduction of floods. For not causing the negative chains of degradations, the needs exist of accomplishing the appropriate interventions to maintain the sustainability of the rice productions in the basins. Especially in the agricultural sector, the introductions of agricultural practice that allow the decreases of the costs and of the productivities increasing

The measures that make possible the increases of the productivities and of economicalization of the costs of productions are the mechanizations of the agricultural practice and the introductions of the resistant varieties in the uses of pesticides that control the weeds or the introductions of the cultivations of rice in added value. In the agricultural practices, the applied pesticides in the paddy fields are prohibited the discharges to the rivers at least 2 weeks. With these conditions, for the farmers, the rice culture continuities are requirements of modernizing them practice agronomic.

With these circumstances, to modernize the agricultural practice, it is necessary the heightening of the paddy ridge and the environmental regularization that demand the uses of the respected lands use in accordance with the environmental codes through the necessary interventions, such as of topographic survey of the lands and of regenerations of RL and APP. Through the structuring of the bases of productions of rice, the producers can be insured of the continuities of the productions of rice.

Therefore, to maintain and to improve the capacity of reduction of the rains in the paddy fields, it is necessary to accomplish interventions and appropriate strategies of planting, so that the producers stay producing his/her cultivation in the referred Basin.

The project of heightening of paddy ridge of the paddy fields, besides contributing in the reductions of the floods, will allow to the farmers to modernize productive system, being adjusted to the forest code, as well as accomplishing necessary activities, such as topographical risings, recoveries of the riprap forests and introduction of integral production of rice. In following Fig, it is indicated the interrelations between the rice culture and the reduction project by the heightening of paddy ridge.



Source: JICA Survey Team

Figure 1.1.1 Relation between the Project Water Storage in Paddy fields and Rice Culture

(4) Compatibility with the Master plan of the Committee of Itajaí

The Itajaí committee, aiming at the uses of the water resources in the basins, had formulated “Water resources Plan in the Itajai Basin” composed by topic and programs. As the topic related to this project exist the theme of recovering of riprap forests and rural managements, being identified the following programs;

TEMA	PROGRAMA
1. Instrumentos de Gestão de Recursos Hídricos	■ Implementação dos instrumento de Recursos hídricos
2. Gestão Ambiental Municipal	■ Programa de Apoio à Gestão Ambiental Municipal
3. Tratamento de Esgotos	■ Programa de Tratamento de Esgotos
4. Recuperação de Matas Ciliares	■ Programa de Recuperação de Matas Ciliares
5. Manejo Rural	■ Programa de recuperação de Matas Ciliares ■ Programa-Produto de Água ■ Programa de regularização de reservas legais ■ Programa de agroecologia ■ Programa de produção integrada de arroz e retenção de água da chuva
6. Desastres	■ Plano de Preservação e Mitigação de Risco de Desastres
7. Redução de Sedimentos	■ Programa de Redução de Sedimentos
8. Reservação de Água	■ Programa de Reservação de Água
9. Unidades de Conservações	■ Programa de Unidades de Conservação
10. Educação Ambiental e Comunicação	■ Programa de Educação Ambiental e Comunicação
11. Comunidades Indígenas	■ Programa para Comunidade Indígena
12. Conhecimento de Gestão	■ Programa de Gestão do Conhecimento da Bacia Hidrográfica
13. Capacitação de Recursos	■ Programa de Captação de Recursos ■ Programa de Neutralização de Carbono

Figure 1.1.2 Compatibility of plan with the Itajaí Basin Water Resources Plan

1.2 General policy of plan

(1) Objective of Plan

This plan, based on the above mentioned background, in purpose to enlarge the flood reduction capacity, being used of the paddy fields expanded in the all basins, will be heighten paddy ridge and gradually to introduce the rice production with better quality and safety. As measures, it will develop the following activities:

Table 1.2.1 Objective of Project and Proposed Action

Increase of capacity of reduction of the floods effect	- Elevation of the paddy ridge.
Use of the land in accordance with Environmental Legislation	- Recovery of the riprap forest. - Incentive to use of farm land in accordance with environmental legislation
Safety foods Supply	- Incentive to introduce the Integrated Rice Production

Source: JICA Survey Team

Through the materialization of the elements above, the following effects will be produced:

Table 1.2.2 Action and Expected Impact

Construction of paddy ridge	- Increase of the attenuation capacity of the rains in the paddy (for an increase from 40.000.000 to 1.000.000.000 retention m³). - Retention of the defensive used inside of the paddy fields. - Decrease of the soil erosion loss
Recovery of riprap forest	- Transformation of the area to the rice production area balanced with the environmental codes. - Protection of riprap area - Make the bases of to acquisition of "Certificate of legalized lands"
Incentive to use land in accordance with environmental regulation	- No penalty - Improvement to accessibility to Official Agricultural Credit
Strengthening of PIA	- Offer of healthy and safe foods. - maintainable and financial Stability of the producers through the production of valued rice

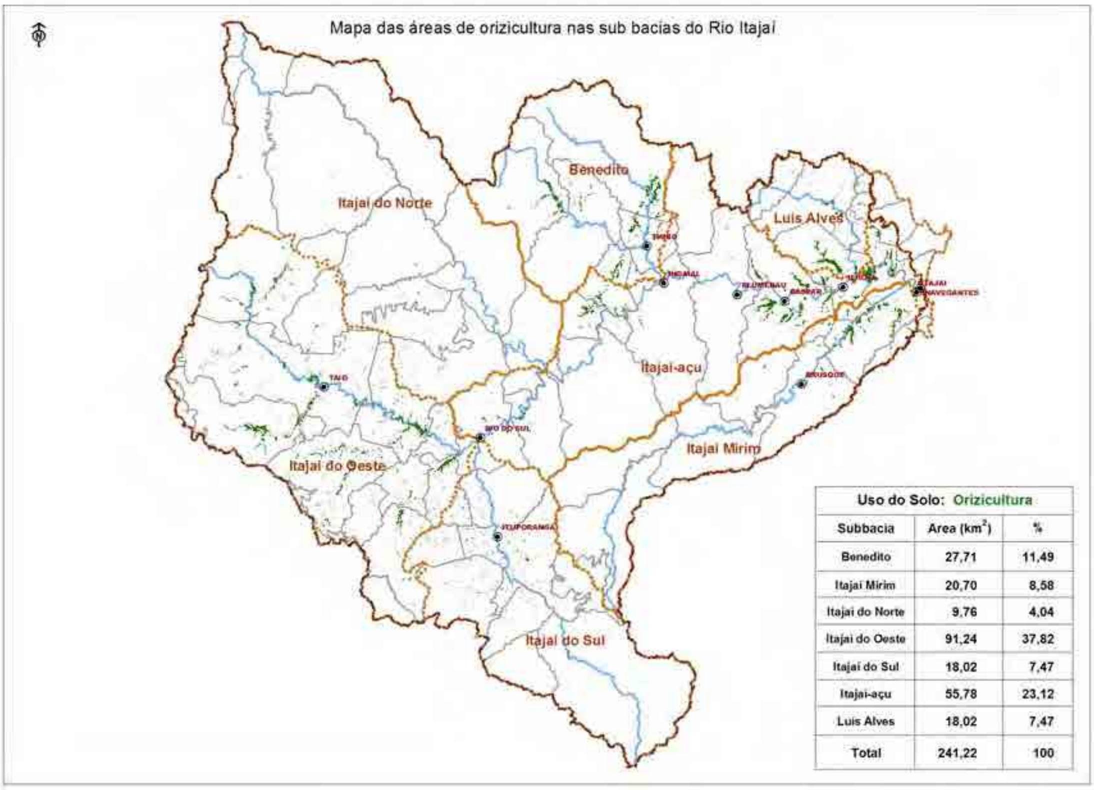
(2) Extents of Plan

The extent of the plan is expansion of the rice field in the Itajaí basin. In the following Table, it is indicated the basin area, the paddy area and the percentage regarding the total area.

Table 1.2.3 – Sub-basin area, Rice field's area and percentage of rice fields area

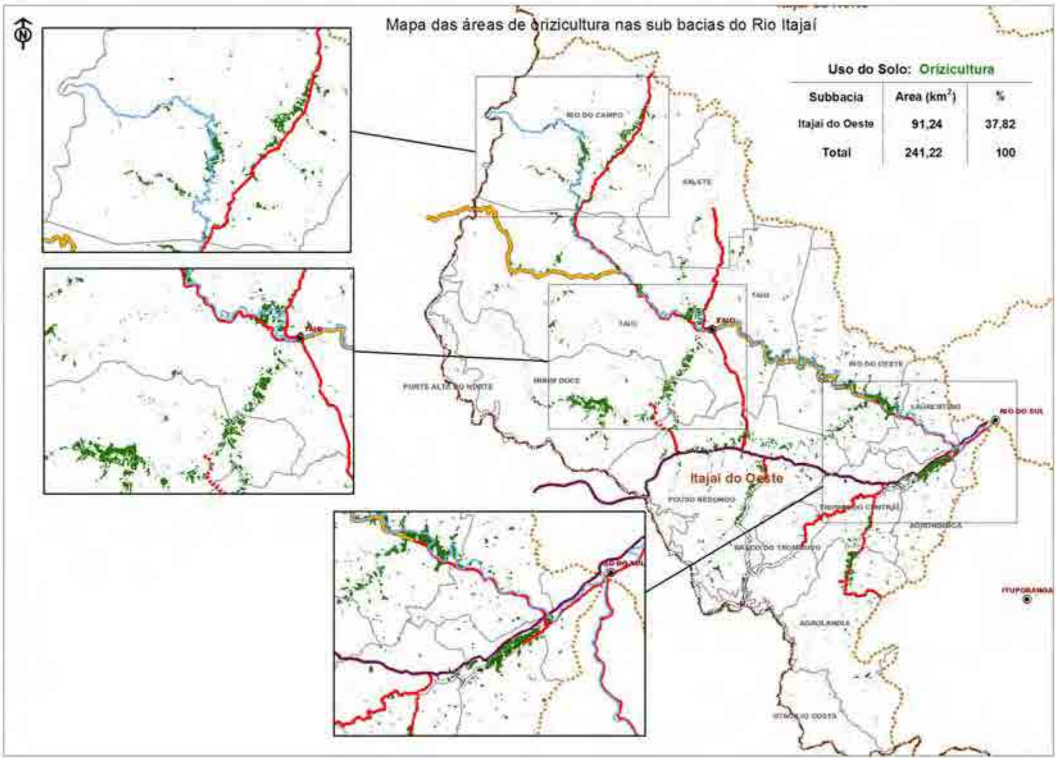
	Basin (km^2)	Paddy fields (km^2)	Percentage
Total	14,933.2	262.95	1.76%
Itajaí do Oeste	3,014.9	99.45	3.30%
Itajaí do Sul	2,026.7	19.64	0.97%
Itajaí do Norte	3,353.8	10.64	0.32%
Benedito	1,500.3	30.20	2.01%
Luis Alves	580.0	19.64	3.39%
Itajaí Açú	3,358.6	60.80	1.81%
Itajaí Mirim	1,678.9	22.57	1.34%

Source: JICA Survey Team



Source: JICA Survey Team

Figure 1.2.1 Paddy Area in the Itajaí River Basin



Source: JICA Survey Team

Figure 1.2.2 Paddy Area in higher Basin of Itajaí River

(3) Implementation method for the Project of Water Storage in Paddy fields

Considering the nature of this type of measure that depend on the paddy fields and the inundation risk falling directly to the producers, the project seeks to benefit both side, so much of production and of inundation mitigation in the same time, without only sacrificing the producers, exploring the paddies potentialities in the mitigation of flood and improvements of the productive infrastructures through the construction of paddy ridge. At the same time, as the compensatory measures of the paddy fields, it will be implemented the introduction of the Integrated rice production (PIA) that the producers can be adjust to the environmental demands, guaranteeing them financial means for so much, facilitating them the obtaining of the CRF.

Inside of this plan, the government will finance the construction of paddy ridge heightening works, and the producers will participate in their activities of implementing the paddy ridge in their property. The government's contribution and of the producers in the heightening of the paddy ridges will be the following:

Table 1.2.4 Activity of the Project Rain containment in the paddy fields

	Governments Support	Producers Contribution
Plan	- Support for formation of the Term of Agreement of the producers. - Support to organizations of the associations. - Support to the obtaining of C.R.F. - Topographical survey - Formulation of the Plan / Project (amount of works, calendar of the works, determination of the participation)	- Agreements between the producers and Establishment of the associations of the producers. - Arrangements of the registrations of the participants' registers. - Agreement among the producers in the accomplishment of the Plan. - Contract of the Execution of the Project
Heightening of paddy ridge	- Dispatch of consultants - Definition of the methodologies of project implementation.	- Execution of the Construction of Heightening of Paddy Ridge - Co-payment of Construction Cost

	Governments Support	Producers Contribution
	- Disbursement of Construction Cost of Heightening of Paddy Ridge (80%) - Supervision of the works	(20%)
Recovering of riprap forest		- Placement of that material in the margins of Rio. - Planting of nursery plants
Adjustment for Environmental legislation	- Certification of the property registration. - Survey of properties. - Emission of APP, R.L. - Establishment of the reach of APP	- Title of the property. - Certificate of Land title
Promotion of the PIA	- Technical Orientation for the PIA	- Introduction of PIA

Source: JICA Survey Team

Due to the paddy fields extend for the whole basin, and there are approximately of 2.000 farmers, there is complexity of the implantation of this plan to make format the implementation plan if it implement in contract base. Besides, because of the characteristics of this plan, and having a lot of stages of processes in materialization, it is suggested that the execution is executed in the following way:

Government
- Lend of services and consultancies for the implementation of the Plan. - Disbursement of construction Cost (80%)

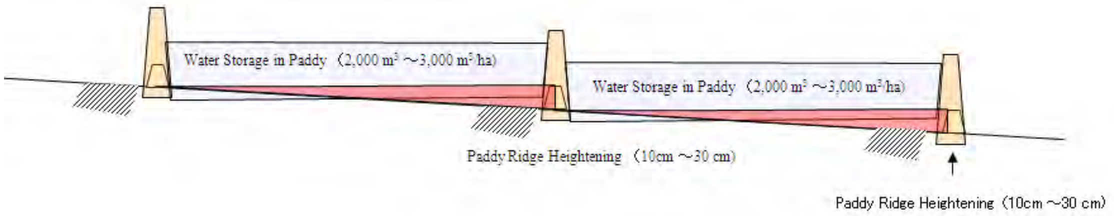
Producers
- Construction of paddy ridge heightening. - Provision of riprap forest area - Co-payment of Construction Cost

CHAPTER 2 WATER STORAGE IN PADDY FIELDS

2.1 Outline

(1) Scheme for the retention of flood water in the paddy fields

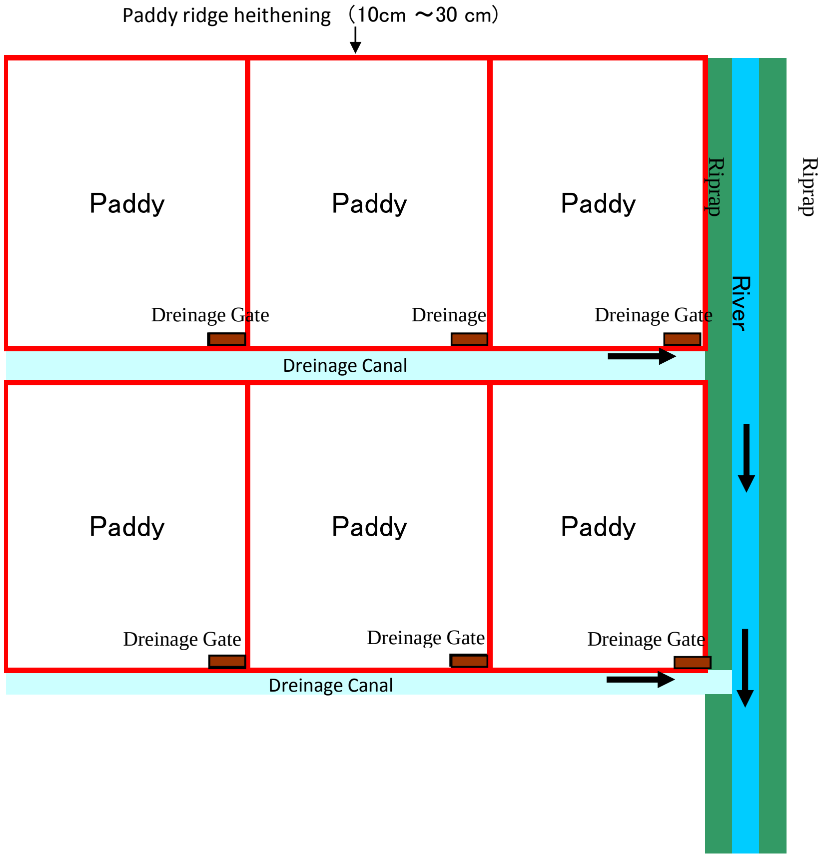
It is foreseen to execute the heightening of the paddy ridge (current 10 cm) for more 10 to 30 cm, hoping to increase the capacity of retention of the rains for more 2.000 ~ 3.000 m3 for hectares, as well as it is indicated in following Fig;



Source: JICA Survey Team

Figure 2.1.1 Retention Methodology of Flood water in paddy fields

The works of paddy ridge heightening will be realized in the margins of the suitable paddy with red lines in following Fig. For not accompanying the damage in the rice production, the gates will be installed capable to control/mitigate the effects of floods, especially in the times of flowering season period of rice when larger risk of loss of the products exists.



Source : JICA Survey team

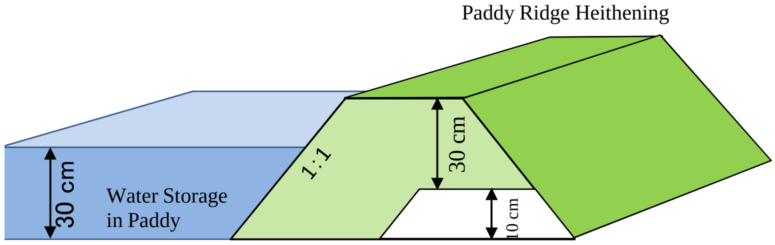
Figure 2.2.2 Retention of Rain in paddy fields

(2) Definition of the flood depth and height of paddy ridge

The rice culture, being the certain resistant culture for flood, it allows to accomplish the control of the floods, through the floods of the fields with the appropriate water managements. However, vulnerable times exist as in the period 20 days before flowering season in that the culture can be damaged by the excess of water. The required height of paddy ridge will be defined, being considered the heights of rice, to minimize the damages that can be caused by the floods by this project. The depths of projected floods depth are the following ones;

- During the period of non cultivation, it settles down 30 cm of water depth. From the period of 20 days before the formation of the ears of rice, until the time of flower of these, the maximum depth will be of 30 cm, being the normal depth of 20 cm.
- After the seedling, given the fact of the young plants does not support for a long time be submerged, one should not leave submerged for more than 4 consecutive days

The height of the heightening is of 30 cm, to make possible the installation of floodgates and to guarantee the depth of until 30 cm of the water.



Source：JICA Survey team

Figure 2.1.3 Cross section of Paddy Ridge

The amount of work for the paddy ridge is the following ones:

Present, (0,5+0,7):2x0.1m= 0,06 m3

After heighten: (0,5+1,3):2x0,4m= 0.36m3

Difference of earth volume: 0,30 m3

(3) Extents of Project Area

The extent of the Project will be the areas where appropriate rice field expansion exists inside of the entire paddy areas expanded in the Basin. As the first phase, it is considered 5,000 ha of the paddy expanded in the basin. The objective areas of project will be selected in the basic study phase and it will be settled down the following goals:

Paddy Areas:	26.295 has
Objective Project Area:	20.000 has
First phase Area of the Project:	5.000 has

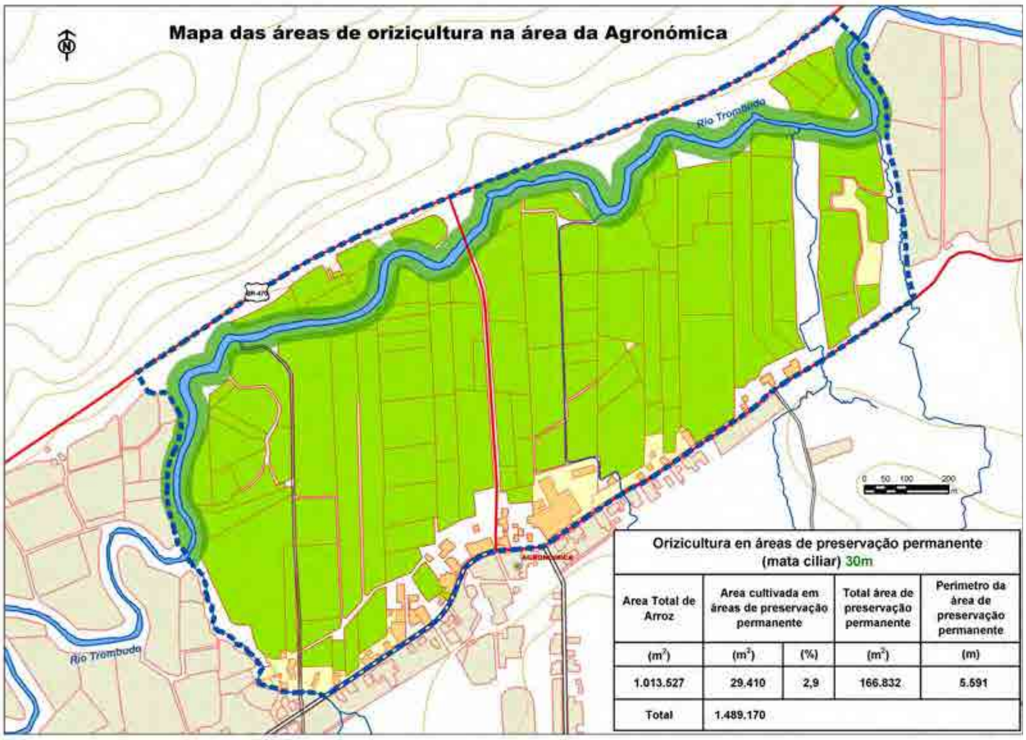
(4) Heightening of Paddy Ridge and required work quantities

The dimensions of the works were estimated, being taken the area of Agronomic as sampling. The estimated amounts of works are;

Table 2.1.1 Agronômica – Quantities of necessary works

	Present	Projected	Cost/ha
Total Area			
Paddy Ares	101.4 ha		
Extension of ridge	29.4 km		
Number of farm	106		
Area of Riprap forest		0.03 ha/ha	R$ 5,000/ha
Length of paddy ridge	m/ha	300 m/ha	
Required volume for heightening of Paddy ridge		90 m3/ha	R$ 3.150/ha
Extension of medium paddy	ha	1 ha	
Required Gate		1 per ha	R$ 250/ha
Specific riprap Forest area for 1 hectare	16.6 ha	0.16 ha/ha	
Length of riprap forest	5,600 m	55 m/ha	

In following Fig, the Agronomic area as a sample is indicated, in order to estimate the amounts of works requested for this measure;



Source: JICA Survey Team

Figure 2.1.4 Paddy Fields in AGRONOMICA

The estimated costs of the project, included topographical survey, formulation of the plan, environmental licensing and the environmental regulation are:

Table 2.1.2 Estimated required budget in a Contract Base

Item	Quant.	unit	Unit Price (R$)	Total Value (R$1,000)
Work for heightening of paddy ridge	5.000	Ha	3,400	17,000
Recovering of riprap forest	200	Ha	5,000	1,000
Subtotal				18.000
Design of paddy ridge	20.000	Ha	100	2,000
Detailed design and Bidding assistance	5,000	Ha	200	1,000
Construction Supervision	5,000	Ha	600	3,000
Topographic Survey	5,000	Ha	200	1,000

Item	Quant.	unit	**Unit Price (R$)**	**Total Value (R$1,000)**
Emission of C.R.F	500	Family	100	500
Support to Environmental Regularization	500	Family	200	1,000
Training	500	Family	1,000	500
Total				27,000

Note: The cost of measures of contention of floods in the paddy was esteemed in the following form: Works of heightening (R$ 3.150 / ha) + Floodgate (R$250/unit. The Cost of riprap forest was estimated in the following form: Plants (1.000 x R$3.0 / unit) + fence
Source: JICA Survey Team

Implementation Method

The work of the heightening/preparation of the paddy ridge will be executed by the contractor, and the implementations work methodology is as follow:

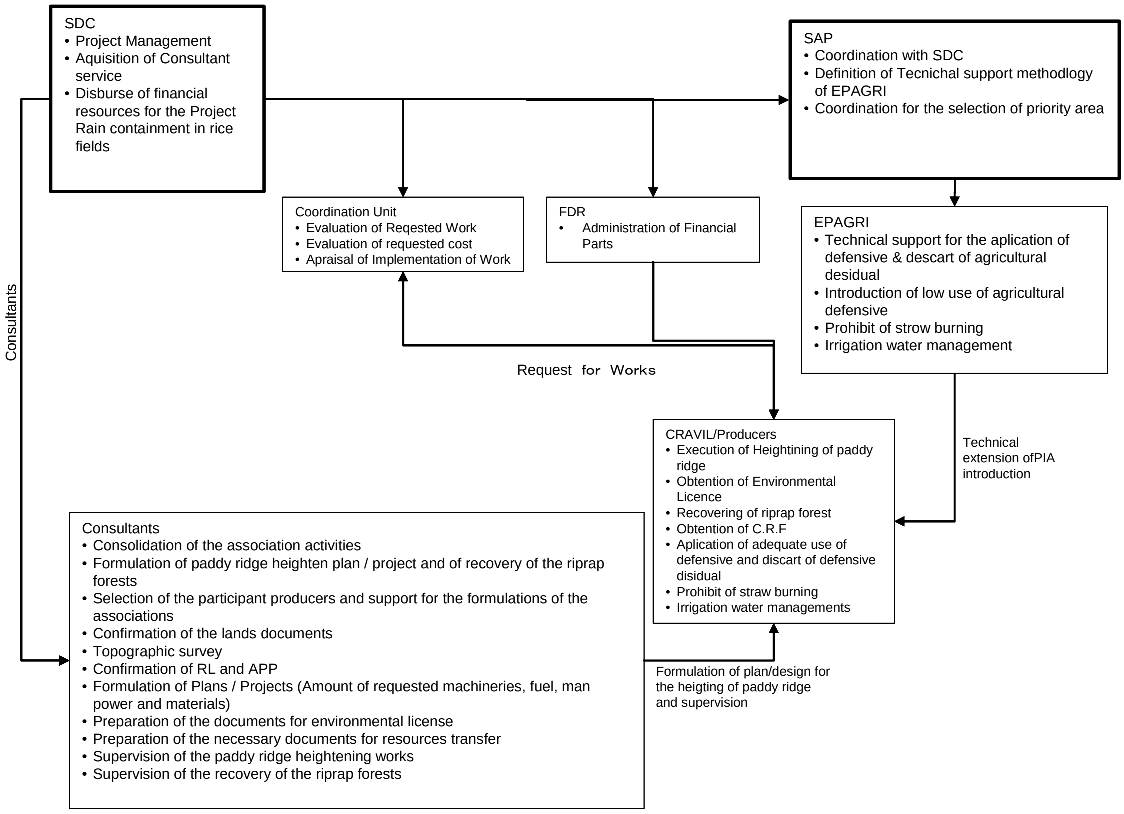
- Promotion to the producers to participate in the project of containment of rains in paddy fields, through the explanations of its benefits and activities to be accomplished by the project.
- Formation of the Agreement among the producers on implementation of the project.
- Realization of the Study, preparation of the Projects and necessary topographic survey.
- Project Design/Estimation of Cost
- Estimation of Cost
- Request for the implementation of Works for the Coordination Unit
- Request for FDR
- Evaluation of the Project, Appraisal and Contract for the Execution of the Project
- Construction by producers
- Disbursement of Construction Cost (80%)

It is foreseen to promote the understanding of the producers, in agreement, projects and survey. This type of service will be accomplished through support activity of contracted consultancy. The place of the borrow pit of the earth for the paddy ridge heightening will be made by own local population. Also the works of the riprap forest plantations will be done as of the beneficiaries' responsibility, so much in the execution, as in the financial area.

2.2 Organization Chart of Project Implementation

(1) Process

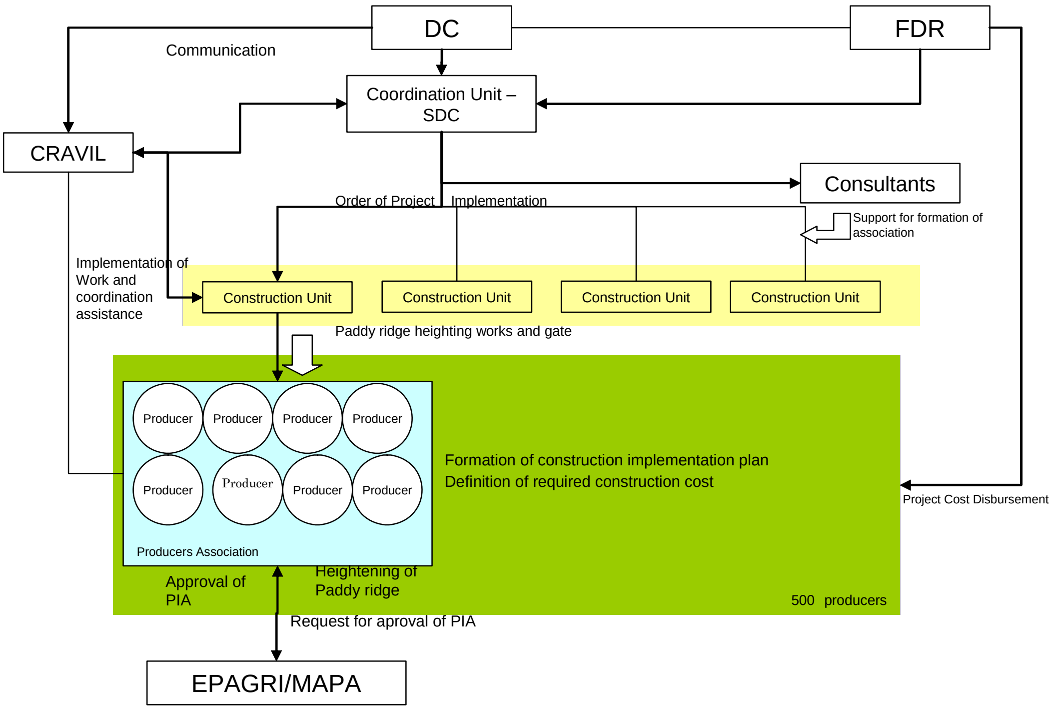
The implementation of the project will have as base, the rational and economical use of the budget, with the wide participation of the producers in the work. The State government will finance the materials and necessary machineries for the work, as well as consultancy services and training. Below, it is the organization chart of the implementation of the works:



Source: JICA Survey Team

Figure 2.2.1 Organization Chart

The executive organization of this project is the State secretariat of Civil Defense (SDC). For the implementation of this project, the SDC will be acquired the Constructor through the bidding.



Source: JICA Survey Team

Figure 2.2.2 Project Implementation Flow

In this Project, the FDR will already be applied established in the SAP, as an operational mechanism of the Project of the World Bank "MICROBACIA."

(2) Consultancy Service

The Consultancy of this operation will have the following attributions:

Formulation Stage for the Rain Water Contention in Paddy Fields

- To determine the outline of execution of the whole plan.
- To chooses of the candidate farmers and organization of the associations (final objective, 500 families of farmers).
- Verification of the participants' (final objective, 500 families of farmers) land titles situation.
- Verification of land title documents
- Topographic Survey and verification of Legal Reserve and Riprap Forest Area
- Detailed design and Cost Estimation
- Cost estimation
- Obtaining of the environmental license.
- Preparation of application document for appraisal
- Supervision and orientation of the work of heightening of walls (paddy ridge).
- Supervision of the countermeasures of APP

(3) Technical support for the introduction of PIA

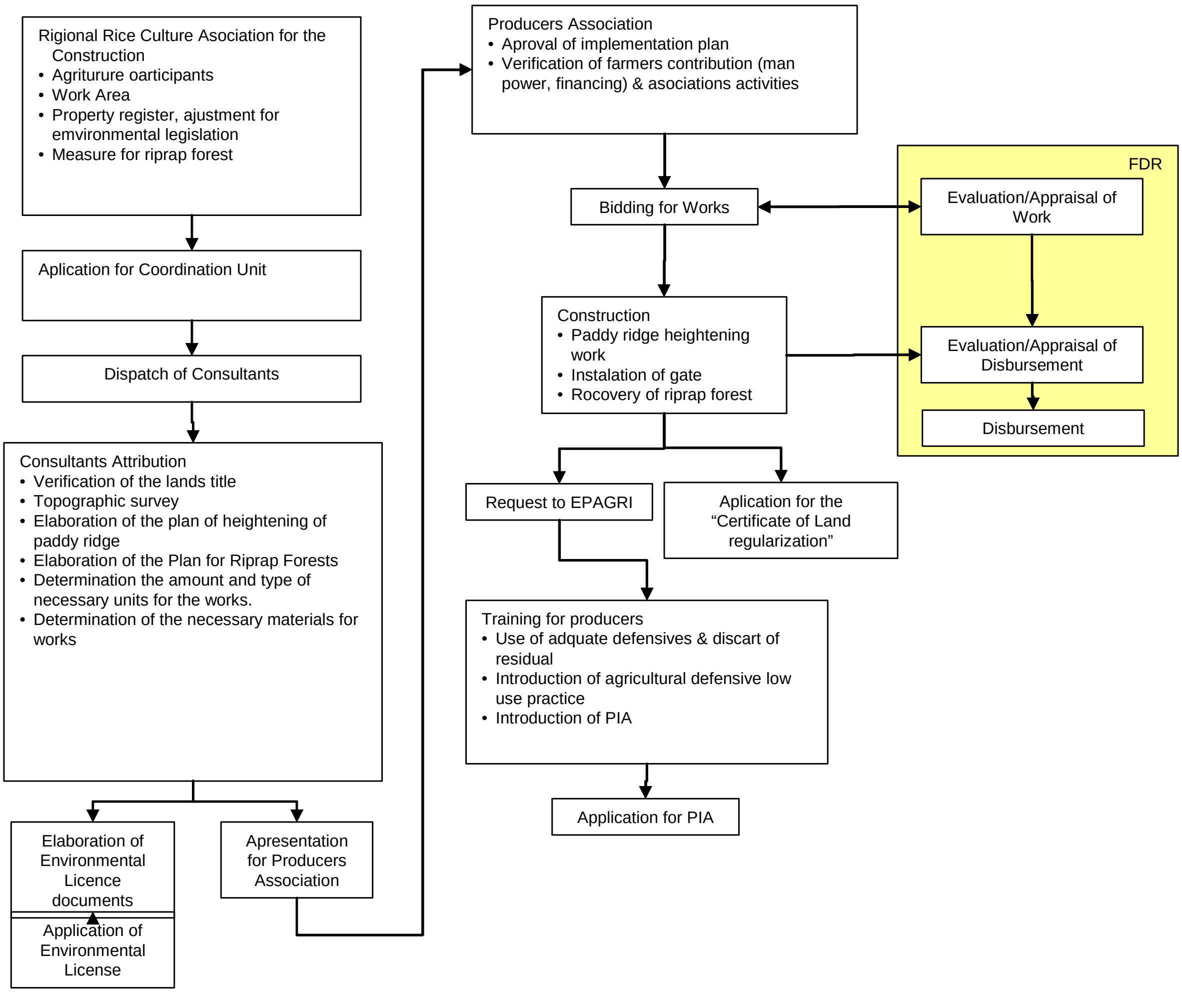
With support of the EPAGRI/SAP, the following necessary activities will be implemented for the introduction of the PIA

- Managements of defensive use and appropriate discard of package of defensive.
- Introduction of low defensive use agricultural.
- Prohibition of the burnings of the straw.
- Administration of water resources

CHAPTER 3 PROJECT IMPLEMENTATION GUIDELINE

3.1 Methodology of Execution

The process of the Project Contain of the rains in the paddy fields will be implemented in the following forms:



Source: JICA Survey Team

Figure 3.1.1 Project Implementation Process

3.2 Activity of each Phase

(1) Assembly of the rice producers of the area for the execution of the works

The assembly of the rice producers of the area, with objective of obtaining support of the municipal districts, will be established at the level of the each municipal district, town or irrigation scheme established already, and it will be unit responsible of the implementation of the work. Like this, each unit of those assemblies will be, hereafter, them unit of implementation of the construction of paddy ridge works, administering the system and also making the devolution of the financing.

It is desirable that each unit has a minimum of 10 families, so that there is a harmonic work in the works. Each assembly will have to decide on the following items:

- Participants of the project.

- Area where the project will be accomplished.
- Outline of the registrations of the earth (for adaptation to the environmental legislation).
- Measures are taken with regard to the riprap forest.
- Place from where will leave earth.

(2) Requirement of work for the Coordination Unit of the Project

In the Project of containments of rains in the paddy fields, each association will make request to the Coordinating Unit of the Project for its implementation. The coordinating Unit will establish the order of service of each work and execution, tends in view the applications done by each area and area. The coordinating Unit will do, also, allocation of the consultants, lend of necessary machines in agreement with the applications made by each area.

(3) Paddy Ridge construction works and recovery of riprap forest

To execute the paddy ridge works and recoveries of the riprap forests will be accomplished the following activities;

- Verification of the lands registrations.
- To execute the measurement (topographic survey).
- To elaborate the project of construction of paddy ridge.
- To elaborate the project of the riprap forest.
- Project of earth (for the paddy ridge) retreat.
- To determine the amount of machines and the necessary days of use.
- Methods of reproduction of seedlings.
- Elaboration of the applications to CIDASC and FDR.

The measurement area will be basically of those foreseen for the heightening of paddy ridge, lands that are out of the Project, the costs for measurement will be covered by the proprietors. As for the riprap forests, through the verification of the land registrations, will be divided in areas of individual responsibility and community. The plan of borrow pit as for the earth will be resolution item in the assemblies of each unit, as for the place from where will be removed, and means of providing resources for such. It is also due to calculate the amount of workers, the number of necessary machines, the cost, the necessary materials, to base the applications to the CIDASC and the FDR.

As for the recovering of the riprap forest, it is just due not to foresee the species and varieties of the nursery plants, production and planting, but also from where the resources will come for so much.

(4) Application for obtaining of the environmental authorization

The allocated consultants will elaborate the document and necessary applications to the obtaining of the environmental license; especially, close to the FATMA (General office of the Environment of the State).

(5) Last confirmations of the Assemblies of the rice producers associations

Through the assemblies, it will be verified the following points:

- Lines as for the methods and means of the elevation of the barriers (of the heightening of paddy ridge).
- Methods for restoration of the riprap Forest, and their limits.
- Preparation of the document for the application for the implementation of the work

(6) Evaluation/Appraisal of requested document for the implementation of works and preparation of contract documents

The Coordination Unit will evaluate the appropriation of the construction cost requested by the producers and, after the appraisal of the document, it will be transferred the request for the FDR. Also, the Coordination Unit will make contract document subscribing the responsibility of farmers and implementation schedule, including the responsibility in the contention of rain water in their paddy fields.

(7) Start of Works by producers and request for the disbursement

The Rice producer will carry out the paddy ridge heightening works and riprap forest recovering. The cost for the work will be requested for the disbursement.

(8) Evaluation of construction cost and disbursement

The Implementation Agency will evaluate requested construction cost and after the appraisal of the requested document will be disbursed through the FDR. The disbursed amount is 80% of the construction cost.

(9) Training for Producers

To optimize the effects of the works in this basin, besides improving the quality of the water the producers they will receive the following trainings:

- Use of defensive / discard of residues and packing.
- Introduction to the low defensive use agriculture.
- Prohibition of burnings of the straw.
- Practice of the agriculture with low use of defensive.
- Handling of water

Supporting Report (F)
Environmental and Social Considerations

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

FINAL REPORT

VOLUME III : SUPPORTING REPORT
ANNEX F: ENVIRONMENTAL AND SOCIAL CONSIDERATIONS

Table of Contents

Tables

Figures

List of Attachment

CHAPTER 1 ENVIRONMENTAL AND SOCIAL CONSIDERATION OF SELECTED PROJECTS

1.1 Introduction

The JICA Survey Team has held a series of meetings and public consultations with the relevant stakeholders in accordance with the JICA's Guidelines on Social and Environmental Consideration (2010) in the course of the master plan study. Comments and suggestions given by the stakeholders in the meetings were fully reviewed and examined in the preparation of the master plan. As a result, the JICA Survey Team adopted the disaster-prevention measures against the 50-year flood as the main framework of the master plan.

As described in the master plan, the Survey Team propsoed the implementation of the master plan in a stage-wise implementation since the implementation of the entire disaster-prevention measures against the 50-year flood would require an enormous amount of money and take a long period of time until its completion.

To this end, the Survey Team evaluated and examined all the measures proposed in the master plan for prioritization. Among others, the Team put emphasis on whether or not a consensus on the implementation of the proposed measure among the stakeholders is easy to get in the evaluation, as the meetings with the stakholders in the master plan study revealed that it would be difficult for them to accept the construction of a diversion chnnel for floodway and dykes along the river due to the likely impacts on the surrounding ecosystems and other natural environment. In fact, the Survey Team expected that the consensus building on the construction of those measures would not be easy and require a long-term process. As a result of the evaluation, the disaster-prevention measures against the 10-year flood were determined as the priority projects in the first phase.

The environemntal and social considerations in this study were carried out with the following limitations:

i) No interview survey on socio-economic conditions, such as land ownership, family structure, and household income, was made in the areas where the land acquisition might be required, as the information of the projects were not able to open during the study due to the uncertanty about the commencement of the projects. In fact, the study team did not disclose as much information as explained in the pulic hearings at the end of the master plan study.

ii) The topographic maps on a scale of 1:10,000 were not available when the feasibility study was made although they were supposed to be ready to use during the study. Alternatively, the topographic map of 1:50:000 and the results of a topographic survey of river conducted in the feasibility study were used for the detailed study and designing. In addition to them, the topographic maps of Itajai City on a scale of 1:2,000 was available and also used for the study on the urbanized areas of Itajai City.

The feasibility study revealed that the existing roads of Itajai Mirim River was sufficiently high and not necessarily heightened as a disaster-prevention measure. Hence, the Survey Team judged that no resettlement or extensive land acquisition except the areas to be affected by the heightening of Oeste Dam would take place.

This chapter describes: i) the results of a review of the initial environmental examination (IEE) made in the master plan; ii) the draft terms of reference for an environmental impact study (EIA) that would be needed for implementation of the priority projects; iii) the results of environmental and social considerations of the priority projects; and iv) the environment-related legislation and necessary steps and surveys for acquisition of an environmental lisense in Brazil.

1.2 Review of Initial Environment Examination (IEE) in M/P Study

1.2.1 Requirement for Envrionmental Lisense

Since the catchment of Itajai river is locate within the territory of SC state, FATMA in SC state is the responsible institution for examining environmental study reports and issuing an envrionemntal license for the project. Therefore, the information on the environmental impact studies, especially crucial points in environmental and social assessments, were collected at FATMA in the study.

In SC state, an environmental assessment report to be submitted to FATMA varies with the extent of expected environmental impact. A proponent of a project shall submit one of the following reports based on the size, type and location of a project as described in Chapter 7 of the master plan study.

- Environmental Impact Assessment (EIA; Estudo de Impacto Ambiental)
- Simplified Environmental Study (EAS; Estudo Ambiental Simplificado)
- Previous Environmental Report (RAP; Relatório Ambiental Prévio)

As this project targets all the water sources in the whole catchment of Itajai river, FATMA is, therefore, obligated to submit an EIA report for a "macrodrenagem[1]" project, which covers all the components proposed as the priority projects in the first phase.

1.2.2 Review of IEE

The Survey Team reviewed the initial environmental examination (IEE) made in the master plan study using the maps and data additionally collected in the feasibility study. The focus of the review was put on the coponents proposed as the priority projects in the first phase. The following sections describe the results of the review and Table 1.1 shows the revised environmental screening and scoping of the priority projects in the first phase. Furthermore, the provisional environmental checklist for the proposed projects is shown in Attachment-1.

(1) Water Storage in Paddy Fields

The environmental and social impacts caused by this component are expected to be negligible, as the component does not require any large-scale engineering work. The component will be implemented by CRAVIL when the topographic maps of 1/10,000 are completed and the implementable areas are determined. Having determined the target areas using the topographic maps, CRAVIL shall prepare an implementation plan and take the necessary procedures for the registration of the target areas as Legal Reserve (RL) areas according to the Forestry Law.

(2) Heightening of Sul and Oeste Dams

The lands used for heightening Sul and Oeste dams and those that might be inundated in the operation phase were considered as the areas affected by this component. In the feasibility study, the progress of the land acquisition made by the Government of SC state and the proposed designs of the dams, which were not available in the master plan study, were able to be collected and, therefore, fully reviewed and analyzed to identify and determine the potentially-affected areas. As detailed in the following sections, the assessment revealed that all the areas to be affected by the heightening of Sul dam and those up to the height of the existing spill way of Oeste dam had already been acquired, although the results of the IEE in the master plan study indicated that land acquisition and involuntary resettlement might be the possible impacts caused by the heightening of both dams.

[1] It is used as the term standing for the project including "integrated flood control" and "basin management", although its literal meaning is "large-scale drainage."

Figure 1.1 shows the potential water level of the dams with and without heightening of dam crests and the progress of the land acquisition that the Government of SC state has made so far.

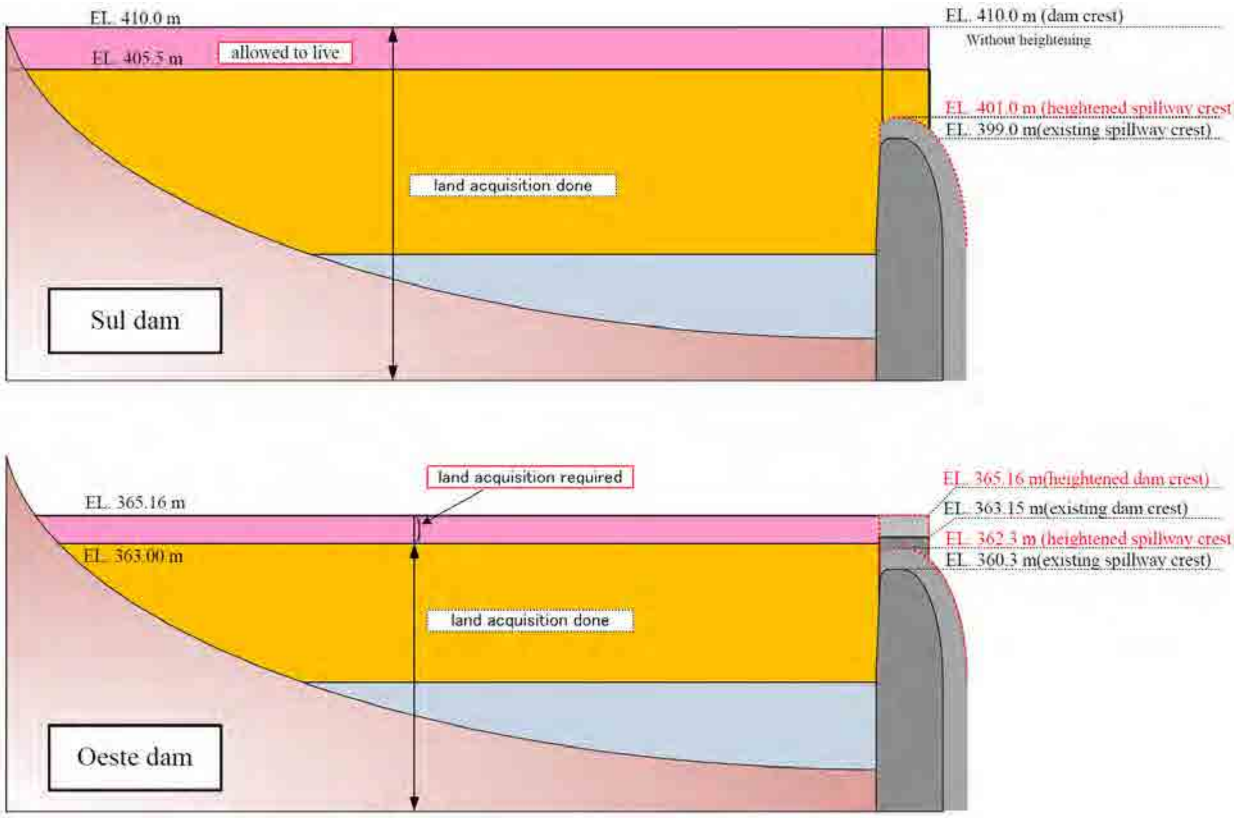


Figure 1.1 Dam Heights after the Heightening of Sul and Oeste Dams and Status of Land Tenure

The heights of spillway of Sul dam will be heightened from 399.0 m to 401 m, but the dam crest will not be heighted. As the Government of SC has already acquired the potential inundation areas up to EL. 410.0 m, no further land acquisition will be required for this component. However, DNOS (former federal government who constructed Sul and Oeste Dam) has made an agreement with COOPERBASUL on the use of the lands extended from EL. 405.0 m to EL. 410.0 m elevation, which have been rarely inundated. As the agreement does not include a compensation clause on any damages caused by inundation, DNOS shall review and revise the current agreement with COOPERBASUL so as to ensure that COOPERBASUL could get compensation when such areas are inundated.

The dam and spillway crests of Oeste Dam will be heightened by 2 m, and therefore the heights of both crests will be EL. 365.16 m and EL. 362.3 m, respectively. Although Deinfra of SC state has already acquired the potential inundation areas up to 363.0 m, there is still a need to acquire the rest of the potential areas up to the height of the planned dam crest (EL. 365.16 m). In other word, the heightening of Oeste Dam is expected to affect households/communities residing in the potential inundation areas from 363.0 m to 365.16 m elevation.

An inventory of existing buildings (houses and barns) in the potential inundation areas of Oeste Dam was also carried out to assess the compensation cost for households who might be affected by heightening its dam crest. The results of the simplified inventory are described in Section 2.1.

Besides, the heightening of Oeste dam might cause the adverse impact on the river environment (e.g., water quality, river bed, and riverine flora and fauna) since the engineering works will be done on the main body of the dam and need to divert the main stream of the Oeste river during the period of its construction works. On the other hand, no adverse impact on the river environment is predicted by the heightening of Sul dam, as the engineering works for Sul dam is to heighten the spillway by two meter and do not require any works on the mainstream of the river.

(3) Utilization of CELESC's Hydro-Electric Dams for Flood Control (Introduction of Pre-release method)

This is the measure taken by CELESC, which is the operation of Rio Bonito and Pinhal Dams in Rio dos cedros river, to mitigate the flood risk by the pre-lease of storage water in the dams when having a flood warning. An alert system needs to be installed to alarm households living in the downstream areas of both dams to cope with a potential flood risk after discharging the storage water from the dams. More details about the pre-release along with an early warning system are described in Chapters 4 and 6 in Part II of the Feasibility Study report.

(4) Installation of Floodgates in Intajai Mirim River

This measure aims to install two floodgates and one back water dyke in the old river canal of Itajai Mirim river as described in Chapter 5 in Part II of Feasibility Study report.

The places of the floodgates are located in the residential area, the construction work for the floodgates might cause a vibration and/or noise or increase trafic. The construction work might also cause turbid water in the river. However, these impacts would be negligible if the necessary mitigation measures for these impacts are to be taken by the contracotr in the construction phase. On the other hand, there is no impact predicted in the operation phase as long as the floodgates are operated properly.

As described in Chapter 5 in part II of the Feasibility Study report, driving sheet pile walls on the right bank was proposed to secure the safety of households living by the river in addition to the floodgates, as such areas on the right bank are subject to flood damage especaiily by high tide and back water. The construction work associated with driving sheet pile walls is exected to generate noise and vibration in the surrounding areas, the contractor should arrange and allocate temporary accommodations for households living in such affected areas in advance.

(5) Structural Measure for Landslide

This measure is to apply slope protection measures to the slopes along the national roads to prevent landslides/slope failures. As the construction work will cause the traffic hindrance by parking a large truck in the road or blocking off one of the lanes, the contractor needs to take safety measures, such as traffic control, during the construction.

(6) Development of Flood Forecasting and Warning System (FFWS)

Since this measure does not include any structural works, no environmental impact is predicted in the construction phase. Furthermore, the system will not change the lifestyle or any socio-economic conditions of households living in the area but help them protect their lives from flood damage. Consequently, no social impact is predicted by the introduction of the flood forecasting and warning system (FFWS). It is however important to ensure that such a system can disseminate the information up to the vulnerable groups and to conduct an emergency drill with the participation of those groups using the system, so as to minimize the risk of a flood.

(7) Development of Landslide and Flush Flood Warning System

Likewise, this measure is not predicted to cause any environemntal or social impact in both construction and operation phases since the measure does not include any structural work or cause any socio-economic change. As in the case of FFWS described above, what would be requisite for ensuring the effectiveness of the system are to disseminate the information to the vulerable groups using the system and conduct suffucuent emergency drills along with the system, so that the group could react properly when having an warning by the system.

Table 1.1 Revised Environmental Screening and Scoping of the Priority Projects in the First Phase.

Construction Phase

	SOCIOECONOMIC IMPACTS, POLLUTION AND NATURAL ENVIRONMENT / PROPOSED MEASURES	Basin Storage Measures: Rain water storage in rice field	Basin Storage Measures: Heightening of Oeste Dam	Basin Storage Measures: Heightening of Sul Dam	Basin Storage Measures: Preliminary discharge of CELESC hydro-electric dams	River Improvement Measure: Flood gate control installation in Itajai Mirim	Others: Measures for landslide disaster	Others: Flood Forecasting and Warning System (FFWD)
Natural Environment	Landscape	-	-	-	-	B-	C	-
	Global issues: Greenhouse gas	-	-	-	-	-	-	-
	Coastal area	-	-	-	-	-	-	-
	Fauna and flora	-	C	-	-	-	-	-
	Groundwater	-	-	-	-	-	-	-
	Solid waste	-	C	C	-	-	C	-
	Bottom sedimentation	-	C	-	-	B-	-	-
	Topography and geology	-	-	-	-	B-	-	-
	Offensive odor	-	-	-	-	-	-	-
Pollution	Land subsidence	-	-	-	-	-	-	-
	Noise and vibration	-	C	C	-	B-	C	-
	Soil contamination	-	-	-	-	-	-	-
	Air pollution	-	-	-	-	-	C	-
	Water pollution	-	C	-	-	B-	-	-
Socio-economic impacts	Impacts of regional infrastructure (Transmission line, roads, bridges, etc.)	-	-	-	-	C	-	-
	Impacts of land / buildings	-	B-	B-	-	-	-	-
	Involuntary resettlement	-	B-	B-	-	-	-	-
	Impact in downstream area	-	-	-	B-	-	-	-
	Impact of agriculture	-	-	-	-	-	-	-
	Change of income, life condition	-	-	-	-	-	-	-
	Traffic / interference of traffic during construction	-	C	C	-	B-	B-	-
	Public health	-	-	-	-	-	-	-
	Sanitation	-	-	-	-	-	-	-
	Cultural heritages	-	-	-	-	-	-	-
	Indigeneous / traditional people	-	-	-	-	-	-	-
	Water use	-	-	-	-	-	-	-
	Effects / prejudice of low income people	-	-	-	-	-	-	-
	Benefit to urban area vs. prejudice of rural area	-	-	-	-	-	-	-
	Regional comflicts	A-	A-	A-	A-	A-	-	-
	Land use and occupation	-	-	-	-	-	-	-
	Economic and Productive activities	-	-	-	-	-	-	-
	Land acquisition	-	B-	B-	-	C	-	-
	Expectation in local and regional people / Mobilization of NGO	A+	A+	A+	A+	A+	A+	A+
	Flood Probability							

Construction Phase

	SOCIOECONOMIC IMPACTS, POLLUTION AND NATURAL ENVIRONMENT / PROPOSED MEASURES	Basin Storage Measures: Rain water storage in rice field	Basin Storage Measures: Heightening of Oeste Dam	Basin Storage Measures: Heightening of Sul Dam	Basin Storage Measures: Preliminary discharge of CELESC hydro-electric dams	River Improvement Measure: Flood gate control installation in Itajai Mirim	Others: Measures for landslide disaster	Others: Flood Forecasting and Warning System (FFWD)
Natural Environment	Landscape	-	-	-	-	-	C	-
	Global issues: Greenhouse gas	-	-	-	-	-	-	-
	Coastal area	-	-	-	-	-	-	-
	Fauna and flora	B+	-	-	-	C	-	-
	Groundwater	-	-	-	-	-	-	-
	Solid waste	-	-	-	-	-	-	-
	Bottom sedimentation	-	-	-	-	C	-	-
	Topography and geology	-	-	-	-	-	-	-
	Offensive odor	-	-	-	-	-	-	-
Pollution	Land subsidence	-	-	-	-	-	-	-
	Noise and vibration	-	-	-	-	-	-	-
	Soil contamination	-	-	-	-	-	-	-
	Air pollution	-	-	-	-	-	-	-
	Water pollution	-	-	-	-	-	-	-
Socio-economic impacts	Impacts of regional infrastructure (Transmission line, roads, bridges, etc.)	-	-	-	-	-	-	-
	Impacts of land / buildings	-	-	-	-	-	-	-
	Involuntary resettlement	-	-	-	-	-	-	-
	Impact in downstream area	-	-	-	B-	-	-	-
	Impact of agriculture	B-	C	C	-	-	-	-
	Change of income, life condition	-	C	C	-	-	-	-
	Traffic / interference of traffic during construction	-	-	-	-	-	-	-
	Public health	-	-	-	-	-	-	-
	Sanitation	-	-	-	-	-	-	-
	Cultural heritages	-	-	-	-	-	-	-
	Indigeneous / traditional people	-	-	-	-	-	-	-
	Water use	-	-	-	-	-	-	-
	Effects / prejudice of low income people	-	C	C	-	-	-	C
	Benefit to urban area vs. prejudice of rural area	-	C	C	-	-	-	-
	Regional comflicts	-	-	-	-	-	-	-
	Land use and occupation	-	C	C	-	C	-	-
	Economic and Productive activities	B-	C	C	-	-	-	-
	Land acquisition	-	-	-	-	-	-	-
	Expectation in local and regional people / Mobilization of NGO	-	-	-	-	-	-	-
	Flood Probability							

Note: A+: Big positive impact, A-: Big negative impact, B+: Medium positive impact, B-: Medium negative impact, C: Impact of measure is unknown, -: No impact. .

1.3 Draft Outlines of TOR for EIA/RIMA Study

1.3.1 Draft Contents of EIA required for the Environmental License

A daft TOR for EIA study on the flood management project in the Itajai river basin was prepared and shown in Attachment-2. In the preparation of the draft TOR, the JICA Survey Team collected the existing TORs for the past EIA studies kept in FATMA in SC state. Although there was no EIA study made for a flood management or landslide disaster management project in SC state, those used for a basin management project in Minas Gerais state and an integrated port development project in SC state were referred for preparation of the draft TOR. The outlines of the draft TOR are shown below.

Contents of Draft Terms of Reference

1. Background
2. Proposed Project
    - 1.1 Objectives
    - 1.2 Project Area
    - 1.3 Outlines of the Project
3. Scope of the EIA Study
    - 3.1 The Study Area
    - 3.2 Environmental Items to be Assessed
    - 3.3 Surveys and Investigations
    - 3.4 Impact Identification and Assessment
    - 3.5 Preparation of Mitigation Measures
    - 3.6 Preparation of Environmental Management and Monitoring Plan
    - 3.7 Stakeholders Meetings
4. Report Making
5. Timeframe of the EIA Study
6. Expected Outputs

1.3.2 Schedule of an EIA Study and Estimated Cost for the Study

An environmental impact assessment study is to be carried out by a consulting firm or consultants registered in the state. In general, an EIA study to be contracted out to a consulting firm/consultants encompasses: i) the preparation and finalization of TOR for the study; ii) the conduct of the study; iii) the preparation of enironmental reports (RIMA report and report for public disclosure); and iv) the arrangement and organization of public audience. Hence, the TOR for an EIA study is to be first drafted by a consulting firm/consultants after it is officially selected. The draft TOR is to be reviewed, examined and approved by FATMA in the state within 45 - 60 days after its submission. An EIA Study shall be carried out in accordance with the approved TOR. A tentative works schedule of an EIA study and estimated cost for the study are shown in Tables 1.2 and 1.3, respectively.

Table 1.2 Tentative Schedule of EIA/RIMA Study

Items	**Months**											
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**(11*)**	**12**
Preparation of TOR	○	○										
EIA Study		○	○	○	○	○	○	○	○			
Preparation of RIMA										○		
Public consultations												○

Note*：The duration from the preparation of RIMA to the public consultations varies with the environmental examination institutions in the respective states
Source：JICA Study Team

Table 1.3 Estimated Cost for EIA STUDY

Unit: R$

Item	Unit Cost	Unit	Quantity	Contingency (20%)	Total
Preparation of TOR	6,880	MM	2	3,430	17,150
EIA Study	57,168	MM	3	114,336	571,680
Preparation of RIMA	36,587	L.S.	1	9,147	45,734
Public consultations	26,676.0	MM	1	6,644	33,220
				Total	667,785

Source：ECSA, Engenharia Socioambiental S/S

CHAPTER 2 NECESSARY LAND ACQUISITION AND RESETTLEMENT FOR PRIORITY PROJECTS

2.1 Results of Field Survey of the Target Area for Dam Heightening

2.1.1 Introduction

Oeste and Sul dams, which respectively are located in Taió city and Ituporanga city, are targeted for dam heightening. In the planning stage of the dam heightening, the state government as an implementing body shall acquire the areas, which will be potentially inundated or impounded by heightining dam crests of both dams, from land owners. Both dams are located in upper tributaries of Itajai River, Oeste dam in Itajaí do Oeste River and Sul dam in Itajaí do Sul River.

This section describes the results on the study on the potential social impacts caused by the dam heightening works and mitigation measures against potential impacts. Oeste and Sul dams both are flood control dams which usually have no strage water during the non-flooded period.

In Brazil, there are some existing Environmental Impact Assessment (EIA) studies for hydro-electric generation dam projects, while EIA for flood control dam[2] had not been made so far[3].

The boundaries of the areas affected by the construction of Oeste and Sul dams were not ble to be determined due to lack of information, such as their design and completion drawings, as the construction works and resettlement was implemented more than 30 years ago. Accordingly, there had been no major complains made by the surrounding communities about land acquisition and dam operations of each dam site.

As described in Section 1.2, the heightening of dam is expected to affect the areas from the elevation of the existing dam crests to that of the heightened dam crests, especially for Oeste dam. In order to grasp the actual conditions of the affected area, the JICA Survey Team conducted a field survey composed of literature study and site reconnaissance as described below.

2.1.2 Survey Method

(1) Literature Study

During the literature study, the following information and data were obtained from Deinfra - SC, which is the responsible agency for operation and maintenance of Oeste and Sul dams.

- Base map of Sul Dam which shows the distribution and locations of areas to be acquired
- Engineering drawings of Sul Dam Body
- Base map of Oeste Dam which shows the distribution and locations of areas to be acquired
- Results of trial evaluation of land prices of the areas to be affected by heightening both dams
- Results of interviews to the responsible agency of dam operations and the agricultural unions

Deinfra − SC has limited data and information relevant to Oeste and Sul dams possibly due to the transfer of the responsibility for operation and maintenance of dams from DNOS to Deinfra - SC.

(2) Site Reconnaissance

Site reconnaissance survey in Oeste and Sul dams was conducted during April 15-17, and April 14-16,

[2] Since flood control dam does not form the inundation area, the dam storage area can be accessible by the communities during non-flooded period.
[3] Environmental licensing system had not been established yet when Oeste and Sul dams were constructed.

2011, respectively with an aim to collect information required for development of a resettlement program with cost estimation. During the survey, the geographical data, such as latitude, longitude, and elevation of the houses and barns located in the affected area, were collected by using the receiver devices, TOPCOM GR-3 under Global Navigation Satellite System (GNSS).

2.1.3 Results of the Field Survey

(1) Current Condition of the Affected and Surrounding Areas of Sul Dam

As for the heightening of spillway of Sul Dam, which is one of the priority projects, the design flood water level was set as EL 410.0m in consideration of the maximum water level of 10,000-year flood and freeboard in accordance with the official design standards of Brazil[4].

According to the design drawings of Sul dam and information obtained from key informants in Deinfra - SC, land acquisition had been already completed up to EL. 410.0 m when existing dam was constructed. Heightening the spillway would not require further land aqcuisition in principle. However, the satelite images covering the affected area indicated that some buildings, such as houses and barns, were located below EL. 410.0 m. Hence, the site reconnaissance survey was conducted. During the site reconnaissance, due consideration was given to keeping the residents in the affected area from having the project information. The results of the site reconnaisance survey is shown in Attachment-3.

The survey revealed that six (6) buildings, four (4) houses with kiosk and two (2) log cabins, were located at between EL. 401.276 m and EL. 409.314 m.

Furthermore, the survey identified the present land use classes in the potentially affected areas under Sul dam as follows:

- Paddy field (class I): 10.0 %
- Onion farm (class III and IV): 25.0 %
- Slope area (class V): 5.0%
- Grassland (class VI and VII): 35.0%
- Permanent Preservation Area (APP) (class VIII): 25.0 %

Details of the land use classification are shown in Section 3.4.

(2) Current Condition of the Affected and Surrounding Area of Oeste Dam

Likewise, the design flood water level of Oeste dam was set as 365.0 m in consideration of the the maximum water level of 1,000-year flood and freeboard in accordance with the official design standards of Brazil. Consequently, heightening the existing dam by 2 m was proposed by raising the dam crest from EL. 363.15 m to EL. 365.16 m. Although the land acquisition was completed up to EL. 363.0 m when the existing dam was constructed, the rough estimation based on the topographic maps of 1:50,000 revealed that the additional 67 ha of lands still need to be acquired.

The present land use in the affected areas are classified as follows:

- Agricultural farm (class III): 10.0 %
- Agricultural farm for short-term crops (class IV): 25.0%
- Perenial crops planted area (class V): 30.0%
- Grassland (class VI): 20.0%
- Steep grassland (class VII): 5.0%
- Permanent Preservation Area APP (class VIII): 10.0 %

The site reconnaissance survey further found that there were two (2) wooden houses, three (3) wooden sheds and one (1) brick house with barn in the potential affected areas between EL. 361.988 m and

[4] Critérios de Projeto Civil de Ucinas Hidroelétricas", October, 2003

354.979 m.

2.2 Necessary Measures to be taken for Minimizing the Possible Impacts by the Heightening of Sul dam

As mentioned in 2.1.3, land acquisition was completed up to EL. 410.0 m by DNOS for construction of the existing dam. In 1981, an agreement on land use concession for the area between EL. 405.5 m and 410.0 m was concluded by DNOS and COOPERBASUL, which was a cooperative organized by the surrounding communities, to allow the members of COOPERBASUL to use the said area for animal husbandry. The contents of the said contract are shown in Supporting Report F.

Currently, Deinfra - SC, which the responsibility for operation and maintenance of the dam was given from DNOS, follows the said contract signed by DNOS without any revision and allows the members of COOPERBASUL to use the area based on the contract.

To date, there have been no serious trouble with COOPERBASUL and Deinfra − SC, despite the fact that the water storage level had sometimes reached to the maximum water level.[5] The heightening of dam, which would increase the possibility of innundation in the concession area, might cause a negative impact on the use of the area, although its possibiloty is least-likely.

It is therefore recommended that Deinfra - SC discuss the possible negative impact with COOPERBASUL to ammend the current agreement on the use of land concession area on this occaision.

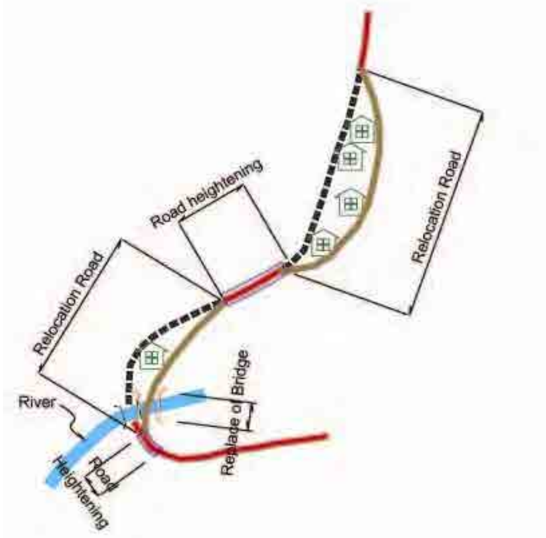
2.3 Mitigation Measures against the Possible Impacts caused by the Heightening of Oeste Dam

As described in 2.1.3, the heightening of Oeste Dam would require new land acquisition of 67 ha between EL. 363.0 m and EL. 365.16 m. The houses, sheds, some part of roads and bridges were located within the potential innundation areas as shown in Supporting Report F. With an aim to mitigate the possible negative impacts on the communities living in the possible innundation areas, JICA Survey Team proposes rerouting the existing roads and using them as dikes to protect the houses from being innundated.

Table 2.1 shows the general features of two (2) alternative measures, one with road relocation and the other with resettlement of the communities, while Table 2.2 compares the estimated costs for both alternatives.

[5] Several overflows from the spillway of Sul dam were recorded by Deinfra-SC.

Table 2.1 General Features of Alternative Measures

	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 ・Reduction of inundation area due to installation of the road	・Resettlement of houses/communities necessary
Project cost	R$ 4,797,000 (100%)	R$ 2,819,000 (58.8%)

Source: JICA survey team

Table 2.2 Cost Estimation

(R$)

			Alternative of Road relocation		Alternative of Compensation		Remarks
	unit	unit cost	quantity	amount	quantity	amount	
Replacement of Bridge	m2	3,000	160	480,000	80	240,000	
Relocation of 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	LS	966,000	1	966,000	1	966,000	
House Compensation	LS	326,000	---	---	1	326,000	3houses+3sheds
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**	

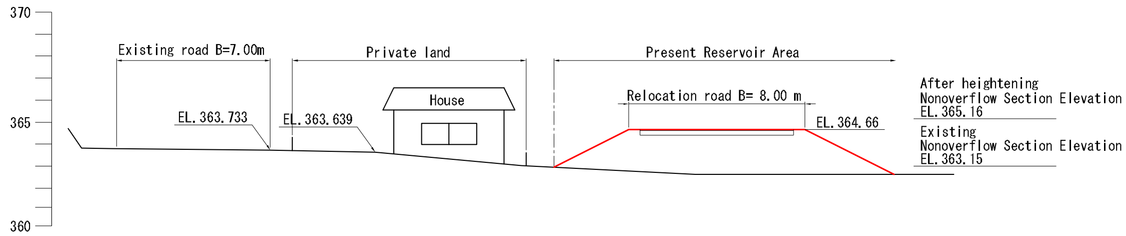
Source: JICA survey team

The cost breakdown for land acquisition and house compensation are also shown in Table 2.3.

Table 2.3 Cost Breakdown for Land Acquisition and House Compensation

Land Acquisition	
7.0ha of Class III and 17.0ha of Class IV exploited with crops (R$ 20,000.00/ha)	480,000.00
20.0ha of Class V of pasture (R$ 15,000.00/ha)	300,000.00
13.0ha of Class VI and 3.0ha of Class VII with restrictions (R$ 7,500.00/ha)	120,000,00
7.0ha of Class VIII of APP (R$ 4,300.00/ha)	30,000.00
24.0ha permanent crops (R$ 1,500.00/ha)	36,000.00
Total	**966,000.00**
House Compensation	
Masonry house (100.0m^2), masonry wall (240.0m), wooden shed (90.0m^2).	165,000.00
Wooden shed (72.0 m^2)	28,000.00
Wooden shed (60.0 m^2)	23,000.00
Wooden house (60.0 m^2)	30,000.00
Wooden shed (96.0 m^2)	36,000.00
Wooden house (90.0 m^2)	44,000.00
Total	**326,000.00**
Reserve +15%	194,000.00
GENERAL TOTAL	**1,486,000.00**

Source: JICA survey team



Source: JICA survey team

Figure 2.1 Typical Section of Relocation Road

<u>Conclusion</u>

In Brazil, compensation for properties affected by public works has been generally conducted in accordance with the relevant laws and regulations, while administrative proceedings have been often taken against the process of resettlement due to lack of the relevant legislation. In fact, there have been many troubles and complaints caused by insufficient and improper support/arrangement in resettlement, such as i) relocation to remote areas and ii) limited opportunities for employment in relocated areas, especially for professions that the resettled people used to engage in.

Although the cost for the alternative measure-1 with road rerouting is 1.7 times higher than that for the alternative measure-2 with resettlement as shown in Table 2.2, JICA Survey Team recommends alternative measure-1 as a more reasonable and justifiable plan in order to avoid and minimize future negative impacts in line with the basic principles of JICA Guidelines for Environmental and Social Considerations.

2.4 Process of Resettlement under Oeste Dam and the Proposed Resettlement Program

2.4.1 Introduction

While the JICA Study Team recommends the relocation of road, the state government might possibly select the alternative measure-2 since the number of target families to be relocated is limited. If so, the preparation and submission of necessary plans, such as resettlement plan and monitoring plan, will be required.

The following sections further describe the processes of resettlement and compensation.

2.4.2 Resettlement Process

(1) Type of Applicable Compensation Measures

The resettlement program aims to secure the livelihoods of the resettled families/people providing necessary support to restore their livelihoods but not to alter their cultural features.

According to the general process of resettlement in the country described in Section 9.3.3, JICA Survey Team proposes the following options as compensation measures for properties to be affected by the heightening of Oeste dam. Nevertheless, the amount of compensation for losses shall be valuated in accordance with World Bank Operation Manual (OP) 4.12, Annex A- Involuntary Resettlement Instruments (WB OP 4.12, Annex A)[6] as stipulated in JICA Guidelines for Environmental and Social Consideration (2010).

1) Compensation for Assets

Value of existing buildings and lands shall be appraised to determine the amount to be paid in cash to the affected families in accordance with the process described in Section 3.3. In valuation, it is important to estimate the compensation in accordance with WB OP 4.12, Annex A as described above.

2) Individual Resettlement (Commitment Letter, CC)

The target public is responsible for searching and selecting the land and other properties equivalent to those they originally owned. If the implementing body judges that the properties selected by the target public meets the criteria set in the agreement between the implementing body and the affected families, the implementing body shall purchase the selected properties.

3) Individual Resettlement of Special Cases (CE)

In case the target public has some limitations/handicaps who need a special care or considerations in its families, special arrangements shall be made such as allocation of urban lots instead of rural ones (but the size of the lots should be lower than the affected one.).

(2) Target Public for Resettlement

As for the heightening of Oeste dam, the following persons are expected to be the target public:

- the person who conducts economic activities in the affected area
- the person who lives in the affected area but has no legal property in the same
- the person who has his/her property/ies in the area remaining unused by heightening the dam
- the person who depends on the affected property/ies for his/her livelihood

[6] Compensation for losses shall be valuated by the replacement cost, which is defied below.
"For agricultural land, it is the pre-project or pre-displacement, whichever is higher, market value of land of equal productive potential or use located in the vicinity of the affected land, plus the cost of preparing the land to levels similar to those of the affected land, plus the cost of any registration and transfer taxes. For land in urban areas, it is the pre-displacement market value of land of equal size and use, with similar or improved public infrastructure facilities and services and located in the vicinity of the affected land, plus the cost of any registration and transfer taxes. For houses and other structures, it is the market cost of the materials to build a replacement structure with an area and quality similar to or better than those of the affected structure, or to repair a partially affected structure, plus the cost of transporting building materials to the construction site, plus the cost of any labor and contractors' fees, plus the cost of any registration and transfer taxes. In determining the replacement cost, depreciation of the asset and the value of salvage materials are not taken into account, nor is the value of benefits to be derived from the project deducted from the valuation of an affected asset. Where domestic law does not meet the standard of compensation at full replacement cost, compensation under domestic law is supplemented by additional measures so as to meet the replacement cost standard." (Source: World Bank OP 4.12, Annex-A)

(3) Resettlement Process

1) Individual Resettlement

In the case of individual resettlement, the implementing body shall issue the Commitment Letter (CC) which shows the estimated cost of the affected properties so that the target public can have an idea on the amount of compensation.

Upon the issuance of CC, the target public shall search and select the properties, and inform the implementing body of the selected properties. The implementing body shall examine the appropriateness of the selected properties prior to the approval of its acquisition. Having validated the legitimacy and appropriateness of the properties, the implementing body will permit the acquisition/purchase of the property and sign on to the official document for purchase.

In the procedures of individual resettlement, the following points shall be considered.

- Size of lot acquired
The size of lot acquired will range from 3.0 ha to 17.0 ha depending on the family structure and its type based on CSE (See Section 3.3). In determination of the size of lot, due consideration shall be paid to the concept of “replacement cost” defined in WB OP 4.12 Annex-A.

- Buildings for acquisition
Likewise, the buildings in each lot shall be determined in consideration of the family structure and type of houses based on CSE (See Section 3.3). Likewise, WB OP 4.12 Annex-A shall be referred for estimation of compensation for losses.

- Option for the commitment letter
The target public can prepare its request in writing on the acquisition of properties. Once such a request is developed in writing, no revision is allowed.

- Technical assistance for recovery of livelihoods
The implementing body is responsible for provision of ad hoc technical and social assistance for the target public upon the acquisition of the new property/ies to restore its livelihoods.

- Mode of reimbursement
In case that the target public is non-owner of the affected area, reimbursement of the resettlement cost might be required. The implementing body and the target public will go into negotiations and determine whether or not the repayment is arranged. In case the repayment is required, the mode of reimbursement such as total or partial reimbursement, should also be discussed between the implementing body and the target public.

2) Special Cases of Individual Resettlement (CE)

The special case of individual resettlement (CE) shall go through similar procedures and give similar considerations in the process of resettlement in principle. It is noted that the size of lot can be determined according to the features of the affected families.

The necessary considerations to be made in determining the size of lot are described below.

- Lots size for acquisition

Rural lots

Area remaining unused shall be preferably utilized for resettlement of the target public in the rural area.

The lower limit of the area of rural lots shall be eighteen (18) ha according to the Minimum Fraction of Parceling (FMP) as defined by the National Institute of Colonization and Agrarian

Reform – INCRA.

Urban lots

The target public who selects urban area to reside shall have the right to acquire the minimum size of lot in accordance with the central and/or local government legislation.

- Mode of reimbursement

The target public, who is the owner, heir, or other claimants of the affected lands and properties, shall be exempted from any payments, except the transfer of the affected lands and properties to the implementing body as a payment.

If the value of the existing properties acquired exceeds the benefit which the target public can obtain from the resettlement, the balance shall be paid in cash to the target public.

CHAPTER 3 ENVIRONMENTAL LEGISLATION, GENERAL RESETTLEMENT PROGRAM, AND MONITORING PROGRAM IN THE POST-RESETTLEMENT IN BRAZIL

3.1 Legislation relating to Compensation

The major legislation relating to environmental and social considerations in Brazil is presented in Section 7.2 in Part I of the master plan study.

The Constitution of the Federative Republic of Brazil in 1988 stipulates that any projects shall pay sufficient monetary compensation to people who own the areas affected by projects prior to its implementation. The same also stipulates that the compensation shall include the costs for the maintenance of the environmental quality, necessary environmental management works, and land acquisition for reforestation in permanent preservation areas (APP).

A proponent of a project shall have the legal responsibility for land acquisition for implementation of a project. In a dam construction project, the implementing body of the government shall be responsible for acquiring the construction sites following the proper process of land acquisition. In case the implementing body and the affected families/people can not reach an amicable agreement on land acquisition, the implementing body shall take the necessary legal measures for compulsory land acquisition.

In Brazil, various laws and regulations relating to land acquisition and compensations have been enacted and implemented since the first regulation was enacted in 1821. The applicable laws and regulations, which are currently effective, are shown below.

- Constitution of the Federative Republic of Brazil, as of 05/10/88;
- Decree-Law No. 3,365, as of June 21, 1941, and as amended, and complementarily the Code of Civil Procedures (CPC);
- Federal Law No. 4,132, as of September 10, 1962;
- Brazilian Association of Technical Standards (ABNT), as of 2004, for appraisal of rural properties (NBR 14,653-3), and of urban properties (NBR 14,653-2).

It is noteworthy that there is no legislation or regulation specifying the procedures for resettlement, although the regulations and guidelines on estimation of compensation and compulsory land acquisition are already in place. Therefore, in most of the projects in the past, the framework for resettlement needed to be determined through negotiations with the affected families/people.

The following sections highlight the relevant points of the existing legislation on land acquisition in Brazil.

3.1.1 Constitution of the Federative Republic of Brazil, 05/10/1988

The Constitution of the Federative Republic of Brazil of 1988, which was enacted on May 10, 1988, brought important innovations to the conditions of land acquisition. Clause 14 of Article 5 in the Constitution is summarized below.

Article 5: Everyone is equal before the law, with no distinction of any nature. The law shall ensure to the Brazilian people and foreign residents in the Country the inviolability of the rights to life, to freedom, to equality, to safety and to property, under the following terms.

Clause XIV: The law shall define the procedure for land acquisition for need or public interest, through a fair and previous compensation in cash, except for the cases provided for in this Constitution.

3.1.2 Federal Decree-Law No. 3,365 dated June 21, 1941.

The Federal Decree-law No. 3,365, as of June 21, 1941, provides for land acquisition for public-interest purposes. This Decree-Law specifies the rules and process of the land acquisition in Brazil and also referred to by the Code of Civil Procedures in Article 271. On January 29, 1999, this Decree-Law was partially amended by Law No. 9,785.

3.1.3 Federal Law No. 4,132 dated September 10, 1962

This Federal Law defines the procedures for land acquisition for public-interest purposes. It was amended by Law 6,513 in December 20, 1977 (art. 31).

3.1.4 Others

Federal Decree No. 24,643 of July 10, 1934, amended by Federal Decree No. 35,851 on July 16, 1954, defines so-called the Code of Waters in item "b" of Article 151 as shown below.

Article 151 item b: to acquire private buildings, and in pre-existing authorizations, the goods, including private waters upon which the concession is granted, and the rights which might be necessary, according to the law regulating land acquisition for public interest, being responsible for the resettlement and payment of compensations.

The Brazilian Association of Technical Standards (ABNT) in 2004 defines the standards for asset appraisal, such as general procedures in NBR 14,653-1, standards for asset appraisal in urban areas in NBR 14,653-2, and the same in rural areas in NBR 14,653-3.

3.2 Comparison between JICA's Guidelines on Environmental and Social Considerations and Relevant Legislation in Brazil

Table 3.1 shows a comparison between JICA's guidelines on environmental and social considerations and relevant legislation in Brazil.

Table 3.1 Comparison between JICA's Guidelines on Environmental and Social Considerations and Relevant Legislation in Brazil

No.	Descriptions	Relevant Legislation in Brazil and their Summaries
1) JICA Guidelines for Environmental and Social Consideration		
1.	Involuntary resettlement and loss of means of livelihood are to be avoided when feasible by exploring all viable alternatives. When, after such an examination, avoidance is proved unfeasible, effective measures to minimize impact and to compensate for losses must be agreed upon with the people who will be affected.	Not available (or follow social convention / protcol)
2.	Peopl who must be resettled involuntarily and people whose means of livelihood will be hindered or lost must be sufficiently compensated and supported by project proponents etc. in a timely manner.	Not available (or follow social convention / protcol)
3	Host countries must make efforts to enable peole affected by projects and to improve their standard of living, income opportunities, and production levels, or at least to restore these to pre-project levels.	<u>Article 5 of Clause 24 in the Constitution of Brasil</u> The article stipulates the procedures for land acquisition with fair and advance monetary compensation in consideration of the public benefit and necessity. <u>Decree-Law No. 3365 (June 21,1941)</u> This dicree stipulates the rules on and processes of condemnation/expropriation for public projects as well as any other purposes for public interest. The decree deifines that an owner of properties expropriated/acquired for pubic interest shall be compensated in cash.
T	Prior comensation, at full replacement cost, must be provided as much as possible.	<u>Article 5 of Clause 24 in the Constitution of Brasil</u> Same as above.
5.	For projects that will result in large-scale involuntary resettlement, resettlement action plans must be prepared and made available to the public. In preparing a resettlement action plan, consultations must be held with the affected	<u>CONAMA Resolution No. 01 (as of Jan. 23, 1986)</u> The resolusion stipulates that an EIA report shall be discosed to the public and SEENV or the municipality government should hold the public

No.	Descriptions	Relevant Legislation in Brazil and their Summaries
	people and their communities based on sufficient information made available to them in advance.	hearings or consultation meetings on a project and its potential impact. CONAMA Resolution No. 09 (as of Dec. 3, 1987) The resolution defines the purpose of the public hearings/consultations, outlines of public hearings/consultations (e.g., timing, timeframe, frequency, and venues), responsible agency, and the necessity of documentation of the hearings/consultations.
6.	When consultations are held, explanations must be given in a form, manner, and language that are understandable to the affected people.	CONAMA Resolution No. 01 (as of Jan. 23, 1986) Same as above. CONAMA Resolution No. 09 (as of Dec. 3, 1987) Same as above.
7.	Appropriate participation by affected people and their communities must be promoted in the planning, implementation, and monitopring of resettlement action plans and measures to prevent the loss of their means of livelihood.	Not available (or follow social convention / protcol)
8.	Appropriate and accessible grievance mechanisms must be established for the affected people and their communities.	CONAMA Resolution No. 01 (as of Jan. 23, 1986) Same as above. CONAMA Resolution No. 09 (as of Dec. 3, 1987) Same as above.
World Bank Safeguard Policy, OP 4.12 and OP 4.12, Annex A		
9.	Upon identification of the need for involuntary resettlement in a project, the borrower carries out a census to identify the persons who will be affected by the project, to determine who will be eligible for assistance, and to discourage inflow of people ineligible for assistance. (WB OP4.12 Para 6) The results of a census survey covers: (i) current occupants of the affected area; (ii) standard characteristics of dispalced households; (iii) the magnitude of the expected loss; (iv) information on vulnerable groups or persons; and (v) provisions to update information on the displaced people's livelihoods and standard of living. (WB OP4.12 Annex A Para 6)	Federal Decree No. 7342 (as of October 26, 2010) This decree institutionalizes the registration of the losses of properties of the persons who will be affected by a dam construction project for hydroelectirc generation. Types of losses to be registered and the organization (the inter-ministrial committee) to administer the registration are defied in the decree.
10.	Displaced persons may be classified in one of the following three groups: (a) those who have formal legal rights to land; (b) those who do not have formal legal rights to land at the time the census begins but have a claim to such land or assets-provided that such claims are recognized under the laws of the country or become recognized through a process identified in the resettlement plan; and (c) those who have no recognizable legal right or claim to the land they are occupying. (WB OP4.12 Para 15)	Not available (or follow social convention / protcol)
11.	Preference should be given to land-based resettlement strategies for displaced persons whose livelihoods are land-based. These strategies may include resettlement on public land, or on private land acquired or purchased for resettlement. (WB OP4.12 Para 11)	No regulation
12.	The resettlement plan or resettlement policy framework also include measures to ensure that displaced persons are: (i) offered support after displacement, for a transition period, based on a reasonable estimate of the time likely to be needed to restore their livelihood and standards of living; and (ii) provided with development assistance in addition to compensation measures, such as land preparation, credit facilities, training, or job opportunities.(WB OP4.12 Para 6)	Not available (or follow social convention / protcol)
13.	To achieve the objectives of this policy, particular attention is paid to the needs of vulnerable groups among those displaced, especially those below the poverty line, the landless, the elderly, women and children, indigenous peoples, ethnic minorities, or other displaced persons who may not be protected through national land compensation legislation.(WB OP4.12 Para 8)	Not available (or follow social convention / protcol)
14.	In case that impacts on the entire displaced population are minor or fewer than 200 people are displaced, an abbreviated resettlement plan may be required.(WB OP4.12 Para 25)	Not available (or follow social convention / protcol)

Source: JICA Study Team

Although legislation and regulations on land acquisition and compensation are in place in Brazil, the resettlement and livelihood support after relocation have been generally based on the social convention or protocol according to Brazilian Constitution so far. However, the environmental examination for environmental licensing has been getting strict and examining a resettlement plan with its monitoring plan in the examination as there have been many troubles and complaints caused by insufficient and improper support and arrangements in resettlement, such as i) relocation to a remote area and ii) limited opportunities for employment in a relocated area, especially those for the professions that the affected people engaged in before resettlement.

It is therefore necessary for the state government to carry out an EIA study and resettlement measures pursuant to the JICA guidelines (JICA Guidelines for Environmental and Social Consideration) and those used by the international funding institutions, such as World Bank, so that the state government could avail of the loan scheme of the international funding institutions for implementation of the Project.

Consequently, it is recommended that an EIA Study should be carried out in accordance with the draft TOR attached to this report. Considering the sound financial status of the state government, the JICA Study Team judges that it would not be much difficult for the state government to finance the conduct of the EIA study for the project.

3.3 General Resettlement Procedures in Brazil

3.3.1 Basic Concepts

Some basic concepts to be considered in the formulation of a resettlent program for the people who would be affected by the construction of a dam are defined:

Socioeconomic records (questionnaire) – CSE: means a structured interview survey with a set of preset questionnaires. As CSE aims to statistically analyze the socio-economic profiles of the affected families in a quantitative and qualitative manner and prepare a resettlement program for all the potential families regardless of the possession of land and other assets in the affected areas, the interview survey shall target all the families living in the potential inundation areas and construction site.

Directly affected area: or simply **"affected area"**: means the area/areas that will be used for construction or heightening of dam crests and those that might be inundated or impounded after the heightening of a dam. A strip of permanent preservation area (APP) around the permanent reservoir area is also included in this category. APP is not necessarily formed in a flood control dam in general, but it might be required when necessary.

Affected property and affected people: means assets and families associated with or located in the affected areas. The terms are used as herein defined except when otherwise explained.

Workforce (FT): means the number of available workers/laborers who are engaged in agricultural development and exploitation works. The current data on workforce are estimated on the basis of the age structure in each family.

"Target Public" of a resettlement program: means the people directly and indirectly affected by the implementation of a project, which include; land owners, illegal occupants, tenants, investor and its partners, community organizations, salaried workers, and children of land owners and the other relevant people. The target public shall be defined in each step of a resettlement program.

3.3.2 Resettlement Measures and Alternatives

In general, the target public of a resettlement program under a dam construction project for hydropower generation will be compensated or provided an alternative land for resettlement. The outlines of the resettlement measures are summarized below.

Compensation consists of the total or partial acquisition, with cash payment, of affected properties and/or areas including the unfeasible remaining areas[7] and any profits to be generated from the economic activities based on/in the affected properties as well as areas. The amount of the compensation will be determined by a mutual agreement between a proponent and the respective affected families/persons.

Provision of alternative lands means to provide the target public with alternative lands and assists them in relocating themselves to such lands. It consists of the following types of measures.

a) Collective Rural Resettlement (RRC): where rural lots with basic social infrastructure will be allotted to the whole communities in the affected areas.

b) Small Rural Resettlement (PR): where individual electrified houses and barns with a water supply system will be allotted to the affected families.

c) Individual Resettlement (Commitment Letter, CC): where the individual affected families will have negotiations with a proponent and determine the value of the affected areas and properties. In principle, each family is responsible for searching his/her relocating land and its associated facilities, which should be equivalent to the values of his/her affected properties/areas. The acquisition and registration of such properties shall be done by a proponent.

d) Resettlement in Remaining Area (AR): where land use rights for farming will be granted to the affected families. The areas that will neither be affected by a project nor be designated as APP among those acquired by a proponent for a project will be used for this purpose.

e) Resettlement in Special Cases (CE): where a special arrangement, such as arrangement for urban lots or downsizing of lots from the original plan, will be made for the affected families that have persons in need of special attention (e.g., the aged and disabled).

Due consideration shall be given to the socio economic aspects of the affected families in the preparation of a resettlement program. In particular, the conservation of customary norms/customs related to land and traditional culture in the affected areas shall be considered in a resettlement program.

Furthermore, in case CSE reveals that any indigenous communities or special social categories (such as Quilombos) might be affected by a project, specific standards should be employed to pay due attention to their traditional and cultural characteristics and peculiarities.

In principle, each target public shall select the resettlement measure by themselves considering the respective socio-economic as well as traditional characteristics and according to the guidelines and criteria based on the case studies in the past.

Furthermore, after the estimated values of the affected areas are presented to the target public, additional options should be determined and selected on the basis of the results of CSE in a participatory manner. Such a participatory process and continuous discussions would enable them to appraise their own conditions and determine appropriate resettlement measures.

3.4 Procedures for Compensation and Land Acquisition

The following sections describe the procedures taken for compensation and land acquisition in a dam construction project for hydropower generation as an example for the administrative procedures for compensation and land acquisition. Although these procedures are considered applicable to a project for the construction of a flood control dam in principle, there is a need to further examine whether or not all the procedures described below can be applied to the proposed project in a further study, since the existing flood control dam projects are rather scarce as compared to those for hydro-generation

[7] The unfeasible remaining area means the area located outside the affected area but owned by the same owner and where the owner will not be able to gain profit from any economic activities based in.

dams.

Based on the mutual agreements on land acquisition between a proponent and the affected families/people, compensation for the affected properties and lands and assets including the lands remaining unused due to a project should be made in money in principle. The close communication and good relationship between a proponent and the affected families/people is crucial to smooth progress of the land acquisition process as the land acquisition is based on negotiations with owners of the affected properties in principle. The amount of compensation for the affected lands and properties will be estimated on the basis of data and information collected through a market research on the prevailing market prices, in addition to the comments from the representatives of the owners of the affected lands and properties.

A field survey is to be carried out to assess the affected lands and properties with the presence of the land owners or their delegates/agents. The survey shall cover the lands that would not be directly affected by a project but should remain unused due to project activities and the properties owned by tenants or illegal occupants who do not have the ownership of the lands as well.

In case a proponent and land owners can not reach a mutual agreement on compensation, the compulsory land acquisition process will be taken based on the Public Declaratory Resolution on Public Facilities in the Affected Areas issued by the National Agency of Electric Energy (ANEEL). The Resolution is applicable to only the case when an amicable agreement with land owners is judged impossible to reach.

The procedures for compensation along with the implementation of a project are summarized below.

(1) Preparation of the Registration Sheet

Prior to the field assessment surveys on the lands and properties that would be affected by a project, project outlines and other relevant information shall be disclosed to the owners or persons responsible for management of the properties as the first step of the process. In simultaneous with the disclosure of the project information, a proponent shall explain the procedures for land acquisition and compensation as well as the applicable guidelines on the same to them. At the same time, a field survey team will take the formal permission from the owners for entering the affected areas for assessment. The survey team shall prepare the Registration Sheet for each owner’s properties filling in data and information of the target properties and obtain informed consent for the sheet from the owner with his/her signature.

(2) Demarcation of Maximum Flood Elevation Line and of the Permanent Preservation Area (APP)

A field survey shall be carried out to delineate the maximum flood elevation level and boundaries of APP along the permanent reservoir[8] so as to clearly demarcate the areas to be inundated/impounded.

(3) Determination of the Acquired Land and Properties

The distribution and areas of the potential land use types in the acquired lands shall be clarified by delineating the boundaries of the acquired lands and classifying the potential land use types in the same.

(4) Market Research on Prices and Determination of Unit Values

Unit values for buildings (non-reproductive immovables) and for the perennial crops (reproductive immovables) will be set to estimate the costs of replacement and reproduction. The information related

[8] There is no official comment from environmental agency (FATMA), it is not clear whether it is necessary to set APP around the temporal reservoir.

to agricultural production (e.g., agricultural input, labor, and other operational costs including sales) and any other economic activities in the area shall be used for setting the unit values.

The value of the bare land will be estimated on the basis of information and data collected from the market research and interviews to available sources, such as realtors, notaries, brokers, unions, municipalities, banks, and other agricultural experts. It is necessary to collect sufficient information to set the reliable prices/values in line with the standard prices in the region.

An inspection survey shall be carried out to clarify the features of the lands, such as potential land capability, current management practices, and accessibility, for land acquisition. The unit value for compensation shall be determined by estimating the price of the bare land based on a statistical analysis of the above-mentioned data and assessing the quantity as well as quality of buildings and perennial crops existing in the lands.

The procedures described above are essential to ensuring the reliability of appraisal of the amount of compensation and avoiding future disputes that might be caused by any speculations.

The market research shall be carried out by an expert. A proponent and the affected families/people shall select the representatives who will verify the process of the survey to ensure the validity of the results of the research. In general, the market research shall target the affected families/people and those who own similar properties in the surrounding areas/municipalities.

Once the surveys for setting the unit values are completed, a matrix table showing the amounts of compensation shall be prepared. The table shall be reviewed and examined by a proponent and the representatives of the affected families/people for approval. The amounts of compensation shall be reviewed every six months and updated whenever the market prices rise drastically.

Table 3.2 shows the sample amounts of compensation described in Commitment Letter for individual houses around Oeste Dam.

Table 3.2 Cost Estimation for Commitment Letters

SITUATIONS	HOUSE VALUE	ROOF VALUE	LAND VALUE	TOTAL (R$)
House Type I = 54.00 m^2	31,398.00	33,480.00	202,550.00	267,428.00
House Type II = 63.00 m²	35,961.00	33,480.00	202,550.00	271,991.00
House Type III = 72.00 m²	40,963.00	33,480.00	202,550.00	276,993.00
House Type I. Minimum S = 40.50 m^2	23,388.00	0.00	45,000.00	68,388.00
House Type II. Minimum S = 45.00 m^2	24,536.00	0.00	45,000.00	69,536.00
House Type III. Minimum S = 50.00 m^2	23,388.00	0.00	45,000.00	68,388.00
House Type I. Maximum S = 40.50 m^2	23,388.00	23,315.00	137,500.00	184,203.00
House Type II. Maximum S = 45.00 m^2	24,536.00	23,315.00	137,500.00	185,351.00
House Type II. Maximum S = 50.00 m^2	26,659.00	23,315.00	137,500.00	187,474.00

Source: JICA survey team

(5) Literature Survey

A literature survey shall be carried out to review and analyze the processes necessary for i) transfer of ownership of the lands and properties and ii) acquisition of easements recorded in the recording office and notary public office. Furthermore, this survey aims to i) confirm the ownership of the affected lands and properties; ii) identify the potential beneficiaries of compensation; and iii) collect other documents relating to lands and properties (e.g., legal documents, payment of taxes, registration of pledge, and registration and transfer of mortgage), to verify the validity of compensation.

(6) Assessment of Affected Properties

To assess the lands and properties affected by a project in a qualitative and quantitative manner, a field survey shall be carried out to clarify the land-related information (i.e., i) classification of potential land capability, ii) present land use, iii) forest classification, iv) vegetation and forest covers, and v) density of forests in the affected areas) and to make an inventory of the existing properties and facilities, such as perennial crops, buildings, roads, electric power lines, telephone lines, wells, ponds, springs being used for water supply, community facilities, and tourism facilities.

The field survey shall be conducted by experts or a professional company with a wide range of expertise, and its results will be evaluated and validated by a proponent.

(7) Appraisal of Affected Properties

The appraisal of properties shall be conducted in accordance with the official standards of the country for land evaluation[9], namely NBR 14653-3 and NBR 14653-2, which shall be applied to rural and urban properties, respectively.

Criteria for appraisal of reproductive immovables, such as perennial crops, and non-reproductive immovables, such as buildings, shall be determined by a evaluation method for immovables and economic values for crops.

The appraisal of buildings shall be estimated on the basis of the costs estimated for wrecking of buildings, transportation of materials, and rebuilding. In the case of residential buildings, the amount of appraisal estimated in the same manner shall be the basis for determining the range of the rental cost.

On the other hand, the appraisal of lands shall be based on the land capability, which has the following eight (8) classes.

Class I: Arable land without any limitations on production of annual and perennial crops, usage of pasture, and planting of trees. The soils are fertile and have a deep effective soil layer with a high capacity of water retention. The area has a low risk of flood and no shallow groundwater.

Class II: Arable land with few limitations on crop production and soil conservation. For example, the soils indicate either less or excess CEC (Cation Exchange Capacity) and needs some amendment for crop production. However, the area can be used for crop production with proper management in general.

Class III: Arable land with some limitations on crop production and soil conservation. The area would be rapidly degraded without application of soil conservation measures or other necessary management practices. The area might need to introduce complex conservation measures for production of annual crops suitable for the climatic conditions. In case of sloping land, the area is further classified into sub-classes according to the slopes. Intensive farming will accelerate the possibility of soil erosion. On the other hand, a risk of flood is the major limitation of this class of area in the plain land.

Class IV: Land only used for cropping in a short period of time and not used for crop production continuously over years. The soil fertility is low to medium and clay content in soil texture is 15~60%. The soils are generally deep and rather well drained.

Class V: Land suitable for perennial crops, pasture, and trees, but not for annual crop. The soils are rather shallow (less than 1.2 m) and have gravel fraction (less than 5 % gravel content). The area is

[9] Developed by Brazilian Association of Technical Standards (ABNT)

rather dried.

Class VI: Land in which perennial crops, pasture and trees can be grown but no annual crop can grow. The soils are infertile, well drained, and with 5~10 % gravel content, although the area is flat with rather deep soils (more than 2 m).

Class VII: Land in which perennial crops and trees may not grow well. Like the area categorized as Class VI, the area needs to introduce appropriate soil conservation measures and other land management practices to minimize the soil erosion/degradation potentials. The soils extending flat to gently rolling terrain are infertile, well-drained, and rather shallow (less than 0.5 m). The area is dried.

Class VIII: Land not suitable for crop production or afforestation/reforestation. Consequently, the area can be used only for habitat for wild animals, sites for recreation, and water storage or harvesting facilities. Inundation area, mangrove forest, and barren or rocky area are calssified as this class.

Even if the area is owned by Navy, the same procedures for appraisal shall be followed. A simplified estimation is not allowed for the land owned by Navy.

If any floras of native species exist in the remaining unused area, the value of such floras shall be appraised and compensated along with other properties. However, those in permanent preservation area (APP) shall be kept untouched and maintained as they are. The value can be appraised but any alternation is not allowed.

Compensation for short-term crops will not be made if the notice of the date of resettlement is made more than six months before. On the other hand, if the notice is made less than six months before and short-term crops can not be harvested by the time of resettlement due to time constraints, the value of short-term crops shall be compensated.

Roads, wells, water supply systems, and electric lines in the affected areas shall be compensated by rebuilding/reconstructing the same based on the cost evaluation method.

(8) Administrative Technical Reports

An administrative technical report, which is to be used for the reference for compensation, shall be prepared for the respective properties. The report describes i) the expected values of land/property, ii) the potential effects caused by a project, and iii) the amount of compensation.

(9) Negotiations

The negotiations for property acquisition shall not involve anyone who might envision obtaining economic or political benefits from the negotiations, but be made through a direct communication with each owner of property

Compensation shall be based on the administrative technical reports on the respective affected properties. A proponent for a project shall be responsible for issuance of a deed of transfer and registration of the lands to be transferred to the affected families/people in the recording office. The final payment of compensation shall be adjusted by deducting the expenses for registration of new properties for the affected families/people.

(10) Payment

A proponent shall make a payment within 30 days, on the condition of the submission of an ownership certificate, from the date of the mutual contract on the amount of compensation. In the cases of Individual Resettlement (CC) and Special Cases (CE), a part of the final payment might be used for procurement of new properties.

(11) Deadline for Transfer of Properties

In case that the payment of compensation is made or Individual Resettlement (CC) is selected as the method for acquisition, the owner shall transfer the occupied land to a proponent by the deadline for transfer. The deadline shall be determined by a proponent in principle, but it should be adjusted with the conditions of the owners, especially when buildings/facilities are removed in the affected area.

As long as an implementation schedule of a project is not affected, the deadlines for transfer of the affected lands can be extended within a certain timeframe as an exceptional case. In that case, a proponent shall make an agreement on free use of the acquired land with the affected families/people, so that they could use such areas until the new deadlines set by the agreement.

(12) Compulsory Acquisition in accordance with Public Utility Declaration Resolution - DUP

Compulsory land acquisition shall be carried out in accordance with the ANEEL Resolution No. 279/2007 otherwise known as the Public Utility Declaratory Resolution. DUP would be applied in case the amicable land acquisition can not be made due the breakdown of the negotiations on compensation or the defect in the documents on land ownership, which are the bases for a proponent to pay compensation. In some projects, compulsory compensation might be undertaken at the state or municipal level.

In the early stage of the project, sufficient deliberation on whether or not the Resolution can be applied to a flood control dam project shall be made in consideration of the nature of the project. In fact, the Resolution stipulates the legitimacy of land acquisition for a hydropower project, and therefore, the same is considered applicable to a flood control dam as its nature is similar to a hydropower dam.

(13) Granting of Ownership

In case the legal process (or compulsory acquisition) for granting of ownership is required due to the breakdown of the negotiations or disputes over the conveyance of estate, an expert report relevant to the issue shall be prepared for granting of ownership at least six months before the start of impounding.

(14) Prioritization of Properties to be acquired

The land acquisition of the inundation areas shall be carried out in both river banks, from the downstream to upstream if possible. Furthermore, the following lands should be prioritized.

- Construction site and access roads to the site
- Areas to be fully affected or fully acquired
- Areas to be partly affected and whose owners prefer to have partial compensation
- Areas identified as partially affected

(15) Criteria to Examine the Possibility of Continuation of Livelihood Activities in the Remaining Areas

The existing livelihood activities in the remaining areas shall be assessed to examine the possibility of the continuation of them. If the assessment reveals that i) the cost incurred for basic infrastructure necessary for the existing livelihood activities in the remaining areas will be higher than that for resettlement or ii) the remaining areas are too remote from basic infrastructure to maintain the existing livelihoods, the continuation of the livelihood activities would be judged impossible. In addition, in case the remaining areas are susceptible to landslide due to its slopes or geological characteristics, the continuation of the livelihood activities would not be allowed.

In case any investment have been made for agricultural development even in the area where no agricultural activity had been undertaken before a project, the possibility of continuation of the agricultural activities in such areas shall be deliberated.

If the existing agricultural activities, namely livelihood activities, can be continued in the remaining area, a proponent shall only acquire the areas to be inundated/impounded and its surrounding areas for protective vegetation strips (APP).

On the other hand, if the continuation of the existing activities is considered impossible or infeasible to support the livelihoods of families, a proponent shall acquire the whole area including the remaining area unless the owners of the areas officially request the exclusion of the remaining areas from land acquisition.

3.5 Typical Resettlement Monitoring Program in Brazil

3.5.1 Introduction

Due consideration should be given to: i) agricultural production and economic activities (e.g., agricultural activities, any complementary activities, market supply, and commercialization); ii) social interaction (e.g., resumption of community activities and reformation/restructuring of social structure); and iii) arrangement of basic infrastructure (e.g., houses, education facilities, health facilities, and transportation facilities) during the process of resettling the affected families/people. To this end, a systematic monitoring program needs to be implemented during the process of resettlement, so as to ensure the transparency of the process and remedy the resettlement activities when necessary.

3.5.2 Justification

The main aim of the monitoring program is to identify the positive and negative aspects of the various measures (monetary compensation, commitment letter, resettlement in the remaining areas, individual resettlement, and special arrangements/cases) taken for development of new communities in the course of resettlement

The monitoring program is to target thee families/people resettled/relocated ("resettled/relocated families/people"). Having analyzed the negative aspects identified, the monitoring program is to provide effective alternatives and means to minimize the negative factors, such as provision of technical and social guidance.

Forcing people to change the living environment by any reasons other than personal interest might cause social disruption or seriously threaten the basis of civil society. It is therefore important to restore the lifestyle of the affected families/people, provide necessary support for restoration, and continue monitoring of the resettled/relocated families/people in new areas. The monitoring program would be helpful in having the feedbacks from the resettled/relocated families/people and identifying the needs of technical and social assistance for them.

3.5.3 Objectives

The main objective of the monitoring program is to collect the information relevant to the resettled/relocated families/people in the different stages of a resettlement program to evaluate the process of resettlement from the economic and financial viewpoints and to propose any improvement when necessary. Specifically, the program aims to:

- evaluate the changes in lifestyle of families/people living in the areas directly or indirectly affected by a project in the different stages of a resettlement program
- validate the effectiveness and validity of a resettlement program
- monitor the families who recognize the discrepancy between the plan of a resettlement program and the results of the same or who propose revising the guidelines adopted for resettlement

3.5.4 Goals

The goal of the monitoring program is to propose the necessary activities for improvement of a resettlement program as described above. The milestones to be achieved by the monitoring program are to monitor and survey all the resettled/relocated families in different locations at the respective stages (i.e., T0, T1, T2, and T3 stages) within three years from resettlement.

3.5.5 Environmental Indicators

Information to be monitored are: i) the level of satisfaction, ii) the level of family income, and iii) the level of solidarity of new communities of the resettled/relocated families. Furthermore, the following environmental indicators shall be monitored for three years.

- Opposition movement of resettled families/people against the results of the resettlement program;
- Proportion (Percentage) of resettled families/people satisfied with the effectiveness of resettlement;
- Proportion (Percentage) of resettled families/people who remove to other places in a short period of time;
- Degree of crop diversification;
- Proportion (Percentage) of resettled families/people who are able to engage in a job/occupation that enable them to enhance their standard of living;
- Level of increase of crop productivities
- Changes in average family income
- State of adaptation of resettled families/people to the respective new locations
- Level of improvement of social indicators
- Degree of discrepancy between the plan and results of the program
- Degree of introduction of new technologies in agricultural production.

3.5.6 Target Groups

The monitoring program is to target: i) directly-affected people, ii) people forced to relocate, iii) people without house or employment, and iv) people who are not able to obtain property compensation.

3.5.7 Basic Concepts

The resettled families/people should be followed up and the resettlement measures and their process shall be evaluatd in the different stages of a resettlement program. The results of CSE/socio-economic survey will be used for evaluation.

Stage	Aims of monitoring
T0-T0 Stage (when the monitoring activity starts):	Survey and grasp the socio-economic conditions of the affected families/people prior to resettlement through CSE.
T1- T1 Stage (six months after resettlement):	Evaluate the current situations of the resettled/relocated families.
T2- T2 Stag (18 months after resettlement):	Assess the socio-economic conditions of the resettled/relocated families.
T3- T3 Stage (30 months after resettlement):	Evaluate the stability of the resettled/relocated families

3.5.8 Methodological procedures

The monitoring program is to employ quantitative and qualitative surveys, a questionnaire survey using a set of questionnaires, and a semi-structured interview survey to grasp the feelings and sentiments of the ressetled/relocated families. A quantitative survey can reveal the level of satisfaction/dissatisfaction and other qualitative socio-economic aspects before and after resettlement, while the quantitative survey can measure the inter-annual changes in the pre-determined milestones and environmental indicators. Those surveys to be employed shall encompass different approaches that have the respective pre-determined timeframe, interrelate each other, and have the respective clear aims and methodologies.

The results of CSE shall be used as the baseline data to clarify the changes in socio-economic conditions of the resettled families/people through periodical monitoring activities.

Monitoring activities will be carried out in accordance with the following timeframe:

- T0: Before resettlement of families
- T1: Six months after resettlement
- T2: One year after T1
- T3: another one year after T2, when the life of the resettled/relocated families would become stable

3.5.9 Development of Program

The monitoring program is to be developed in consideration of its timeframe and methodologies required.

(1) "T0" STAGE – Before Resettlement

An interview survey will be carried out to determine the baseline of the ressetled/relocated families before resettlement. Hence, the families to be affected by a project will be targeted by this monitoring activity. The changes in the socio-economic conditions will be assessed on the basis of the data collected in this stage.

(2) "T1" STAGE – Six Months after Resettlement

A semi-structured interview using questionnaires will be conducted six month after resettlement to assess if the resettled/relocated families are adaptable to their new environment and evaluate if the unification of the resettled/relocated families as a new community progress as planned. Feelings and sentiments of resettled/relocated families along with good and bad points of the results of resettlement will be grasped through such an interview survey.

(3) "T2" STAGE – One Year after "T1"

The survey at this stage aims to grasp the socio-economic conditions of the resettled/relocated families considering the vulnerability of the respective families. Data on the second year cropping, such as area cultivated, crop yields, and sales of products, will be collected and analyzed for this purpose. The same questionnaires used in T0 Stage will be used in this stage.

(4) "T3" STAGE – One Year after "T2"

This stage aims to assess the degree of social stabilization of the resettled/relocated families by evaluating the effect of compensation payment on the household economy at the third year.

Hence, a structured questionnaire survey, which would enables a quantitative economic and financial analyses with cross-checking and social interaction analysis of the resettled/relocated families, will be carried out in this stage.

3.6 Environmental Management and Monitoring Programs

All the activities relating to envrionmental management, supervision and monitoring in the construction and operation phases shall be planned in EIA/RIMA as the environmental management and monitoring programs in accordance with the methods/procedures defined by the relevant environmental legislation in Brazil. Contents and composition of envoironmental management and monitoring programs will be determined and finalized in consulation with the relevant organizations in the process of the preparation of EIA/RIMA.

The following items shall be included in the envoironmental management and monitoring programs.

i) Environmental Lisence:	It shall describe the environmental impact assessment study and the proposed measures to avoid and mitigate the expected impact.
ii) Environmental Management and Control:	It shall include the descriptions about environmental training for laborers, health management of laborers, and pollution preventive measures (e.g., dust, traffic, noise, and vibration).
iii) Enviromental Monitoring:	It describes a monitoring plan in the construction and operttaion phases.

A proponent shall be responsible for environmental monitoring, but its implementation (e.g., field monitoring activities) is generally contracted out to a consulting firm/consultants or university.

Attachment-1 Environmental Checklist for the Priority Projects

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
1 Permits and Explanation	(1) EIA and Environmental Permits	(a) Have EIA reports been already prepared in official process? (b) Have EIA reports been approved by authorities of the host country's government? (c) Have EIA reports been unconditionally approved? If conditions are imposed on the approval of EIA reports, are the conditions satisfied? (d) In addition to the above approvals, have other required environmental permits been obtained from the appropriate regulatory authorities of the host country's government?	(a) N (b) N (c) N (d) N	(a) The proposed projects has been just identified and selected by the JICA Preparatory Survey as the proposed measures to be implemented in the first phase, recently. Hence, the official process of EIA has yet to start so far. (b) ditto (c) ditto (d) ditto
	(2) Explanation to the Local Stakeholders	(a) Have contents of the project and the potential impacts been adequately explained to the Local stakeholders based on appropriate procedures, including information disclosure? Is understanding obtained from the Local stakeholders? (b) Have the comment from the stakeholders (such as local residents) been reflected to the project design?	(a) N (b) N	(a) Same as above. Explanation of the proposed project to the local stakeholders shall be made in the course of a EIA study. (b) ditto
	(3) Examination of Alternatives	(a) Have alternative plans of the project been examined with social and environmental considerations?	(a) Y	(a) The JICA Preparatory Survey has assessed the alternatives with social and environmental considerations.
2 Pollution Control	(1) Water Quality	(a) Is there a possibility that changes in river flow downstream (mainly water level drawdown) due to the project will cause areas that do not comply with the country's ambient water quality standards?	(a) Y	(a) There is a possibility of polluting the quality of the downstream water by the construction of two floodgates on the Itajai Mirim River; however such a potential impact could be negligible by the application of a proper construction method, such as a prevention measure to treat turbid water. The heightening of Oeste dam might also cause the adverse impact on the river environment since the engineering works will need to divert the main stream of the river during the construction works. However, such an effect could also be minimized by the application of a proper construction method.
	(2) Wastes	(a) In the case of that large volumes of excavated/dredged materials are generated, are the excavated/dredged materials properly treated and disposed of in accordance with the country's standards?	(a) N	(a) No potential impact can be expected.
	(3) Subsidence	(a) Is there a possibility that the excavation of waterways will cause groundwater level drawdown or subsidence? Are adequate measures taken, if necessary?	(a) N	(a) No potential impact can be expected.
3 Natural Environment	(1) Protected Areas	(a) Is the project site located in protected areas designated by the country's laws or international treaties and conventions? Is there a possibility that the project will affect the protected areas?	(a) N	(a) No protected area exists around the project sites.
	(2) Ecosystem	(a) Does the project site encompass primeval forests, tropical rain forests, ecologically valuable habitats (e.g., coral reefs, mangroves, or tidal flats)? (b) Does the project site encompass the protected habitats of endangered species designated by the country's laws or international treaties and conventions? (c) If significant ecological impacts are anticipated, are adequate protection measures taken to reduce the impacts on the ecosystem? (d) Is there a possibility that hydrologic changes, such as reduction of the river flow, and seawater intrusion up the river will adversely affect downstream aquatic organisms, animals, vegetation, and ecosystems? (e) Is there a possibility that the changes in water flows due to the project will adversely affect aquatic environments in the river? Are adequate measures taken to reduce the impacts on aquatic environments, such as aquatic organisms?	(a) N (b) N (c) N (d) Y (e) Y	(a) No valuable forests (e.g., primeval forests and tropical rain forests) or ecologically valuable habitats (e.g., coral reefs, mangroves, or tidal flats) are encompassed by the projects. (b) No habitat of endanger species is confirmed in and around the project sites. (c) No significant ecological impact is expected. (d) Errors in operation of the floodgates on the Itajai Mirim River might cause the drastic reduction of the downstream flow of the river and eventually affect the aquatic organisms in the flow adversely. Nevertheless, such a potential effect is considered negligible as long as the floodgates are properly operated. (e) Ditto
3 Natural Environment	(3) Hydrology	(a) Is there a possibility that hydrologic changes due to the project will adversely affect surface water and groundwater flows?	(a) Y	(a) Pre-release of storage water in Rio Bonito and Pinhal dams may drastically increase the downstream flow of the Rio dos Cendros River. Hence, an early warning system to alert households living in the downstream areas about the potential risks is proposed as a project component.
	(4) Topography and Geology	(a) Is there a possibility that excavation of rivers and channels will cause a large-scale alteration of the topographic features and geologic structures in the surrounding areas?	(a) N	(a) No excavation of rivers and channels is planned in the project.
4 Social Environment	(1) Resettlement	(a) Is involuntary resettlement caused by project implementation? If involuntary resettlement is caused, are efforts made to minimize the impacts caused by the resettlement? (b) Is adequate explanation on compensation and resettlement assistance given to affected people prior to resettlement? (c) Is the resettlement plan, including compensation with full replacement costs, restoration of livelihoods and living standards developed based on socioeconomic studies on resettlement? (d) Is the compensations going to be paid prior to the resettlement? (e) Is the compensation policies prepared in document? (f) Does the resettlement plan pay particular attention to vulnerable groups or people, including women, children, the elderly, people below the poverty line, ethnic minorities, and indigenous peoples? (g) Are agreements with the affected people obtained prior to resettlement? (h) Is the organizational framework established to properly implement resettlement? Are the capacity and budget secured to implement the plan? (i) Are any plans developed to monitor the impacts of resettlement? (j) Is the grievance redress mechanism established?	(a) Y (b) N (c) N (d) Y (e) Y (f) N (g) N (h) N (i) N (j) N	(a) Although the relocation of road along with the heightening of Oeste dam is proposed to avoid involuntary resettlement and land acquisition, the State Government might decide to acquire about 67 ha of potential inundated area and relocate a total of three houses and four barns in the said acquired area. It is recommended that a proper compensation acceding to the relevant legislation in Brazil and OP. 4.12 of World Bank should be made in case that the government select the option of land acquisition and involuntary resettlement. (b) The project has been just formulated, and the state government has not made the final decision whether or not the project will result in involuntary resettlement. (c) Ditto. (d) Compensation prior to the resettlement is defined by the federal legislation. (e) Decree Law No. 3365 defines the rules on compensation. (f) No resettlement plan has been prepared yet as the project has been just formulated and the state government has not made the final decision whether or not the project will result in involuntary resettlement. (g) Ditto (h) Ditto (i) Ditto (j) Ditto
	(2) Living and Livelihood	(a) Is there a possibility that the project will adversely affect the living conditions of inhabitants? Are adequate measures considered to reduce the impacts, if necessary? (b) Is there a possibility that the amount of water (e.g., surface water, groundwater) used by the project will adversely affect the downstream fisheries and other water uses? (c) Is there a possibility that water-borne or water-related diseases (e.g., schistosomiasis, malaria, filariasis) will be introduced?	(a) Y (b) Y (c) N	(a) The heightening of Oeste and Sul dams might affect the living conditions of a few households. The necessary compensation measures shall be taken as proposed in the feasibility study prepared by JICA Preparatory Survey. (b) The project on pre-release of stored water in Rio Bonito and Pinhal dams might affect the downstream areas if the dams are not properly operated. However, as long as the pre-releasing is properly done, the adverse effect on the downstream area is expected to be negligible. (c) There is no possibility of outbreak of water-borne or water-related diseases owning to the priority projects.
	(3) Heritage	(a) Is there a possibility that the project will damage the local archeological, historical, cultural, and religious heritage? Are adequate measures considered to protect these sites in accordance with the country's laws?	(a) N	(a) There is no heritage site in and around the project sites.

Attachment-1 Environmental Checklist for the Priority Projects

Category	Environmental Item	Main Check Items	Yes: Y No: N	Confirmation of Environmental Considerations (Reasons, Mitigation Measures)
	(4) Landscape	(a) Is there a possibility that the project will adversely affect the local landscape? Are necessary measures taken?	(a) Y	(a) The construction work of the new floodgates on the Itajai Mirim River might affect the landscape of the town, as its proposed sites are located in the center of the town. Nevertheless, the expected impact would be minimal as the construction work will be only temporary and not result in any essential alteration of the cityscape.
4 Social Environment	(5) Ethnic Minorities and Indigenous Peoples	(a) Are considerations given to reduce impacts on the culture and lifestyle of ethnic minorities and indigenous peoples? (b) Are all of the rights of ethnic minorities and indigenous peoples in relation to land and resources to be respected?	(a) N (b) N	(a) No ethnic minority lives in and around the project sites. (b) Ditto
	(6) Working Conditions	(a) Is the project proponent not violating any laws and ordinances associated with the working conditions of the country which the project proponent should observe in the project? (b) Are tangible safety considerations in place for individuals involved in the project, such as the installation of safety equipment which prevents industrial accidents, and management of hazardous materials? (c) Are intangible measures being planned and implemented for individuals involved in the project, such as the establishment of a safety and health program, and safety training (including traffic safety and public health) for workers etc.? (d) Are appropriate measures taken to ensure that security guards involved in the project not to violate safety of other individuals involved, or local residents?	(a) Unknown (b) Unknown (c) Unknown (d) Unknown	(a) As long as the contractor follows the construction environmental management plan, which will be prepared in the course of the EIA study, any malpractice on working conditions is not predicted. (b) Ditto (c) Ditto (d) Ditto
5 Others	(1) Impacts during Construction	(a) Are adequate measures considered to reduce impacts during construction (e.g., noise, vibrations, turbid water, dust, exhaust gases, and wastes)? (b) If construction activities adversely affect the natural environment (ecosystem), are adequate measures considered to reduce impacts? (c) If construction activities adversely affect the social environment, are adequate measures considered to reduce impacts?	(a) Y (b) Y (c) Y	(a) The feasibility study prepared by JICA Preparatory Survey proposed adequate measures to mitigate possible impacts during the construction. (b) Ditto (c) Ditto
	(2) Monitoring	(a) Does the proponent develop and implement monitoring program for the environmental items that are considered to have potential impacts? (b) What are the items, methods and frequencies of the monitoring program? (c) Does the proponent establish an adequate monitoring framework (organization, personnel, equipment, and adequate budget to sustain the monitoring framework)? (d) Are any regulatory requirements pertaining to the monitoring report system identified, such as the format and frequency of reports from the proponent to the regulatory authorities?	(a) N (b) N (c) N (d) N	(a) The monitoring plan will be prepared in the course of the EIA study which will be initiated by the State Government in future. (b) Ditto (c) A monitoring framework will be prepared as a part of the monitoring plan, which will be prepared in the course of the EIA study. (d) A monitoring report system will be prepared in the part of the monitoring plan.
6 Note	Reference to Checklist of Other Sectors	(a) Where necessary, pertinent items described in the Forestry checklist should also be checked.	(a) N	(a) The Forestry checklist is not applicable to any of priority projects.
	Note on Using Environmental Checklist	(a) If necessary, the impacts to trans-boundary or global issues should be confirmed (e.g., the project includes factors that may cause problems, such as trans-boundary waste treatment, acid rain, destruction of the ozone layer, or global warming).	(a) Not applicable	(a) No impact to trans-boundary or global issues is expected.

1) Regarding the term "Country's Standards" mentioned in the above table, in the event that environmental standards in the country where the project is located diverge significantly from international standards, appropriat environmental considerations are required to be made.
In cases where local environmental regulations are yet to be established in some areas, considerations should be made based on comparisons with appropriate standards of other countries (including Japan's experience
2) Environmental checklist provides general environmental items to be checked. It may be necessary to add or delete an item taking into account the characteristics of the project and the particular circumstances of th country and locality in which the project is located.

ATTACHMENT-2: DRAFT TERMS OF REFERRENCE (TOR) FOR AN EIA STUDY ON THE IMPELMENTATION OF THE PRIORITY PROJECTS

1. Background

The Itajai River basin with a catchment area of 15,221 km^2 locates in the center of the State of Santa Catarina in the southern part of Brazil. Riparian areas along the Itajai River and its tributaries have been suffering from flood damage due to repeated inundation. After the consecutive attacks by large flood in both years 1983 and 1984, the following studies were carried out under the technical cooperation between the Government of Federative Republic of Brazil and the Government of Japan.

- The Itajai River Basin Flood Control Project (1986-88) (Master plan study and feasibility study)
- The Lower Itajai River Basin Flood Control Project (1988-90) (Feasibility study)

The Government of the Sate of Santa Catarina requested the Japanese ODA Loan for implementation of the Itajai River Flood Control Project. However, the Loan Agreement (L/A) was not concluded due to lack of guarantee of the Government of the Federative Republic.

A catastrophically heavy rainfall hit the Sate of Santa Catarina from November to December in 2008, resulting in serious impacts due to flood and sediment-related disasters in the Itajai River basin. The Government of the Sate of Santa Catarina showed the willingness to implement the the disaster prevention project for the Itajai River basin with technical and financial assistance by the Government of Japan. The execution of the Preparatory Survey for the Project on Disaster Prevention and Mitigation Measures for the Itajai River Basin was agreed between the Government of the Sate of Santa Catarina and the Government of Japan on November 5th 2009.

The JICA Preparatory Survey formulated a master plan for flood and sediment disaster prevention and mitigation measures for the Itajai River basin, and proposed implementing several priority projects after assessment of their viability.

This document is the draft terms of reference (TOR) for an EIA study on the selected priority projects, which specifies the scope of the Study to be fulfilled by an institution/organization that will be engaged in the conduct of the Study.

2. Proposed Projects

2.1 Objectives

The main objective of the proposed priority projects is to reduce and minimize the risk of flood and sediment disasters in the entire Itajai river basis. Specifically, the priority projects aim to prevent the towns and human life from the 10-year level flood.

2.2 Project Area

The priority projects cover the whole basin of the Itajai River. The following sections describe the natural and socio-economic conditions of the Project Area.

2.2.1 Socio-economic Conditions

The total population in the Itajaí River basin in 2009 was recorded at 1.23 million, which is about 20% of the total population in the Santa Catarina State. The average annual population growth in the period of 1970-2009 was 2.0 % as shown in the table below. Cities of Itajaí, Blumenau and Brusque show higher population growth. On the other hand, population growth in the upper Itajaí River basin shows a stagnation or decreasing trend, indicating a significant migration towards the middle-scale cities. The services sector prevails in terms of GRDP in the Itajaí River basin, which accounts for around 50.2% of the GRDP. In Itajaí city, the port services sector is the most important economic activity. The industrial sector is has been the major engine of economic growth in the regions of Brusque, Timbó, Blumenau and Ibirama. The services sector has been recently growing at an average growth rate over 20% in all the regions.

2.2.2 Topography and Geology

The Itajai River Basin is surrounded by mountains with elevations varying from 200 to 1,750 m, except on the Atlantic Ocean side. In the whole Itajai River basin, area rate of altitude range below 100 m is approximately 11%, the range 500 m to 1000 m is predominately 53%, and the range above 1000 m does not reach 1%.The geology of Itajai River Basin has the base from Archean to Proterozoic eons, which compose the stable continent of South America, and above it, there are sedimentary rocks from the Paleozoic and Mesozoic eras, and in the upper layer, there are basaltic rocks run off in the Mesozoic era. Except for the alluvial portion that stretches out in the lowland of the Atlantic coast and the lowland of the banks of rivers, in general, the geology are old in the northeast region and young in the southwest region. In the upstream areas of the basin, there are rocks from the Paleozoic to Mesozoic eras. In the middle and lower portions, there are sedimentary rocks in the Paleozoic era, and metamorphic rocks from the Archean to Proterozoic eons.

2.2.3 Meteorology and Hydrology

The average annual basin mean rainfall in the period of 1950-2008 is 1,560 mm. The maximum annual rainfall is 2,632 mm in 1983 and the minimum is 2,632 mm in 1983. However, in 2008 when the most serious flood disaster recently occurred, the annual basin mean rainfall is 1,899 mm. This is due to the concentration of the rainfalls in the lower part of Itajaí River basin from Indaial city during the 2008 flood. The monthly rainfall shows relatively low from April to August, gradually increasing from September onward, and the highest occurs in January and February. However, historical large floods occurred both in July 1983 and August 1984 even during the period of relatively low rainfalls. The annual mean discharges in 1980-2004 are 40 m^3/s at the Ituporanga station, 131 m^3/s at the Rio do Sul station, and 269 m^3/s at theIndaial station. The annual mean discharge at the Blumenau station is 340 m^3/s. Although wet and dry seasons are not clearly divided, the monthly mean discharges from September to February are generally higher than the annual mean discharge.

2.2.4 Land Use

The forest area accounts for 64.6% of the whole basin, followed by the agricultural land use of crops and pastures with 36.7%. Table 1 presents the current land use in the flood vulnerable area along the Itajaí River. Major urban areas in the basin are located in the flood vulnerable areas, and thus most of the basin population lives in these flood prone areas.

Table 1 Present Land Use within the Itajai River Basin in 2000

Land Use Category	Area (km²)	Ratio (%)

Crops/pastures	4,591.69	36.7
Forests	9,644.44	64.6
Rice paddies	241.22	1.6
Urban region	367.13	2.5
Water bodies	88.75	0.6
Total	**14,933.23**	**100.0**

Source: JICA Survey Team (based on IBGE data)

2.3 Outlines of the Project

2.3.1 Priority Projects for the First Stage

The State Government in the Santa Catarina decided to set of the 50-year flood as the final goal of flood security level in the master plan. In view of the required huge investment and long period for realization, the State Government decided to takes a stage-wise development approach. Consequently, the State Government adopted a security level for around 10-year flood level for the first stage of implementation.

Along this line, the following projects were finally selected by the Sate Government as priority ones for the first stage of implementation:

i) Water storage in the paddy fields

ii) Heightening of the existing flood control dam and change of gate operation method (2 dams)

iii) Utilization of the existing hydropower generation dam for flood control (2 dams)

iv) Strengthening of the existing flood forecasting and warning system (FFWS)

v) Installation of two floodgates on the Itajai Mirim River in Ttajai city

Together with the above, the following two projects were chosen for the first stage of implementation to prevent any sediment disasters.

i) Slope protection of roads at 13 locations

ii) Installation of early warning system for sediment disaster and flush flood

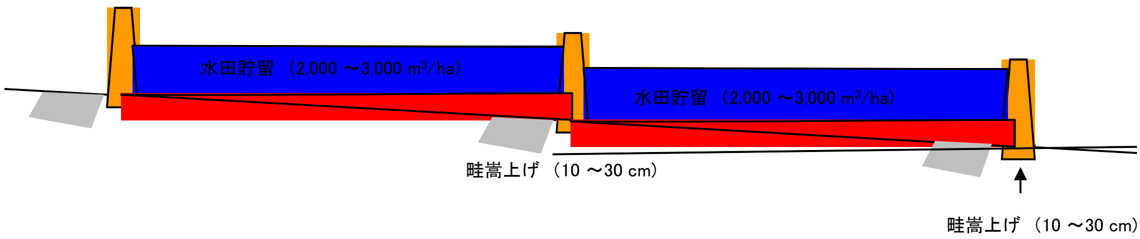
2.3.2 Outlines of the Priority Projects

(1) Water Storage in Paddy Fields

This plan aims at enlargement of the flood retention capacity, being used of the paddy fields expanded in the all sub-basins with provision of heightened paddy ridge and gradually to introduce the rice production with better quality and safety. As measures, it will develop the following activities:

Increase of capacity of reduction of the floods effect:	- Elevation of the paddy ridge.
Use of the land in accordance with environmental legislation:	- Recovery of the riprap forest. - Incentive to use of farm land in accordance with environmental legislation
Safety foods Supply:	- Incentive to introduce the Integrated Rice Production

It is foreseen to execute the heightening of the paddy ridge (current 10 cm) for more 10 to 30 cm, hoping to increase the capacity of retention of the rains for more 2.000 ~ 3.000 m3 for hectares, as well as it is indicated in following figure:



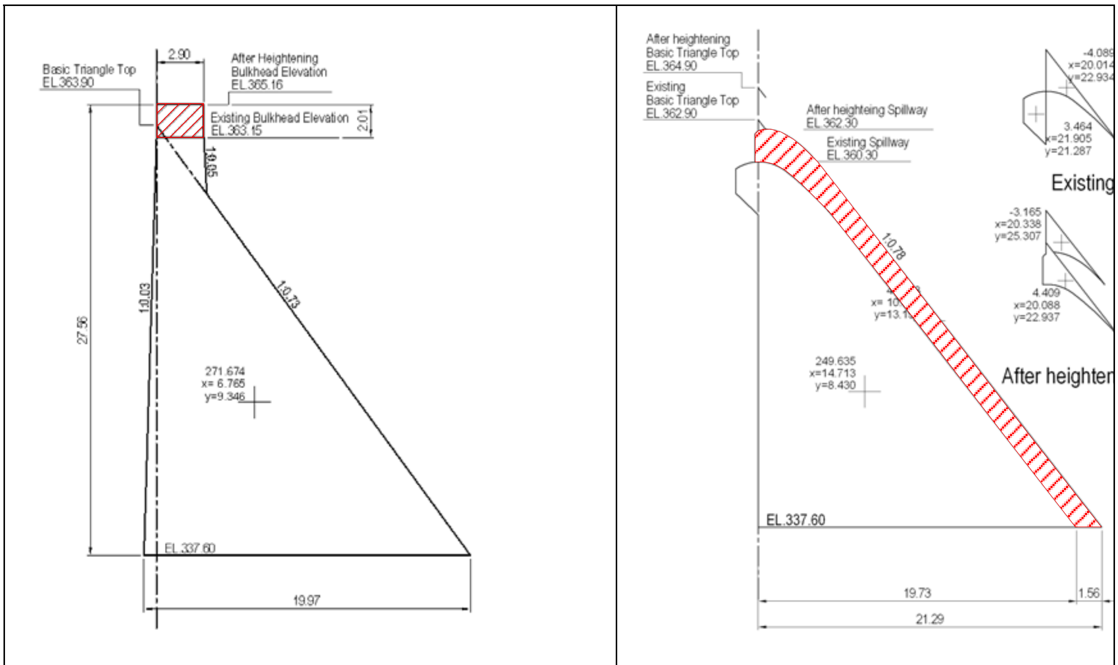
Source: JICA Survey Team

Retention Methodology of Flood water in paddy fields

(2) Heightening of the existing flood control dam and change of gate operation method (2 dams)

a. Heightening of Oeste Dam

It appeared by the topographic survey and hydraulic calculation of the design discharge that the non-overflow section of the dam body should be heightened by 2.01 m, although the spillway section is by 2.0 m. The following figure shows the designed sections.

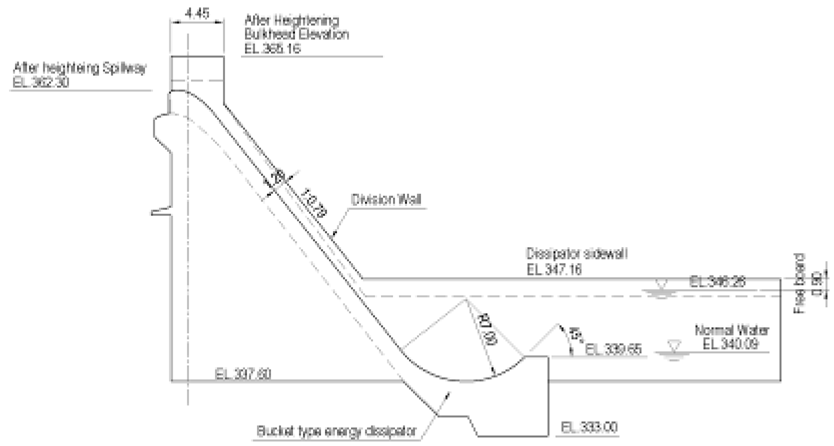


Non-overflow Section	Spillway Section

Source : JICA Survey Team

Designed Sections of the Oeste Dam

From the hydraulic viewpoints, it was proposed to install an energy dissipater along with the heightening of the dam. The energy dissipater is generally installed at the outlet of spillway to dissipate large energy of the overflowed water of spillway. Heightening of the spillway might cause larger energy since the overflow head becomes higher. The proposed dissipater is of the submerged bucket type considering that river water level immediately downstream of the dam is always high enough as illustrated below.



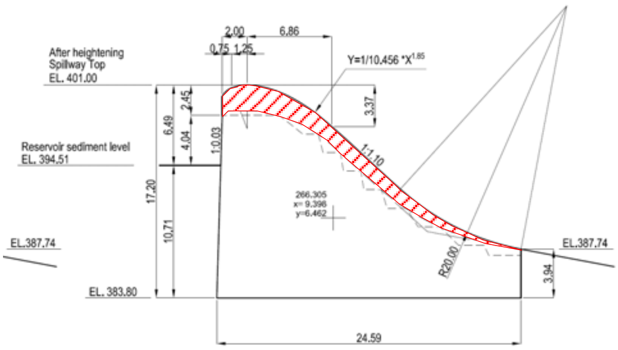
Side wall elevation EL.347.16 m = 346.26 + 0.90

Source：JICA Survey Team

Proposed Energy Dissipater at the Oeste Dam

b. Heightening of Sul Dam

The Sul dam is proposed to heighten only the spillway section by 2.0 m because of sufficient freeboard to the dam crest after heightening as illustrated below.



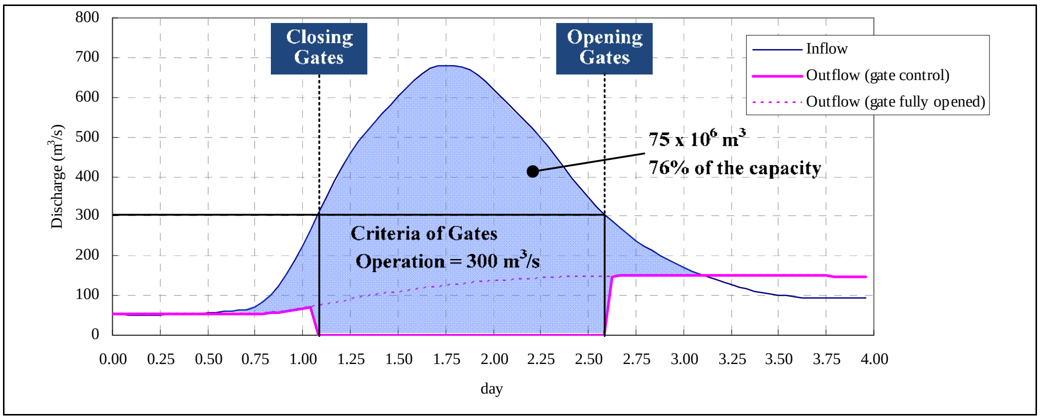
Source：JICA Survey Team

Figure 23 Comparison of Overflow Depth of Design Discharge on the Spillway of Sul Dam

As shown in the figure, the maximum overflow depth of design discharge of 2,570 m^3/s (= 10,000-year flood) is 7.0 m under the present spillway. However, even though the spillway is heightened by 2.0 m, there would be more than 1.0 m space as a freeboard.

c. Modification of the Operations

The following figure shows the proposed flood control operation at the Oeste dam.

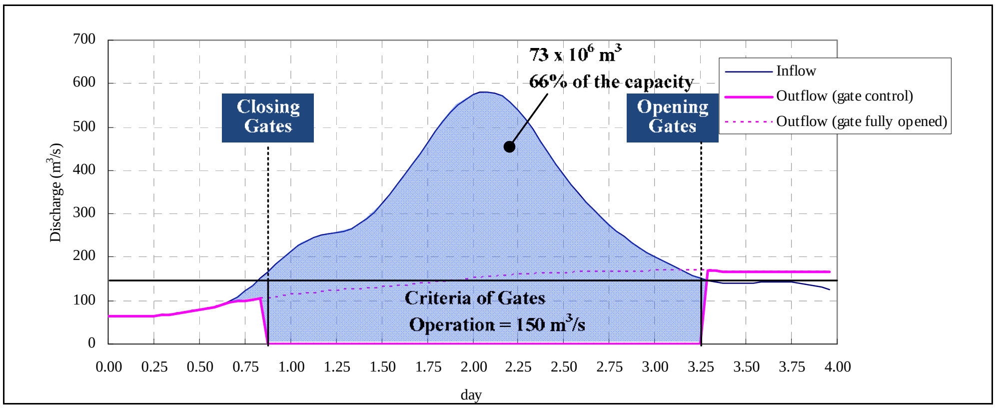


Source: JICA Survey Team

Method of Flood Control at the Oeste Dam

If all the gates are fully opened during the 10-year flood, maximum flood discharge in Taio city is estimated 520 m^3/s. This discharge exceeds the current flow capacity of 440 m^3/s in Taio city. Therefore, the required peak cut for the 10-year flood at the Oeste Dam is estimated 80 m^3/s. It is desirable to close fully the gates during the flood as long as possible, expecting flood control effect in Rio do Sul city and simplification of the operation.

Likewise, the following figure shows the proposed flood control operation at the Sul dam. If all the gates are fully opened during the 10-year flood, maximum flood discharge in Rio do Sul city in the Itajai do Sul River is estimated 570 m^3/s. This discharge exceeds the current flow capacity of 440 m^3/s in Rio do Sul city. Therefore, the required peak cut for the 10-year flood at the Sul dam is estimated 130 m^3/s.



Source: JICA Survey Team

Method of Flood Control at the Sul Dam

(3) Utilization of the existing hydropower generation dam for flood control

The existing two hydropower generation dams of CELESC in the Rio dos Cedros River, named the Rio Bonito and Pinhal dams, are proposed to be used for flood control by means of pre-releasing when an impending flood is predicted. The proposed pre-releasing aims at creation of flood control space in reservoir by means of lowering the reservoir water level by releasing the stored water before flood inflow into the reservoir.

In order to regulate the outflow discharge from both dams not to exceed 140 m^3/s for the

10-year flood with a peak discharge of 210 m^3/s, the required flood control volume to be created by pre-releasing was examined by simulation of reservoir operation at both dams. The required volume for pre-releasing was estimated to be 1.4 million m^3 for the Rio Bonito dam and 3.2 million m^3 for the Pibhal dam, respectively, as shown below.

Table 19 Required Flood Control Volume to be Created by Pre-releasing at Two Dams

	Rio Bonito Dam	Pinhal Dam
Maximum water level in operation	EL.589.5 m	EL.652.0 m
Drawing down by pre-releasing	0.5 m	1.0 m
Water level after pre-releasing	EL.589.0 m	EL.651.0 m
Volume for flood control by pre-releasing	1.4×10^6 m^3	3.2×10^6 m^3
Maximum inflow discharge	85 m^3/s	125 m^3/s
Maximum outflow discharge	60 m^3/s	85 m^3/s
Reduction of discharge at the peak time of inflow	25 m^3/s	45 m^3/s
Operation of gates during flood control Gate opening of the spillway Gate opening of the intake	Constant opening 0.5 m 2.6 m	Constant opening 1.0 m 2.6 m
Operation of gates before flood control	Keep the water level at EL. 589 m (inflow = outflow) by operating intake gate	Keep the water level at EL. 651 m (inflow = outflow) by operating intake gate
Operation of gates after flood control	Keep the water level at EL. 589.5 m (inflow = outflow) by operating spillway gate	Keep the water level at EL. 652 m (inflow = outflow) by operating spillway gate

Source: JICA Survey Team

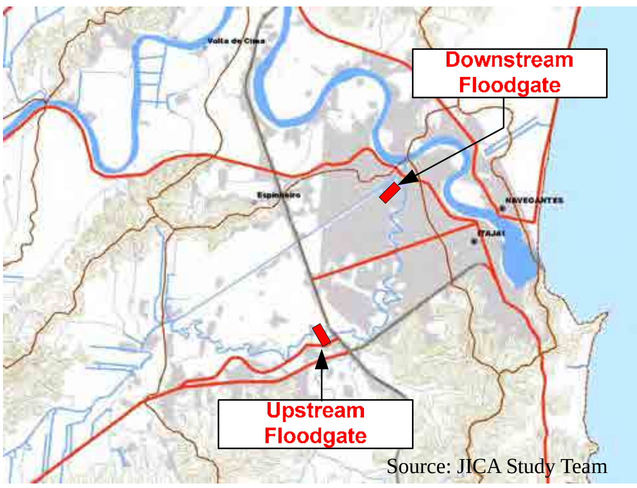
(4) Strengthening of the Existing Flood Forecasting and Warning System (FFWS)

Setting 13 new rainfall and water level gauging stations and 3 CCTVs is proposed in order to strengthen the existing FFWS. Moreover, 3 gauging stations and 2 CCTVs shall be added at the 2 flood control dams (Sul and Oeste dams) and 2 hydropower dams (Rio Bonito dam, Pinhal dam) through the meeting with concerned organizations (Defesa Civil, SDS and CEOPS/ FURB) and the workshop held on Apr. 29, 2011 regarding FFWS and dam operation.

The warning system based on flood forecasting is also required for Itajai city. However flood water level at Itajai city is rather difficult to forecast due to tidal effects of sea water and flood flow from the Itajai Mirim River.

(5) Installation of two floodgates on the Itajai Mirim River in Ttajai city

Riparian area along the Old Mirim is generally low varying EL.1.0 to 3.0 m. On the other hand, the area along the Canal is relatively higher elevation in around EL.3.0 to 4.0 m. Though the Canal has larger flow capacity, the Old Mirim has caused frequent flooding and inundation to its riparian area. Two floodgates on the Old Mirim are proposed to mitigate inundation along the Old Mirim as shown below.



Location Map of Floodgates in Old Mirim

The upstream gate would be closed when the discharge from the Itajai Mirim reaches to the flow capacity of the Old Mirim, in this respect, operation needs the information on the water level of the Old Mirim in the urban area (in the downstream area of the BR-101).On the other hand, the downstream gate would be closed when the water level at the downstream end of the Old Mirim reaches to the critical water level. Therefore, the operation of downstream gate also needs the information on the water level at the downstream end of the Old Mirim.

The effectiveness of the flood gates is evaluated in the following table.

Estimated Inundation Area along the Lower Old Mirim

Inundation depth (m)	Area (m^2) without gate control	Area (m^2) with gate control	Effectiveness (m^2)
< 0.5	2,216,400	564,400	1,652,000
0.5 – 1.0	1,299,600	527,600	772,000
1.0 – 1.5	848,800	242,000	606,800
1.5 – 2.0	431,600	22,000	409,600
2.0 – 2.5	441,200	0	441,200
2.5 – 3.0	40,000	0	40,000
Total	5,277,600	1,356,000	3,921,600

Source: JICA Survey Team

Main features of the designed floodgates are summarized below.

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
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

(6) Slope Protection of Roads at 13 Loations

All of the priority sites are road slops, and the structural measures will be planed to ensure full width road traffic against 60 years heavy rain. Because that high possibility of human lives lost is recognized on 13 priority sites. Type for measures will be selected by learning from existing measures of similar condition slopes, which have not been occurred disaster even under 60 years heavy rain

Comparison of alternatives of cutting slope reinforcement measure for 1,000 m2 is carried out. Reinforced earth method of PP fiber/ cement/sand is recommendable, because it is advantage of all evaluation items of the cost, construction period, and landscape. The selection criteria of structural measures against valley side slope collapse are shown below. The vegetation works, and open ditch is included in basic measures to prevent sediment discharge.

Structural Measures against Valley Side Collapse

<table>
<tr><th colspan="3">Slope condition</th><th>Typical measure alternatives</th><th>Common item</th></tr>
<tr><td rowspan="2">Height (H) of collapse H>10</td><td colspan="2">Height (H) and width (W) of collapse H/W > 0.5</td><td>Pilling, or large block placing</td><td rowspan="2">Tree planting
Open ditch</td></tr>
<tr><td colspan="2">Height (H) and width (W) of collapse H/W ≦0.5</td><td>Gully filling by gabion and longitudinal drainage</td></tr>
<tr><td rowspan="3">Height (H) of collapse H≦10</td><td rowspan="2">Height (H) and width (W) of collapse H/W > 0.5</td><td>Embankment on slope foot is Possible</td><td>Embankment</td><td rowspan="3"></td></tr>
<tr><td>Embankment on slope foot is Impossible</td><td>Pilling, or large size block placing</td></tr>
<tr><td colspan="2">Height (H) and width (W) of collapse H/W ≦0.5</td><td>Gully filling by gabion and longitudinal drainage</td></tr>
</table>

Source: JICA Survey Team

(6) Installation of Early Warning System for Sediment Disaster and Flush Flood

An automatic rain gauge will be installed in each city for the early warning purpose. Location of automatic rain gauge will be determined by following procedure. Redundant data communication will be established by both of VHS (very high frequency connection) of CELESC system and GPRS (general packet radio services) to secure information communication even under stormy condition.

The early information for the public is important, Defesa Civil-SC delegates EPAGRI/CIRAM the announcement of the rainfall level of attention/warning by web-page and/or mass media, as a part of routine or emergency weather report. The computer system of the early warning shall be included the function of automatic sending electronic mail to Defesa Civil-SC, mayor/Defesa civil staff of each city, and EPAGRI/CIRAM staffs in charge.

The Defesa Civil of municipalities will prepare the detailed hazard map (S=1:10,000), and will designate the risk areas/houses, emergency evacuation building such as schools and/or churches, evacuation route. The disaster education about the evacuation will be also conducted. Santa Catarina State shall clarify the responsibility of the municipalities/mayors about evacuation order in a law. The capacity of a municipality is not enough for the evacuation order generally. The Defesa Civil-SC shall coordinate the support of the municipalities, using human resources of universities, engineer of public/private, and/or international technical assistance. The early warning system shall be started as soon as possible. And then, the risk areas/houses which shall be evacuated would be designated one by one by the maximum effort of municipalities to make mature the early warning system.

3. Scope of the EIA Study

3.1 The Study Area

The EIA study shall generally cover the whole basin of the Itajai river in Santa Catarina. Specifically, the study shall focus on the areas to be affected by the priority projects, such as those used for heightening of the existing dams and potential inundation areas by the heightening of the dams, etc.

3.2 Environmental Items to be Assessed

The JICA Preparatory Survey conducted the initial environmental examination (IEE) to identify the potential environmental impacts that the priority projects would cause in the construction and operation phases. Table 1 shows the results of the IEE made in the feasibility study of the JICA Preparatory Survey. Based on the results of the IEE, the items to be assessed in the EIA Study are identified as follows.

Items to be Assessed in the EIA Study

Items	Items	Construction	Operation	Related Projects
Natural Environment	Landscape	TBA	TBA	Floodgate construction Landslide measures
	Fauna and Flora	TBA	TBA	Heightening of Oeste dam Rainwater storage in rice field Floodgate construction
	Bottom sedimentation	TBA	TBA	Heightening of Oeste dam Floodgate construction
	Topography and geology	-	TBA	Floodgate construction
Pollution	Noise and vibration	TBA	-	Heightening of Oeste dam Heightening of Sul dam Floodgate construction Landslide measures
	Solid waste	TBA	-	Heightening of Oeste dam Heightening of Sul dam Landslide measures
	Air Pollution	TBA	-	Landslide measures
	Water Pollution	TBA	-	Heightening of Oeste dam Floodgate construction
Socio-economic impact	Impact on regional infrastructure	TBA	-	Floodgate construction
	Impact on lands/buildings	TBA	-	Heightening of Oeste dam Heightening of Sul dam
	Involuntary resettlement	TBA	-	Heightening of Oeste dam Heightening of Sul dam
	Land acquisition	TBA	-	Heightening of Oeste dam Heightening of Sul dam Floodgate construction
	Impact on downstream areas	TBA	TBA	Preliminary discharge from dam
	Impact on agriculture	-	TBA	Heightening of Oeste dam Heightening of Sul dam Rainwater storage in rice field
	Change of income	-	TBA	Heightening of Oeste dam Heightening of Sul dam
	Traffic during construction	TBA	-	Heightening of Oeste dam Heightening of Sul dam Floodgate construction Landslide measures
	Effects on low income groups	-	TBA	Heightening of Oeste dam Heightening of Sul dam
	Expansion of economic disparity	-	TBA	Heightening of Oeste dam Heightening of Sul dam
	Regional conflicts	TBA	-	Heightening of Oeste dam Heightening of Sul dam Rainwater storage in rice field Floodgate construction Landslide measures
	Land use and occupation	-	TBA	Heightening of Oeste dam Heightening of Sul dam Floodgate construction

Items	Items	Construction	Operation	Related Projects
	Impact on economic and productive activities	TBA	-	Heightening of Oeste dam Heightening of Sul dam Rainwater storage in rice field

Note : TBA: To be assessed.

3.3 Surveys and Investigations

3.2.1 Overall Methods

The surveys and investigations in the EIA study shall be composed of two ways, literature reviews and field investigations. The literature reviews aim to collect data and information relevant to the priority projects as well as the natural and social conditions of the study area, while the field investigations encompass several types of specific surveys that aim to collect the detailed information on the ground.

3.2.2 Review of the Proposed Projects

The final report prepared by the JICA Preparatory Survey shall be reviewed in the beginning of the EIA study to get a clear picture of the proposed projects as well as the natural and social conditions of the project sites.

3.2.2 Overall Framework of the Data Collection

The following information shall be collected for assessment of the aforementioned items in the EIA study. Some data and information shall also be verified and supplemented by field surveys as specified in this TOR.

Data and Information to be Collected

Item	Scope and Coverage	Action to be taken	Study area
Topography, Geology and Soil	• Terrain pattern • Regional geological and soil characteristic • Land subsidence condition	• Collection of additional relevant documents as necessary	Itajai river basin
Hydrology	• River flow discharge	• Collection of secondary data of river flows	Itajai Mirim river Rio dos Cedros river Main streams of Oeste and Sul dams
	• Rain fall data	• Collection of additional secondary data of rainfall (if necessary)	Itajai river basin
Air Quality	• Air quality parameter determined by the regulations of the Federal and/or State Government • Predicted load of air pollutants from the construction works for slope protection measures	• Collection of regulations of the Federal/State government on air quality standards • Collection of additional air quality monitoring data collected in the proposed sites for slope protection measures (if any)	Proposed sites of construction of slope protection measures
Traffic	• Existing vehicle traffic amount on relevant trunk road at the construction sites for Oeste dam, Sul dam, Floodgates on the Itajai Mirim river, and slope protection measures	• Traffic survey at the construction sites	Construction sites for Oeste dam, Sul dam, Floodgates on the Itajai Mirim river, and slope protection measures

Item	Scope and Coverage	Action to be taken	Study area
Water Quality	• Physical, chemical and biological parameters determined by the regulations of the Federal and/or State Government • Water quality of the downstream flow of Oeste dam and Itajai Mirim river	• Collection of regulations of the Federal/State government on air quality standards • Water quality sampling survey in the downstream flow of Oeste dam and Itajai Mirim river	Downstream flow of Oeste dam Itajai Mirim river
Noise and Vibration Level	• Equivalent Sound level (Leq) • Predicted result of noise and vibration level by construction works for the heightening of Oeste and Sul dams, floodgates in the Itajai Mirim river, and slop protection measures. • Existing noise and vibration levels around the construction site for the floodgates in the Itajai Mirim river	• Collection of monitoring data of noise and vibration levels around the construction site for the floodgates in the Itajai Mirim river	Construction sites for Oeste dam, Sul dam, Floodgates on the Itajai Mirim river, and slope protection measures
Flood Condition	• Past record of flooding in the downstream of Rio dos Cedros river	• Collection of secondary data or records of past floods (if available)	Rio dos Cedros river
	• Past record of inundation in the areas potentially affected by the heightening of Oeste and Sul dams	• Collection of secondary data or records of past floods (if available)	the areas potentially affected by the heightening of Oeste and Sul dams
Terrestrial Ecology	• Existing vegetation and its general characteristic in the existing paddy fields and the construction site for Oeste dam • List of major aquatic organisms in the Itajai Mrim River and the downstream flow of Oeste dam.	• Collection of general information of flora and fauna in the existing paddy fields and the construction sites for Oeste dam • Collection of secondary data of aquatic organisms in the Itajai Mrim River and the downstream flow of Oeste dam	Paddy fields for rainwater storage Construction site for the heightening of Oeste Dam Itajai Mirim river Downstream flow of Oeste dam
Land Use	• Existing land use in the areas potentially affected by the heightening of Oeste and Sul dams • Existing land use in the downstream area of the Itajai Mirim river	• Collection of secondary data on the existing land use in the areas potentially affected by the heightening of Oeste and Sul dams and the downstream area of the Itajai Mirim river • Collection of high resolution satellite image analysis of the areas mentioned above • Site reconnaissance survey in the areas mentioned above	Areas potentially affected by the heightening of Oeste and Sul dams Downstream area of the Itajai Mirim river
Transportation	• Network and mode of transportation in and around the construction site for the floodgates in the Itajai Mirim river • Traffic volumes and composition • Traffic congestion and capacity of road network	• Traffic survey to grasp the current traffic volume, composition, traffic congestion, and assess the capacity of road network in the construction site for the flood gates in the Itajai Mirim river	Construction site for the flood gates in the Itajai Mirim river

Item	Scope and Coverage	Action to be taken	Study area
Socio economic conditions	• Current socio economic conditions (agricultural production, land use, major livelihoods, assets, other livelihood activities, income and expenditures, etc.) of i) communities/households who use the paddy fields which will be used for water storage and ii) those who live in the areas potentially affected by the heightening of Oeste and Sul dams • Statistic socio economic data of the household economy of those living in urban and rural areas in the State	• Household interview survey to all the households living in the affected areas and using the paddy fields for water storage • Collection of statistic socio-economic data of typical households in the State	Households living in the areas potentially affected by the heightening of Oeste and Sul dams Households using the paddy fields for water storage
Inventory of land and buildings	• Delineation of the areas that will be potentially affected by the heightening of Oeste and Sul dams • Inventory of existing buildings and other assets in the potentially affected areas • Prevailing market prices of buildings and other associated assets in the potentially affected areas	• Topographic survey in the potentially affected areas • Inventory of existing buildings and other associated assets in the potentially affected areas • Cadastral data of the affected areas • Market research on buildings and other assets that should be compensated by the project	Areas potentially affected by the heightening of Oeste and Sul dams Households usin
	• Delineation of the areas that will be potentially affected by construction of the floodgates in the Itajai Mirim river	• Topographic survey in the potentially affected areas • Cadastral data of the affected area	Areas which need to be acquired for construction

The results of the examination of the collected data and information shall not only be described in the main text of the report, but also summarized in tables, graphs, drawings and maps so that the implications of the collected data and information would be understandable. Furthermore, the environmental standards and regulations, legal systems relating to the implementation of EIA in Brasil shall be briefly described in the EIA report.

3.2.3 Collection of Existing Data

As specified in section 3.2.2, the following data and information shall be collected at the relevant government offices as well as other institutions/organizations.

a. Tographic maps covering the Itajai river basin
b. Geological maps covering the Itajai river basin
c. Soil maps covering the Itajai river basin
d. Land use map covering the Itajai river basin
e. River flow data of the Itajai Mirim River, Rio dos Cedros River, Downstream flows of Oeste and Sul dam
f. Government regulations on air quality standards

g. Any air quality monitoring data in the vicinity of the proposed sites for slope protection measures

h. Government regulations on air quality standards

i. Any monitoring data of noise and vibration levels around the construction sites for the floodgates in the Itajai Mirim River

j. Secondary data or records of past floods of the Rio dos Cerdos River and the areas potentially affected by the heightening of Oeste and Sul dams

k. Information of flora and fauna in the existing paddy fields for water storage and the construction site for Oeste dam

l. Secondary data on aquatic organisms in the Itajai Mirim River and the downstream flow of Oeste dam

m. Secondary data on the existing land use in the areas potentially affected by the heightening of Oeste and Sul dams and the downstream area of the Itajai Mirim River

n. Statistic soico-economic data of typical households in the stage

l. Cadastral data of the areas potentially affected by the heightening of Oeste and Sul dams

3.2.4 Procurement/Collection of High Resolution Satellite Images

The latest high-resolution satellite images covering the areas potentially affected by the heightening of Oeste and Sul dams shall be procured or collected to assess the present land use in the affected areas and confirm the existing buildings and other assets or infrastructure in the areas.

3.2.5 Field Surveys

The following field surveys shall be carried out to collect detailed and updated data and information on the ground.

(1) Traffic Survey

In order to set the baseline of the traffic conditions in the construction sites for the floodgates in the Itajai Mirim River, a traffic survey specified below shall be carried out.

No. of survey plots:	2 points in each construction sites (The locations of the sampling points shall be identified when this draft TOR is finalized.)
Mode of survey:	Traffic count survey (to count the type and nuumber of vehicles passing by the survey plots from 6:00 to 18:00)
Frequency:	One week in the dry season
Other information recorded:	Hours when traffic conjunction takes place in a day

(2) Water Quality Sampling Survey

A water quality sampling survey shall be carried out in the downstream flow of Oeste dam and the Itajai Mirim River to fix the baselines of the water quality of both flows. The specifications of the survey are as follows.

No. of sampling plots:	5 points in each flow (The locations of the sampling points shall be identified when this draft TOR is finalized.)
Mode of sampling:	Sampling shall be made at three layers, namely surface, middle and bottom layers.
Analytical items in the field:	pH, Color, Odor, DO
Analytical items in the laboratory:	BOD (5days at 20^{o}C), Total Conliform Bacteria, Fecal Coliform Bacteria), COD, Suspended Solids, NO_3-N, NH_4-N
Timing of sampling:	Dry season and rainy season (two times in total) (All the samplings shall be taken at the same time.)
Sampling and Analytical Methods:	Water sampling and analytical methods shall follow the methods internationally accredited.

(3) Field Reconnaissance Survey

A field reconnaissance survey shall be carried out in the following areas and for the following aims:

a. Grasp the present land use and existing flora and fauna in the areas potentially affected by the heightening of Oeste and Sul dams and the downstream area of the Itajai Mirim River

b. Confirm the existing flora and fauna in the paddy fields to be used for water storage

(4) Topographic Survey

A topographic survey shall be carried out to demarcate the area to be acquired for the heightening of Oeste dam.

(5) Socio Economic Survey (Household Interview Survey)

A household interview survey shall be carried out to clarify the socio economic conditions of the following households/families:

1) Households who live in or have their assets in the areas potentially affected by the heightening of Oeste and Sul dams; and

2) Households who own the paddy fields to be used for storage of flood water.

The scope of the household interview survey is outlined below.

Target households:	All the households categorized into those specified above
Mode of survey:	Questionnaire survey
Survey Items:	Structure of family, History of family, Occupation, Ethnicity, Land use, Land tenure, Land holding size, Major agricultural products, Current livelihood activities, Income level, etc.
Others:	The awareness level about the proposed projects (the heightening of Oeste and Sul dams) among those who would be potentially affected shall be

confirmed with their intentions.

(6) Inventory of Existing Buildings and Other Assets

Simultaneously with the household interview survey, an inventory of existing buildings and other assets shall be carried out in the areas potentially affected by the heightening of Oeste and Sul dams. The scope of the inventory is outlined below.

Targets:	All the buildings and assets existing in the potentially affected areas.
Mode of survey:	Direct measurements and interview survey to the households
Survey Items:	Size of land occupied by the households, Type of house, Appearance of house, Size of house, Year of construction, Any immovable properties except house in the area, and Any movable properties in the house and area

(7) Market Research on Buildings and Other Assets

A market research shall be carried out to clarify the current prevailing market prices of: i) lands similar to those to be acquired for the heightening of Oeste dam; ii) buildings existing in the potentially affected areas; iii) other immovable properties owned by the affected families/households; and iv) farm inputs needed for establishment of productive farms.

3.4 Impact Identification and Assessment

Having analyzed the data and information gathered through the collection of data and field surveys, the magnitude and extent of potential environmental impacts caused by the implementation of the priority projects shall be estimated and evaluated as quantitatively as possible. As specified in the JICA Guidelines for Environmental and Social Considerations (2010), the impacts with regard to environmental and social considerations including the derivative, secondary and cumulative impacts shall be assessed.

3.5 Preparation of Mitigation Measures

Feasible and cost effective mitigation measures shall be prepared for all the project activities likely to have adverse impacts. The aim of the mitigation measure is to prevent or reduce the negative impacts predicted in the course of the priority projects. In the formulation of the mitigation measures, the following aspects should be taken into account.

a. level of mitigation
b. method of mitigation
c. expected result and effect of the mitigation measures
d. timing of application
e. duration of application
f. institutional arrangement necessary for application
g. cost necessary for application of the mitigation measures

In case that any impacts (residual impacts) that can not be prevented or reduced by the mitigation measures are identified and predicted, the necessary compensation for such impacts shall be estimated instead of the mitigation measures.

3.6 Preparation of Environmental Management and Monitoring Plan

An environmental management and monitoring plan (EMMP) shall be prepared and compiled in the EIA report. The EMMP shall include but not be limited to the following.

a. Environmental construction management plan
b. Information sharing and dissemination plan
c. Resttelement and rehabilitation plan with social development plan
d. Pollution control plan (water, air quality, noise, and vibration)
e. Flood management plan
f. Environmental management plan in the construction and operation phases
g. Institutional arrangement for implementation EMMP

Furthermore, the outlines of the EMMP shall be compiled in a/ summary table/s described below.

Sample of Summary Table of the Environmental Management Plan (EMP)

Project activities	Impact Description	Mitigation/ Enhancement Measure	Cost of mitigation/ Enhancement	Institutional Responsibility	Schedule	Guarantees Understanding/ Contract
I. Construction A. Social Environment B. Natural Environment C. Pollution						
II. Operation and Maintenance A. Social Environment B. Natural Environment C. Pollution						

Sample of Summary Table of the Environmental Monitoring Plan (EMP)

Project activities	Parameters	Location	Frequency	Responsibility	Estimated Cost
I. Construction Phase	A. Social Environment B. Natural Environment C. Pollution				
II. Operation and Maintenance Phase	A. Social Environment B. Natural Environment C. Pollution				

3.7 Stakeholders Meetings

In order to comply with the JICA Guidelines for Environmental and Social Considerations (2010), the stakeholders meetings shall be organized to consult with the relevant stakeholders on the respective priority projects. The specifications of the stakeholders meetings are as follows:

Timing of the meetings:	Two times per priority program (In the beginning of the EIA study and When the EMMP is drafted) (A total of 12 times for 6 priority projects)
Place of the meetings:	Strategic places where key stakeholders for each priority project can gather
Aims of the meetings:	Introduction of the project activities, hearing of the opinions of local stakeholders, comments on the project as well as EMMP, etc.

The discussions made in the stakeholders meeting shall be notated and compiled in the EIA report.

4 Report Making

The EIA report (RIMA) shall be prepared in the language accessible to the public as specified by Article 3 of CONAMA Resolution No. 237/97. The report shall include figures, maps, tables, graphs, and other means to make the contents of the report understandable. The suggested table of contents of the report is as follows.

1) Introduction and background of the study
2) Objectives and rationale of the study
3) Scope and methodologies of the study
4) Results of the environmental study
5) Results of the stakeholders meetings
6) Environmental impact assessment
7) Proposed mitigation measures against likely adverse impacts
8) Environmental management plan
9) Compensation / resettlement plan (if necessary)
10) Environmental monitoring plan
11) Conclusion

5 Timeframe of the EIA Study

The entire work for the EIA study shall be completed xxx days (to be specified based on the schedule of the project) from the commencement of the study.

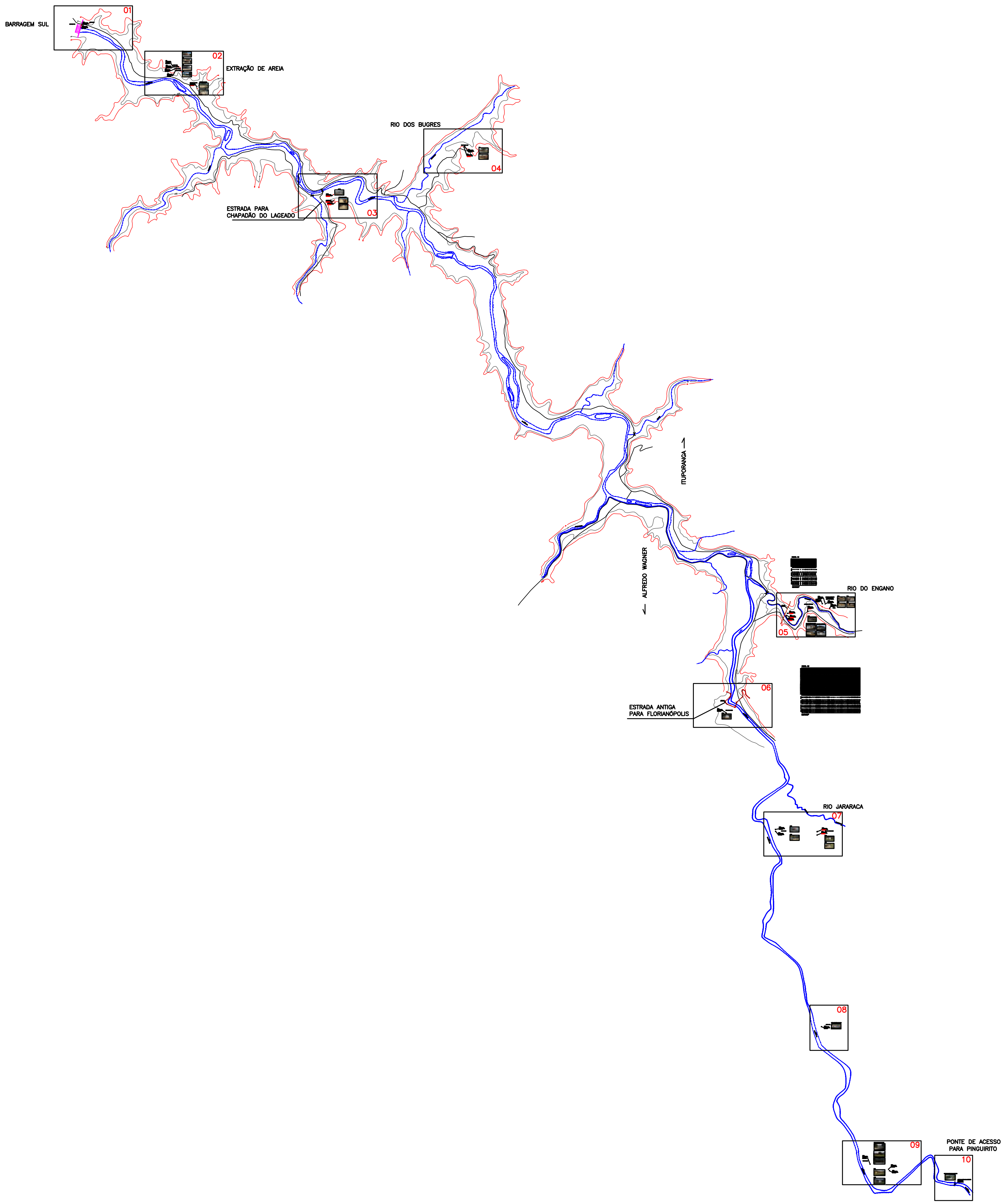
6. Expected Outputs

The following outputs shall be prepared and submitted at the end of the Study.

a. Three (3) hard copies of the EIA report with one (1) soft copy
b. One (1) soft copy of raw data collected through the EIA study
c. One (1) soft copy of database of the project affected households and their properties

ATTACHMENT - 3

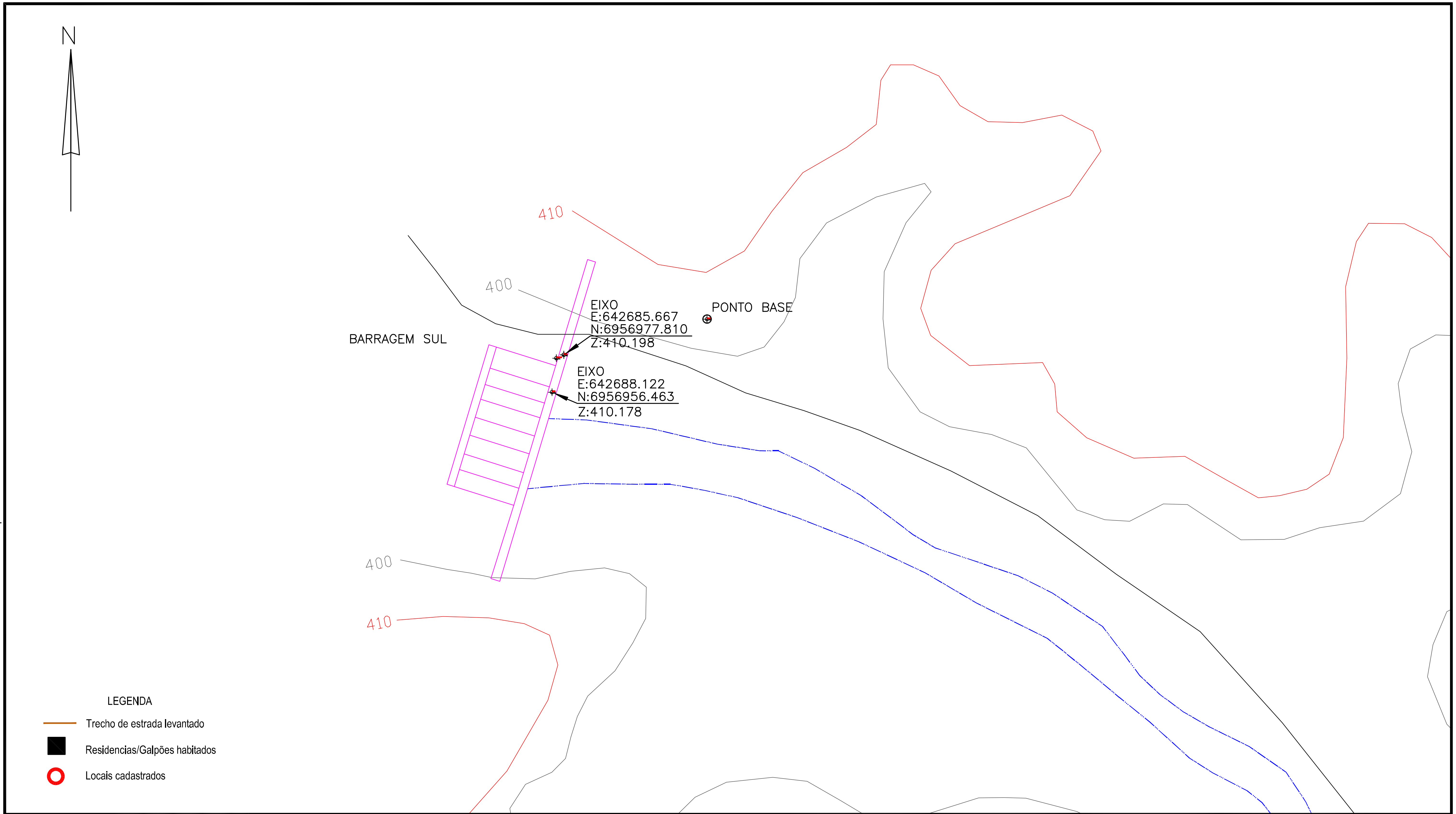
Sul Dam



PROPRIEDADES CADASTRADAS - BARRAGEM SUL

NOME	DESCRIÇÃO	N (m)	E (m)	Z (m)	OBSERVAÇÕES
BASE	PONTO BASE	6957007.418	642804.525	413.292	MARCO DE APOIO
ALÍRIO RODE	CA1	6956368.893	644048.549	423.335	CASA
NILTA KUSTER	CA2	6956351.789	644060.331	422.652	PAIOL E ESTUFA
NADIR KUSTER	CA3	6956320.452	644078.267	421.531	PAIOL E ESTUFA
NADIR KUSTER	CA4	6956296.238	644092.898	421.950	CASA
OSNI VIRGILIO LUIZ	CA5	6955161.104	648421.599	412.536	CASA
OSNI VIRGILIO LUIZ	CA6	6955153.814	648409.210	410.578	RANCHO E ESTREBARIA
PROPRIETÁRIO NÃO IDENTIFICADO	CA7	6948175.215	653250.231	408.527	CONJUNTO DE CASAS
PROPRIETÁRIO NÃO IDENTIFICADO	CA8	6948144.395	653235.867	409.011	CASA E COMÉRCIO
PROPRIETÁRIO NÃO IDENTIFICADO	CA9	6948126.473	653229.632	409.314	CASA E OFICINA
HINKEL	CA10	6948251.314	653882.203	413.479	CASA - FOI OBTIDO APENAS O SOBRENOME DO PROP.
ANDRELINO LUSTEMFELD	CA11	6948318.347	653813.247	412.875	CASA
ANDRELINO LUSTEMFELD	CA12	6948341.862	653806.099	411.920	RANCHO
ANDRELINO LUSTEMFELD	CA13	6948320.455	653769.248	411.866	RANCHO E ESTUFA
ANDRELINO LUSTEMFELD	CA14	6948210.623	653552.180	401.276	RANCHO
MARIA LIDIA KISTER	CA15	6946682.560	652333.010	411.605	CASA
LUIZ TAVARES	CA16	6944914.639	653070.136	419.079	CASA
LUIZ TAVARES	CA17	6944943.138	653067.350	413.514	ESTREBARIA
HONORIO STEINHAUSER	CA18	6944859.859	653693.151	410.218	CASA
HONORIO STEINHAUSER	CA19	6944909.887	653694.293	404.776	RANCHO E ESTREBARIA
LUCAS DIAS	CA20	6941988.432	653758.654	414.151	CONJUNTO DE CASAS COMPOSTO POR DUAS CASAS, UM RANCHO E UM RANCHO COM ESTREABARI
PROPRIETÁRIO NÃO IDENTIFICADO	CA21	6939860.005	654760.957	421.208	CASA
PROPRIETÁRIO NÃO IDENTIFICADO	CA22	6939836.909	654772.980	419.286	RANCHO
TADEU WALTER	CA23	6939935.844	654484.026	412.928	CONJUNTO COMPOSTO POR DUAS CASAS, TRÊS RANCHOS E UM RANCHO COM ESTREBARIA
EIXO	EIXO	6956946.858	642675.940	410.172	EIXO BARRAGEM
EIXO	EIXO	6956977.810	642685.667	410.198	EIXO BARRAGEM
EXTRAÇÃO DE AREIA	EXTRAÇÃO DE AREIA	6956025.651	644421.362	387.354	
PONTE EM CONSTRUÇÃO	PONTE	6954440.742	646454.696	404.136	PONTO PEGO ALEATÓRIAMENTE EM CIMA DA PRIMEIRA VIGA - FOTO
ACESSO PARA PONTE	ACESSO PARA PONTE	6954359.824	646471.263	406.405	
ACESSO PARA PONTE	ACESSO PARA PONTE	6954319.047	646480.768	406.873	
PONTE ACESSO PINGUIRITO	PONTE ACESSO PINGUIRIT	6939620.556	655752.251	414.226	O PONTO FOI COLETADO EM CIMA DO APOIO DA PONTE, O QUAL ESTÁ BEM ABAIXO DAS CASAS DA

Att. 3 - 2



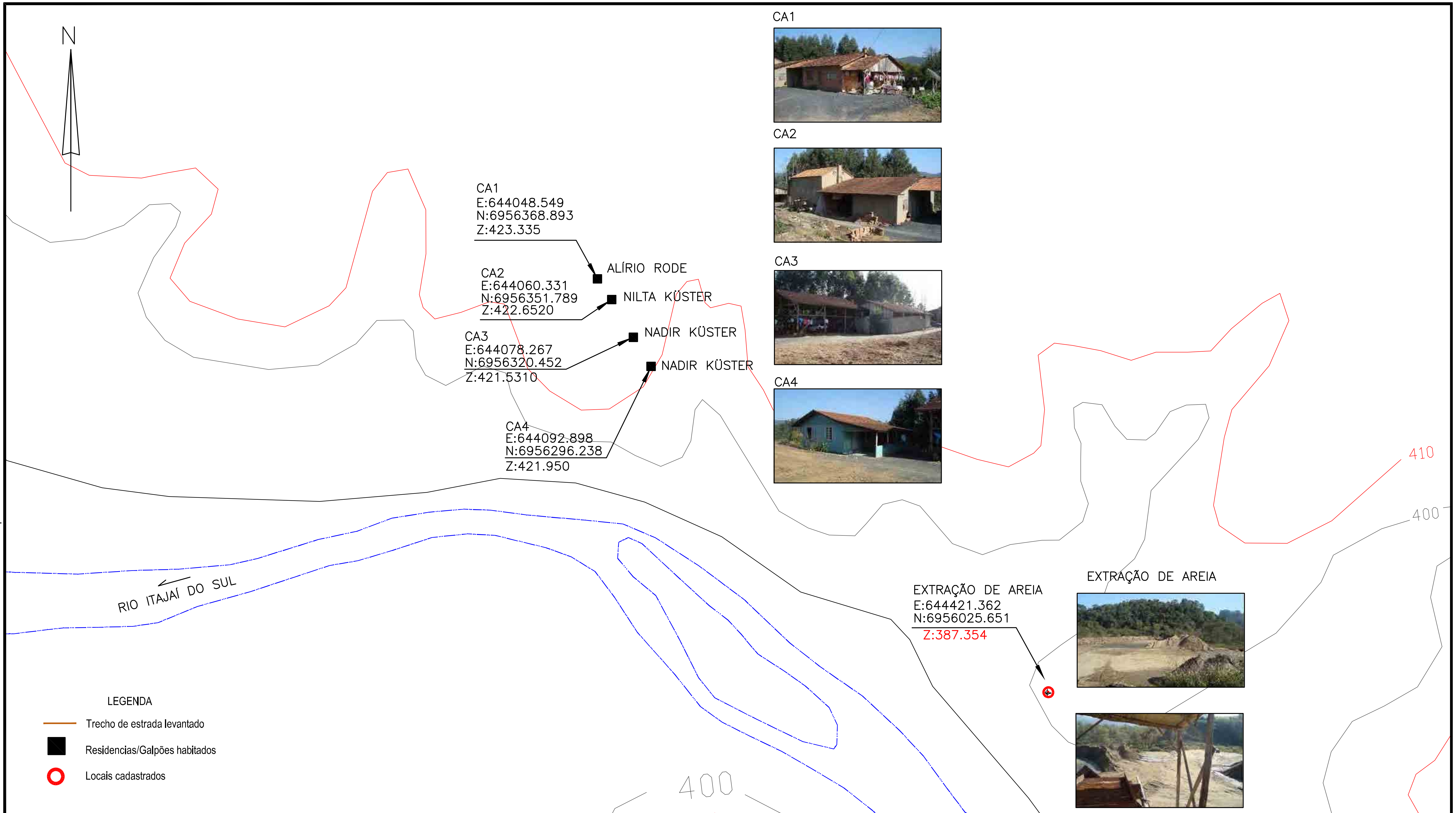
SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO:	LOCAL: ITUPORANGA	FOLHA:
Marinalva Ramos CREA-SC 88806-7	MUNICÍPIO: ITUPORANGA	01

DATA:	OPERADOR:	ESCALA:	ESTADO:
JUNHO/2011	SERGIO	1:3000	SC

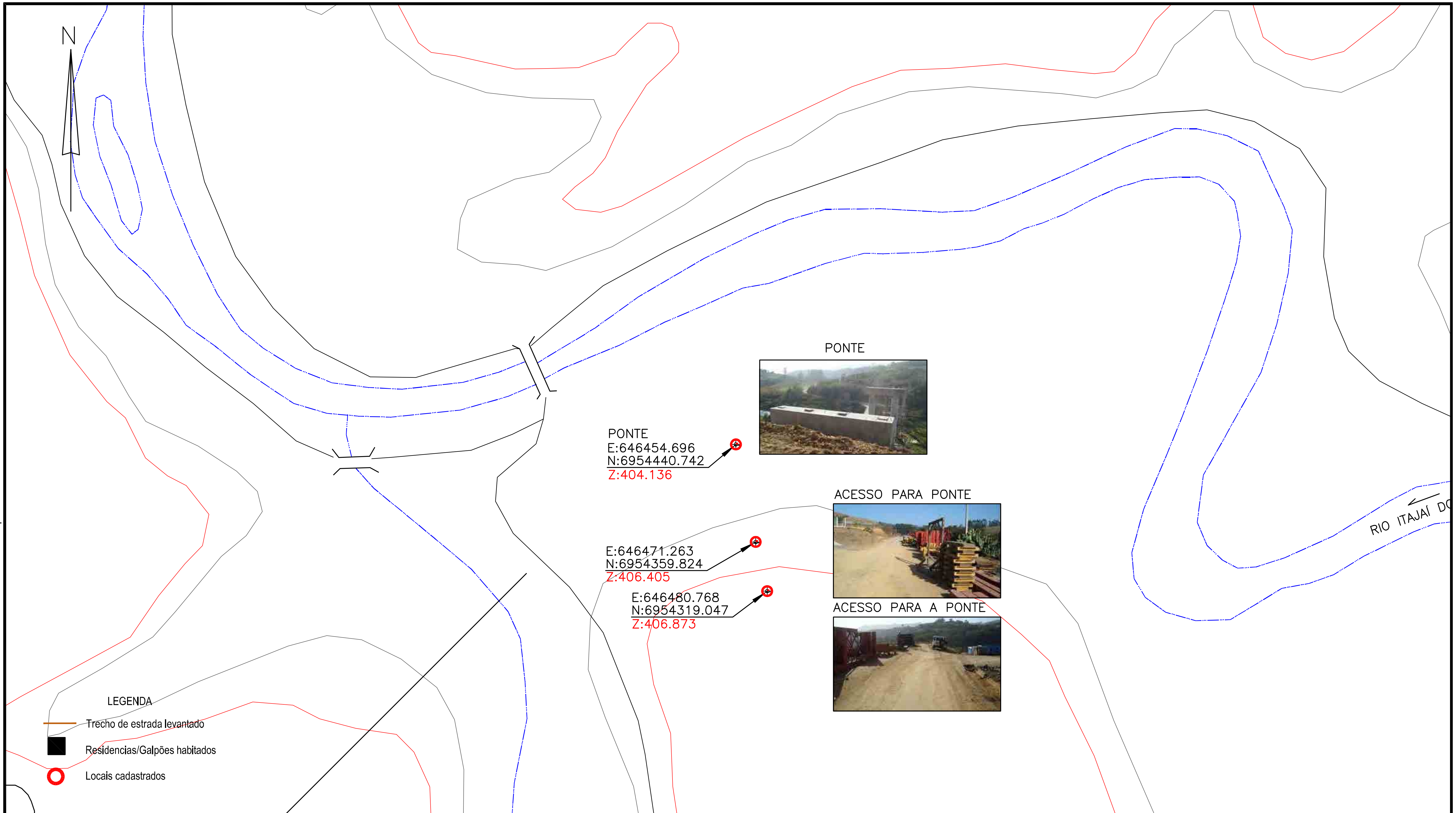
Att. 3 - 4



SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

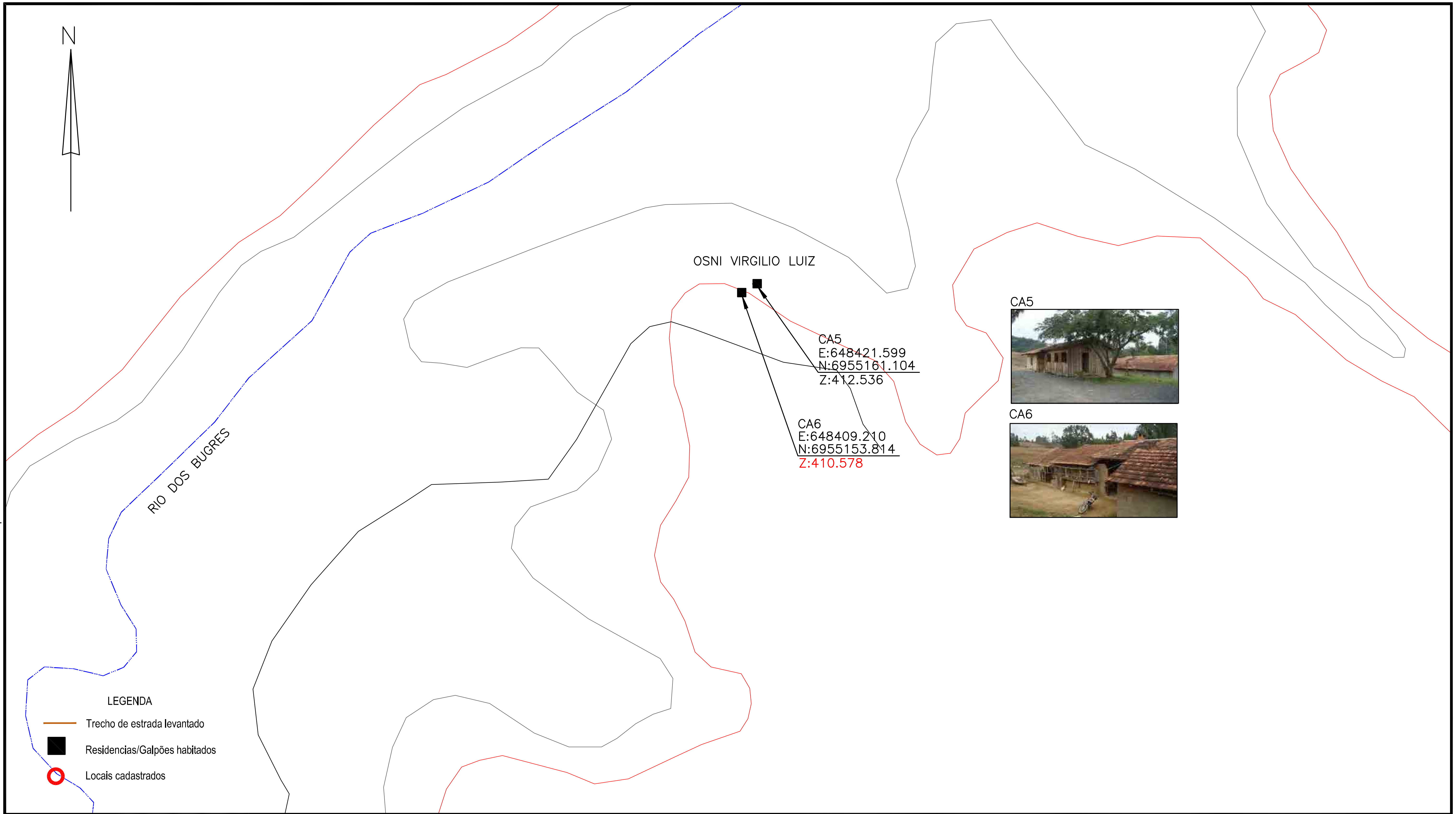
RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7		LOCAL: ITUPORANGA / MUNICÍPIO: ITUPORANGA	FOLHA: 02
DATA: JUNHO/2011	OPERADOR: SERGIO	ESCALA: 1:3000	ESTADO: SC



SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7	LOCAL: ITUPORANGA MUNICÍPIO: ITUPORANGA	FOLHA: 03
DATA: JUNHO/2011 · OPERADOR: SERGIO · ESCALA: 1:3000	ESTADO: SC	



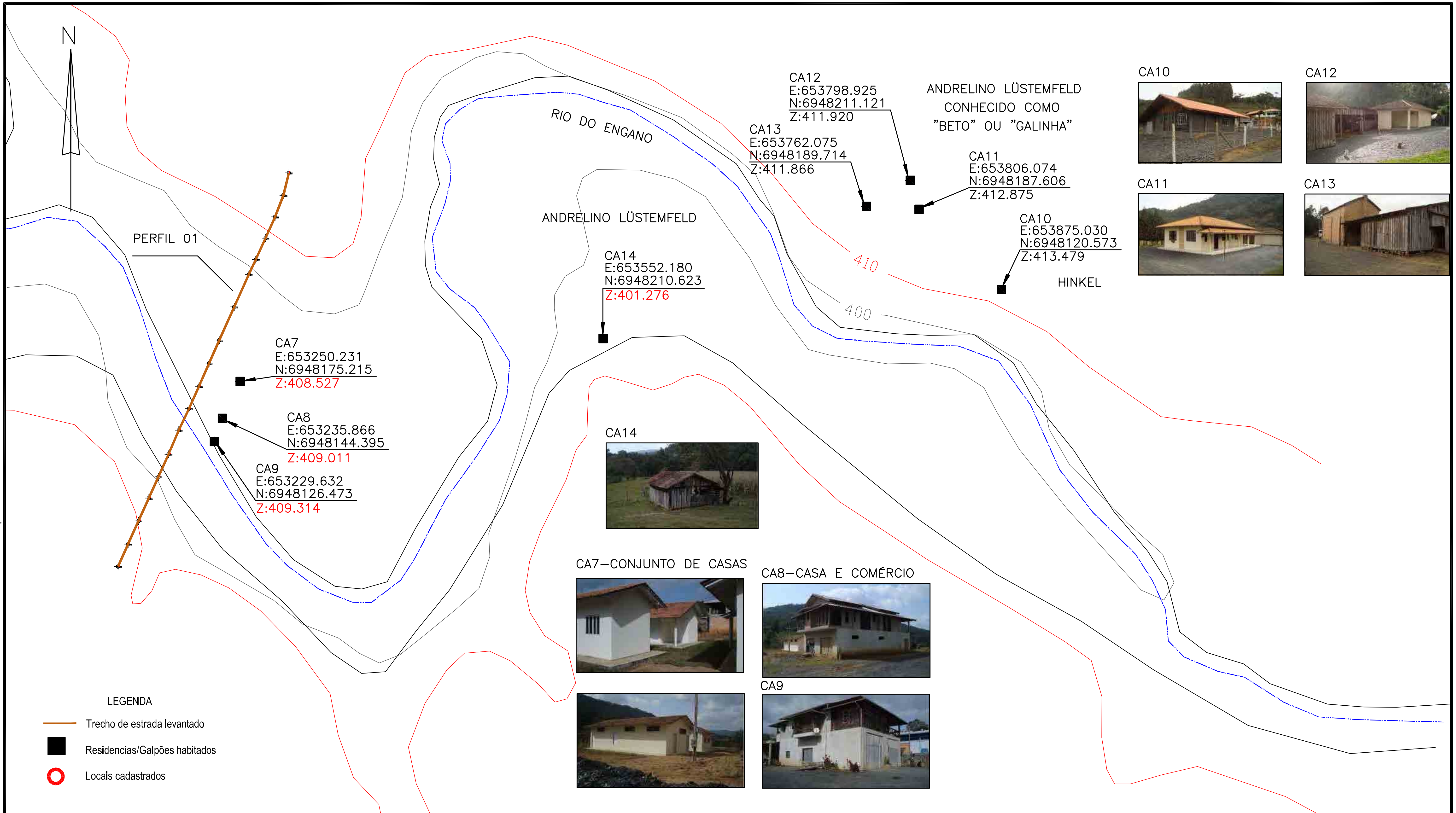
Att. 3 - 6

SERVIÇO:

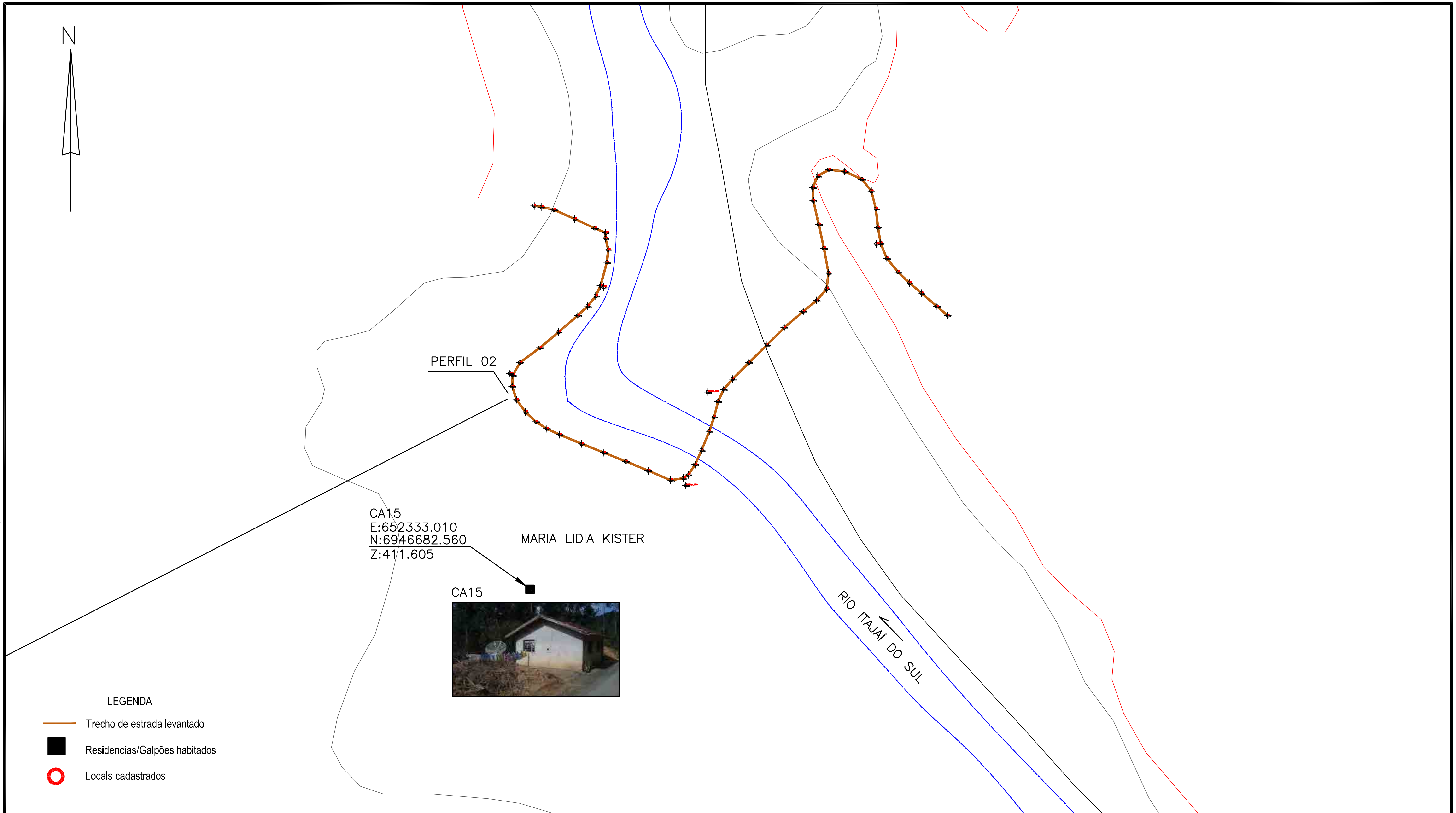
Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO:	LOCAL: ITUPORANGA	FOLHA:
Marinalva Ramos CREA-SC 88806-7	MUNICÍPIO: ITUPORANGA	04

DATA:	OPERADOR:	ESCALA:	ESTADO:
JUNHO/2011	SERGIO	1:3000	SC



SERVIÇO:	Estudo realizado para verificação das áreas atingidas pela cota de inundação 410m - Barragem Sul Município de Ituporanga - SC
RESPONSÁVEL TÉCNICO:	Marinalva Ramos CREA-SC 88806-7
LOCAL:	ITUPORANGA
MUNICÍPIO:	ITUPORANGA
ESTADO:	SC
FOLHA:	05
DATA:	JUNHO/2011
OPERADOR:	SERGIO
ESCALA:	1:3000

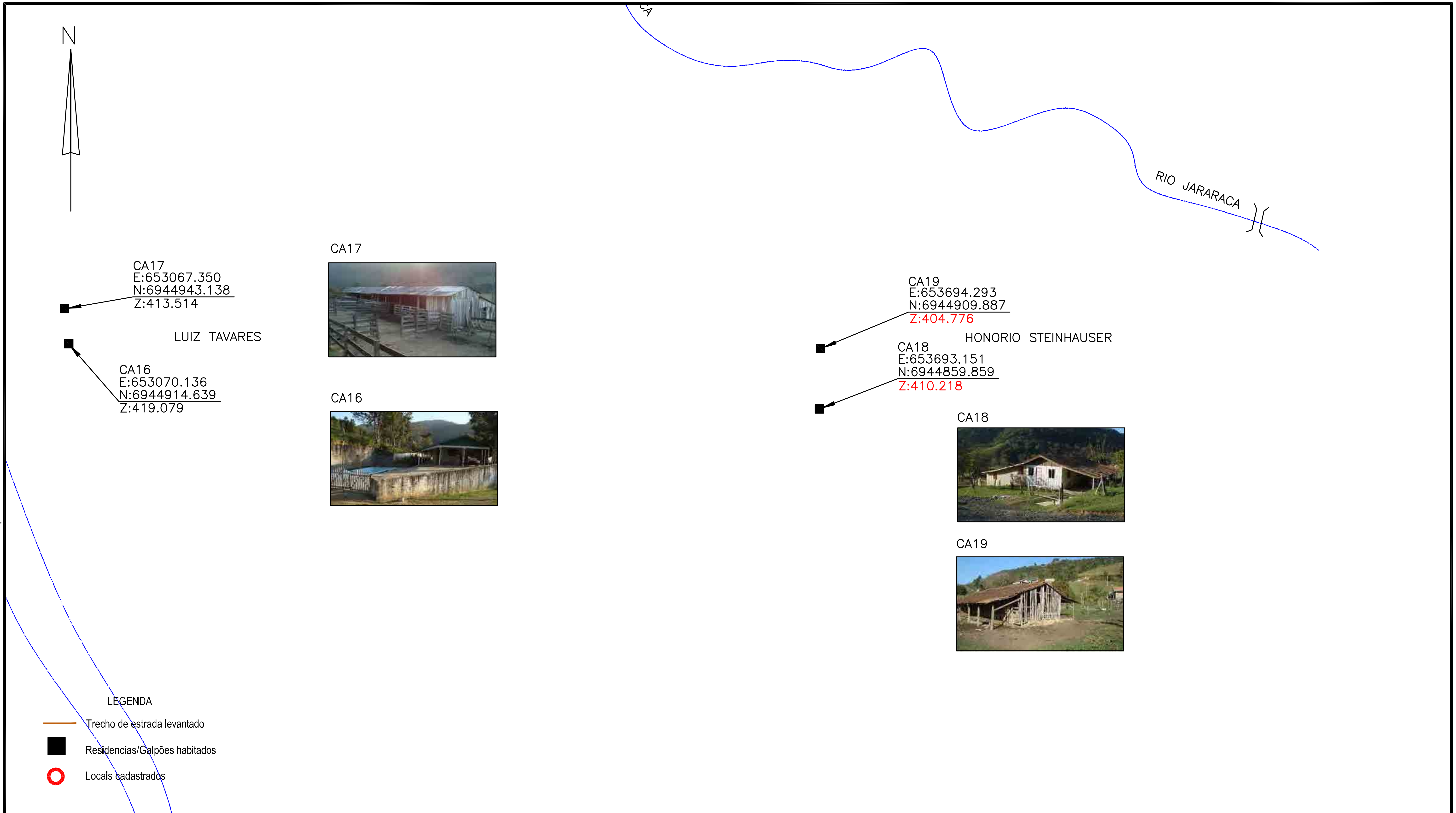


SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7		LOCAL: ITUPORANGA MUNICÍPIO: ITUPORANGA	FOLHA: 06
DATA: JUNHO/2011	OPERADOR: SERGIO	ESCALA: 1:3000	ESTADO: SC

Att. 3 - 8

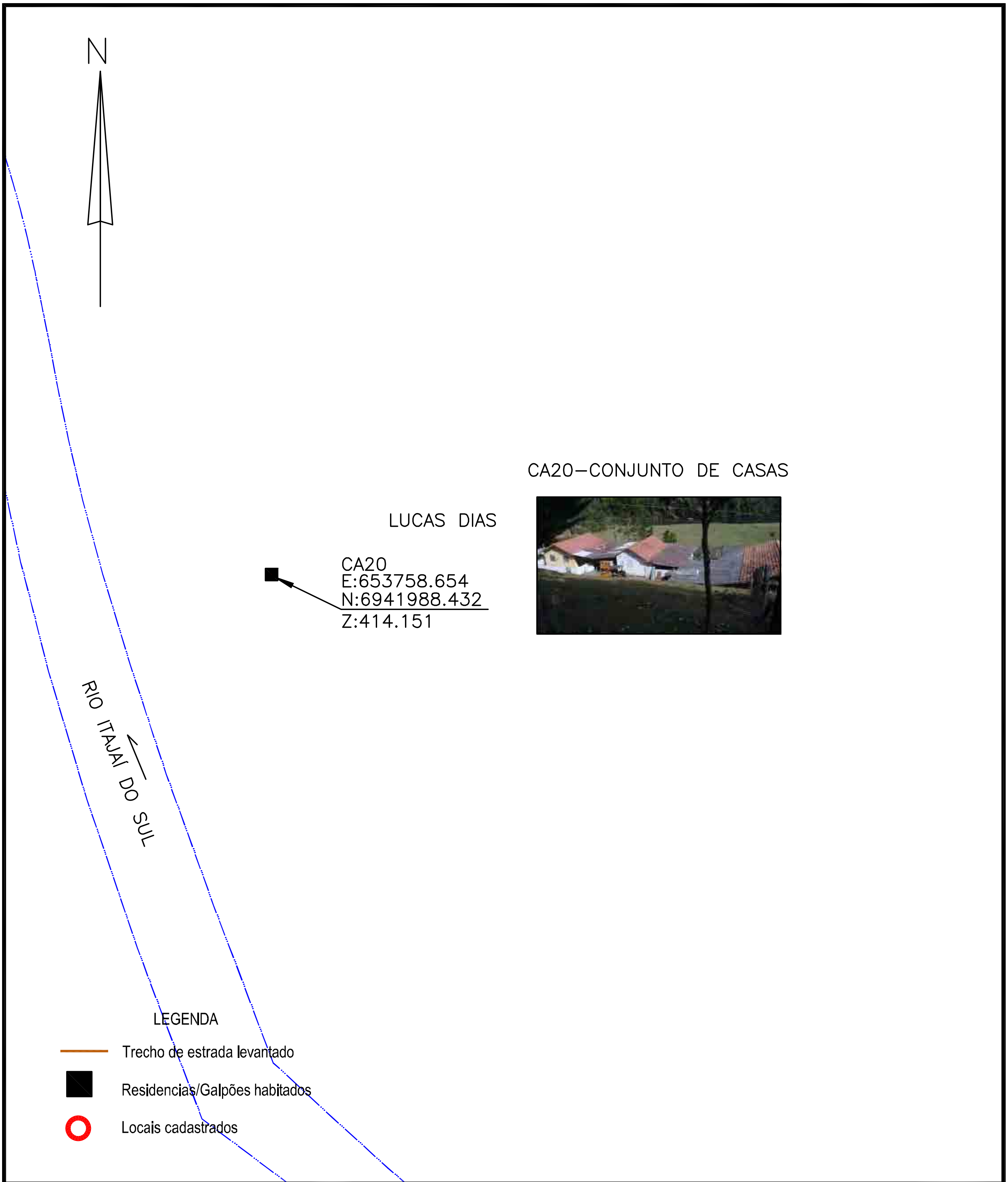


Att. 3 - 9

SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7		LOCAL: ITUPORANGA / MUNICÍPIO: ITUPORANGA	FOLHA: 07
DATA: JUNHO/2011	OPERADOR: SERGIO	ESCALA: 1:3000	ESTADO: SC

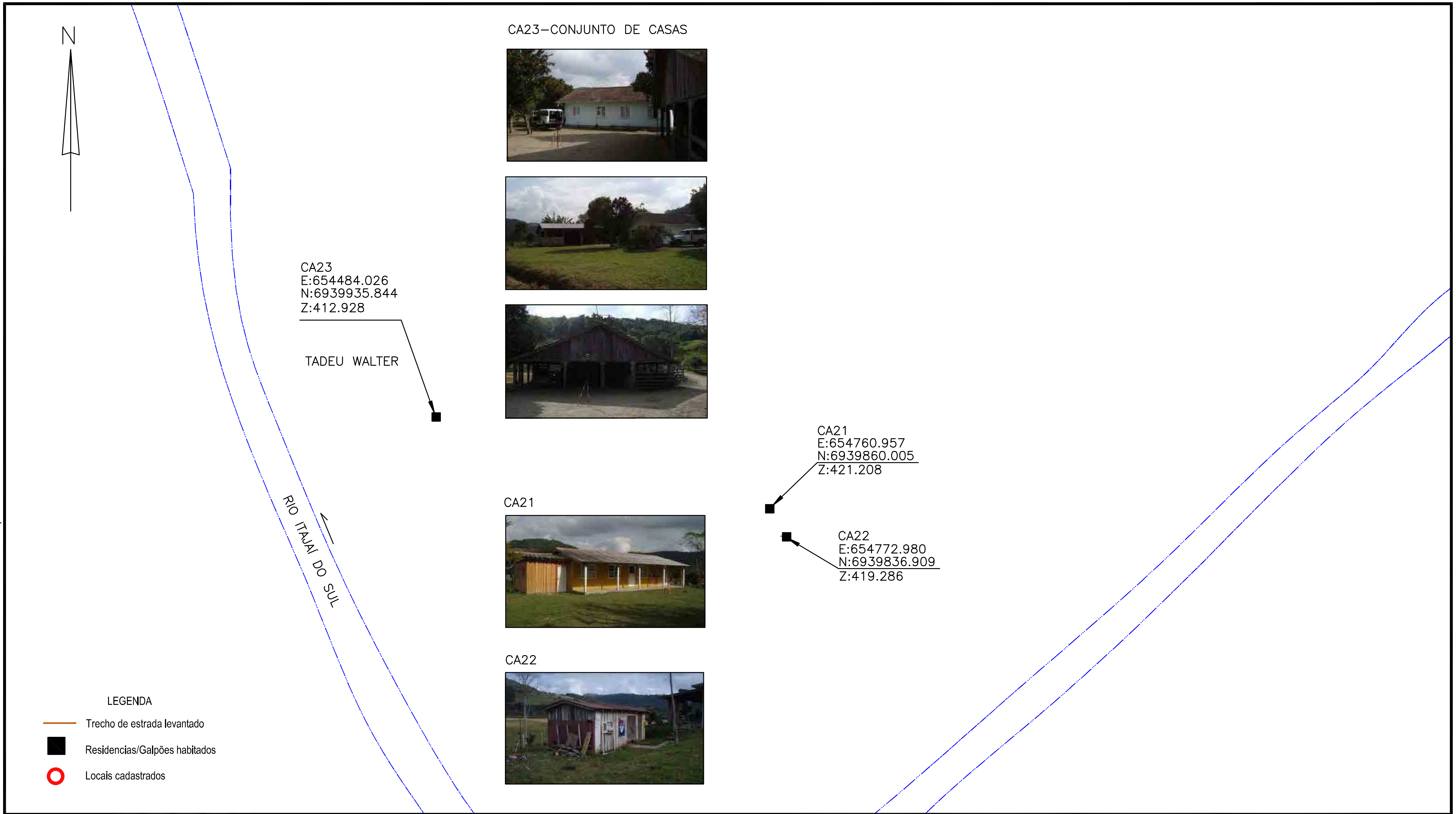


SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7	LOCAL: ITUPORANGA	FOLHA: 08
	MUNICÍPIO: ITUPORANGA	
DATA: JUNHO/2011 · OPERADOR: SERGIO · ESCALA: 1:3000	ESTADO: SC	

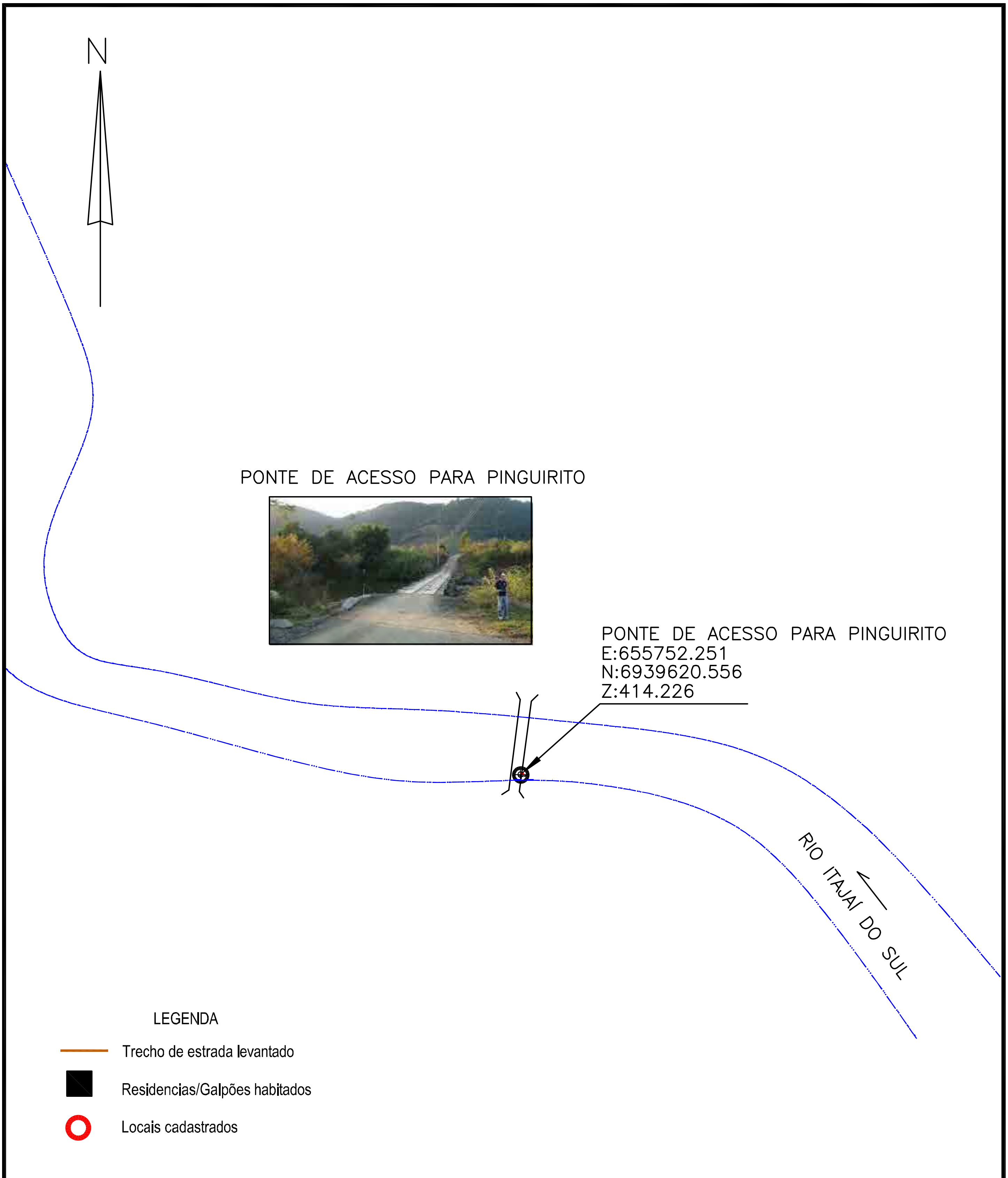
Att. 3 - 11



SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7		LOCAL: ITUPORANGA / MUNICÍPIO: ITUPORANGA	FOLHA: 09
DATA: JUNHO/2011	OPERADOR: SERGIO	ESCALA: 1:3000	ESTADO: SC

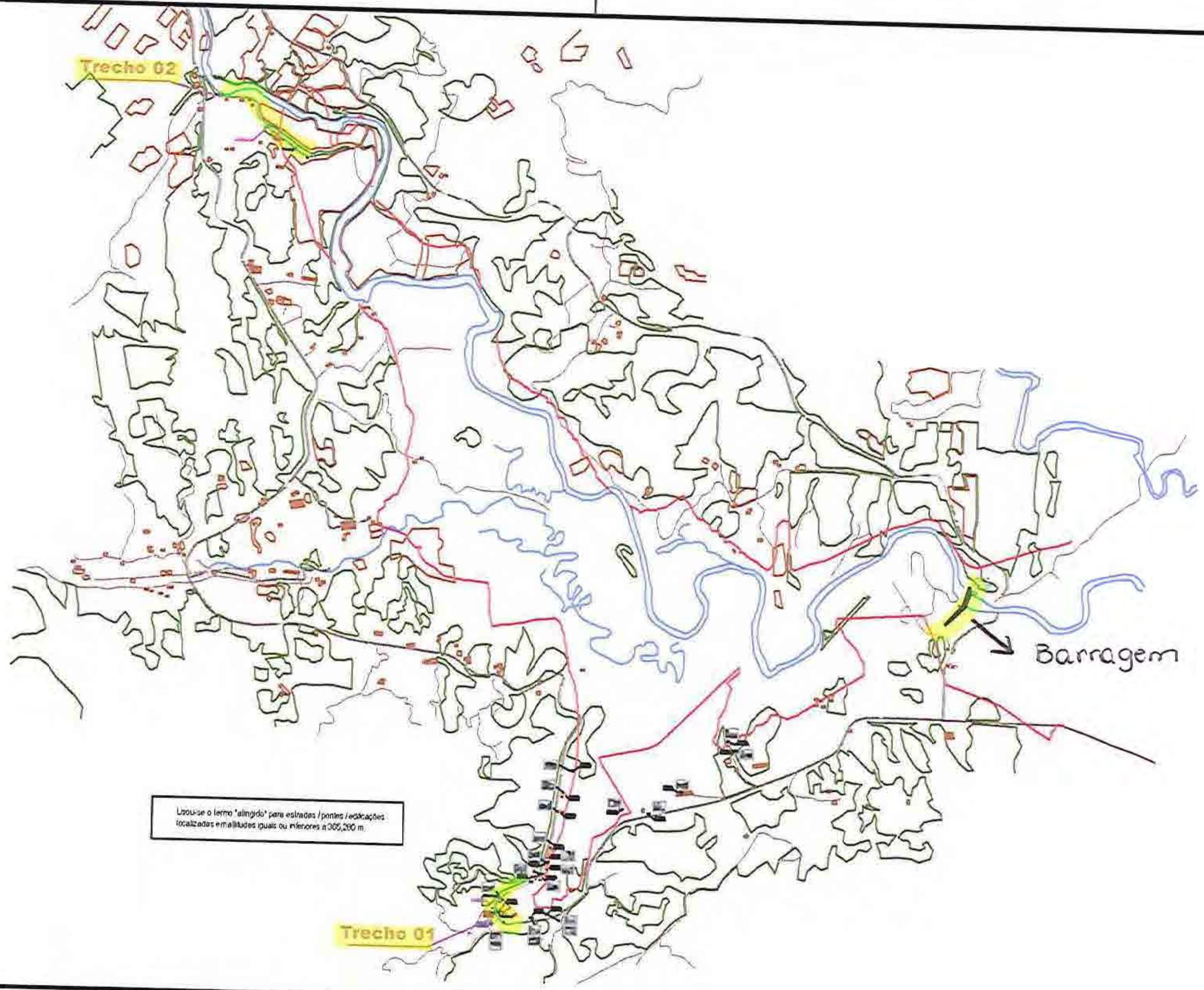


SERVIÇO:

Estudo realizado para verificação das áreas atingidas
pela cota de inundação 410m - Barragem Sul
Município de Ituporanga - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos, CREA-SC 88806-7	LOCAL: ITUPORANGA	FOLHA: 10
	MUNICÍPIO: ITUPORANGA	
DATA: JUNHO/2011 · OPERADOR: SERGIO · ESCALA: 1:3000	ESTADO: SC	

Oeste Dam

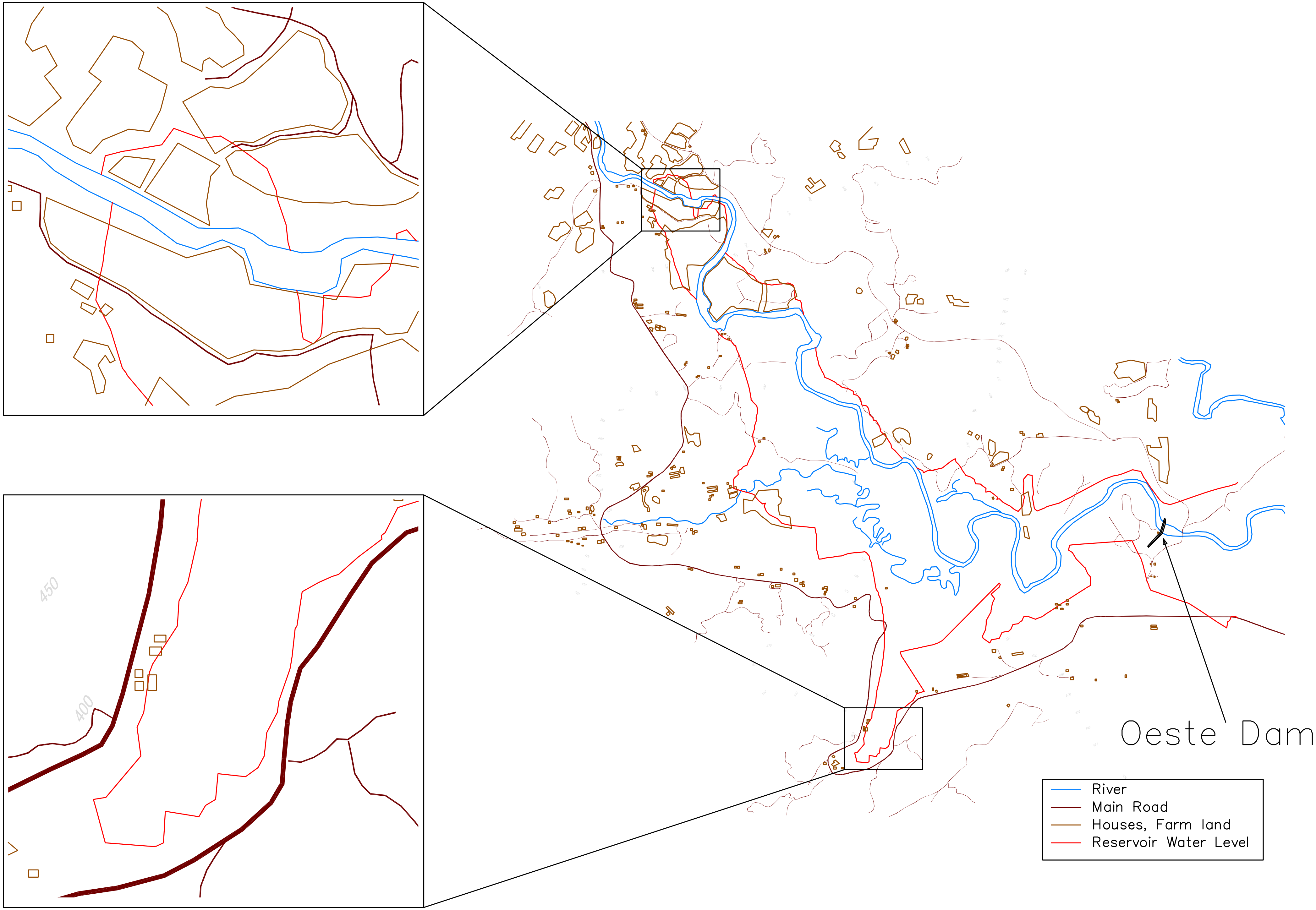


Legenda

- Estradas levantadas
- Trecho de estrada atingido
- Residencias/Galpões atingidos
- Residencias/Galpões não atingidos
- Ponte atingida
- Residência com altitude próxima a cota máxima

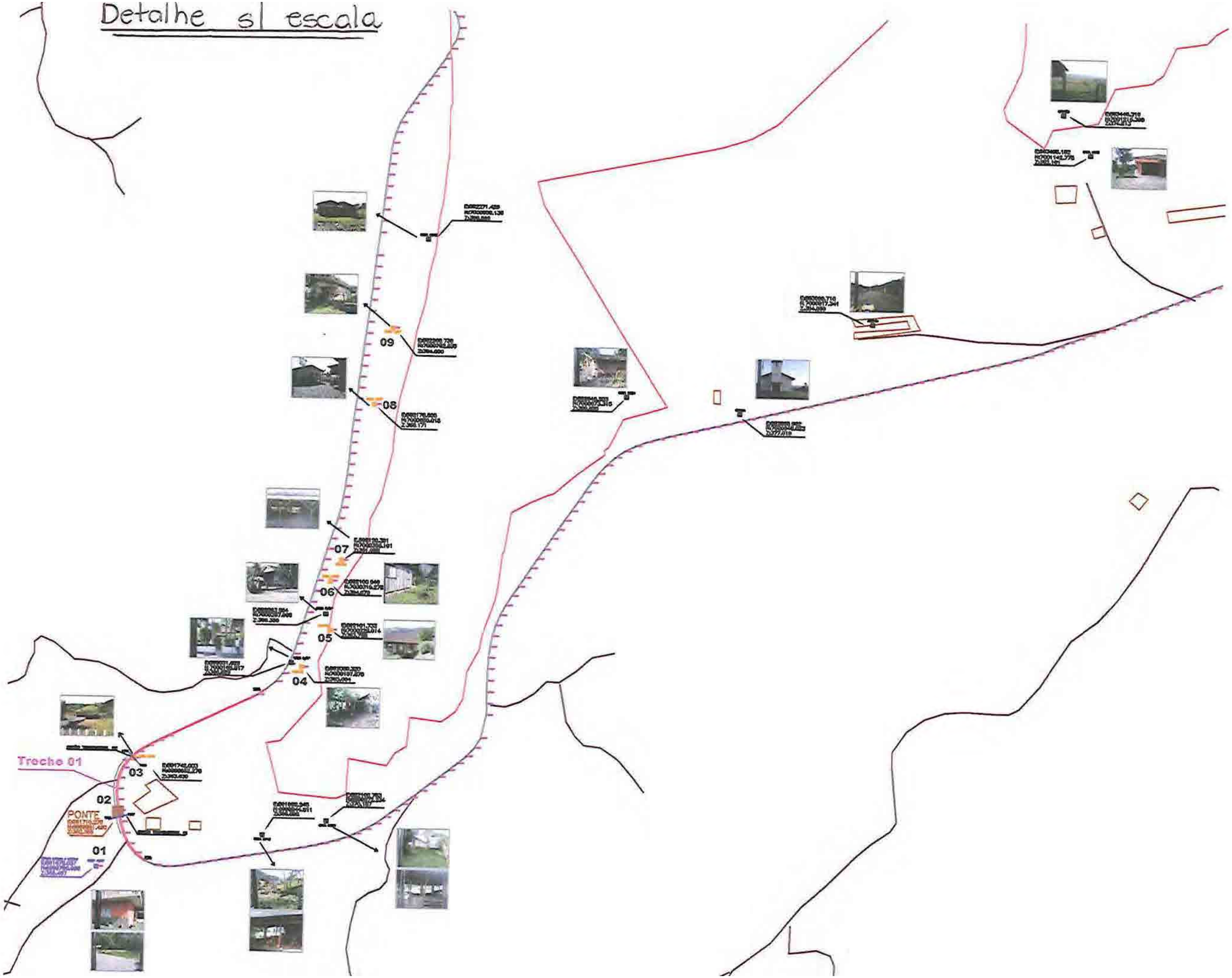
SERVIÇO:	Estudo realizado para verificação das áreas atingidas em razão da elevação de 2 metros da crista da barragem localizada no município de Taió - SC
RESPONSÁVEL TÉCNICO:	Marinalva Ramos CREA-SC 88806-7
LOCAL:	TAIÓ
MUNICÍPIO:	TAIÓ
FOLHA:	00
DATA:	ABRIL/2011
OPERADOR:	LUCAS
ESCALA:	SEM ESCLA

Att. 3 - 14

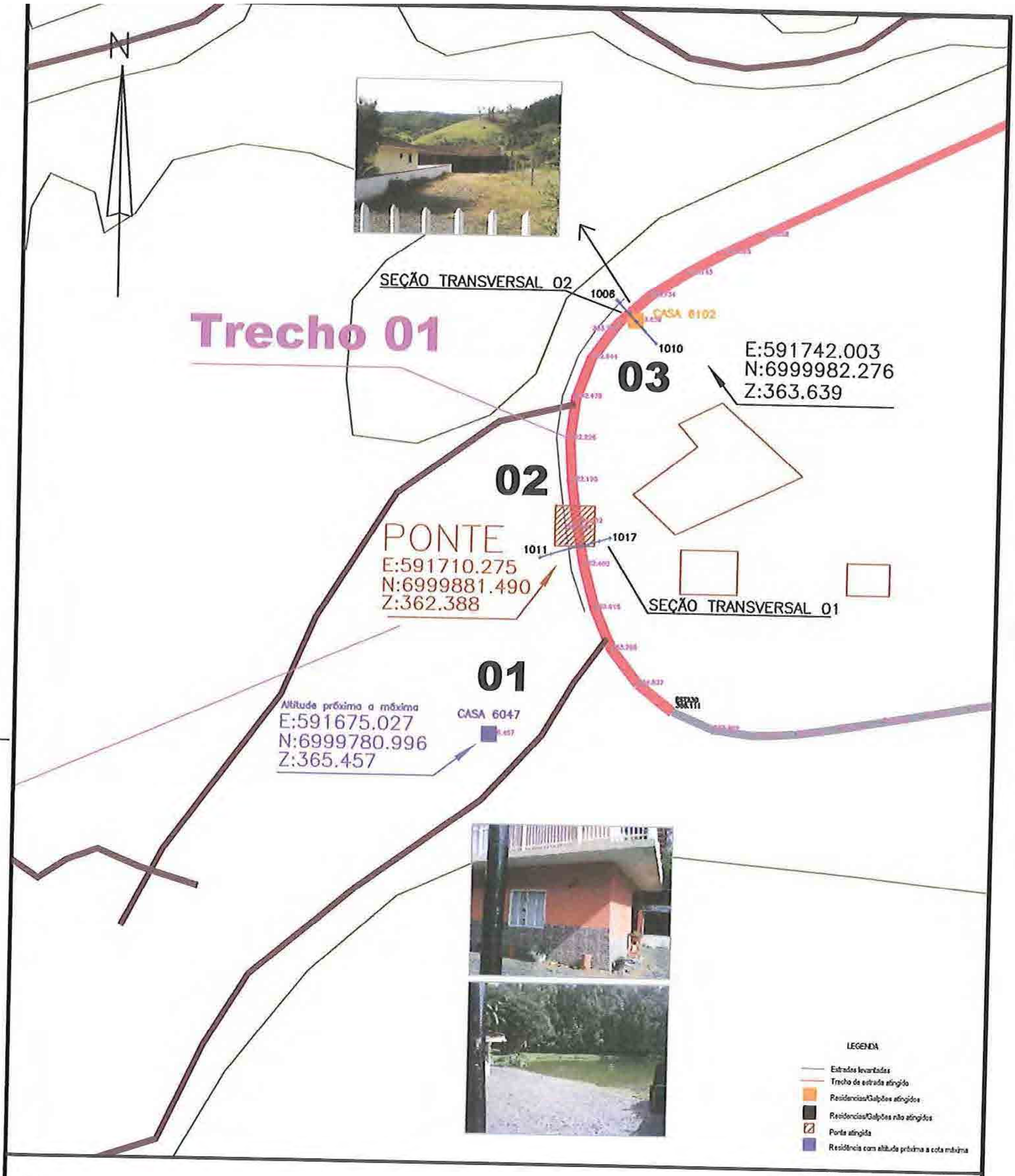


450
400
Oeste Dam
River
Main Road
Houses, Farm land
Reservoir Water Level

Detalhe s/ escala



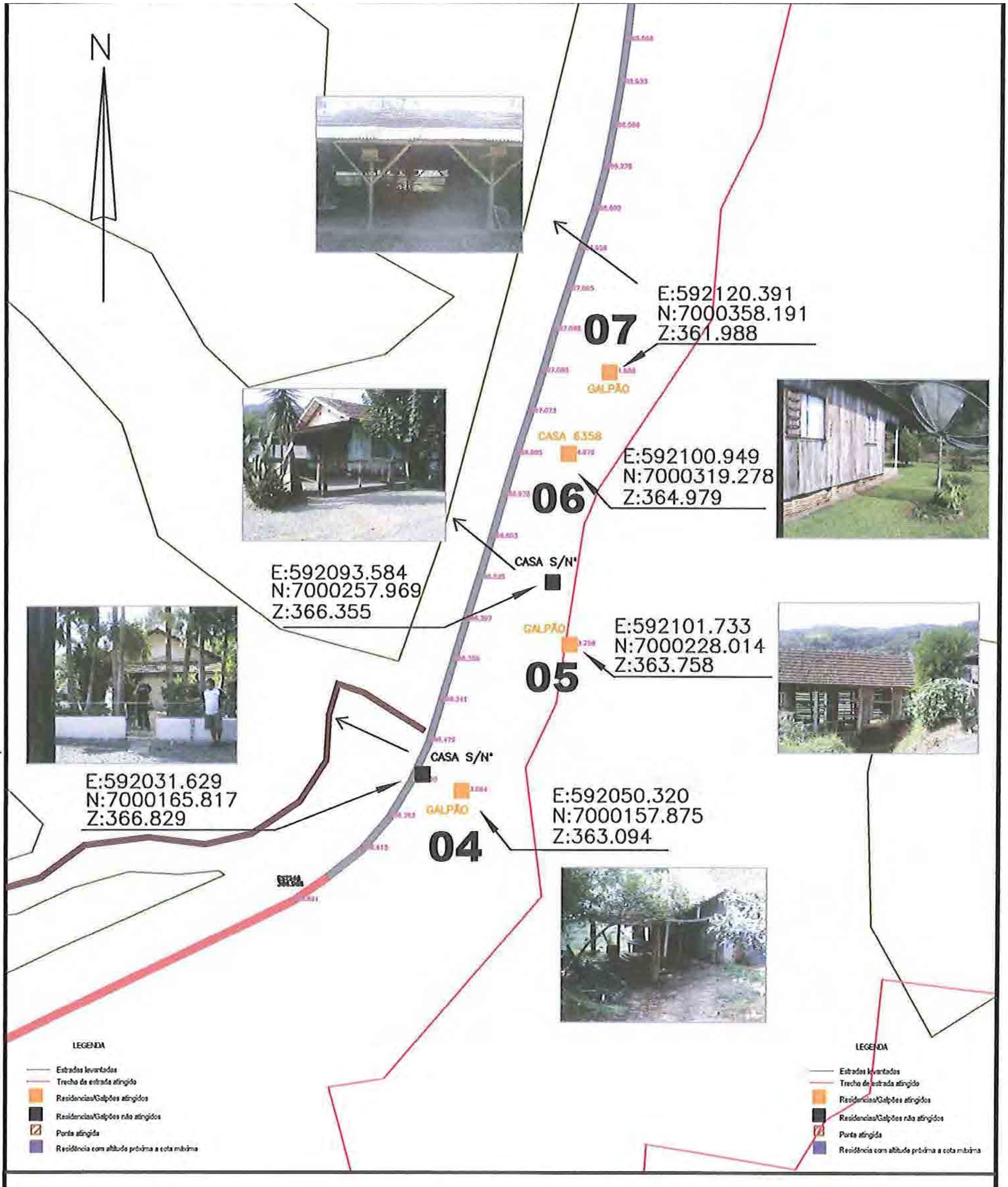
Att. 3 - 15



SERVIÇO:

Estudo realizado para verificação das áreas atingidas em razão da elevação de 2 metros da crista da barragem localizada no município de Taió - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos, CREA-SC 88806-7		LOCAL: TAIÓ	FOLHA: 05
		MUNICÍPIO: TAIÓ	
DATA: ABRIL/2011	OPERADOR: LUCAS	ESCALA: 1:2000	ESTADO: SC

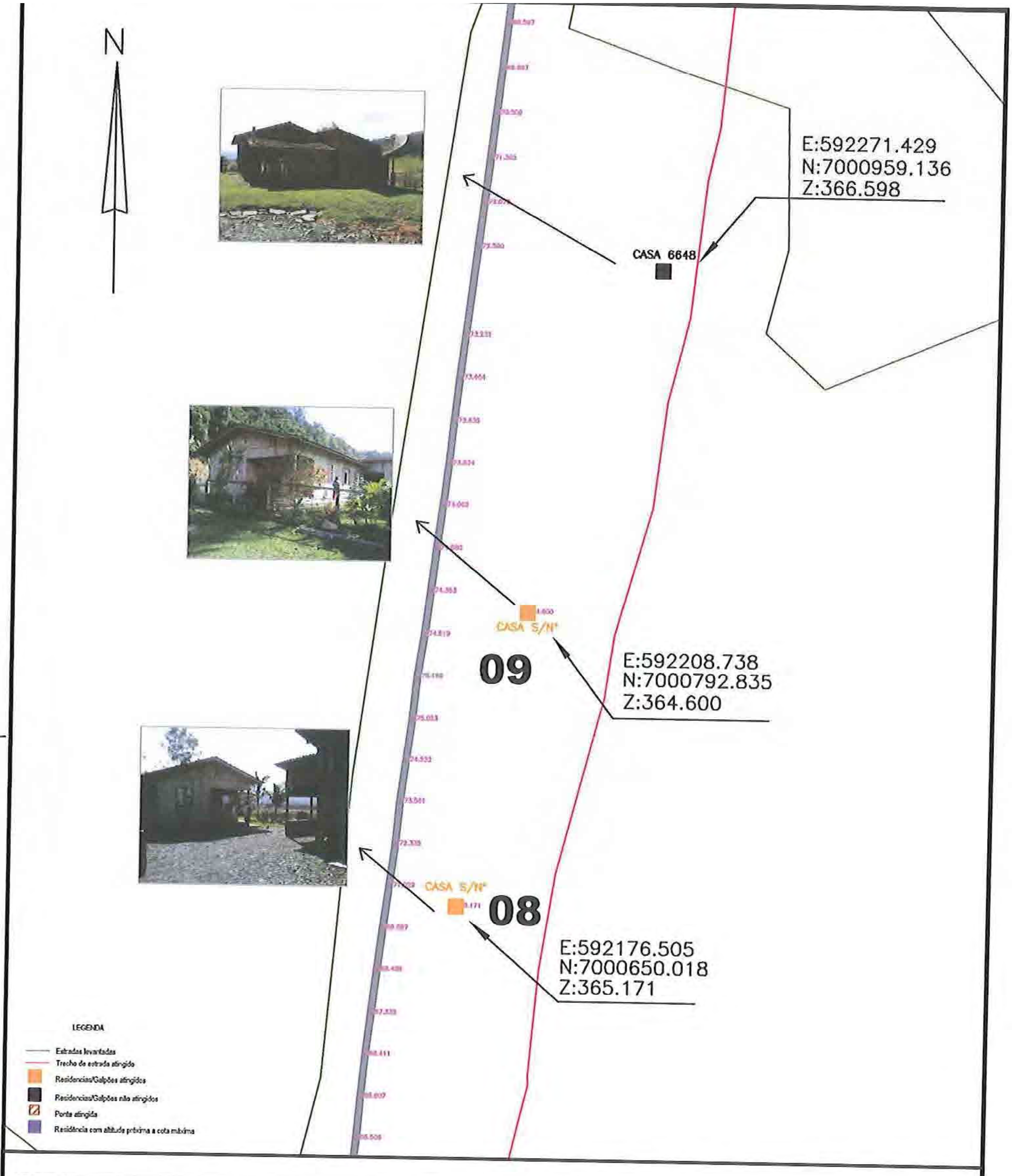


SERVIÇO:

Estudo realizado para verificação das áreas atingidas
em razão da elevação de 2 metros da crista da
barragem localizada no município de Taió - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7	LOCAL: TAIÓ MUNICÍPIO: TAIÓ	FOLHA: 06

DATA: ABRIL/2011	OPERADOR: LUCAS	ESCALA: 1:2000	ESTADO: SC



SERVIÇO:

Estudo realizado para verificação das áreas atingidas em razão da elevação de 2 metros da crista da barragem localizada no município de Taió - SC

RESPONSÁVEL TÉCNICO: Marinalva Ramos CREA-SC 88806-7	LOCAL: TAIÓ	FOLHA: 07
	MUNICÍPIO: TAIÓ	
DATA: ABRIL/2011 · OPERADOR: LUCAS · ESCALA: 1:2000	ESTADO: SC	

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

Table of Contents

<u>Tables</u>

Figures

Page

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.

Mater Plan as structural measures

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

Item	Condition	Dam Type/Flood	Criteria
Freeboard	Normal	Rock fill dam	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.
		Concrete dam	At least 1.5 m shall be secured as the minimum freeboard.
	Flood	Rock fill dam	The minimum freeboard shall be secured 1.0 m above the maximum flood water level in reservoir.
		Concrete dam	The minimum freeboard shall be secured 0.5 m above the maximum flood water level in reservoir.
Extraordinary flood	Normal	Probable maximum flood	For dam higher than 30 m, or there are permanent residents downstream and danger of dam failure
	Small scale dam	1000-year flood	For dam lower than 30 m, or reservoir capacity of smaller than 50 million m^3 and there are no permanent residents downstream.

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.

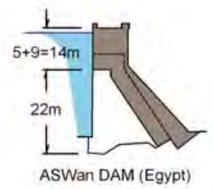
The Oeste dam is a concrete gravity dam, corresponding to the dam type to easy to be raised. As the dam is planned to be raised by 2 m at both the overflow and non-overflow portions, the form of existing spillway is just to be slid upward. The Sul dam is a rock fill dam of the zoned type. In case of 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

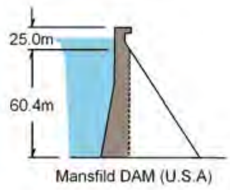
1) Mode of Heightening of Concrete Dam

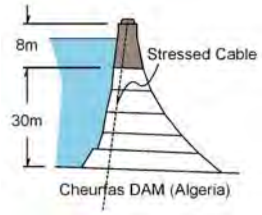
The table below presents the comparison of methods for heightening concrete gravity dams. As the planned heightening is as small as 2 m at the Oeste dam, raising the dam crest was selected.

Table 2.2.2 Heightening Method of Concrete Gravity Dam

<table>
<tr><th>Covering of New Dam</th><th>Raising of Dam Crest</th><th>Thickening of Upstream Dam Body</th><th>Anchoring</th></tr>
<tr><td>


</td><td>


</td><td>


</td><td>


</td></tr>
<tr><td>Placing new concrete on the downstream face of existing dam and forming unified dam body of the new and old concretes</td><td>Placing new concrete on the dam crest and forming unified dam body of the new and old concretes</td><td>Placing new concrete on the upstream face of the existing dam and forming unified body of the new and old concretes</td><td>Placing new concrete on the dam crest and connecting to the upstream dam foundation by stress cable</td></tr>
</table>

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

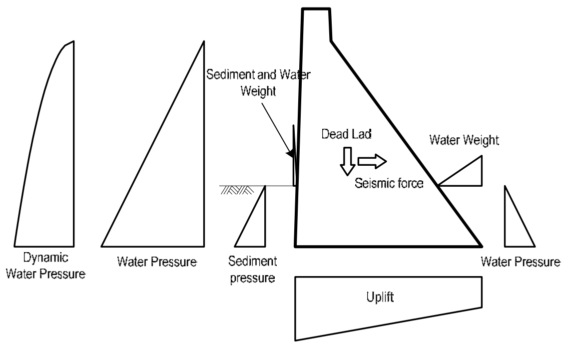
f. Combination of Loads Condition

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.

$$Fh = 0.05 \cdot P \quad \text{(Horizontal)}$$

$$Fv = 0.03 \cdot P \quad \text{(Vertical)}$$

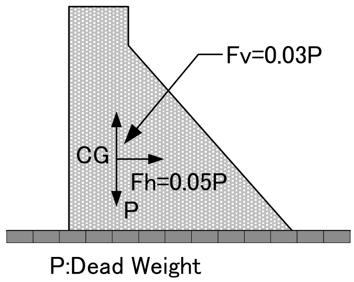


Figure 2.2.2 Diagram of Seismic Factor

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

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)

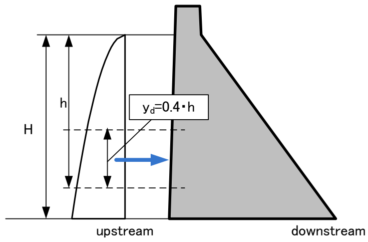


Figure 2.2.3 Diagram of Dynamic Water Pressure

- 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.

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

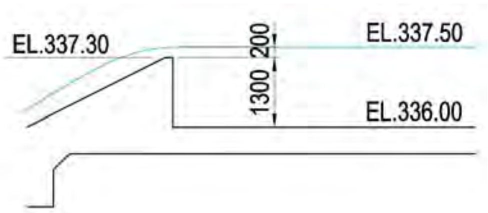


Figure 2.2.4 Water Level at Downstream (Oeste Dam)

(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

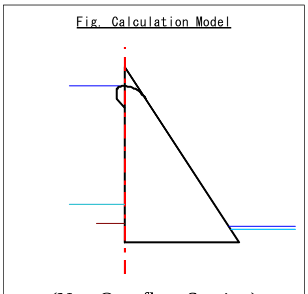
The coefficient of uplift is 1/3 because of the foundation of dam is supposed to be the rock.

iii) Stability Analysis of Existing Oeste dam

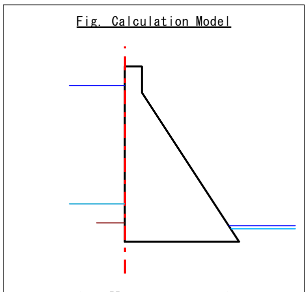
As mentioned earlier, there are no geology date of dam foundation available, the case of existing dam is calculated to estimate the physical properties. The result of analysis, the angle of internal friction and shearing stress are φ=45° and c=50 kN/m^2 is satisfied the result. The definitive loading 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



(Non Overflow Section)



(Spillway Section)

(Result) <u>Non-overflow Section</u>

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.

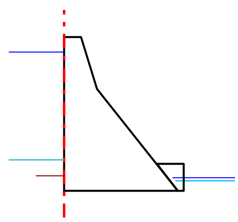
The definitive condition is that the angle of internal friction and shearing stress are φ=45° and c=50 kN/m^2 and loading condition is CCL(Flood＋Earthquake). The critical bearing capacity of foundation ground is requied qu=2,000 kN/m^2.

- 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
	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 2.2.12 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

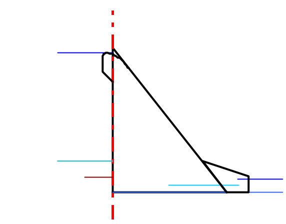
Source : JICA survey team

- Spillway Section

(Calculation Condition)

1.	Elevation of Top of Dam	H_0 =	365.000 m
2.	Downstream Slope	1 : n	0.750
3.	Dam base elevation	H_0 =	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
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 2.2.13 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.

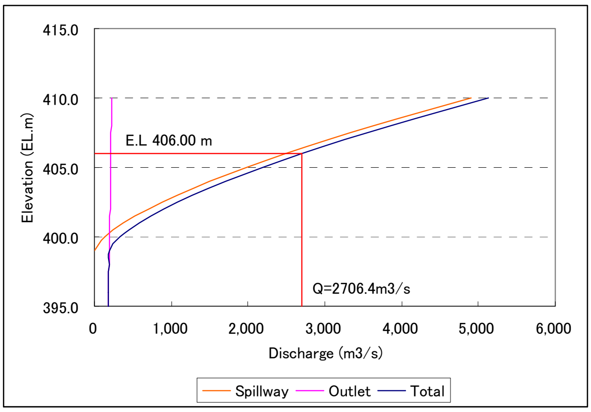
The elevation of the crest of spillway and dam body is 399.0 m and 410 m respectively.
The elevation difference is 11.0 m. The overflow height of spillway is maximum 7.0 m and the probable water lever is 406.m, and freeboard is estimated 4.0 m. The rockfill dam is required 1.0 m for freeboard, so that 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

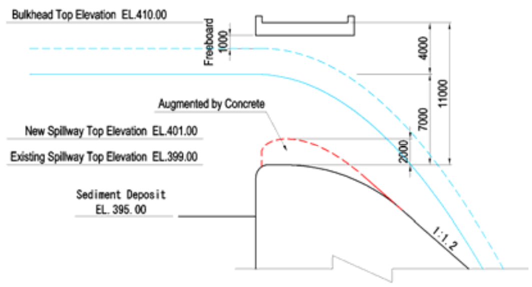


Source : JICA Survey Team

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

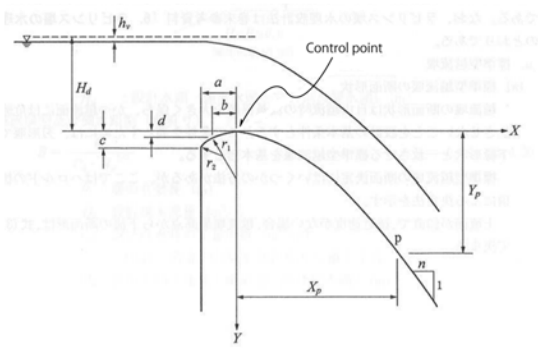
The shape of the crest spillway is basically required to keep the coefficient of discharge is high with free overflow and not to occur the suction at the overflow section. To meet those conditions is the shape of typical spillway.

Standard Shape of Overflow Spillway of Curve

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

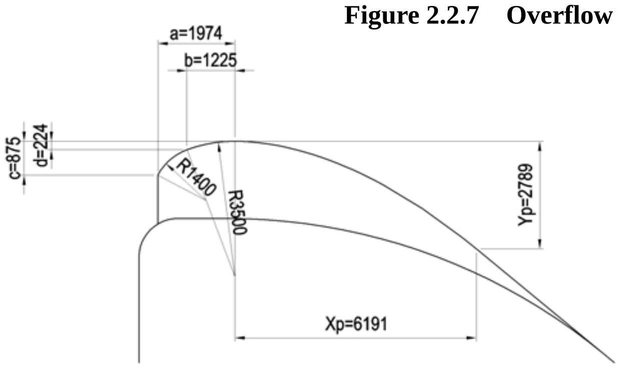
Hd=	7.000m
a=0.282*Hd →	1.974m
b=0.175*Hd →	1.225m
c=0.125*Hd →	0.875m
d=0.032*Hd →	0.224m
r1=0.5*Hd →	3.500m
r2=0.2*Hd →	1.400m

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

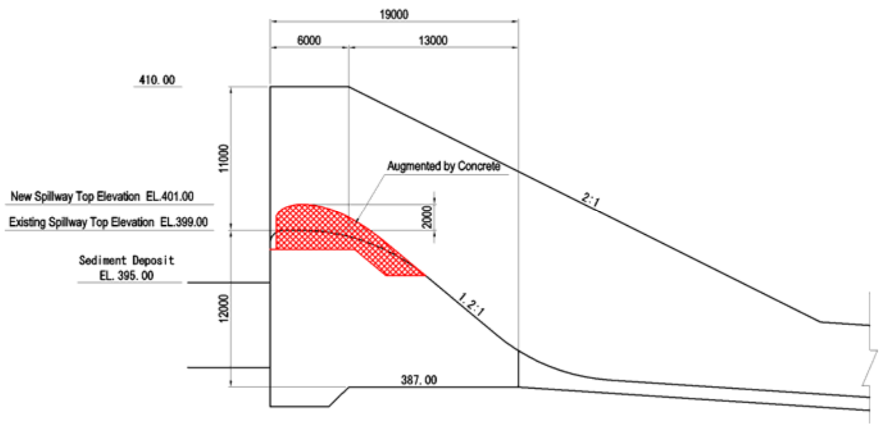
vi) Results of Stability Analysis (2.0 m heightening)

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/m³
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/m³
9. Weight of sediment in the air	γ_s =	17.5 kN/m³
10. Weight of sediment in water	γ_s =	8.5 kN/m³
11. Unit weight of water	γ_w =	10.0 kN/m³
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/m²
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/m²)	Downstream (kN/m²)
[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.

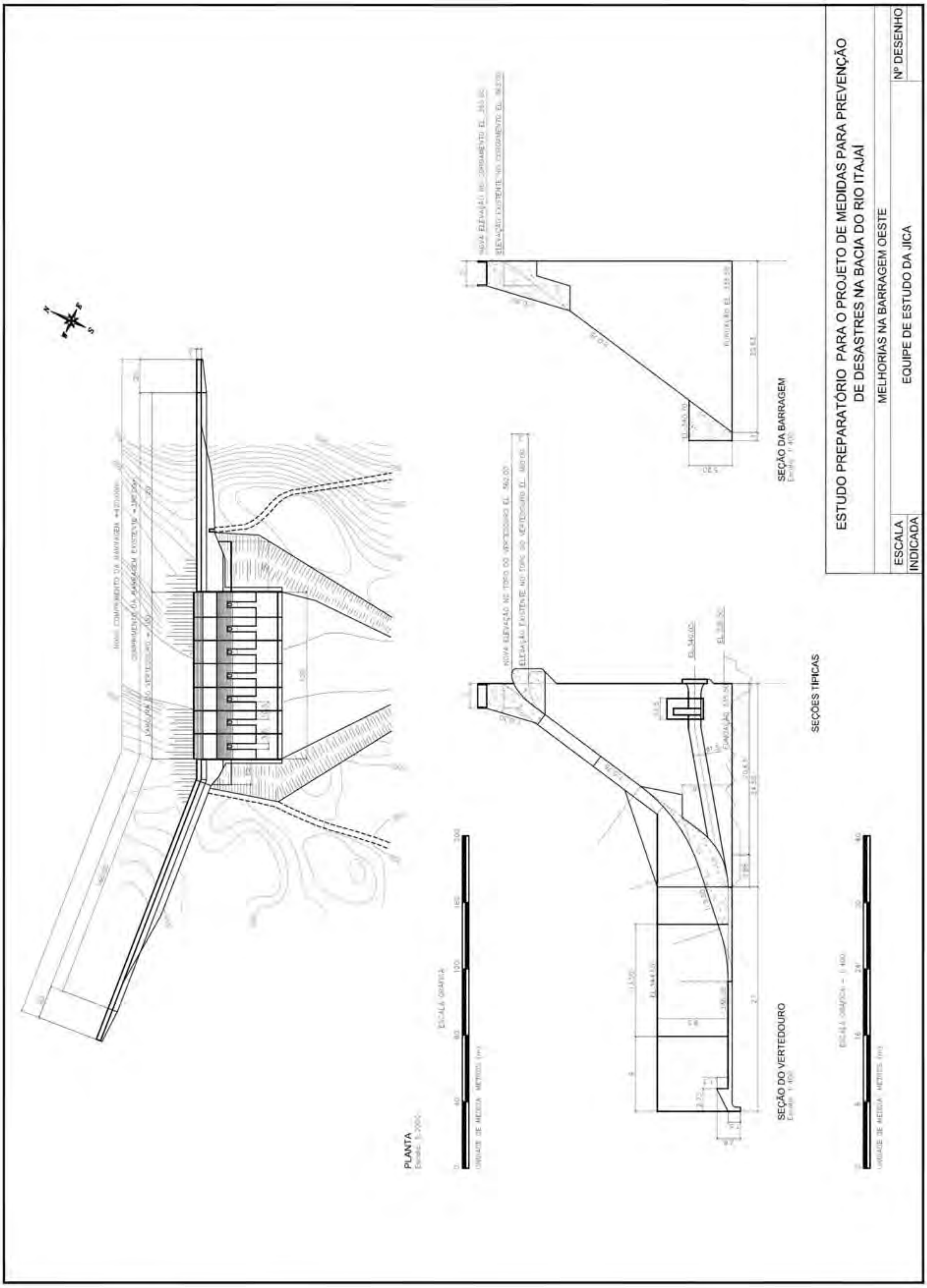


Figure 2.2.10 Drawing on Heightening of Oeste Dam

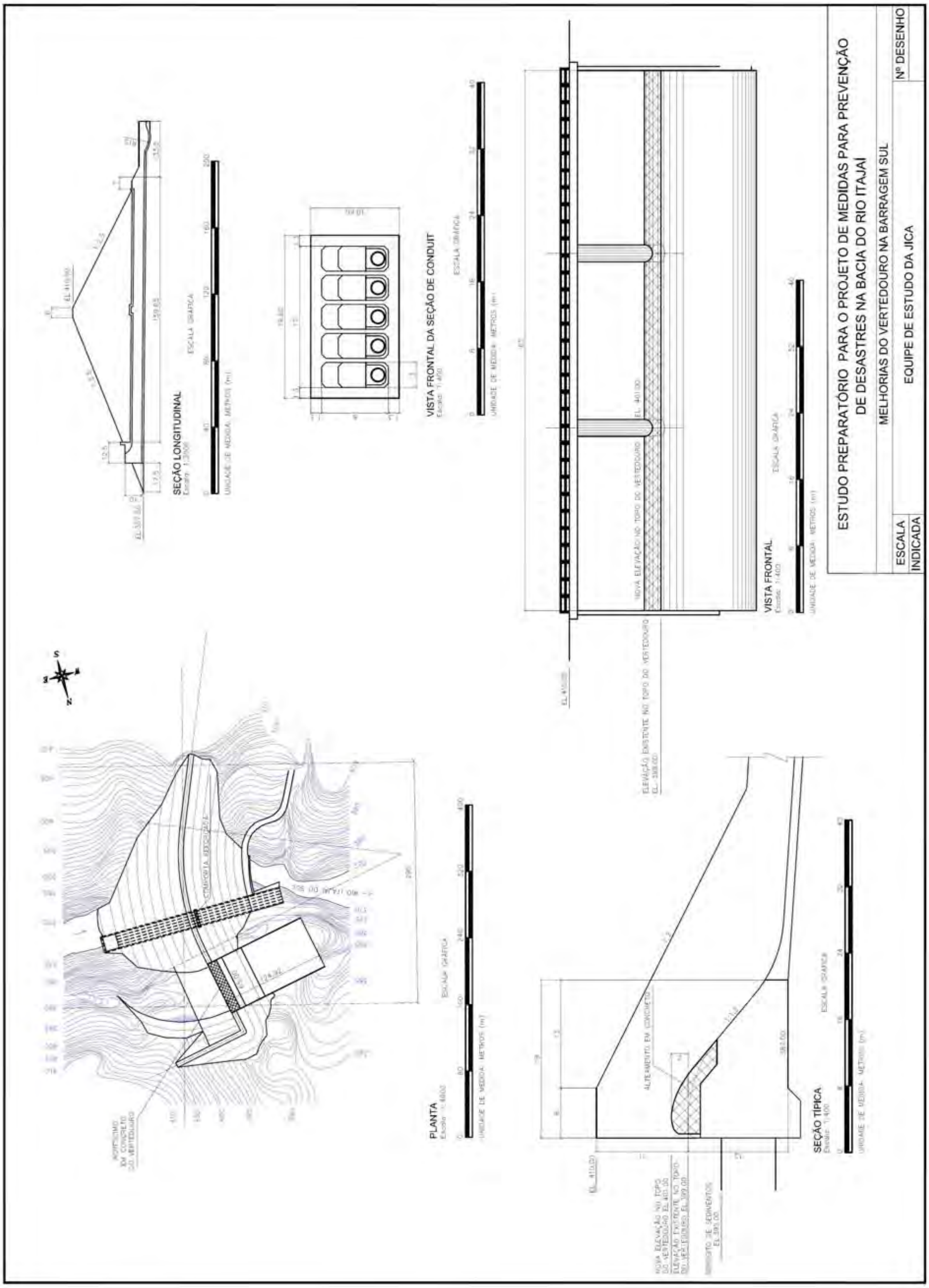


Figure 2.2.11 Drawing on Heightening of Sul Dam

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

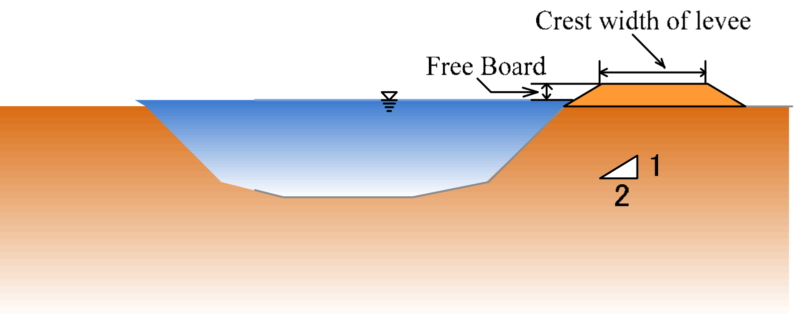
River / City		5 year	10 year	25 year	50 year
Itajai River	Itajai		Dyke (3)* (L=12,830m)	Dyke (3)* (L=12,830m)	
	Ilhota			Ring Dyke (3)* (L=8,000 m)	Ring Dyke (3)* (L=8,000 m)
	Blumenau				Dyke (3)* (L=15,800m)
	Rio do Sul			Channel Excavation (L=10,270m)	Dyke (2)* (L=4,500m)
Benedito River	Timbo			Channel 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.



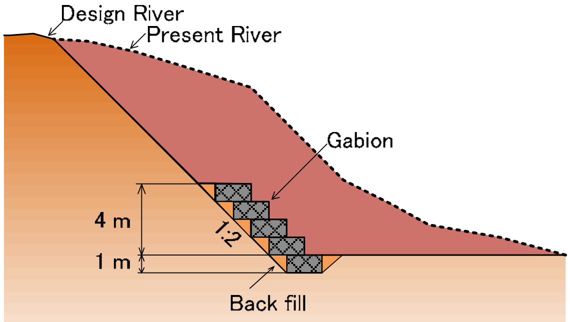
Category No.	Design Discharge (m3 s)	Free Board (m)	Crest width of levee (m)
1	200 ≤ Q < 500	0.8	3.0
2	500 ≤ Q < 2000	1.0	4.0
3	2000 ≤ Q < 5000	1.2	5.0

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.



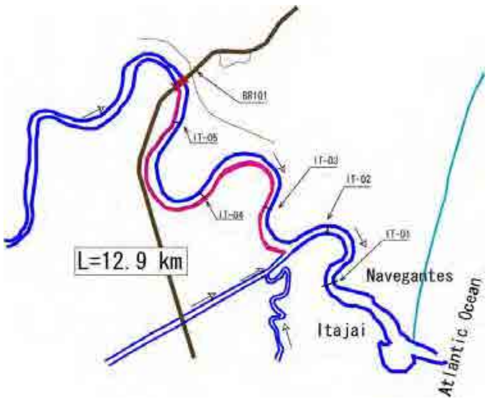
Source：JICA Survey Team

Figure 2.3.2 Design Conditions for Channel Excavation

(3) River Improvement Plan at Cities

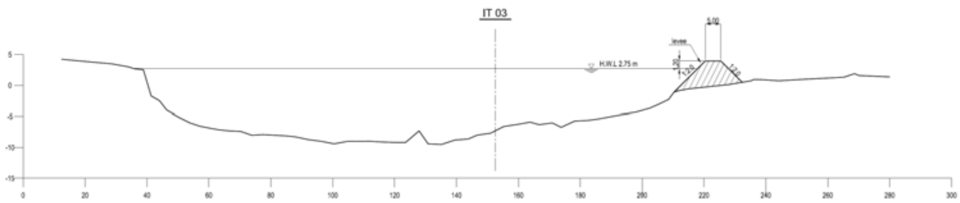
a) Itajai City, Itajai River

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

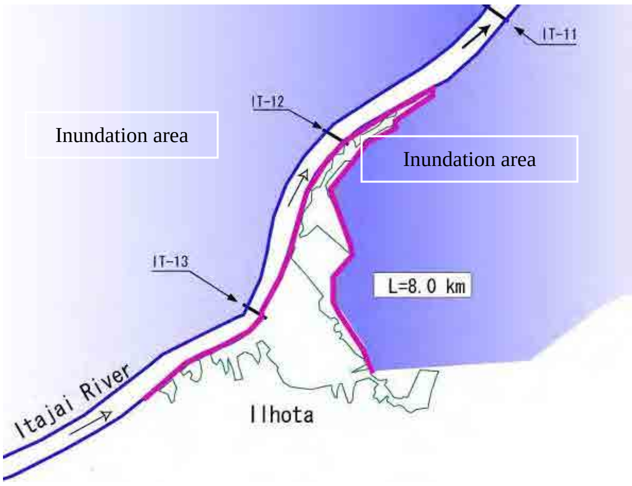


Source：JICA Survey Team

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

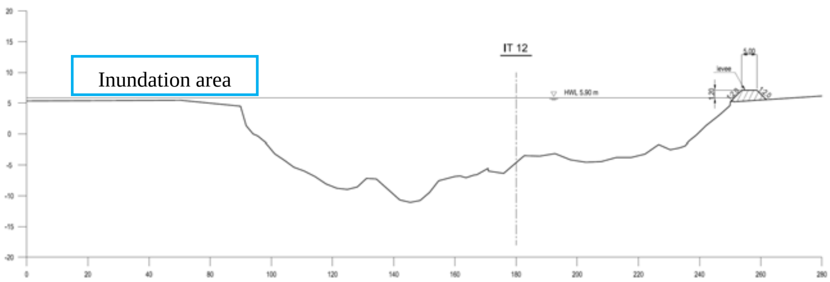
b) Ilhota City, Itajai River

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

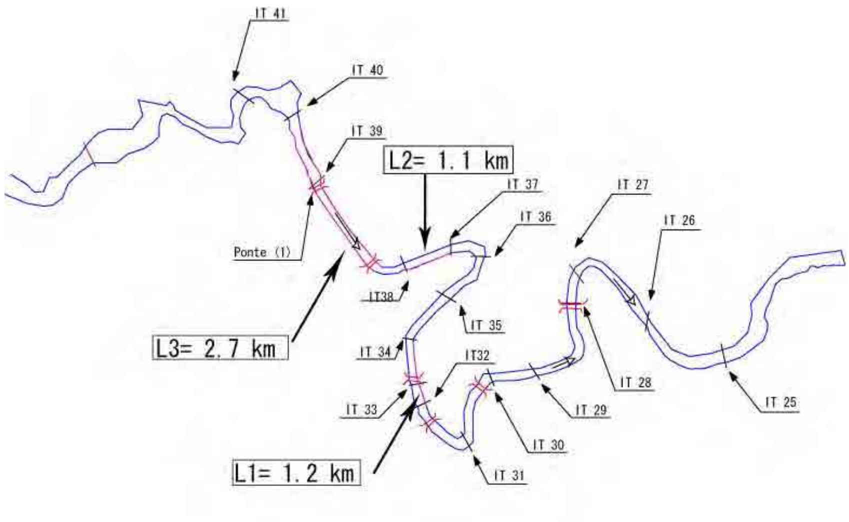


Source：JICA Survey Team

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

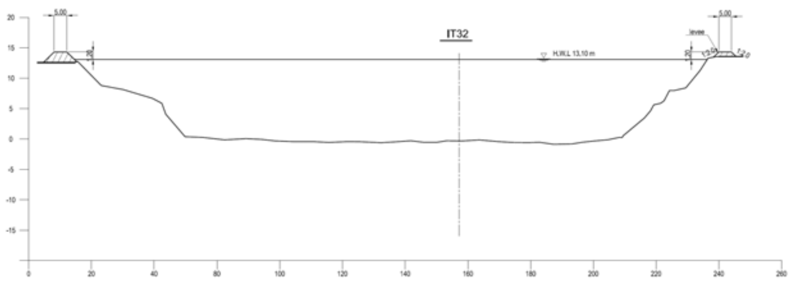
c) Blumenau City, Itajai River

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

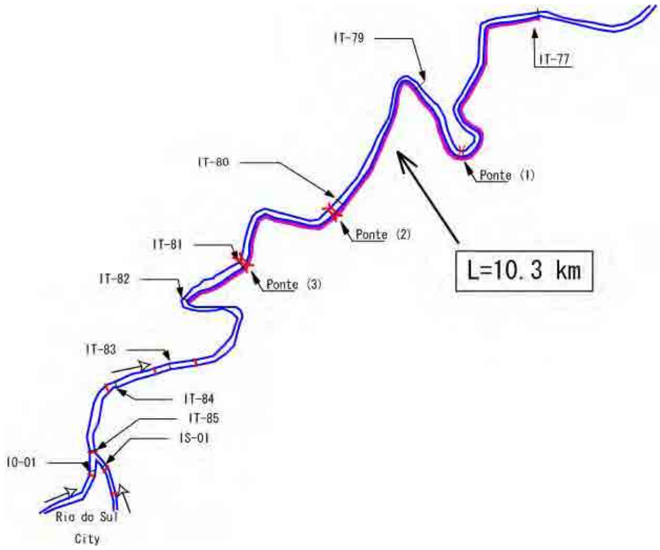
Figure 2.3.8 River Improvement Section in Blumenau City (Section IT-32, 50-year flood)

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.

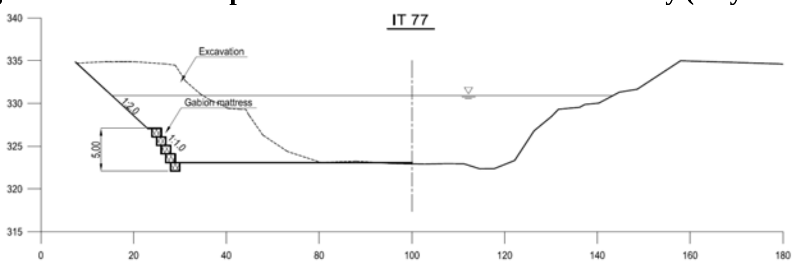
River improvement for the 25-year flood:

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)

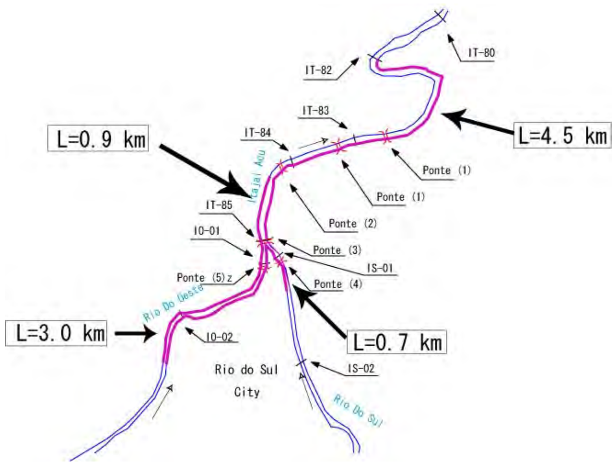


Source：JICA Survey Team

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

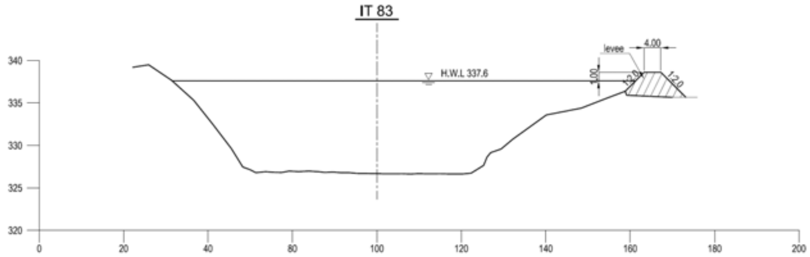
<u>River improvement for the 50-year flood:</u>

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)

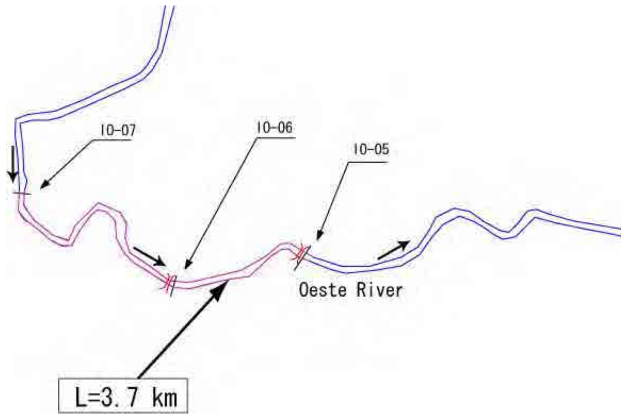


Source：JICA Survey Team

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

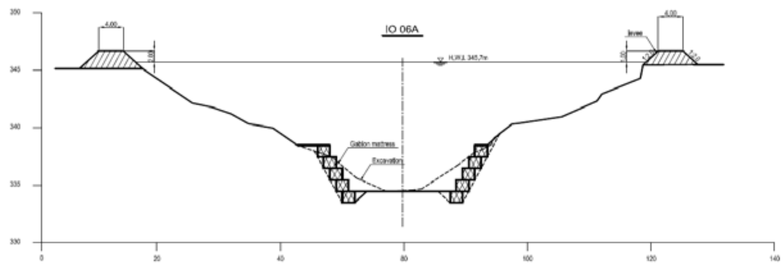
e) Taio City, Itajai do Oeste River

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

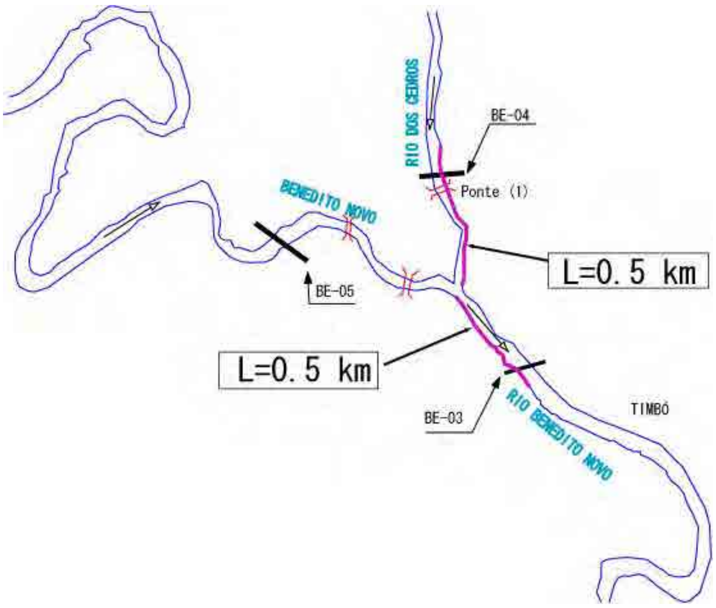


Source：JICA Survey Team

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

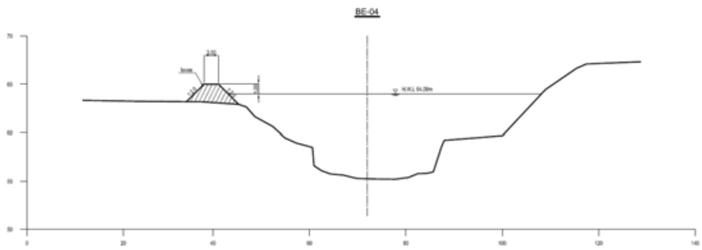
f) Timbo City, Cedros River

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)

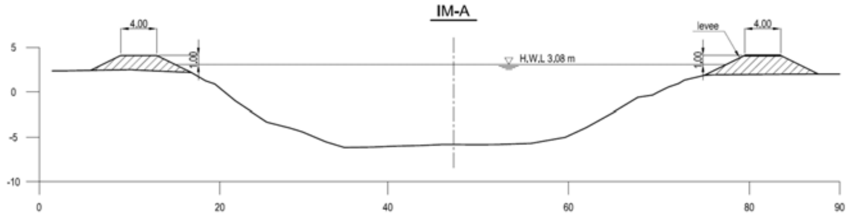
g) Itajai City, Itajai Mirim River

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



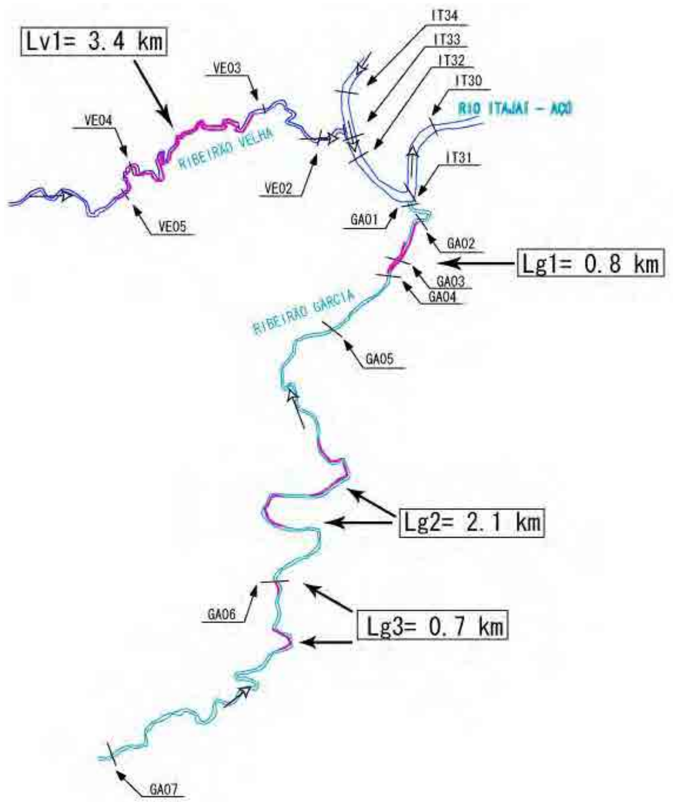
Source : JICA Survey Team

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

h) Urban Rivers in Blumenau City

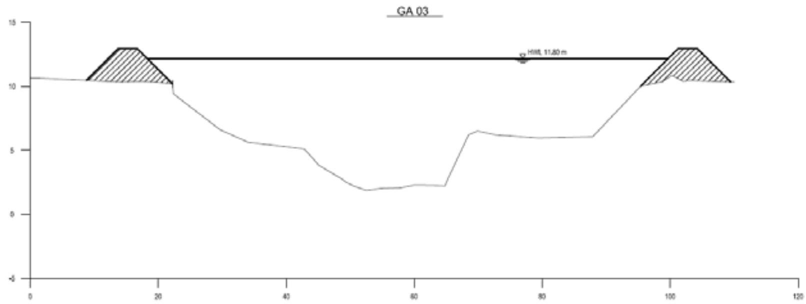
The Garcia River is seriously influenced by back water of the Itajai River in times of flood. Since the urban area along the Garcia River has been suffering from habitual flooding due to the back water effect, this area is planned to be protected by embankment against the 25-year flood. The stretches 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.



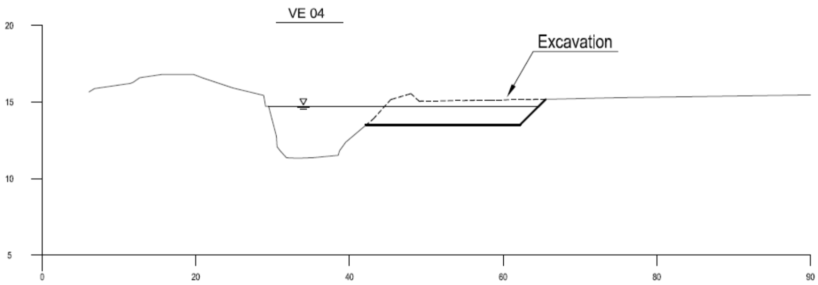
Source：JICA Survey Team

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)

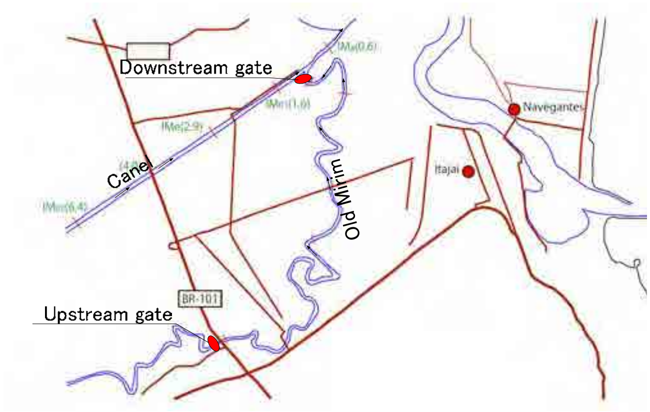


Source：JICA Survey Team

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.



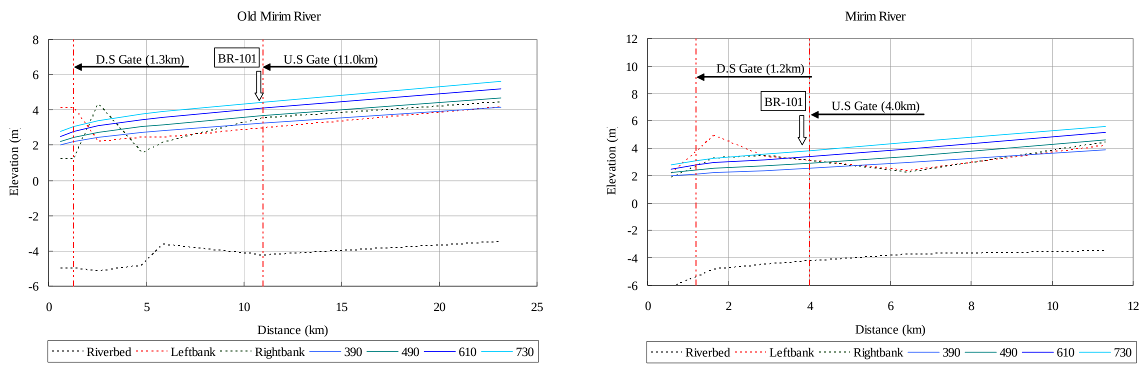
Source : JICA Survey Team

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

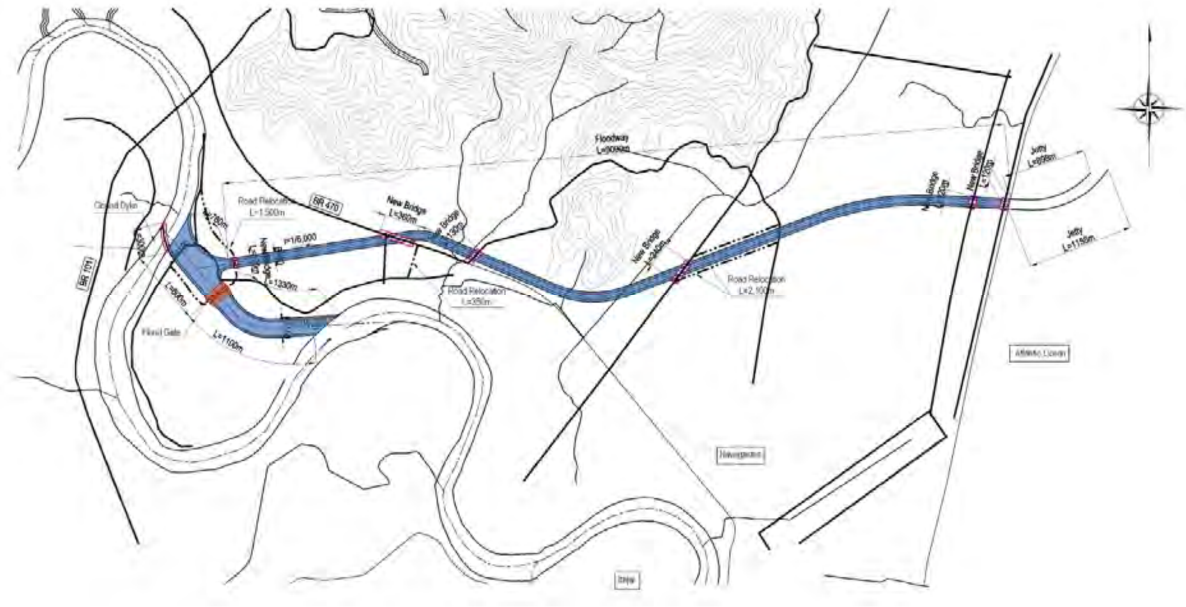
The design floodgates is required to closed in times of flood and after flood it is opened to drainage the own-basin discharge quickly. In addition, since the floodgates are under constant influence of the tide level, it is designed that the crest of floodgate is EL-0.50 m (=minimum tide level: -0.5 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

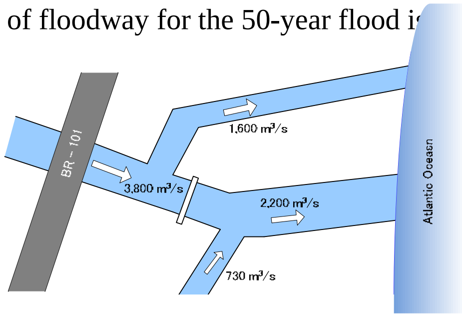
Table below presents the general features of the planned floodway and diversion weir.

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.

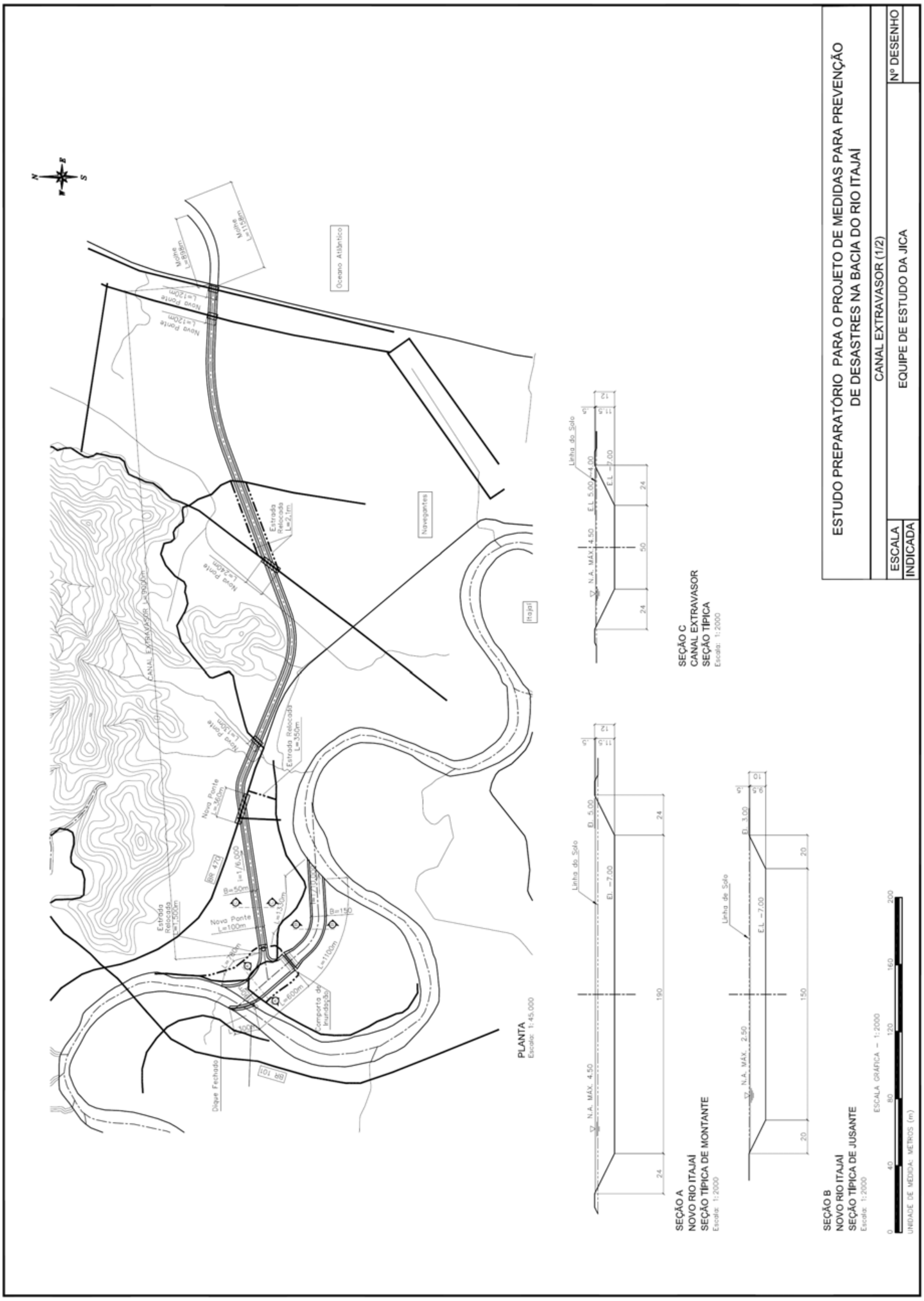


Figure 2.3.26 Structural Drawing of Water Gate on the Old Mirim River

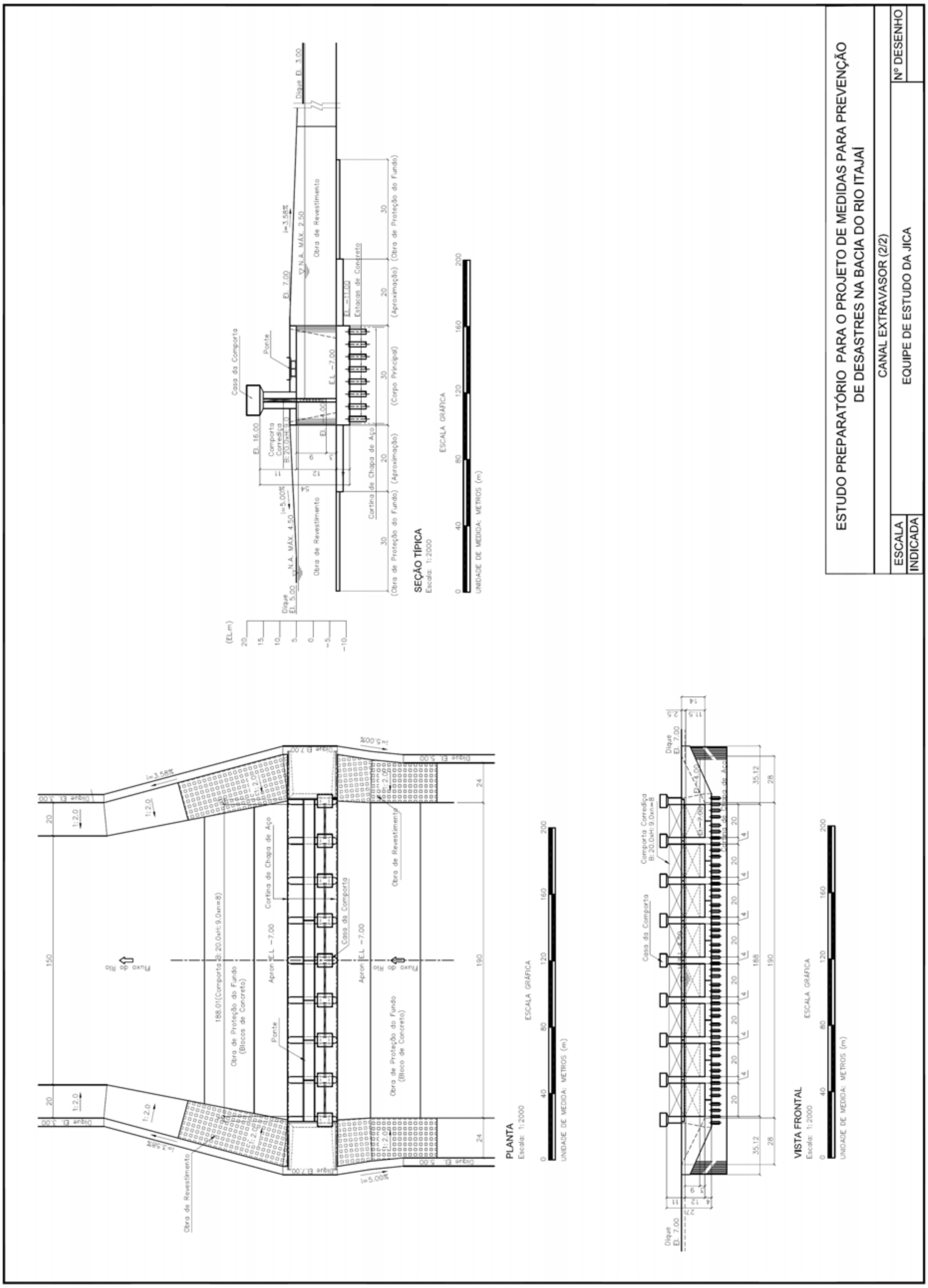
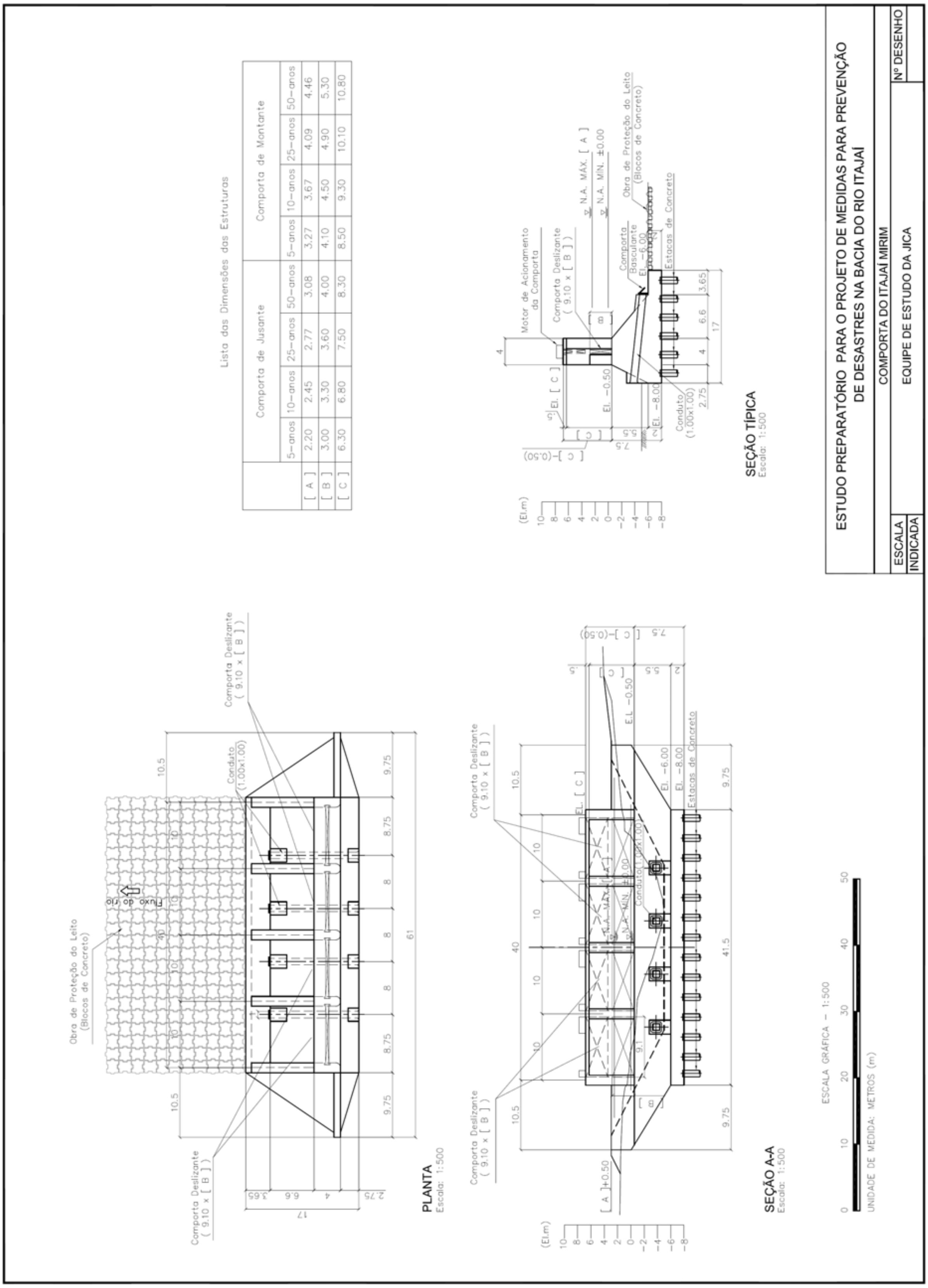


Figure 2.3.27 Structural Drawing of Floodway



Lista das Dimensões das Estruturas

	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

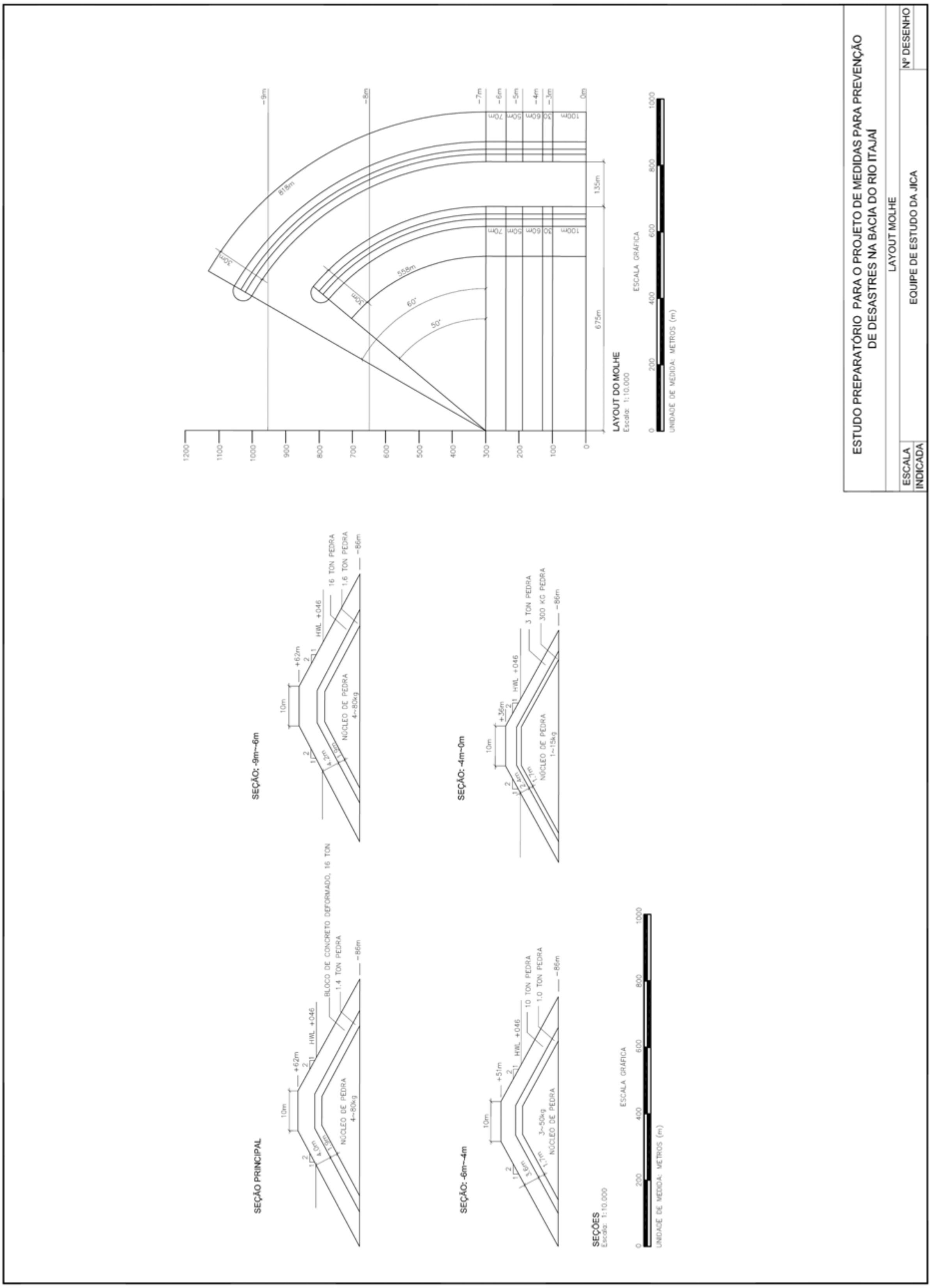


Figure 2.3.29 Layout of Jetty

(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

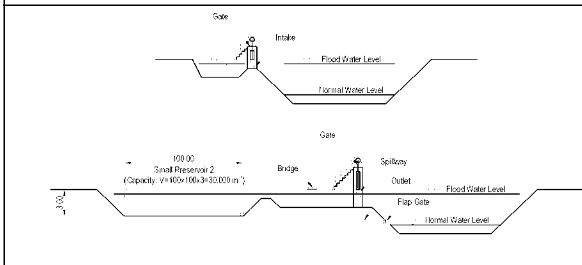
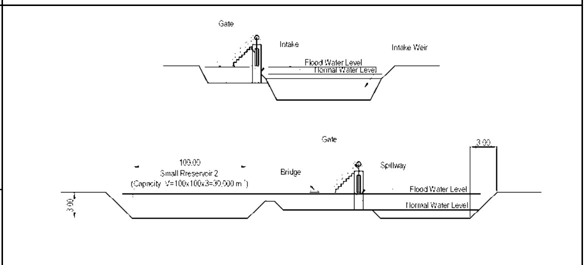
	5-year	**10-year**	**25-year**	**50-year**
nos	2	5	7	7

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).

FREE INTAKE	WEIR INTAKE
	
- 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.	- 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.

Source: : JICA Survey Team

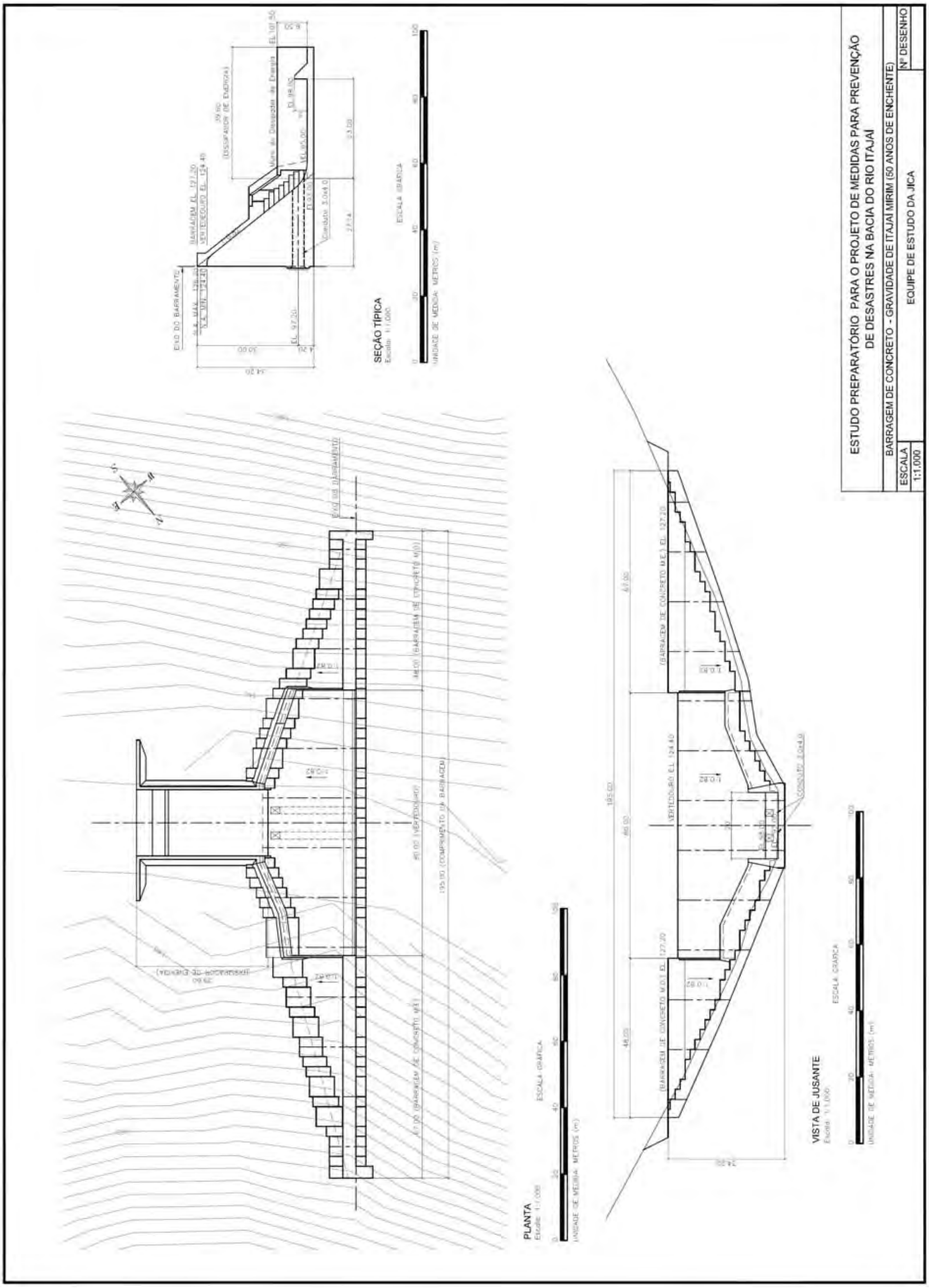


Figure 2.3.30 Utilization for Agriculture's small dam

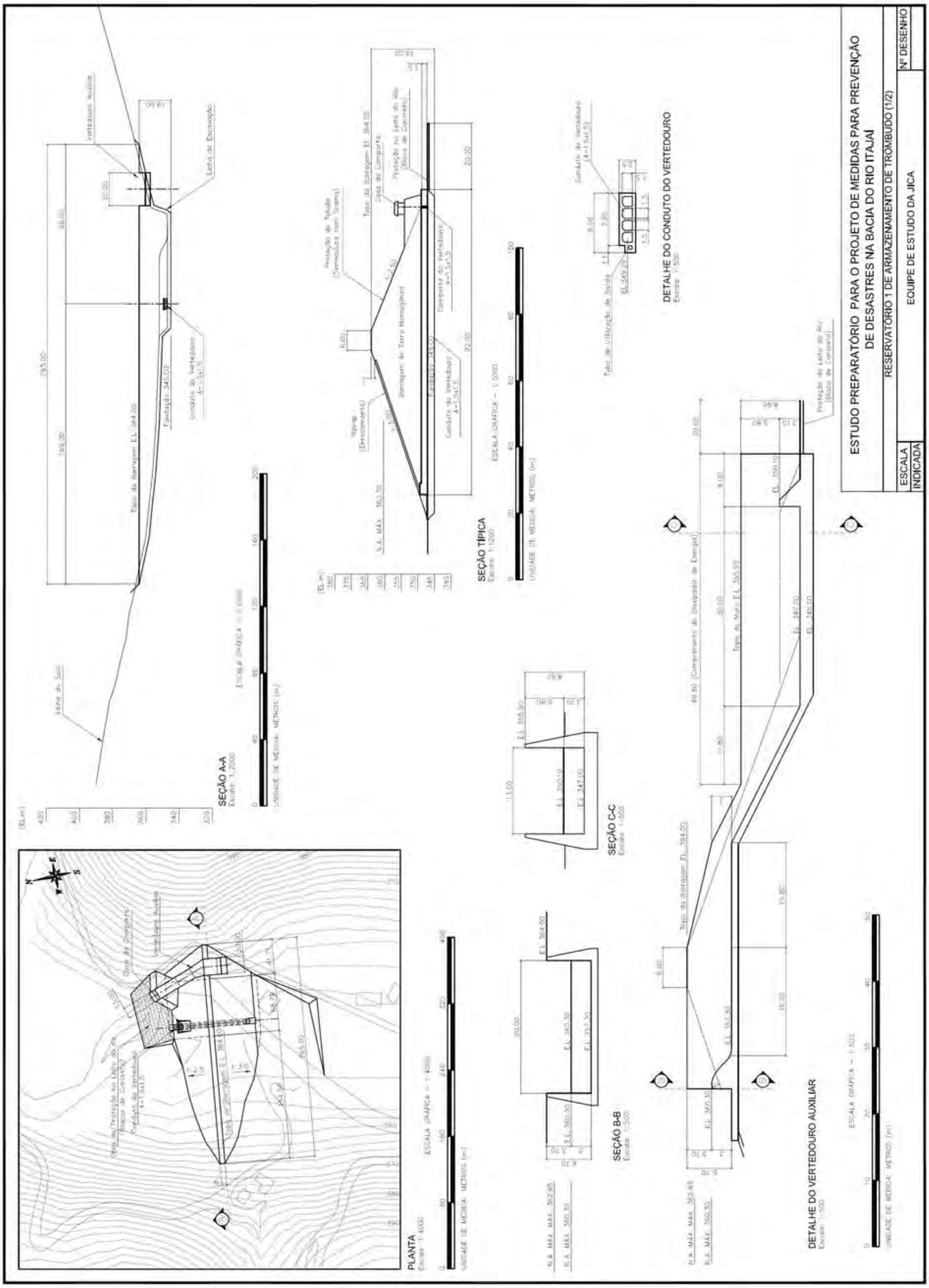


Figure 2.3.31 Structural Drawing of Small Dam (Site-1 on Trombudo River)

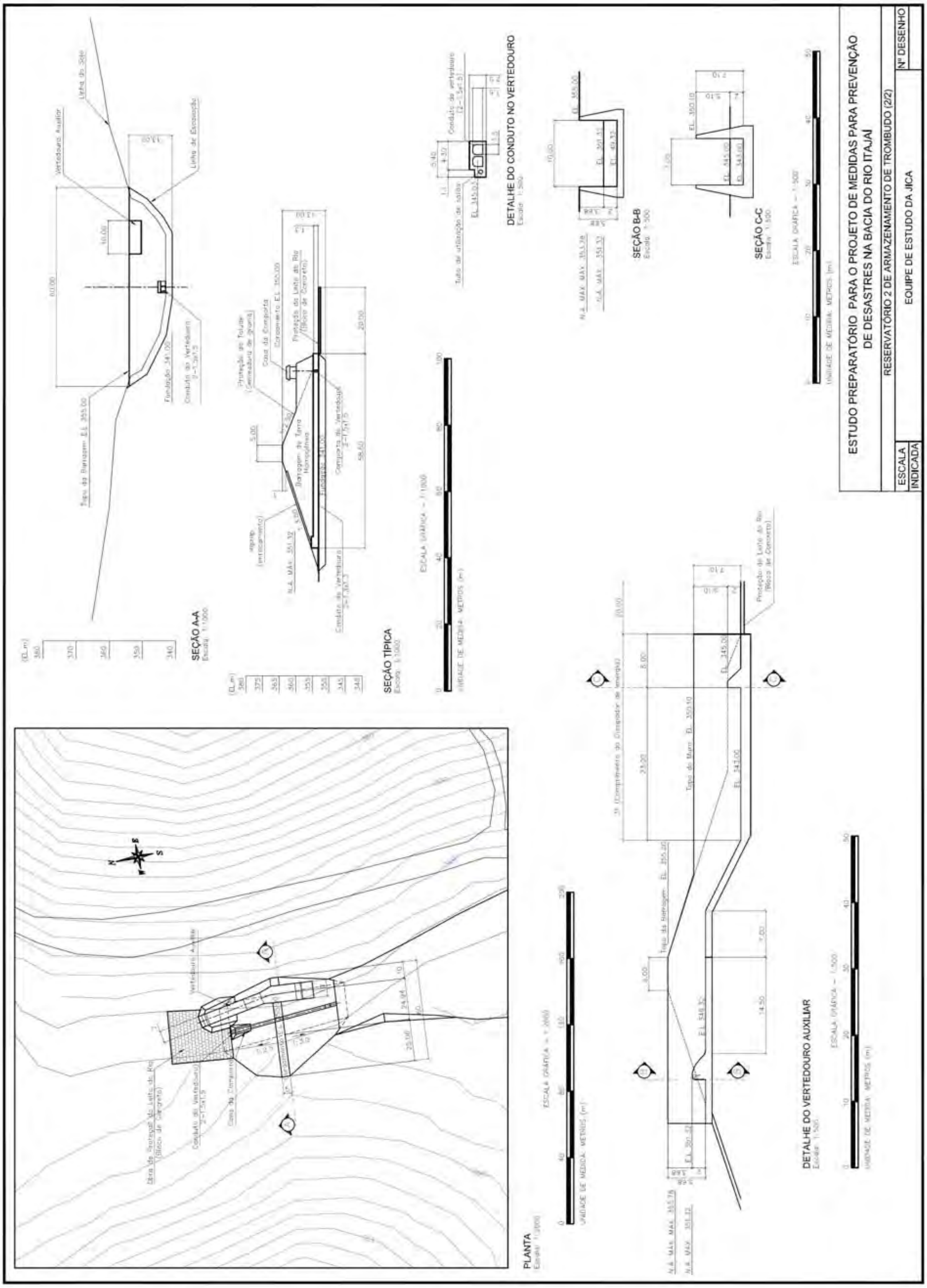


Figure 2.3.32 Structural Drawing of Small Dam (Site-1 on Trombudo River)

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

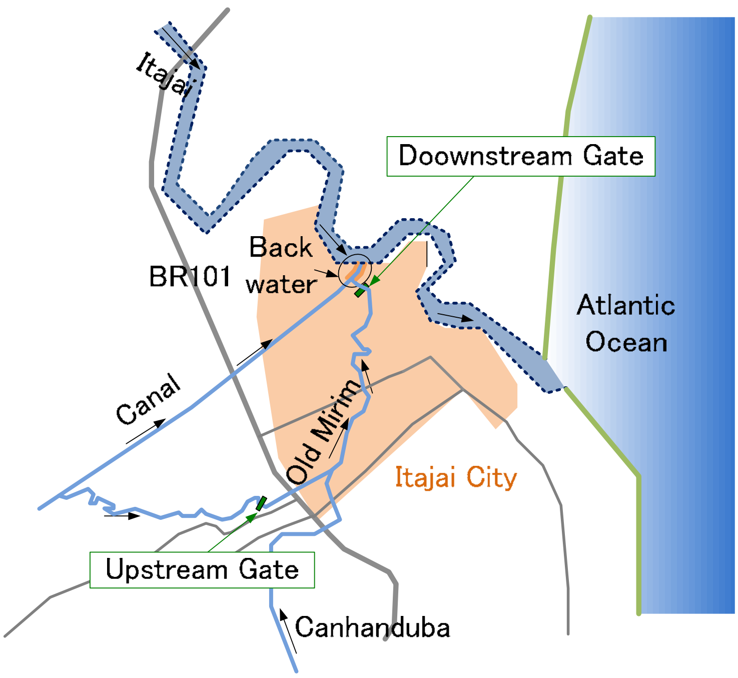
Two(2) floodgates are proposed to be installed in the Old Mirim River as shown in Figure 4.1.1 below. The floodgates located downstream and upstream are called, in this report, “downstream floodgate” and “upstream floodgate” respectively.

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

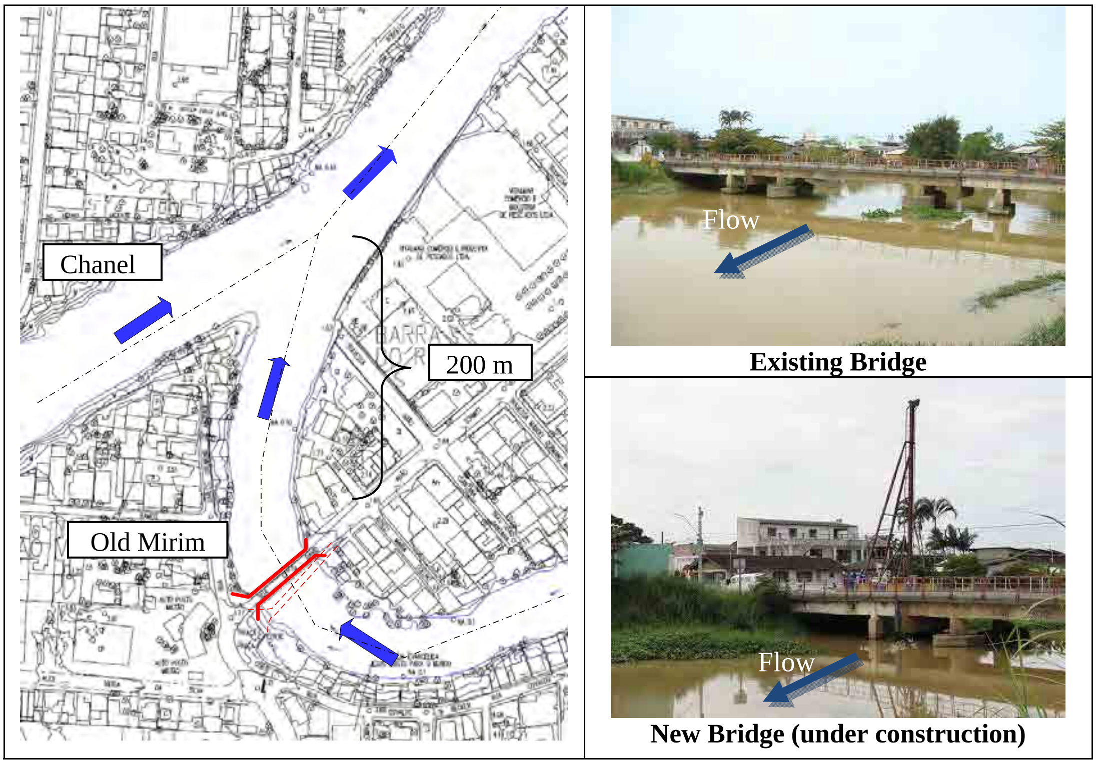
Figure 4.1.1 Location Map

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.

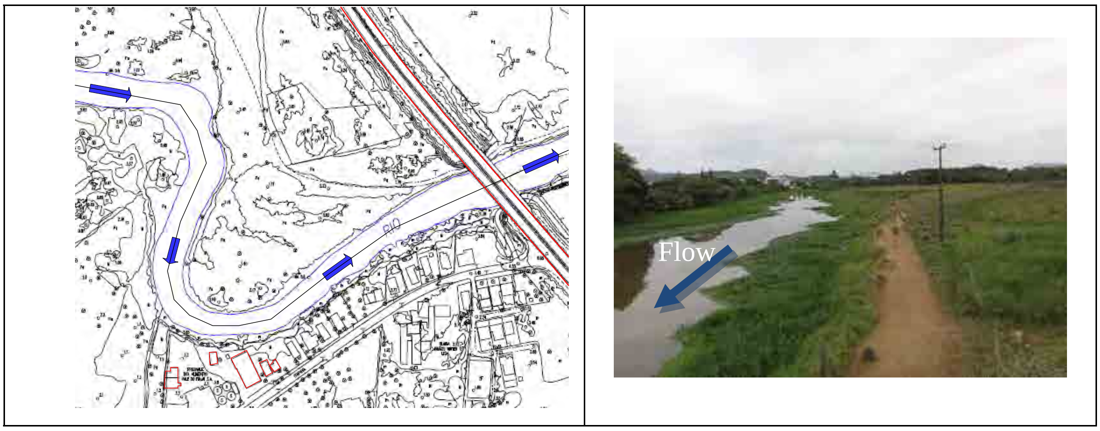


Source: JICA Survey Team

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

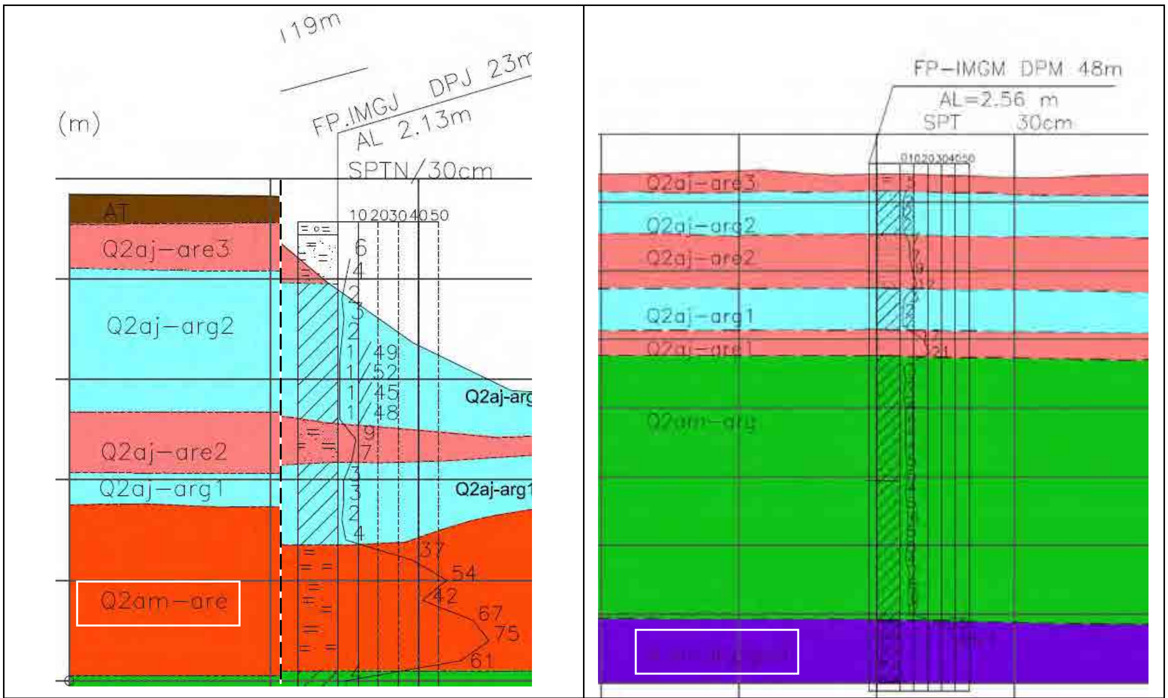
4.2.2 Geological

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

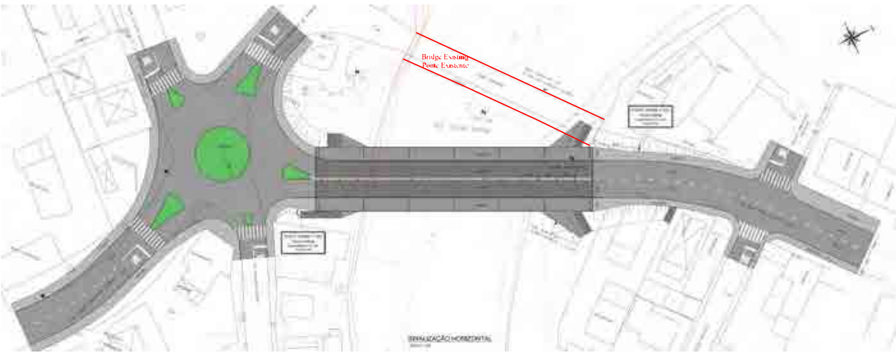
4.2.3 Environment and neighboring structure

(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

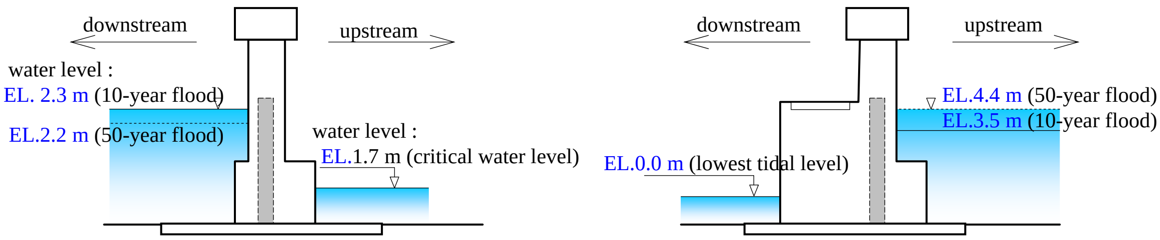
The access road to the site is available from BR101, so that it is not difficult for vehicles to transport. Also there are no residences around the planning site and the site for temporary diversion facility. So it is not necessary to consider the neighbors so far.

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.



(a) downstream gate (b) upstream gate

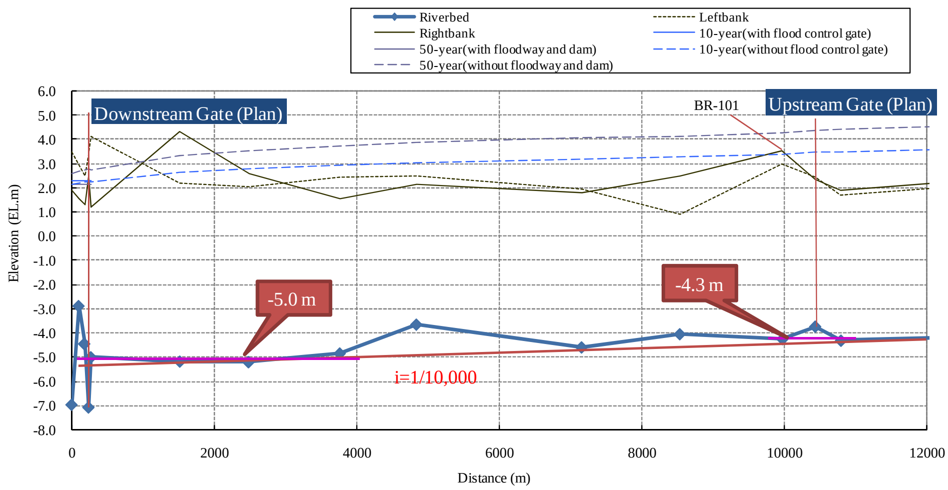
Source: JICA Survey Team

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).

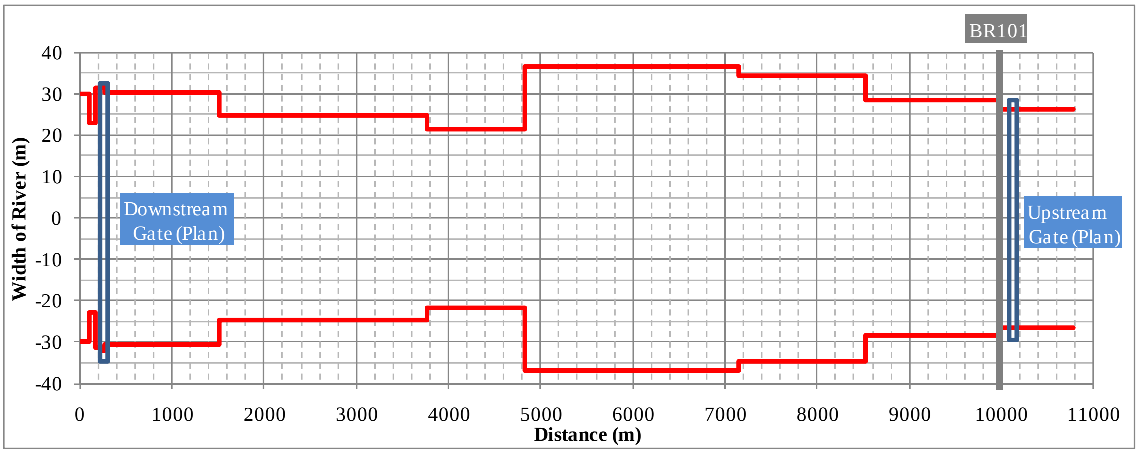


Source: JICA Survey Team

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

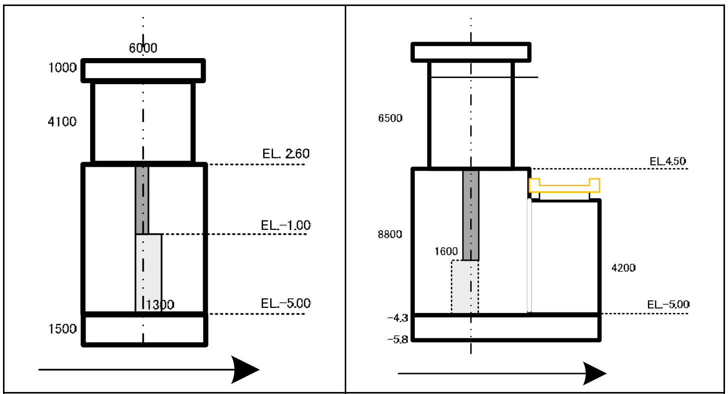
Main features of the designed floodgates are summarized in Table 4.4.1 below.

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

<u>Downstream</u>

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.

<u>Upstream</u>

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

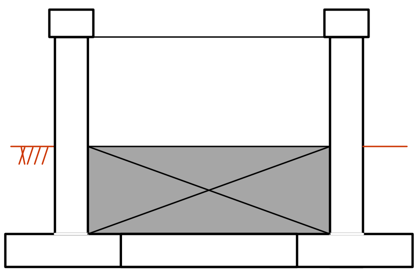


Figure 4.4.2 Image of Separate Type of Gate

(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

Downstream

6.00 m

Upstream

11.20 m

3) Length

Downstream

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

Upstream

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.

Downstream Gate

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

Upstream Gate

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	$\lvert e \rvert \leq \frac{B}{6}$, B = base width	at normal condition
Construction	Fs=1.2	$\lvert e \rvert \leq \frac{B}{3}$, 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

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

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

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

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

<u>Upstream</u>

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 or 149.22 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 or 193.86 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

$e = \dfrac{\sum N}{B \cdot L}\left(1 \pm \dfrac{6 \cdot e}{B}\right) = \dfrac{8633.83}{11.20 \cdot 4.50}\left(1 \pm \dfrac{6 \cdot 0.31}{11.20}\right) = 171.31 \pm 28.45$ = 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

$e = \dfrac{\sum N}{B \cdot L}\left(1 \pm \dfrac{6 \cdot e}{B}\right) = \dfrac{4037.6}{11.20 \cdot 4.50}\left(1 \pm \dfrac{6 \cdot -0.56}{11.20}\right) = 80.11 \pm 24.03$ = 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

$e = \dfrac{\sum N}{B \cdot L}\left(1 \pm \dfrac{6 \cdot e}{B}\right) = \dfrac{7886.69}{11.20 \cdot 4.50}\left(1 \pm \dfrac{6 \cdot -0.12}{11.20}\right) = 156.48 \pm 10.06$ = 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.

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

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)

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

(2) Calculation Result

The calculation sheets were shown below.

<u>Downstream</u>

The allowable bearing capacity is 627.98 kN/nos. as ϕ 400, and 369.67 kN/nos as ϕ 300. The length of piles is 11.0 .m.

The required number of sheet pile – ϕ 400 is more than 10 nos for pier.

The required number of sheet pile – ϕ 300 is more than12 nos for slab.

<u>Upstream</u>

The allowable bearing capacity is 588.94 kN/nos. as ϕ 400, and 359.24 kN/nos. as ϕ 300. 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

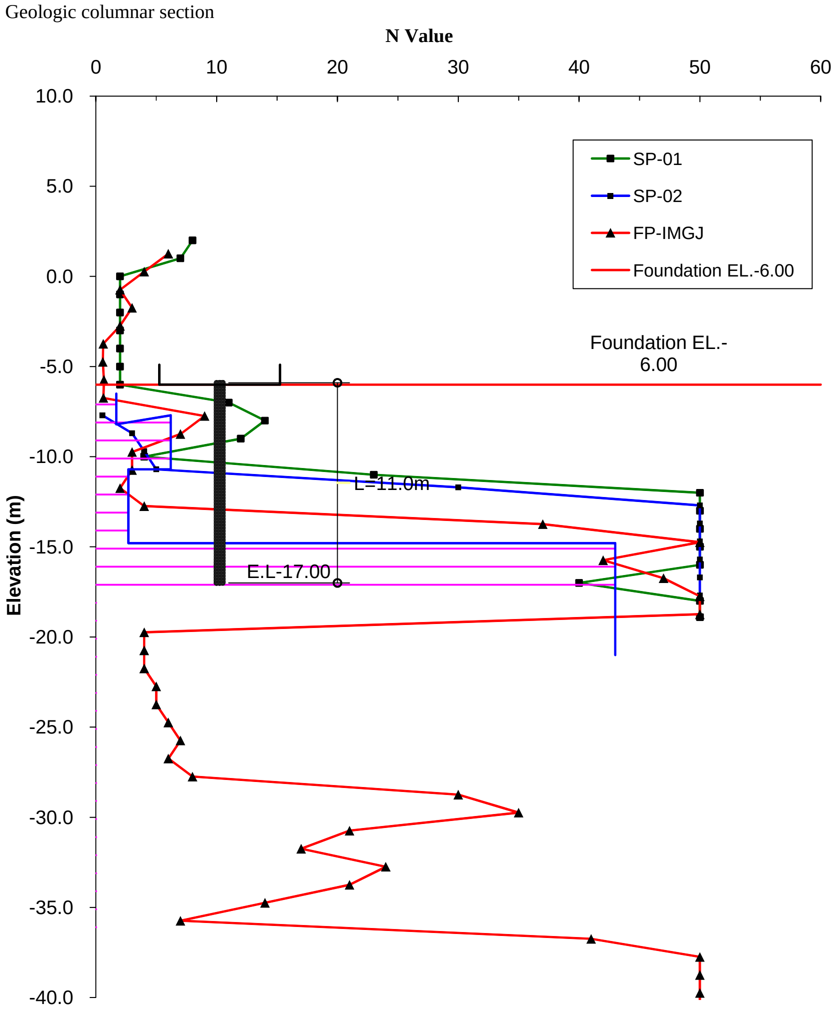
Data :			
	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ300	(mm)
	Thickness	60	(mm)

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)

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		
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

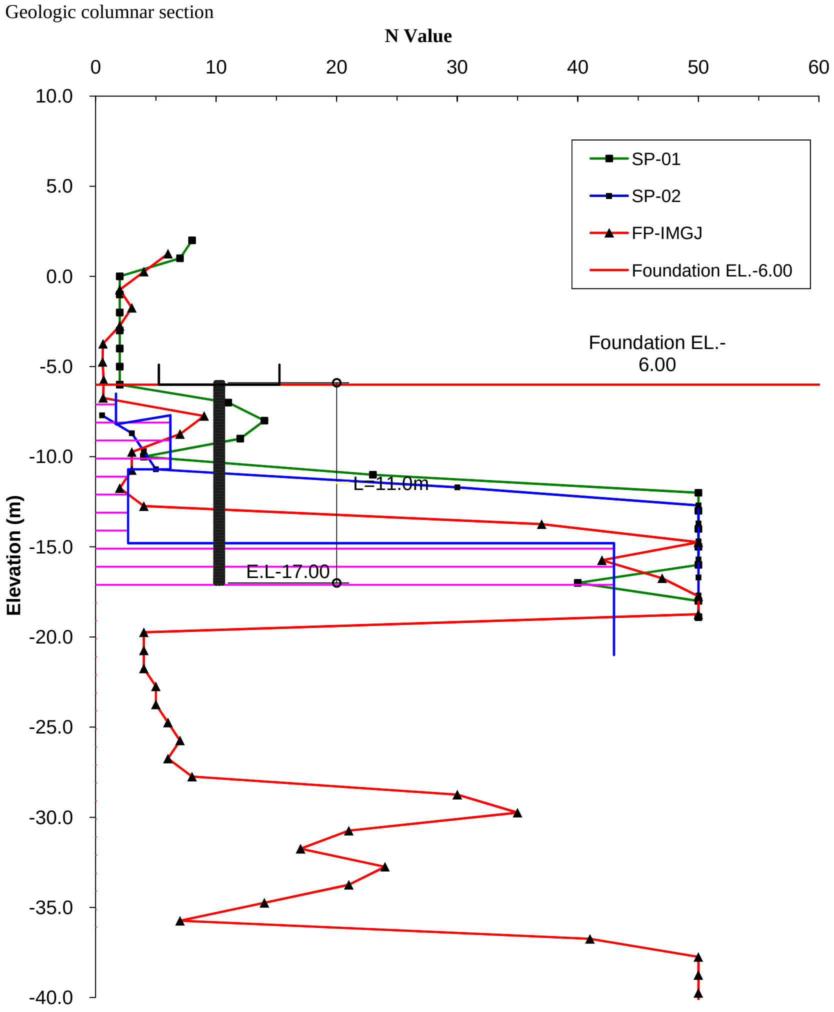
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ400	(mm)
	Thickness	75	(mm)

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

Under normal condition ,Under flood condition

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

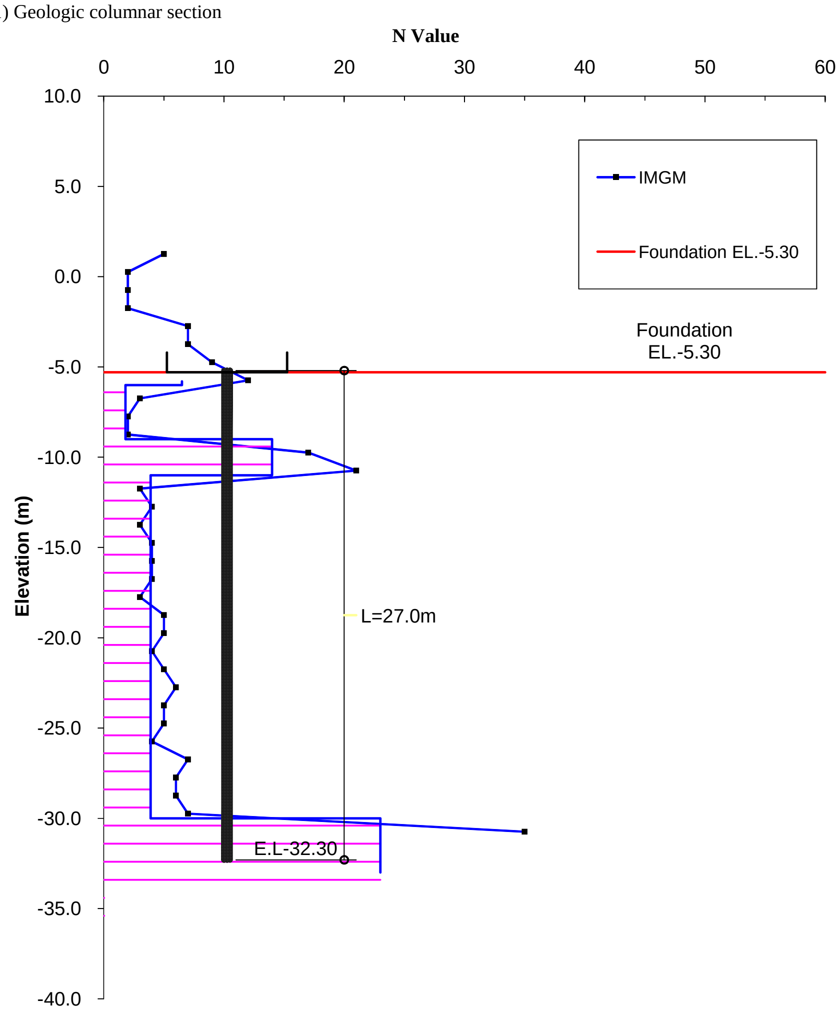
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ300	(mm)
	Thickness	60	(mm)

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

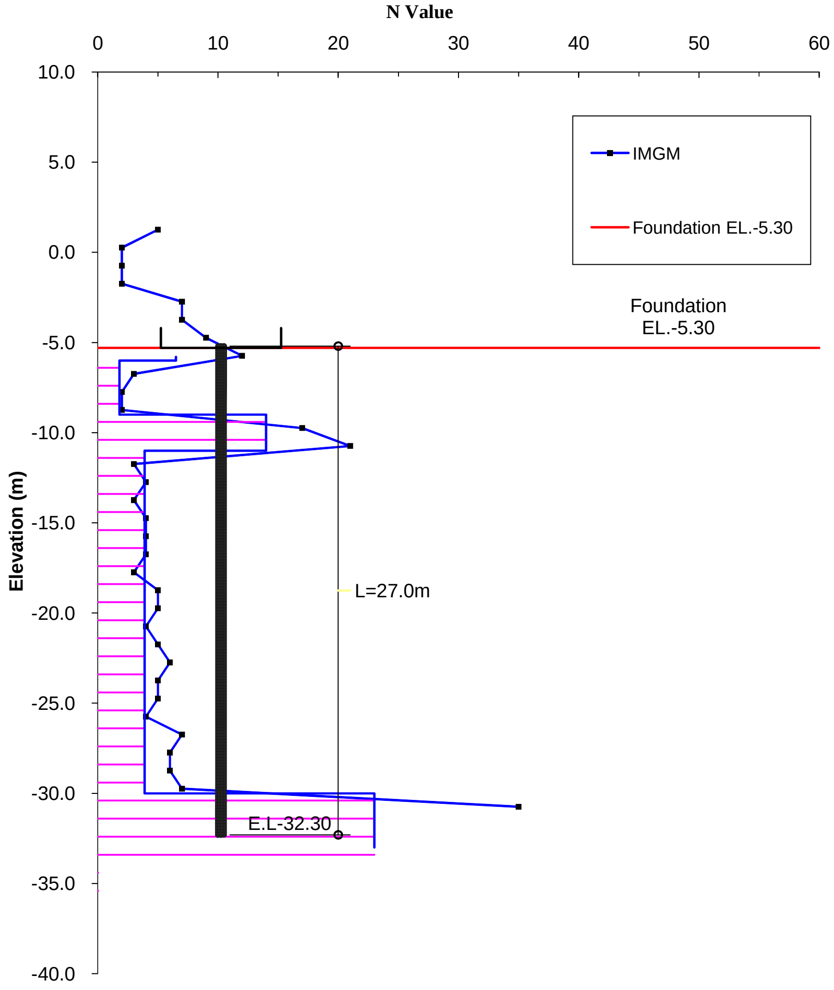
Data :	Pile type	PC pile	
	Condition of Tip of Pile	Rigid	
	Diameter	Φ400	(mm)
	Thickness	75	(mm)

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)

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		
---	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

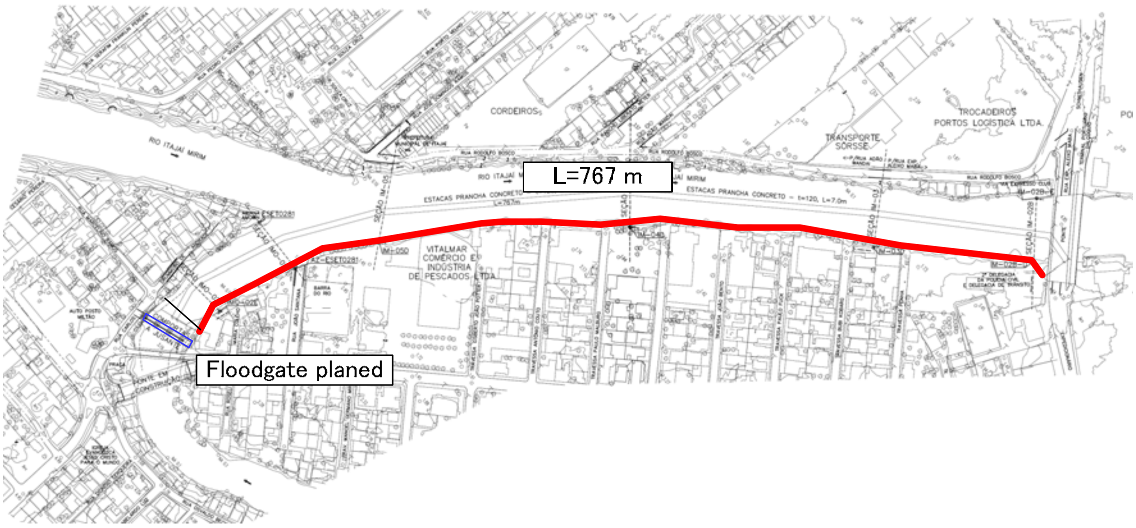
Around 1.0 km long river stretch along the Itajai Mirim River between the confluence with the Itajai River and the downstream floodgate is subject to backwater effect of the Itajai River. The river cross section survey along the Itajai Mirim River has revealed the following findings (see Figure 4.1.6):

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.

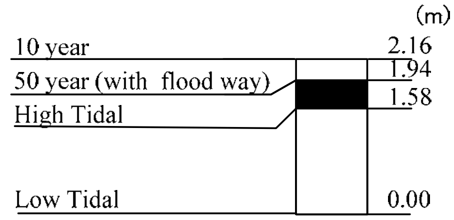


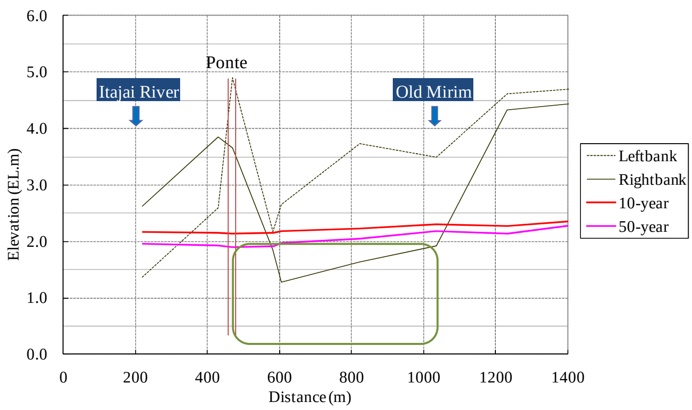
Figure 4.5.2 Water Lever Condition at Downstream

(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

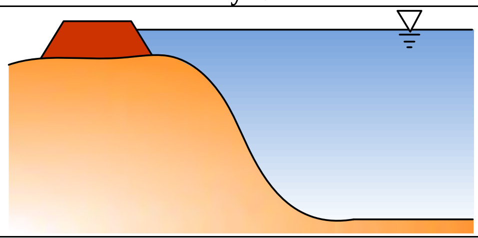
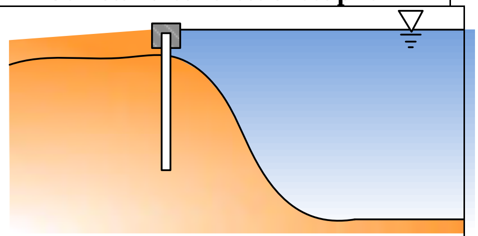
(1) Design condition

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.

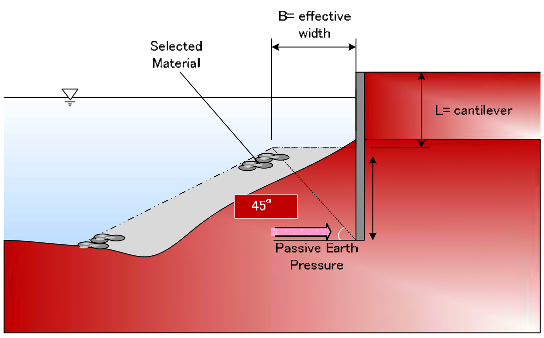
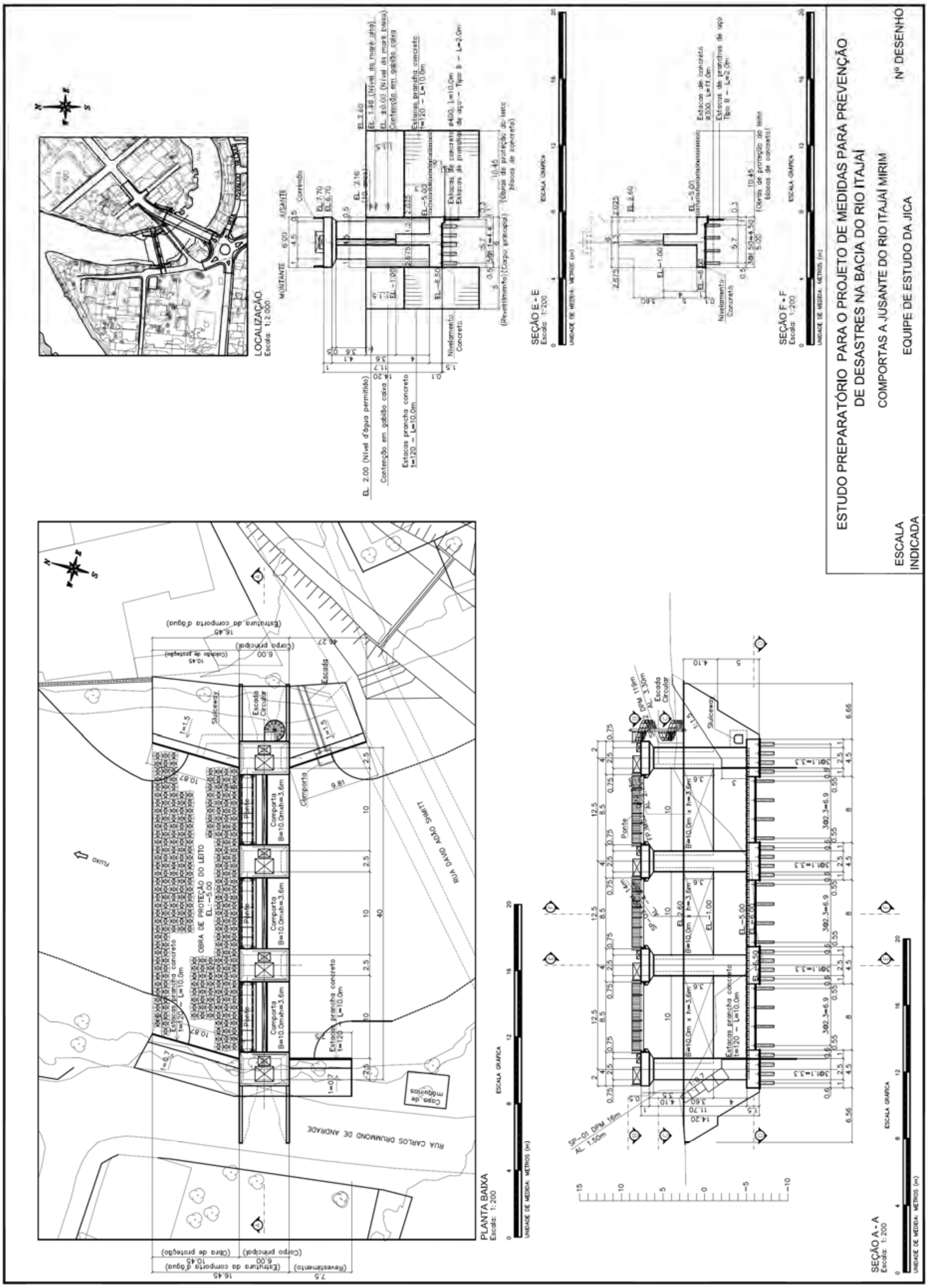


Figure 4.5.4 Water Level of longitudinal Profile



Figur 4.5.5 Downstream Floodgate in Itajai Mirim (1)

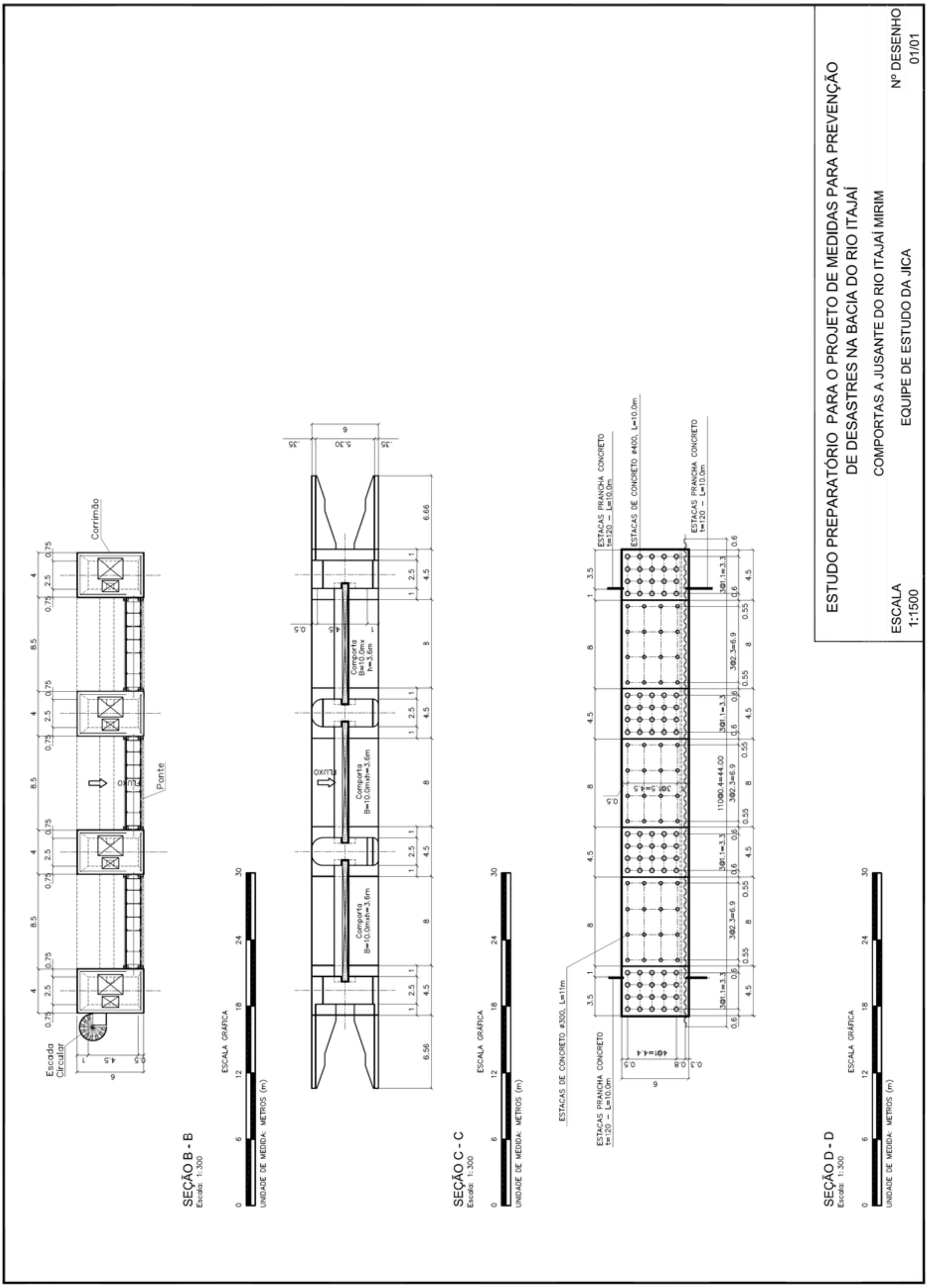


Figure 4.5.6 Downstream Floodgate in Itajai Mirim (2)

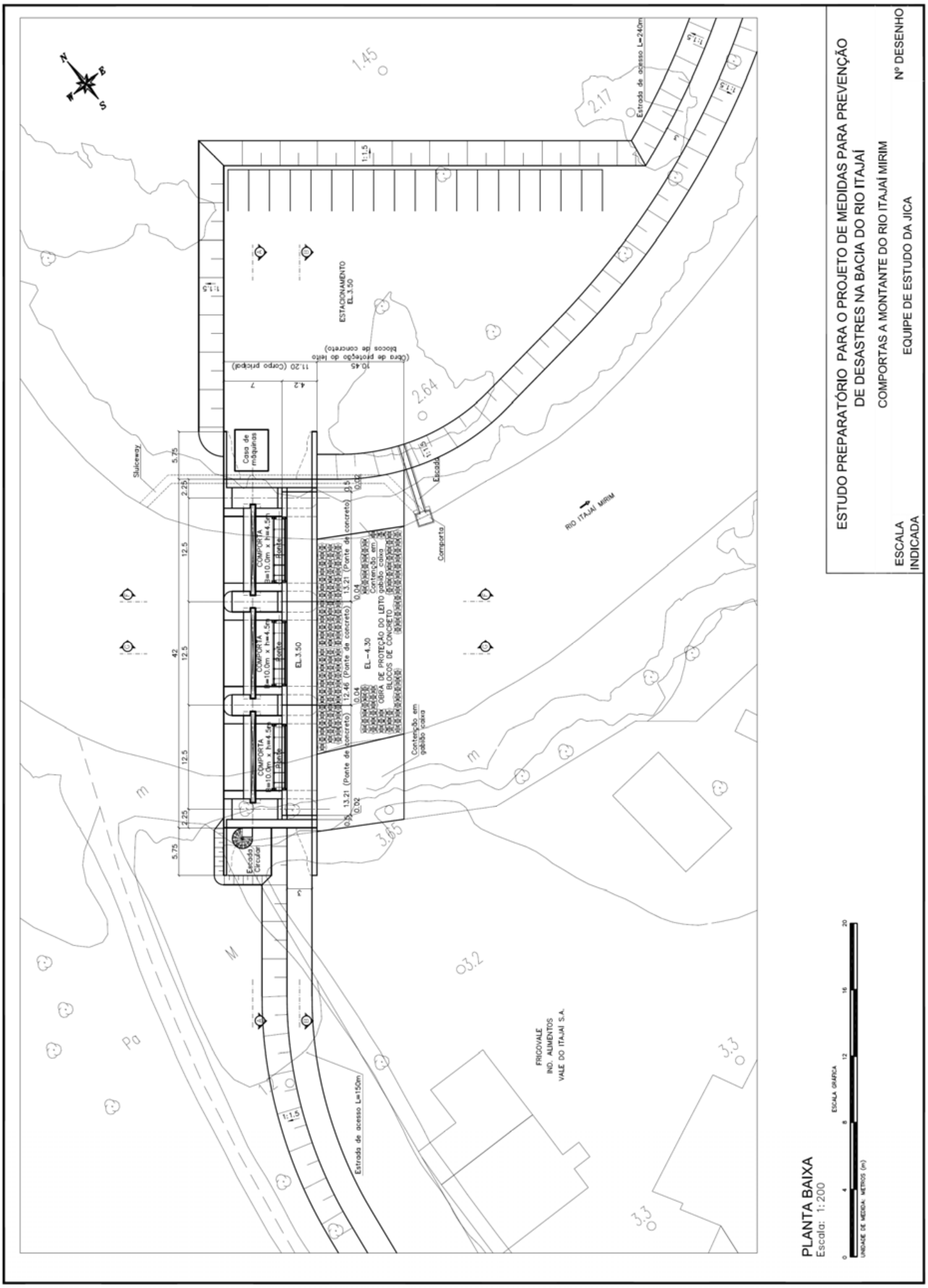


Figure 4.5.7 Upstream Floodgate in Itajai Mirim (1)

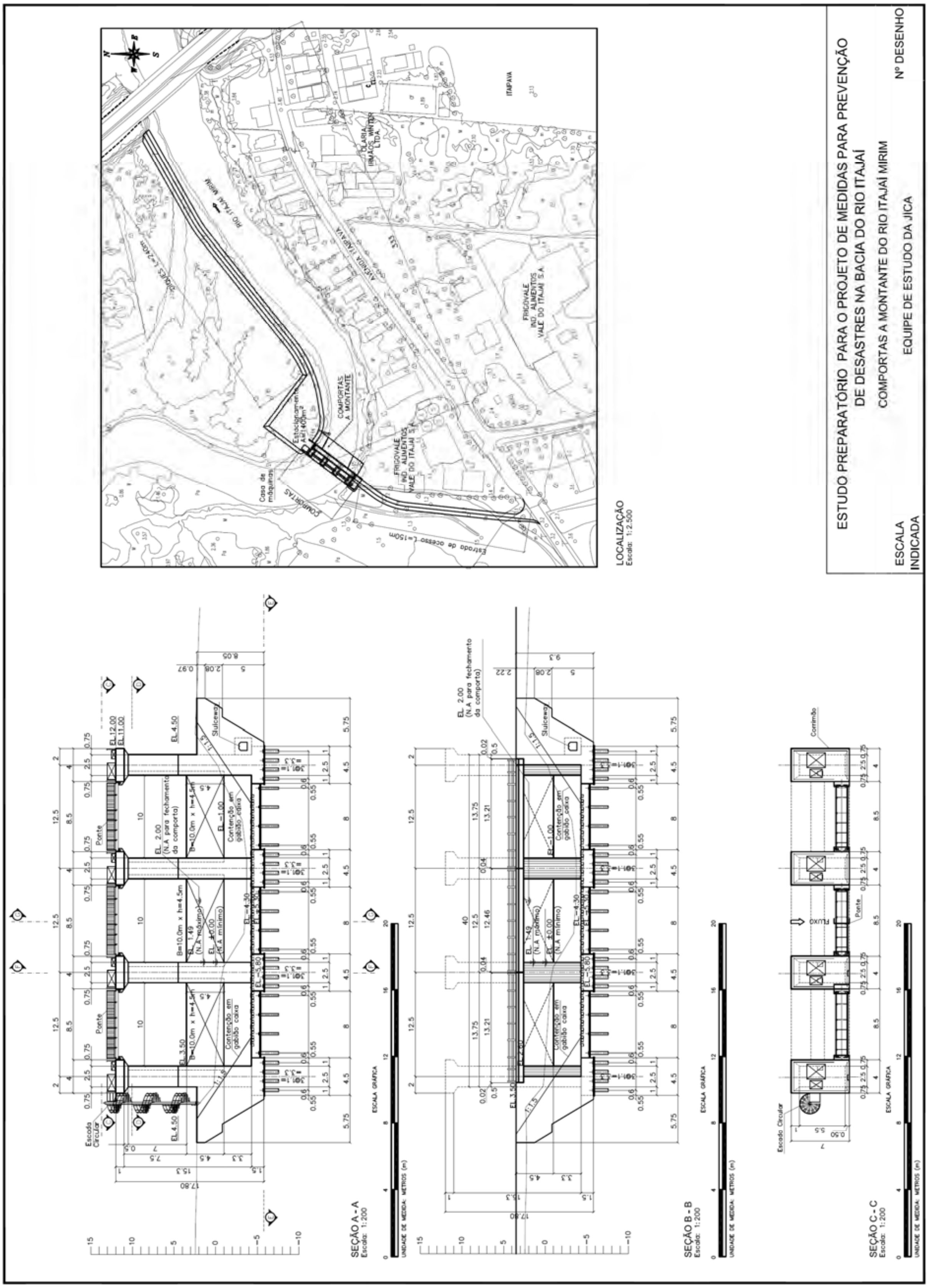


Figure 4.5.8 Upstream Floodgate in Itajai Mirim (2)

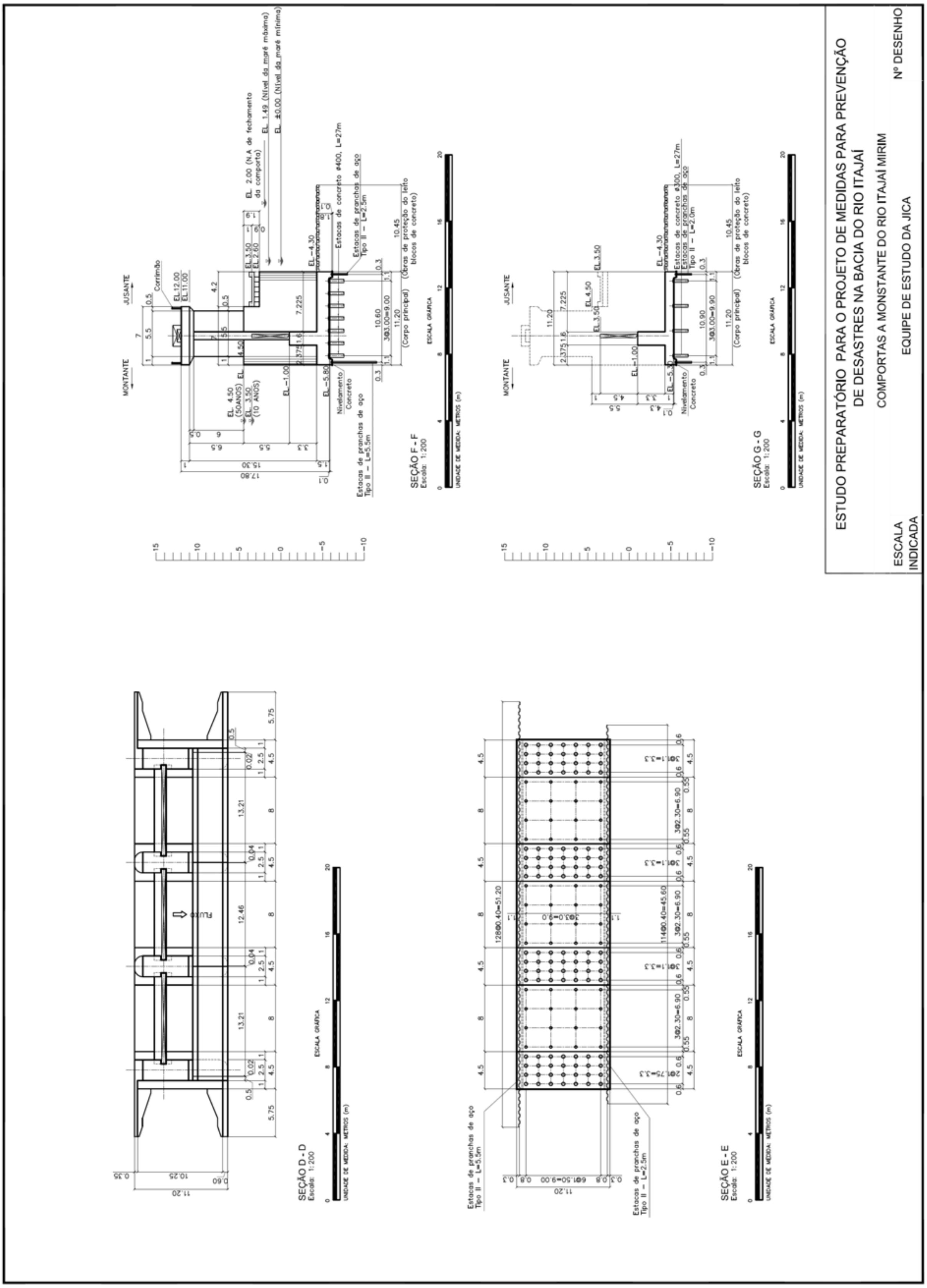


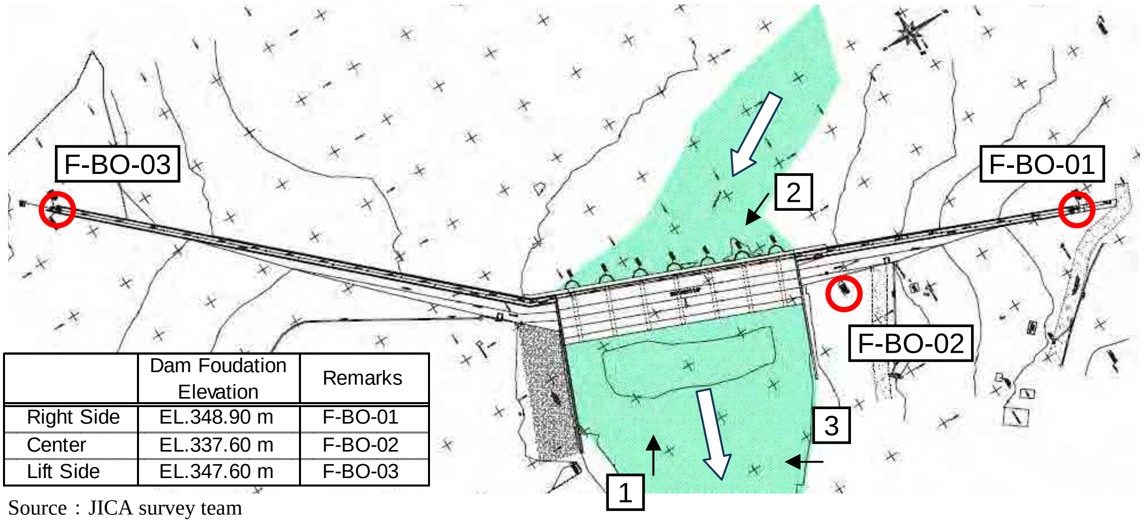
Figure 4.5.9 Upstream Floodgate in Itajai Mirim (3)

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

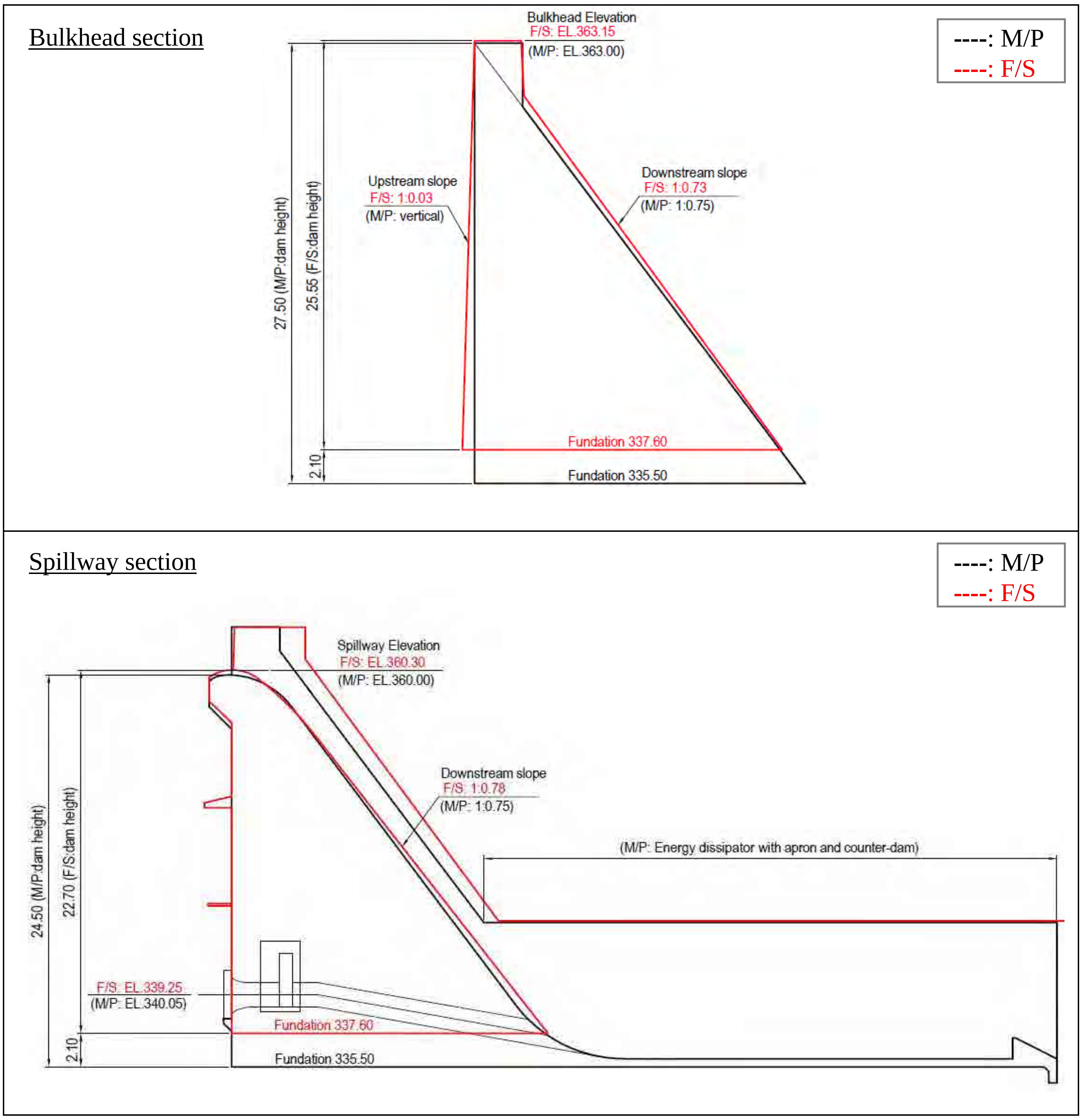
Photo Dam Site of Oeste Dam

(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

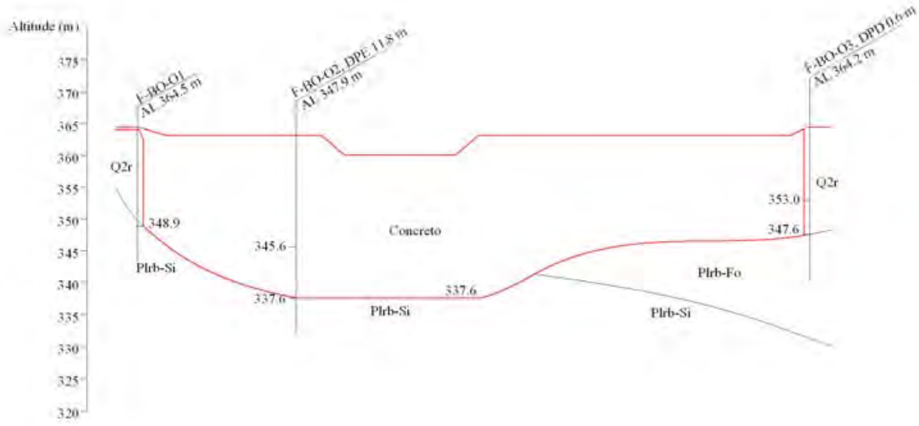
The geological property of the foundation of the Oeste dam is detailed at Annex C. The table on the right is about the geological condition.

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

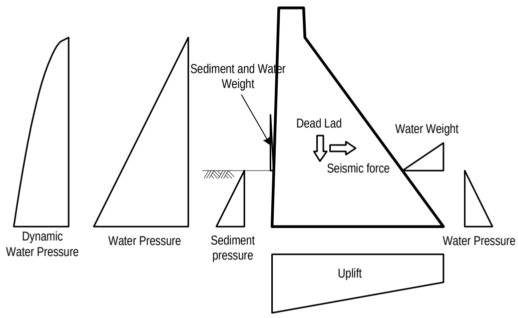
The four (4) safety calculations are these equations as the follows.

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)

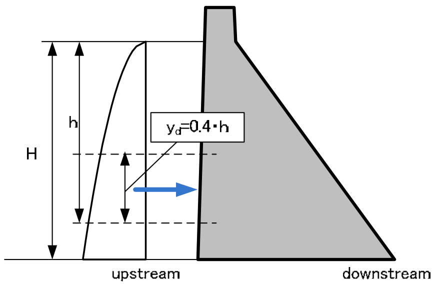


Figure 5.1.5 Diagram of Dynamic Water Pressure

Seismic factor

Seismic force is calculated based on the formula below.

$$Fh = 0.05 \cdot P \quad \text{(Horizontal)}$$

$$Fv = 0.03 \cdot P \quad \text{(Vertical)}$$

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

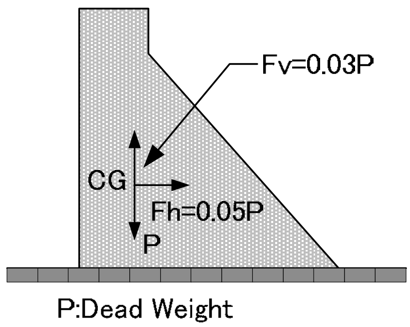


Figure 5.1.6 Diagram of Seismic Factor

Table 5.1.7 Seismic factor

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

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

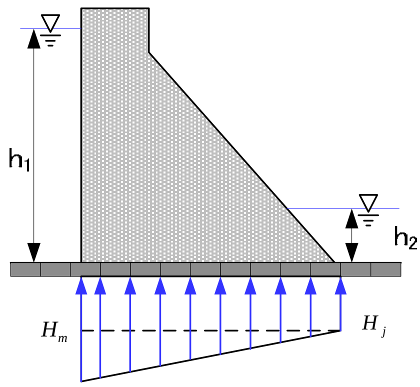


Figure 5.1.7 Diagram of Seismic Factor

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.

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

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

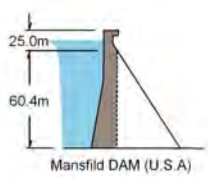
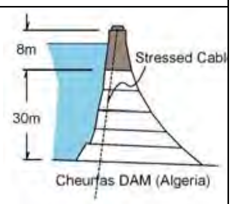
Notes
Internal friction angle: 25 deg (Soft clay)
Height of sediment deposited: EL.338.50 (Inlet of Conduit)

(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.

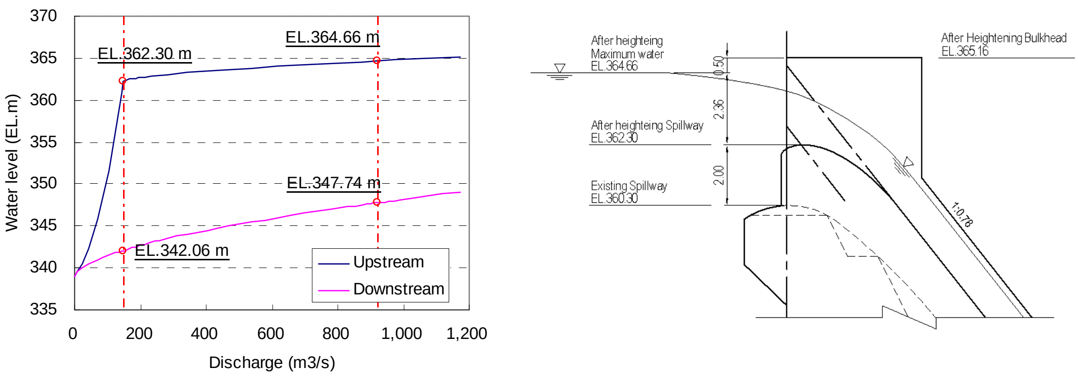
Source : JICA survey team

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 :

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

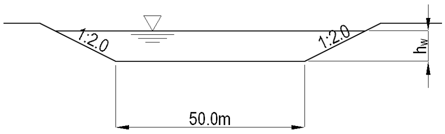
$V : Velocity\ (m/s)$

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

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

$R : Hydraulic\ radius\ (m)$

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



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		---	---	

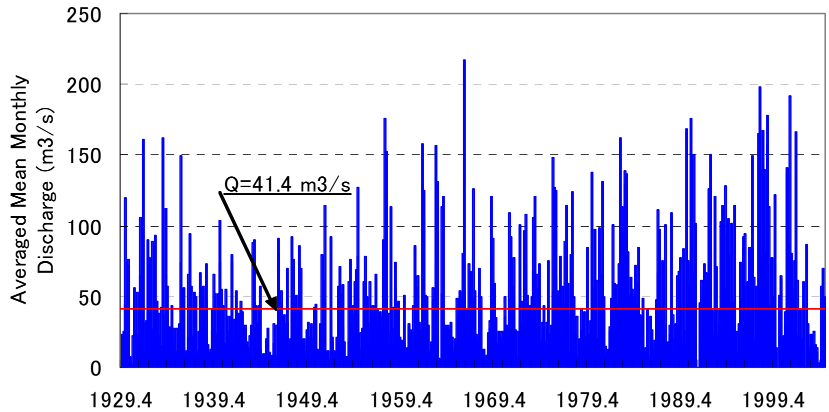
Source：JICA survey team

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



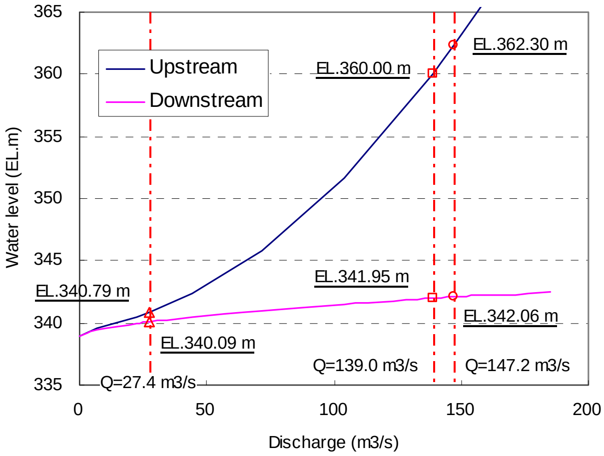
Source：JICA survey team

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

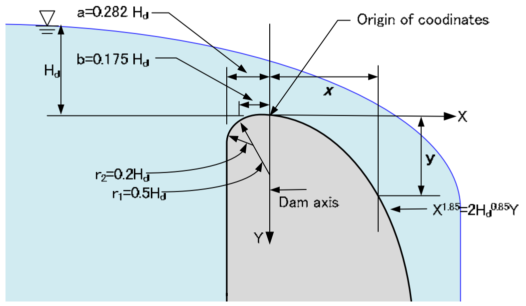
5.1.3 Structure Design

(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.

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

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



Source : JICA survey team (Based on XXX)

Figure 5.1.11 Standard Dimensions and Flow Parameter

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

determined 3.46 (m)

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

determined 0.70 (m)

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

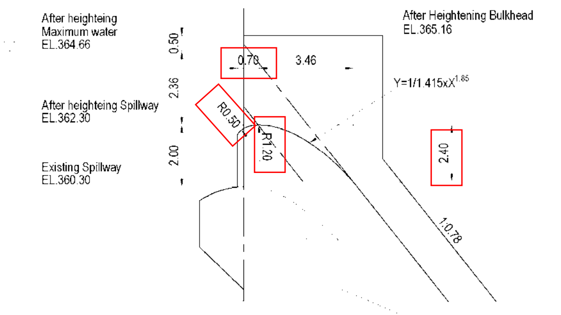
determined 0.45 (m)

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

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.

Design Discharge

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

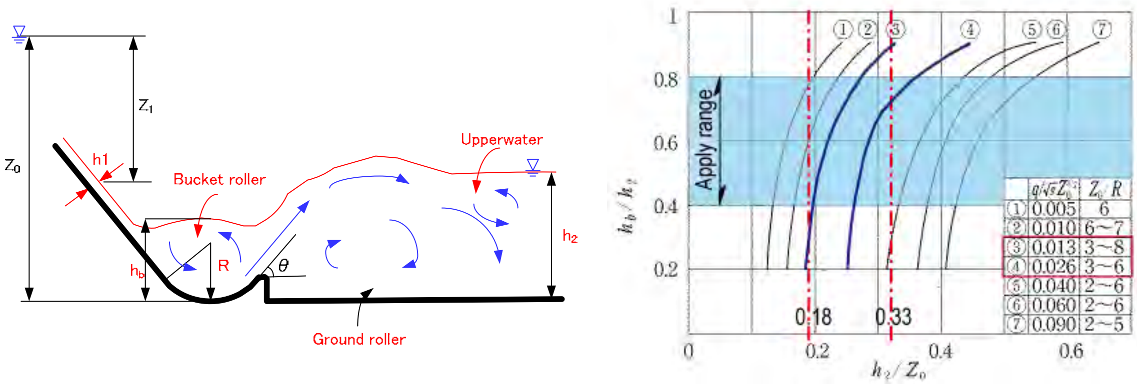
	Taió catchment area =	1570.13	km2
	Barragem Oeste catchment area =	1042.00	km2
	Füller equation :	Qti=Qt(1+2,66/(A**0,3))	

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

Analysis Result of Bucket Type Energy Dissipater

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.



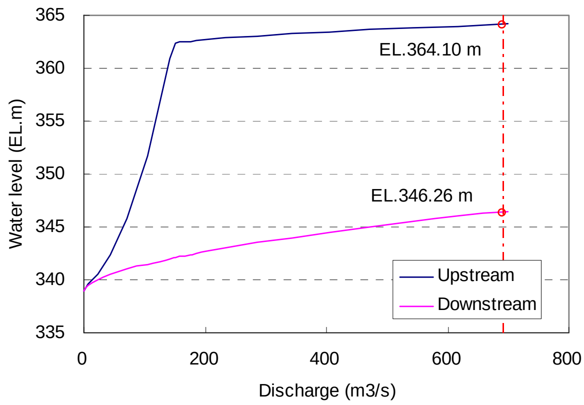
Source：JICA survey team

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

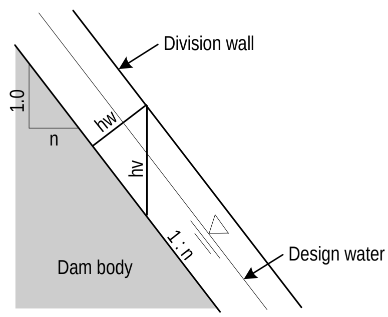


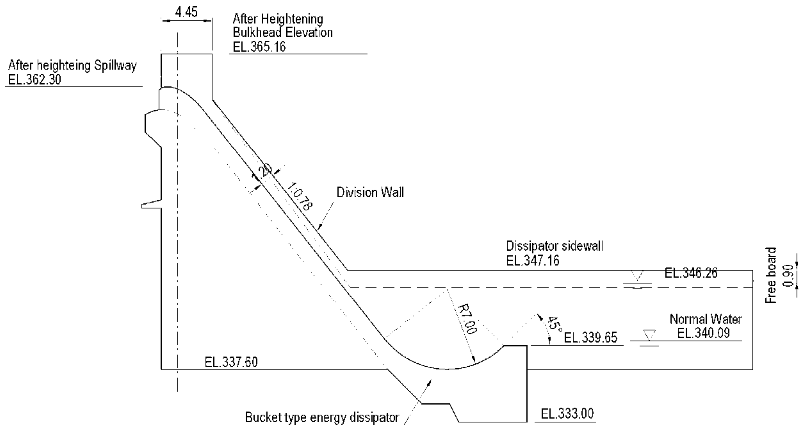
Figure 5.1.15 Diagram of Division Wall

The Height of Dissipator sidewall

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

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

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.

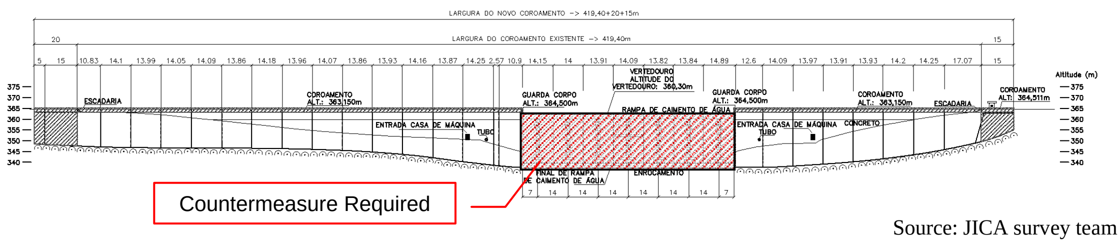
Table 5.1.13 Stability analysis results

	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

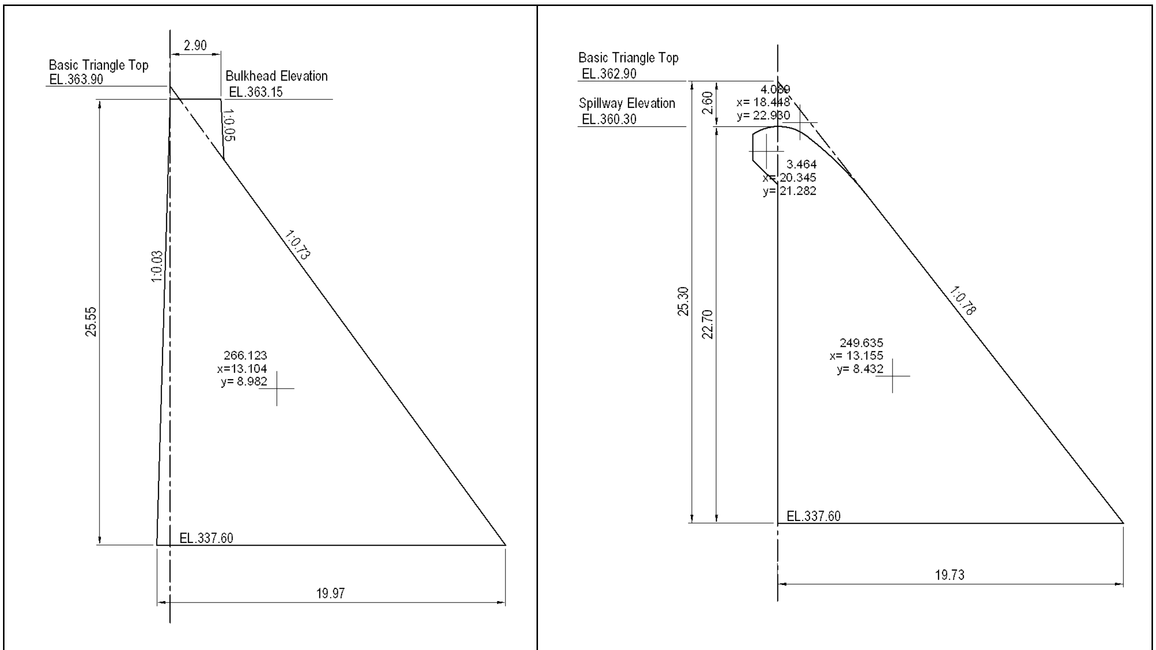
1) Design condition

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

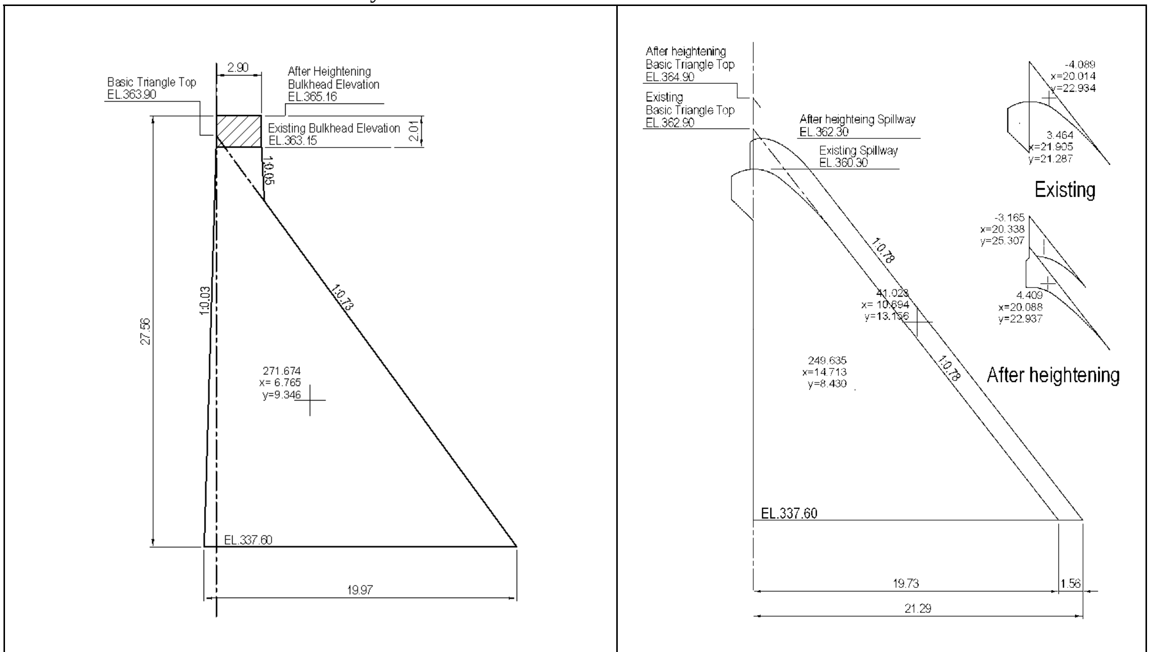
All conditions of stability are satisfied under the condition that the downstream sloop is 1:0.78.The bearing capacity requirement is satisfied sufficiently (σ_a=10 M N/m^2 σmax=0.62 M N/m^2, 1 M=10^6).

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

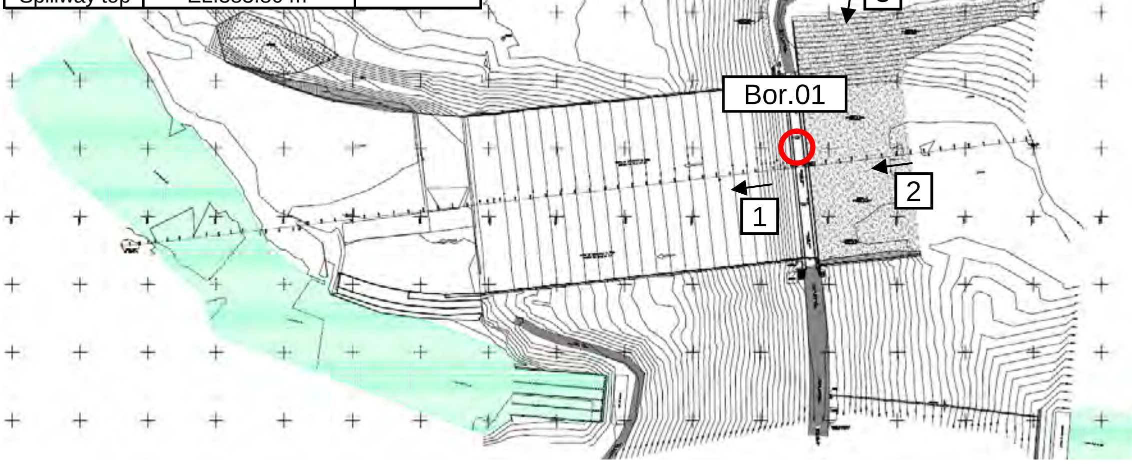
Figure 5.1.19 Determinating Heighten Spillway Section

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

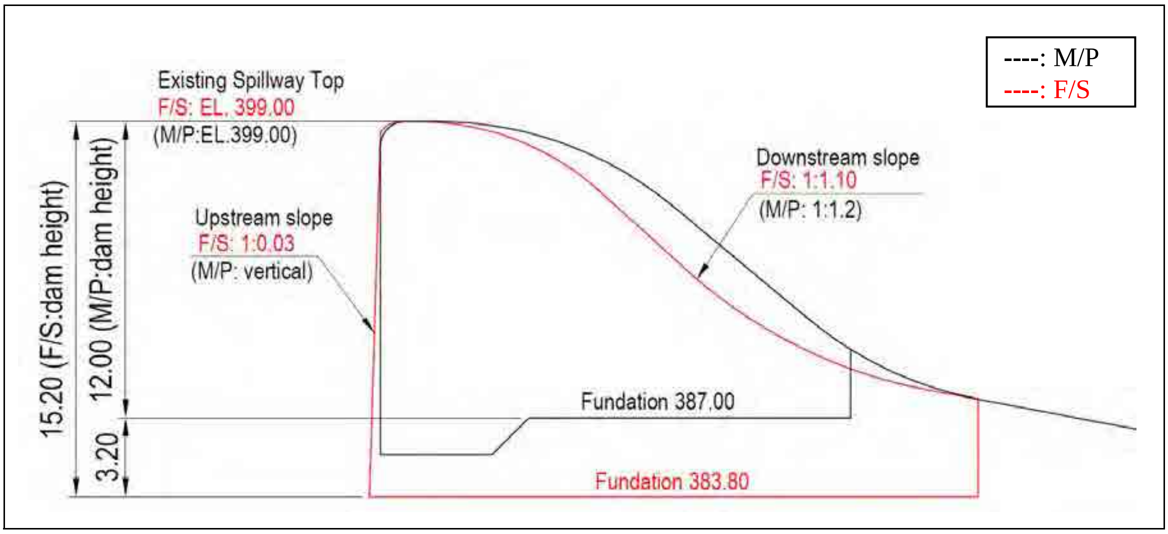
(1) Topography

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.

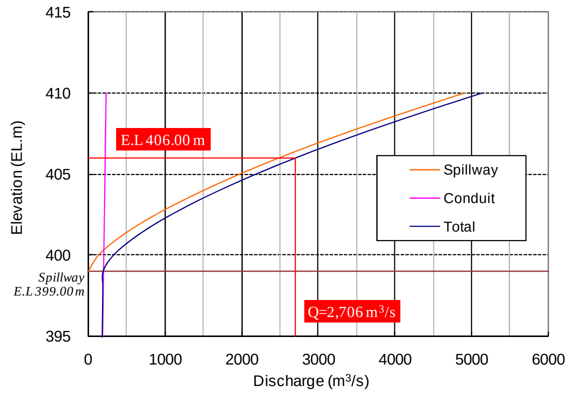
$$Q_{conduit} = C_1 \times N \times C_2 \times (2 \times g \times H_{conduit})^{0.5}$$

where

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

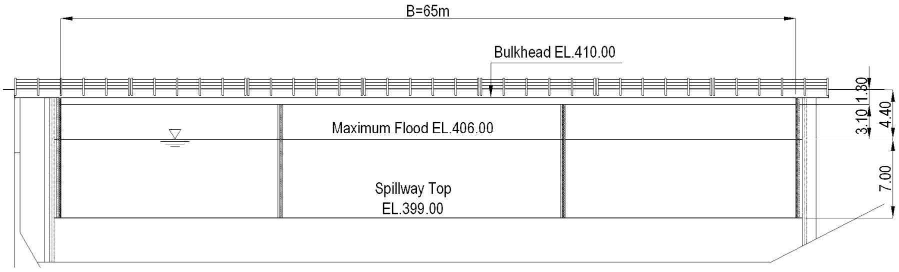
$H_{conduit}$:the head on the conduit

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

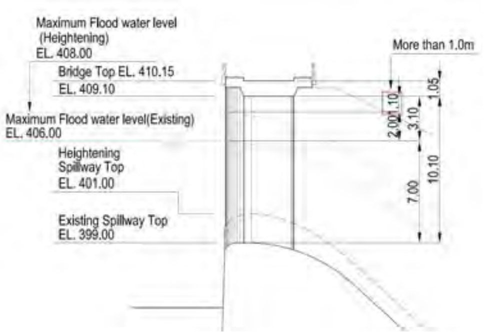
Figure 5.2.4 Front View of Sul Dam Spillway

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).

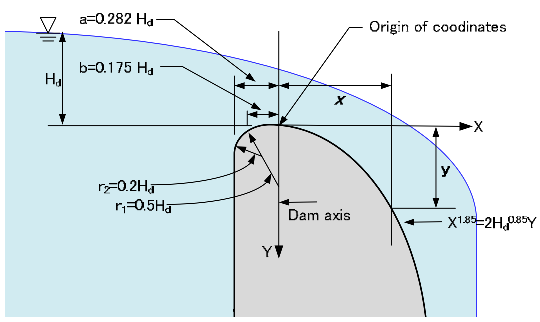
(3) Structure design of Heightening Overflow Section

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)

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

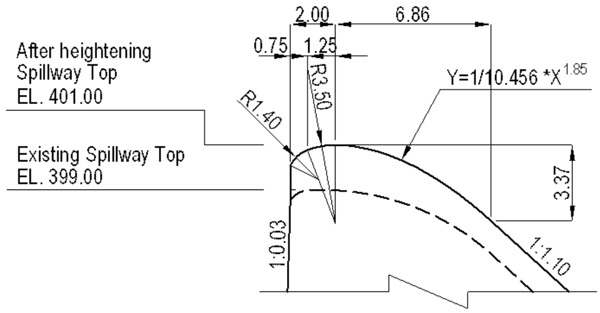
determined 1.25 (m)

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

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

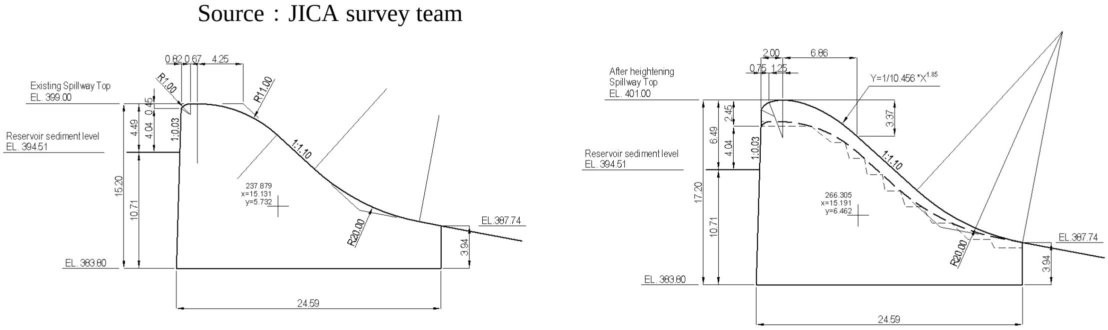
(1) Design condition

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

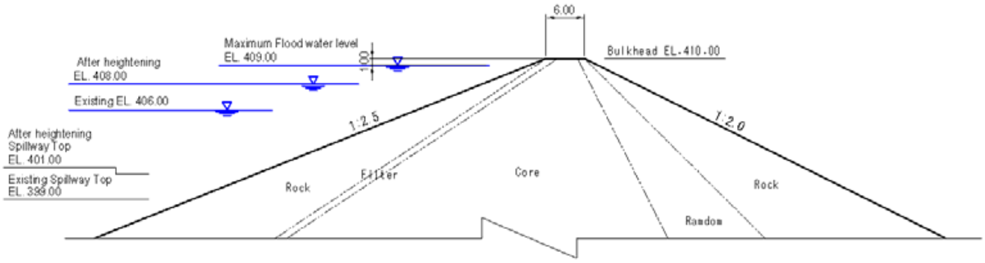
2) Water Level Condition

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

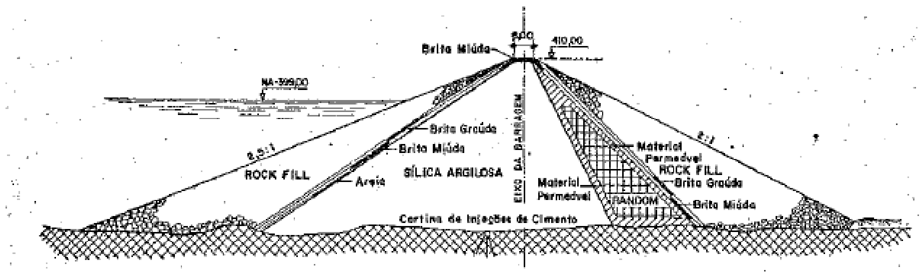


Figure 5.2.10 Traced Old Drawing

3) Analysis method for seepage flow

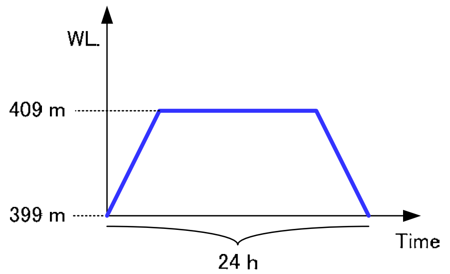
Two-dimensional

The finite element method

Unsteady – flow

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

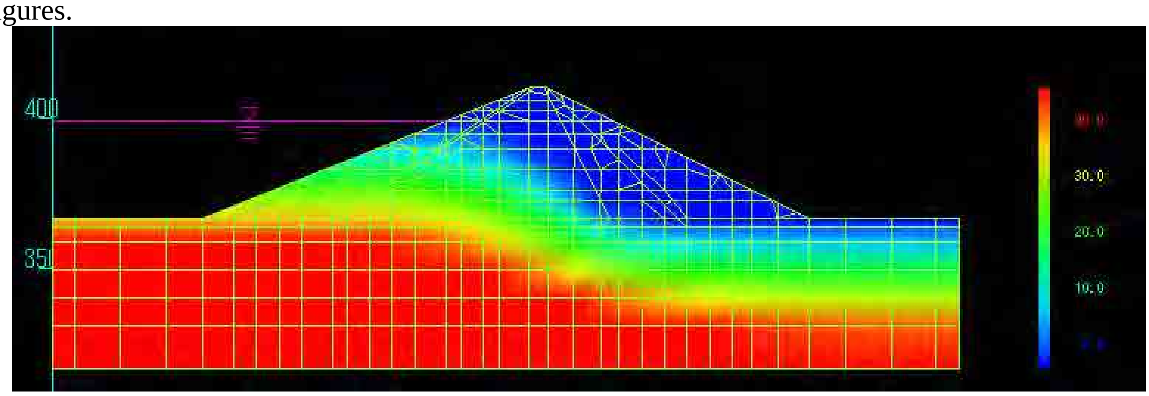
downstream 366.5 m (ground level)



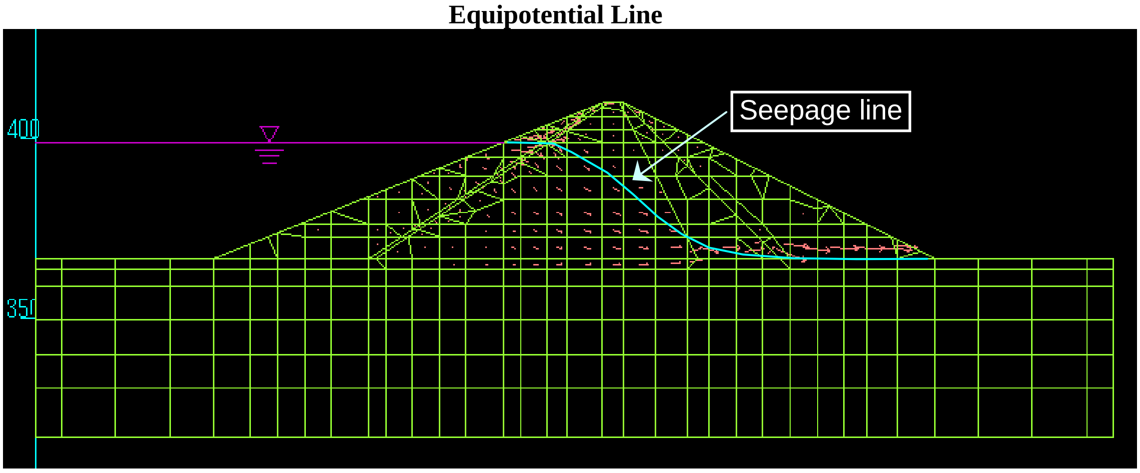
(3) Seepage flow analysis

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) ($P = P_w/\rho g = \rho g h/\rho g = h$)

$$\frac{G}{W} = \frac{19x84.0}{10x45.5} = 3.51 > 1.0$$

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 ~ 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

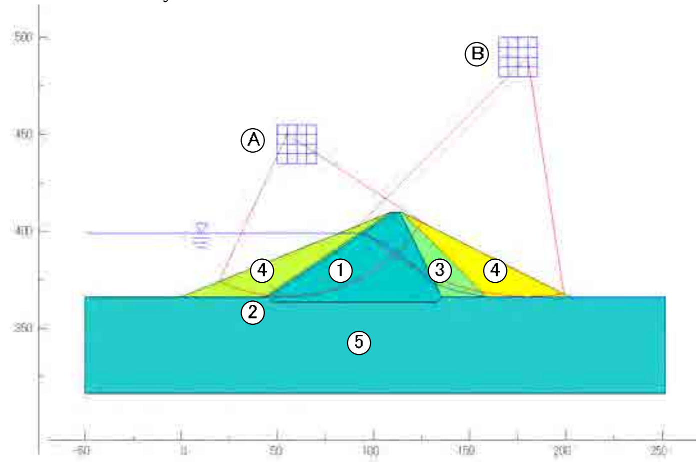
2) Result and Assessment

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

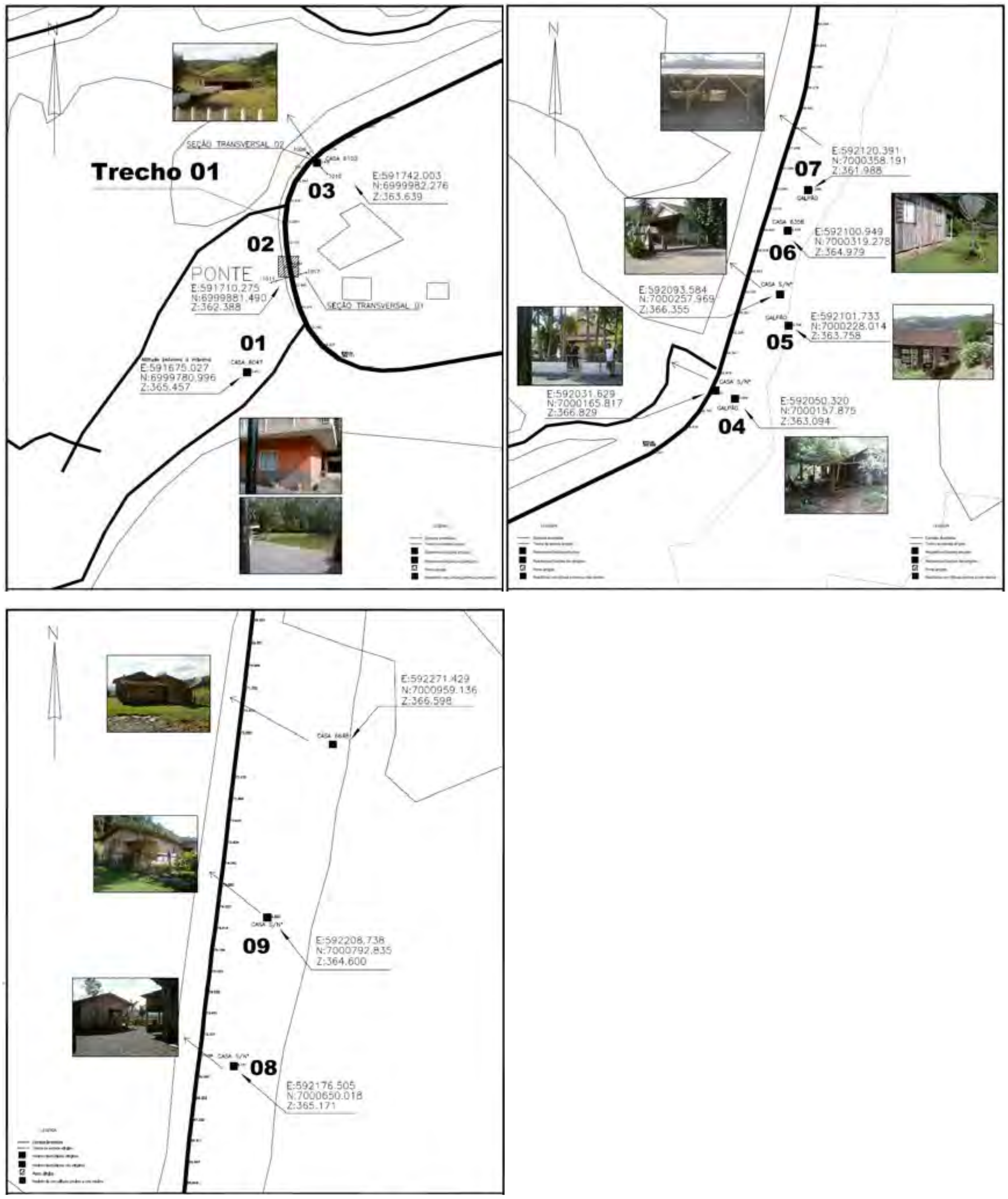
Figure 5.2.12 Result of Slip Circle

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.



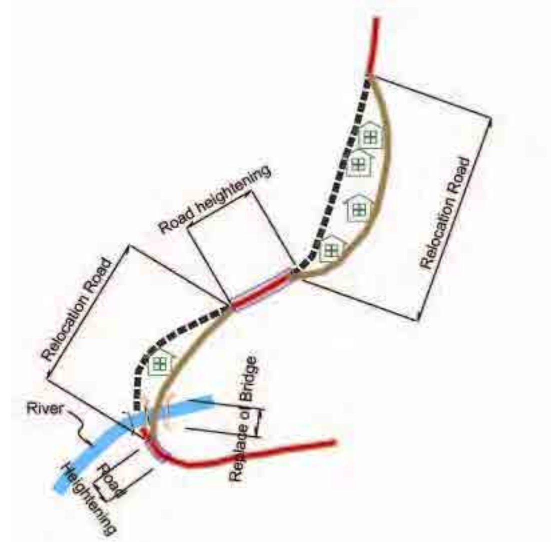
Source: JICA survey team

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

<table>
<tr><th></th><th>Alternative measure-1: with road relocation</th><th>Alternative measure-2: with resettlement</th></tr>
<tr><td>Chart</td><td>


</td><td>


</td></tr>
<tr><td>General description</td><td>・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.</td><td>・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</td></tr>
<tr><td>Merit</td><td>・ No resettlement of the communities</td><td>・Less cost due to decrease of volume of construction works</td></tr>
<tr><td>Demerit</td><td>・Increase of construction cost due to road relocation education of inundation area due to installation of the road</td><td></td></tr>
<tr><td>Project cost</td><td>R$ 4,797,000 (100%)</td><td>R$ 2,819,000 (58.8%)</td></tr>
</table>

Source: JICA survey team

Table 5.3.2 Implementation Cost for Countermeasure

(R$)

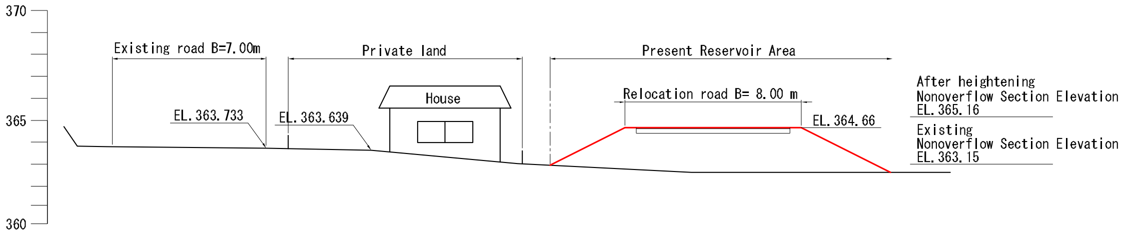
	unit	unit cost	Alternative of Road relocation: quantity	Alternative of Road relocation: amount	Alternative of Compensation: quantity	Alternative of Compensation: amount	Remarks
Replace of Bridge	m2	3,000	160	480,000	80	240,000	
Relocation Road	m	1,570	1,500	2,355,000	500	785,000	
Other works	%	30	---	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.

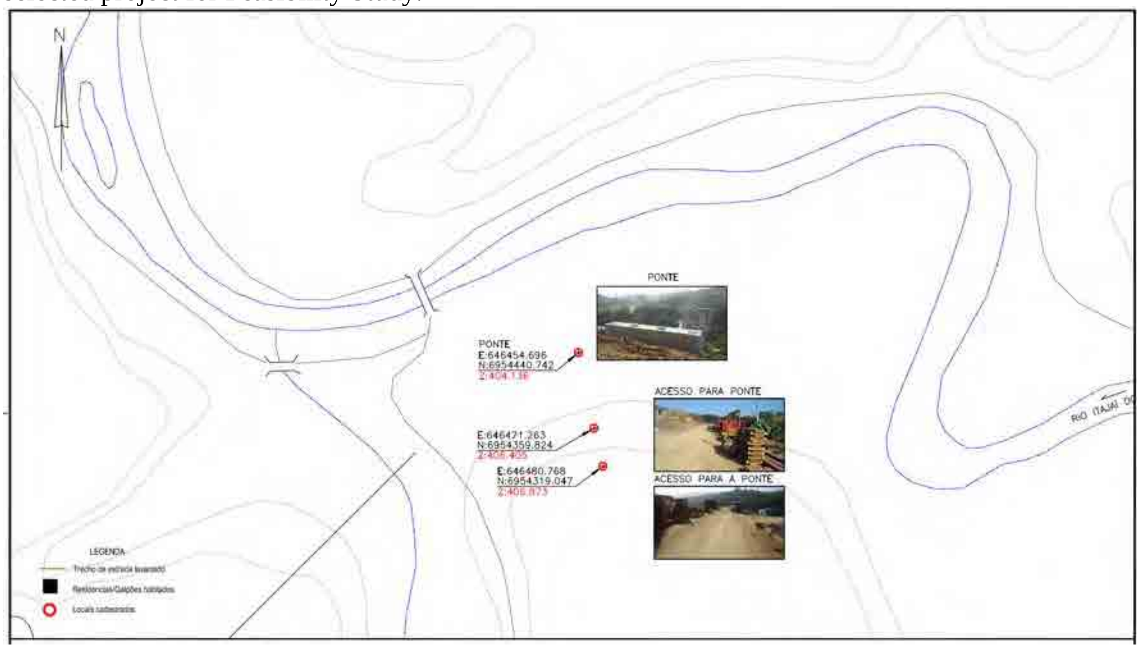


Figure 5.3.3 Survey Result on Sul Dam

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.

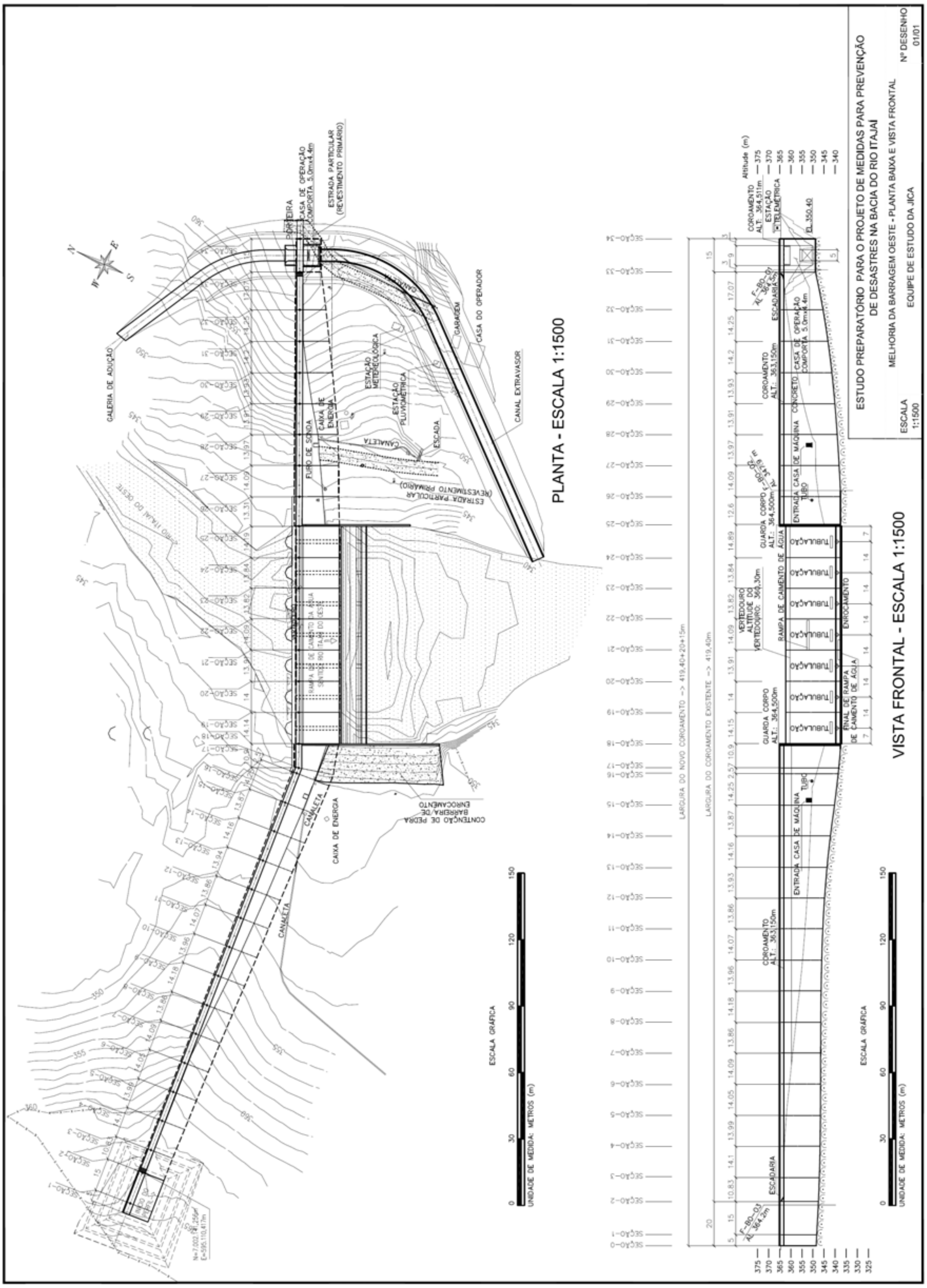


Figure 5.3.4 Heightening Oeste Dam (1)

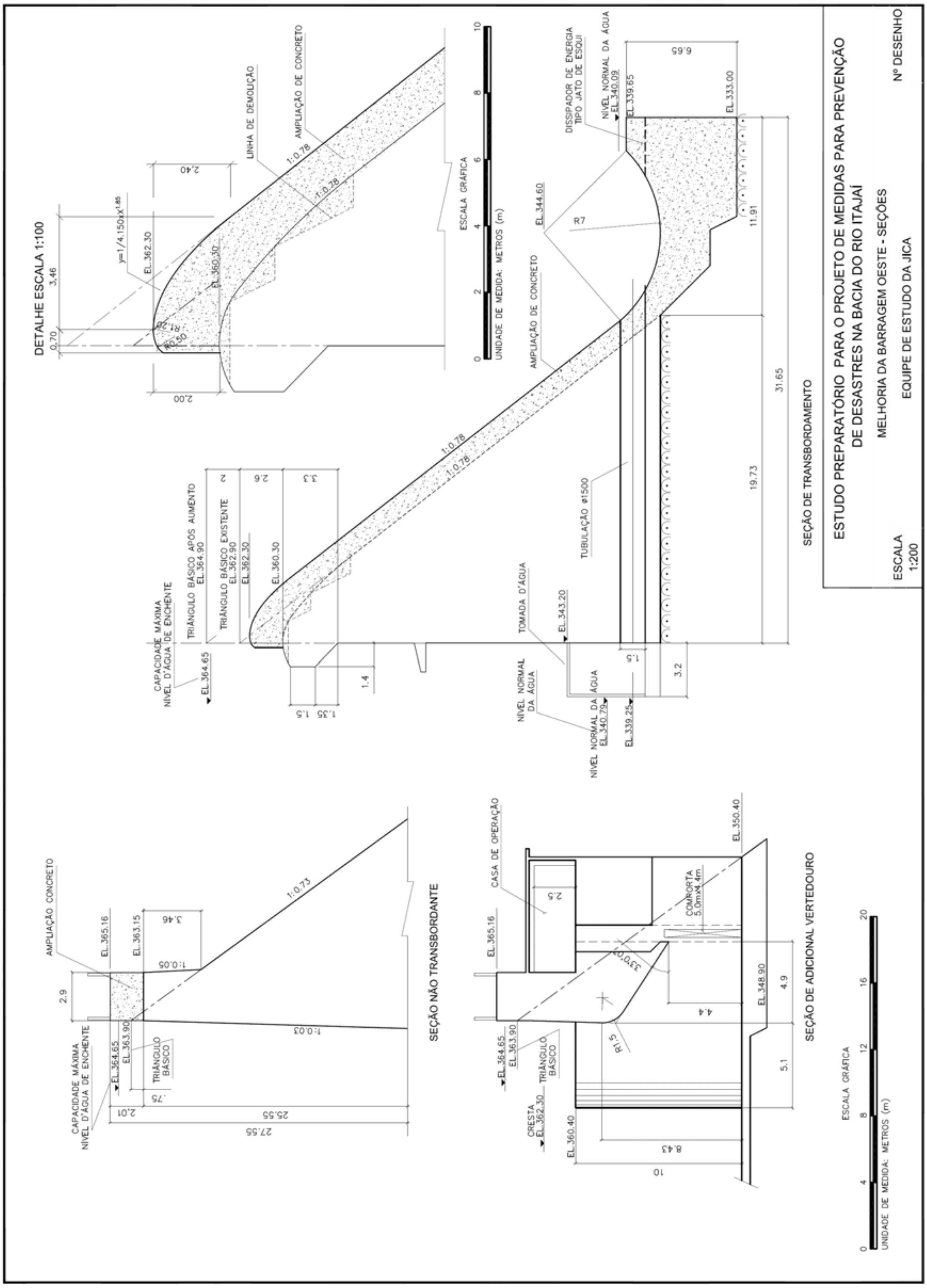


Figure 5.3.5 Heightening Oeste Dam (2)

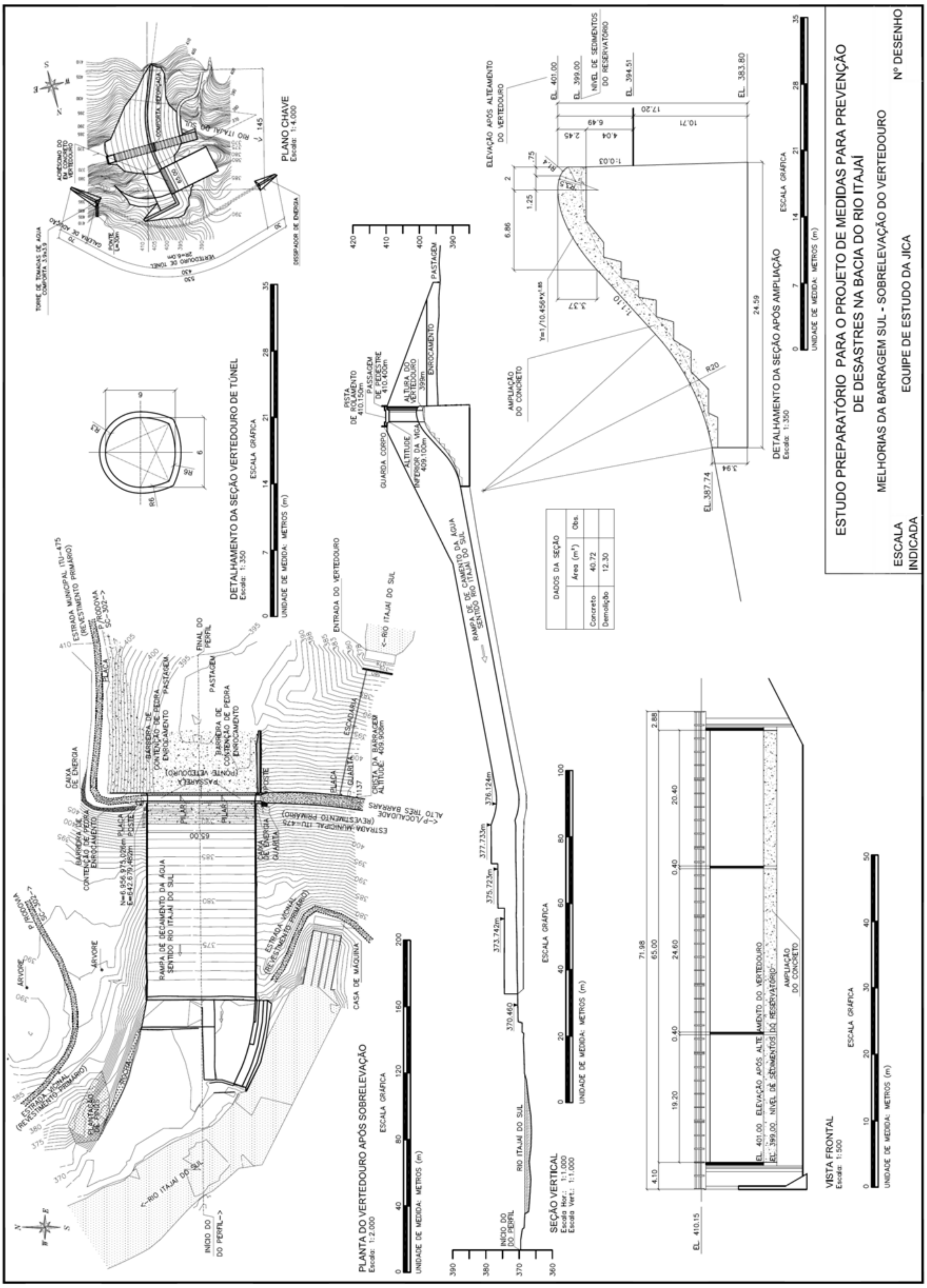


Figure 5.3.6 Heightening Sul Dam

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

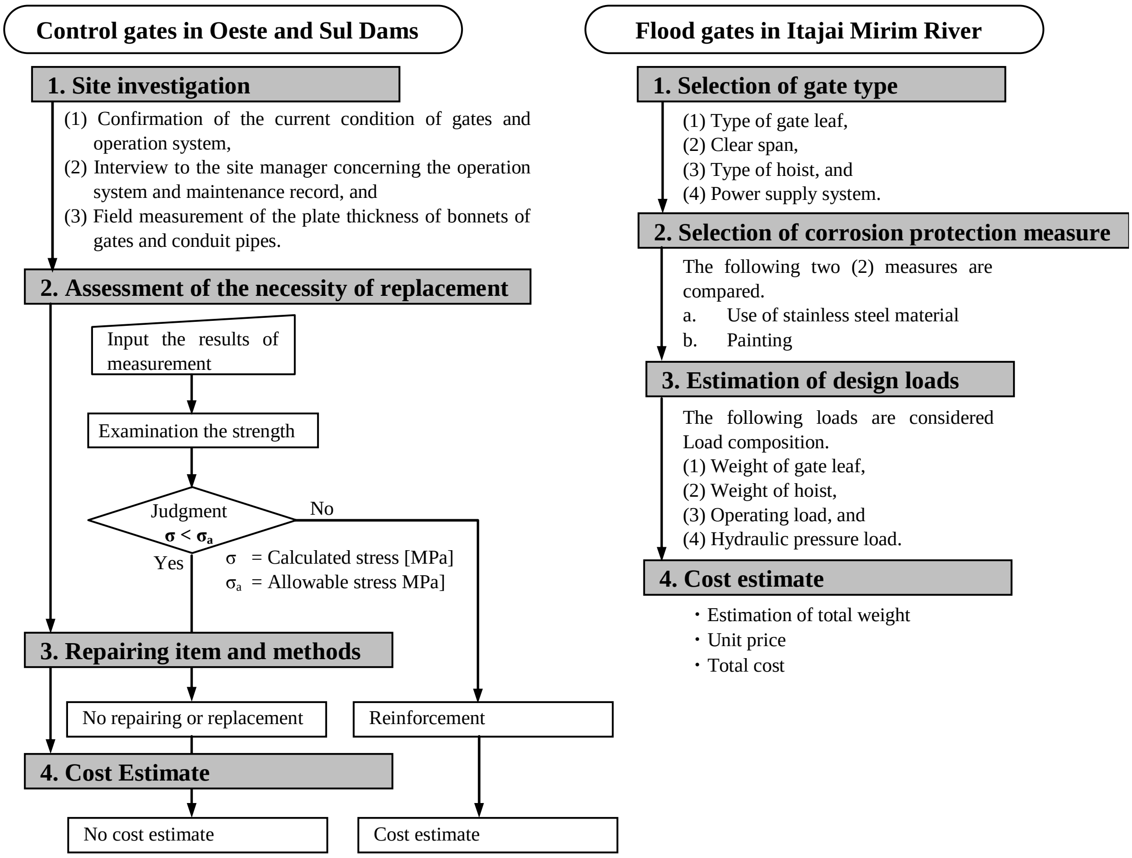
The work flow under examination is shown below.

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

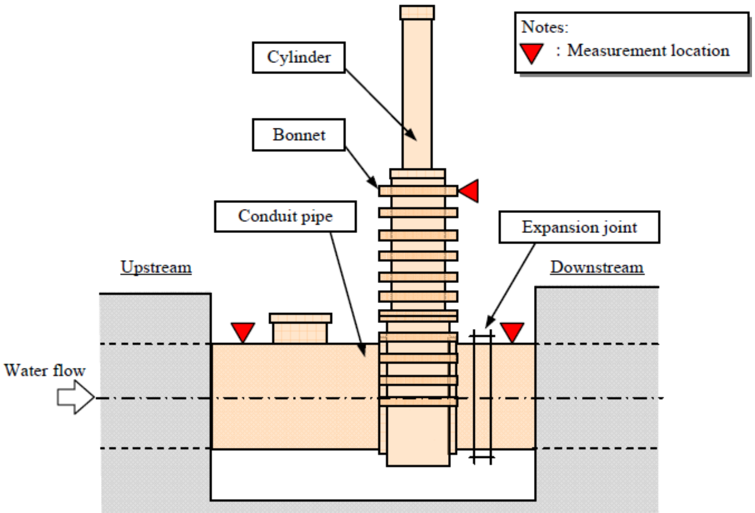
a-1 Thickness of stiffener girder (Bonnet)

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

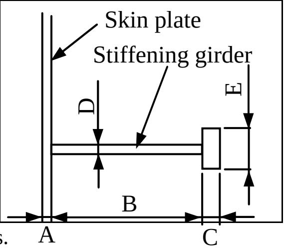
Source: JICA Survey Team

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

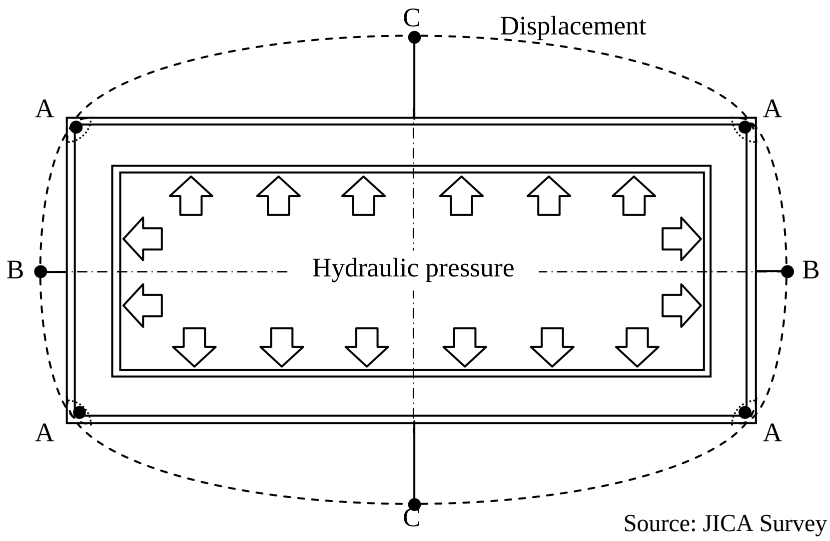
CCE1: Normal water level + Dynamic water pressure during earthquake

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

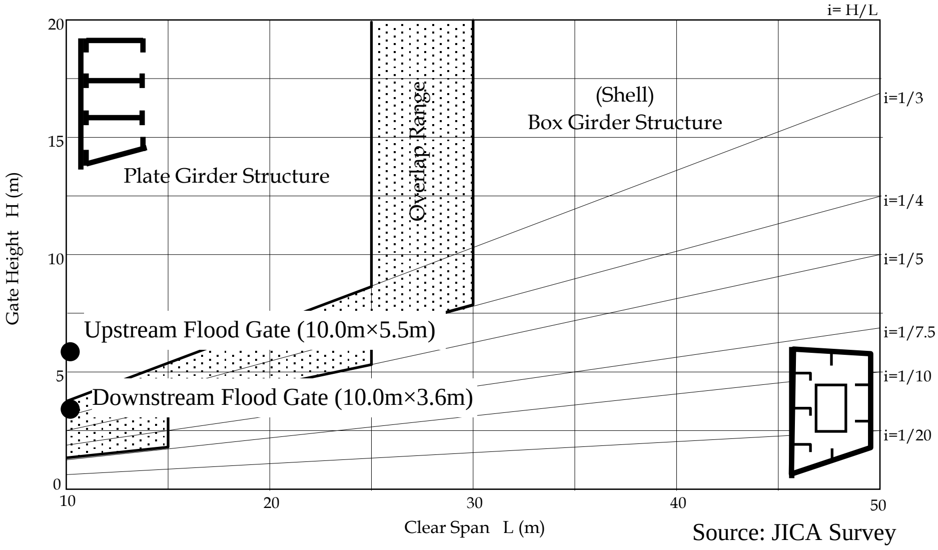
Particulars	Upstream Flood Gate	Downstream Flood Gate
Type of gate	Fixed wheel gate	Fixed wheel gate
Quantity	4 sets	4 sets
Clear span	10.0m	10.0m
Gate height	5.5m	3.6m
Sill elevation	EL.-1.00m	EL.-1.00m
Type of hoist	Wire rope winch hoist	Wire rope winch hoist

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:



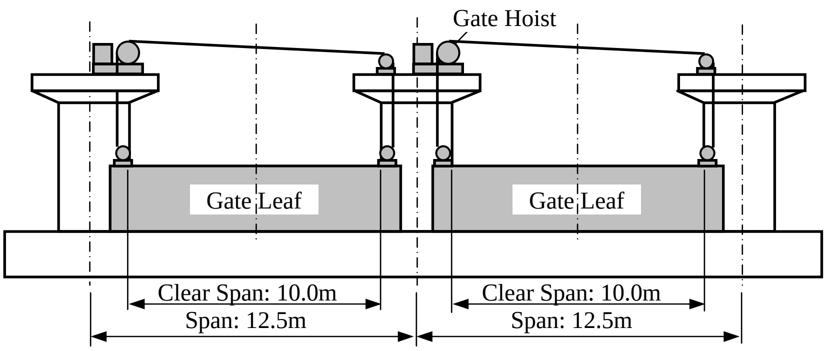
Source: JICA Survey

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

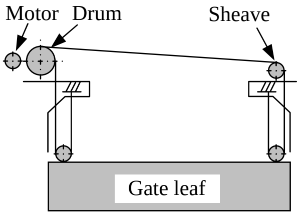
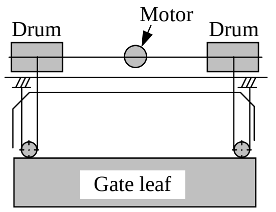
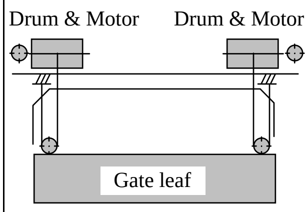
Figure 6.3.2 Clear Span and Span of Gate

(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.

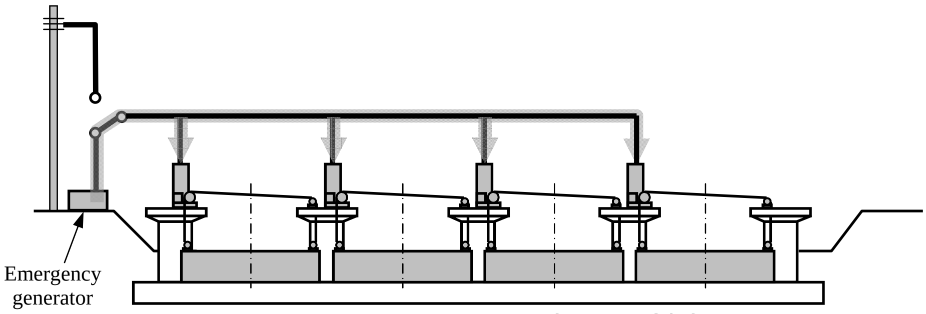
Table 6.3.2 Type of Hoist

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

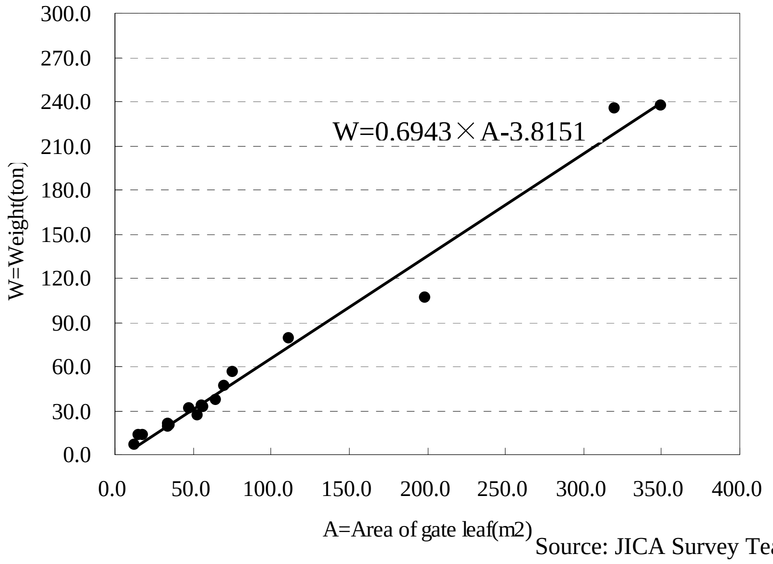
Notes: The unit price in Brazil depends on the HISA hearing survey (May, 2011).

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 × 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

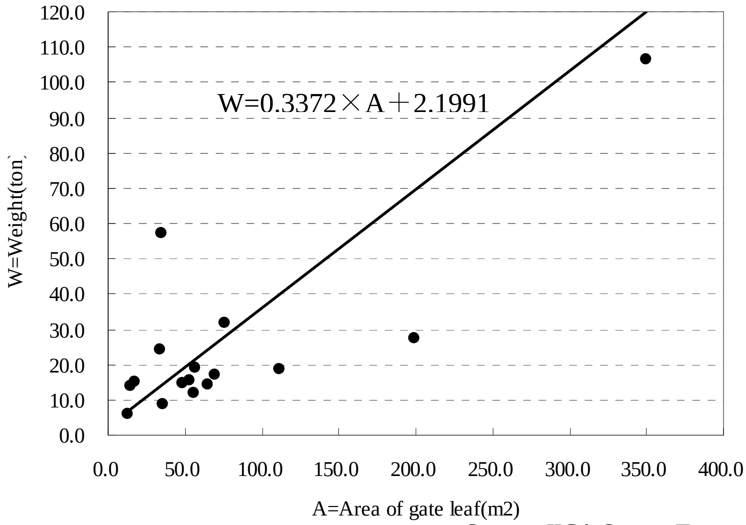
Gate	Clear span (m)	Gate height (m)*	Area (m2)	Weight (ton)	Weight (kN)
Upstream flood gate	10.0	5.5	55.0	42.0	412.1
Downstream flood gate	10.0	3.6	36.0	28.8	282.6

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

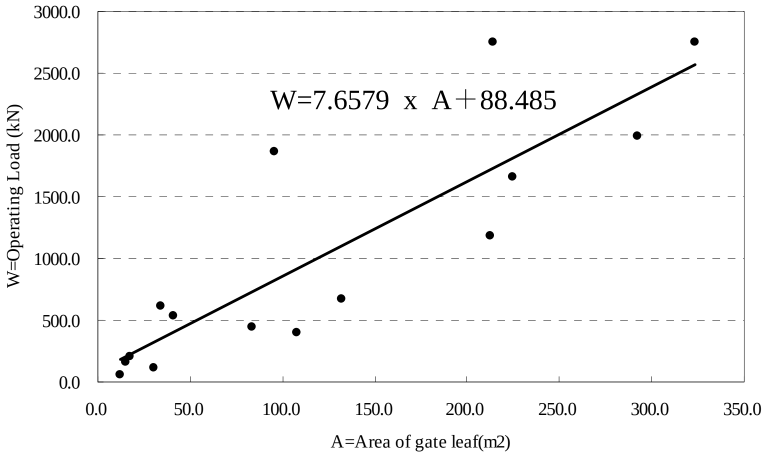
Gate	Clear span (m)	Gate Height (m)*	Area (m2)	Weight (ton)	Weight (kN)
Upstream flood gate	10.0	5.5	55.0	22.8	223.7
Downstream flood gate	10.0	3.6	36.0	15.8	155.0

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:

$W = 7.6579 \times A + 88.485$

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.

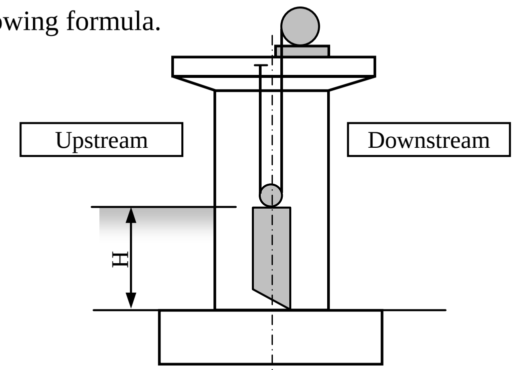
$$WG4 = \frac{1}{2} \times (H^2 \times w_0) \times B$$

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:

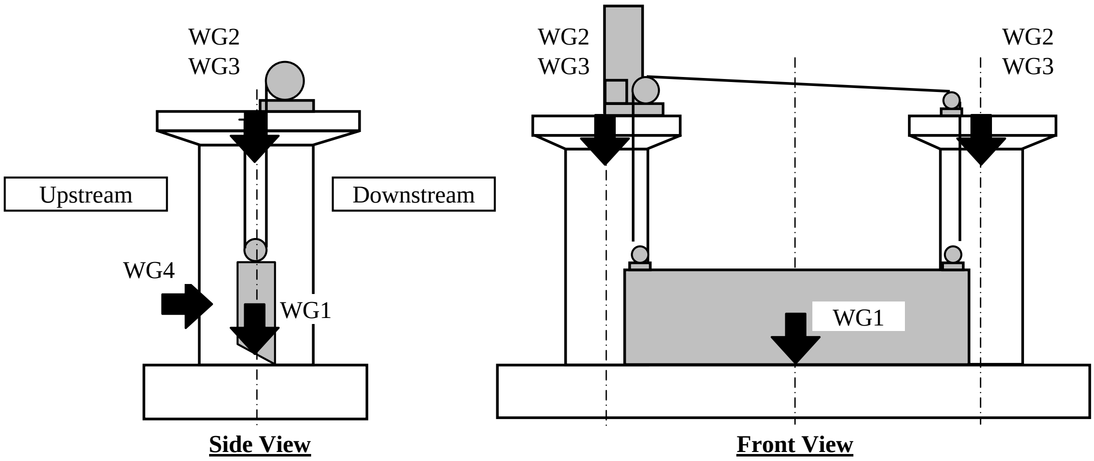
Table 6.3.7 Hydraulic Pressure Load

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

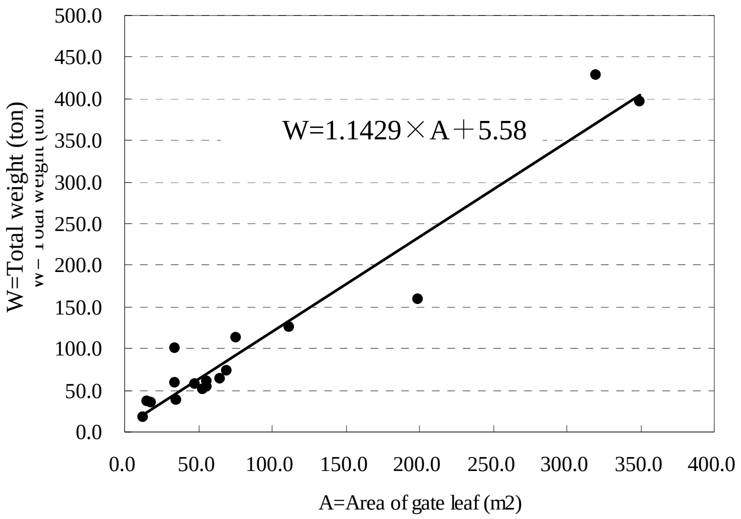
Gate	WG1 (kN)	WG2 (kN)	WG3 (kN)	WG4 (kN)
Upstream flood gate	412.1	111.9	254.9	1909.7
Downstream flood gate	282.6	77.5	182.1	818.2

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

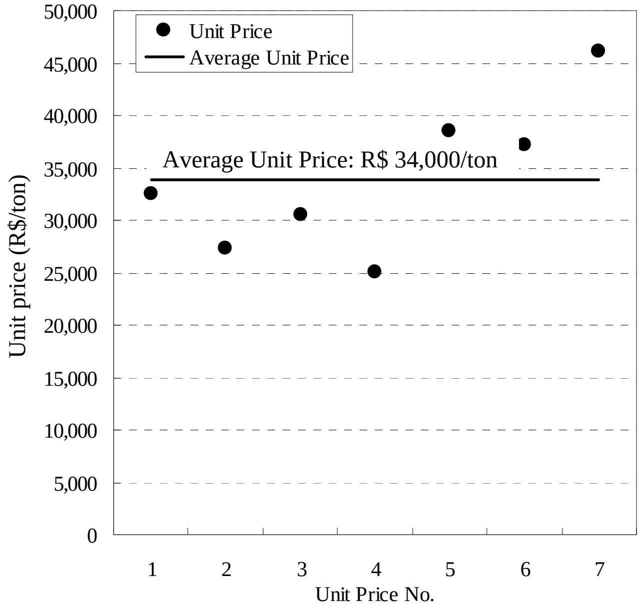
The total weight of gate is estimated by the following formula:

$W = 1.1429 \times A + 5.58$

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

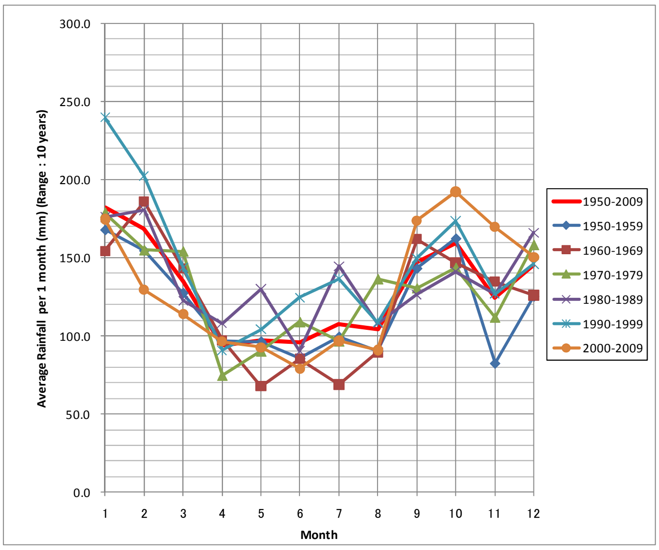
In Brasil, working hours are 44 hours per week and the typical working hours are eight(8) hours. And holidays and weekends are not included. Working days per one month are 20 days calculated by using the equation below, considering 3 days off such as rainy days.

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

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* ($EL.360.00m$)

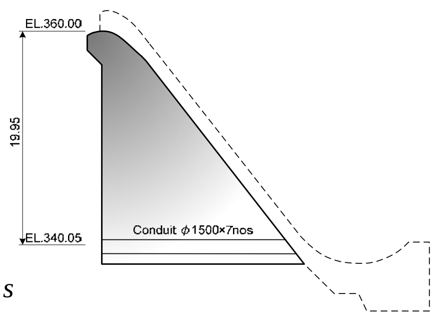
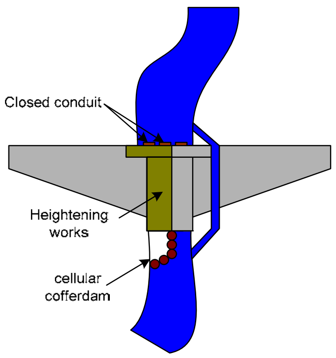
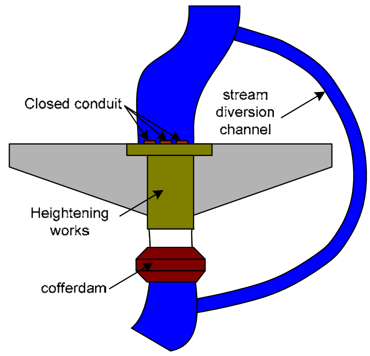


Figure 7.2.2 Image of Calculation of Design Discharge

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

	Multiple-stage Diversion	**Diversion Tunnel**
Outline	 The construction work space is divided two parts alternately.	 Make the tunnel as much size as the conduit. In construction duration, the tunnel is used as water path.
Dimension	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	horse shaped tunnel φ6.0m, i=1/200, L=200m
Construction term	short	long
Construction cost	R$2.9×10^6	R$7.7x10^6
Adjudication	good	---

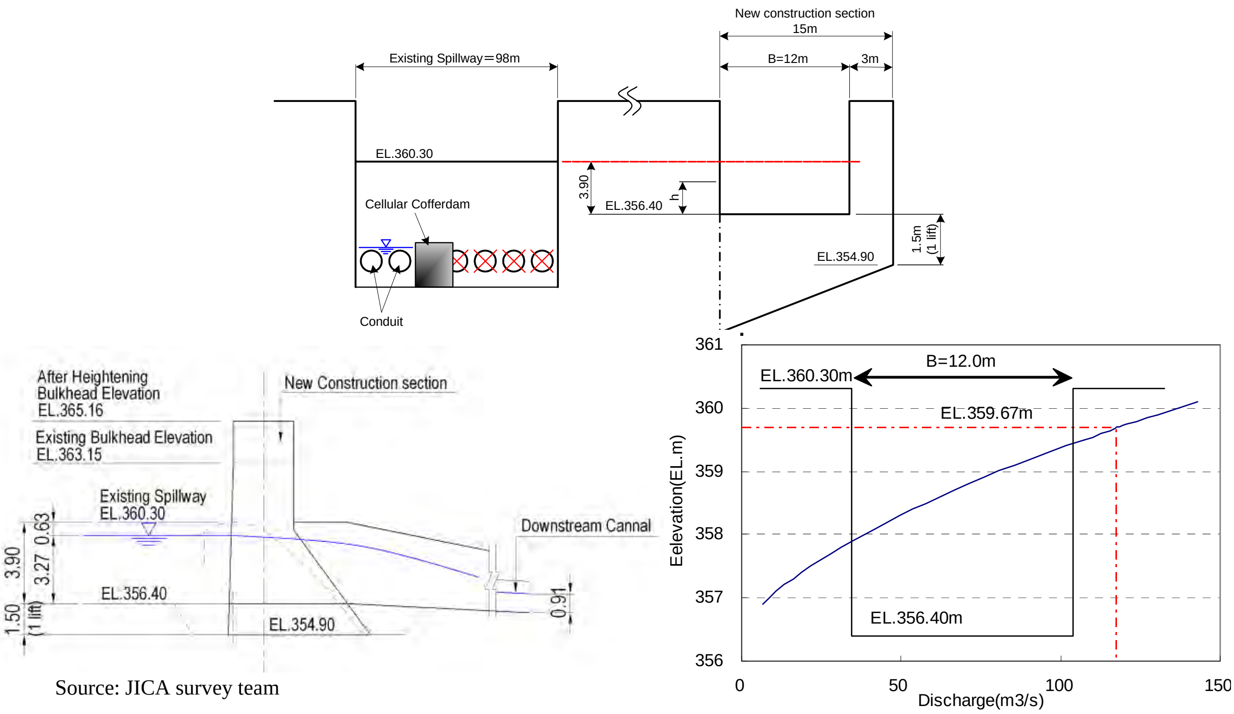
Source: JICA survey team

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

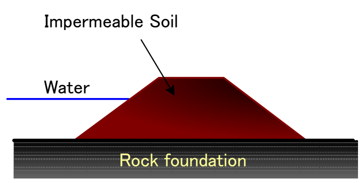
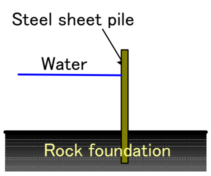
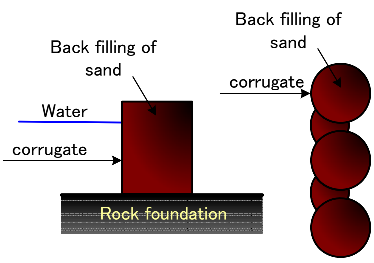
Figure 7.2.3 Scale of Excavation of Wing of Dam Body

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

	Earth Type	Steel Sheet Pile Type	Cellular Cofferdam Type
Figure			

Source: JICA survey team

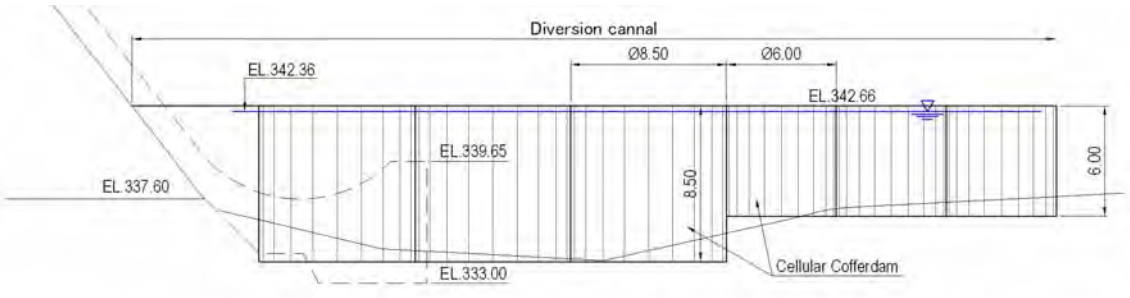
The following figure shows an example of the cellular cofferdam under construction.

Source: MLIT tsugaru dam construction work office

Figure 7.2.4 Example of Construction Cellular Cofferdam

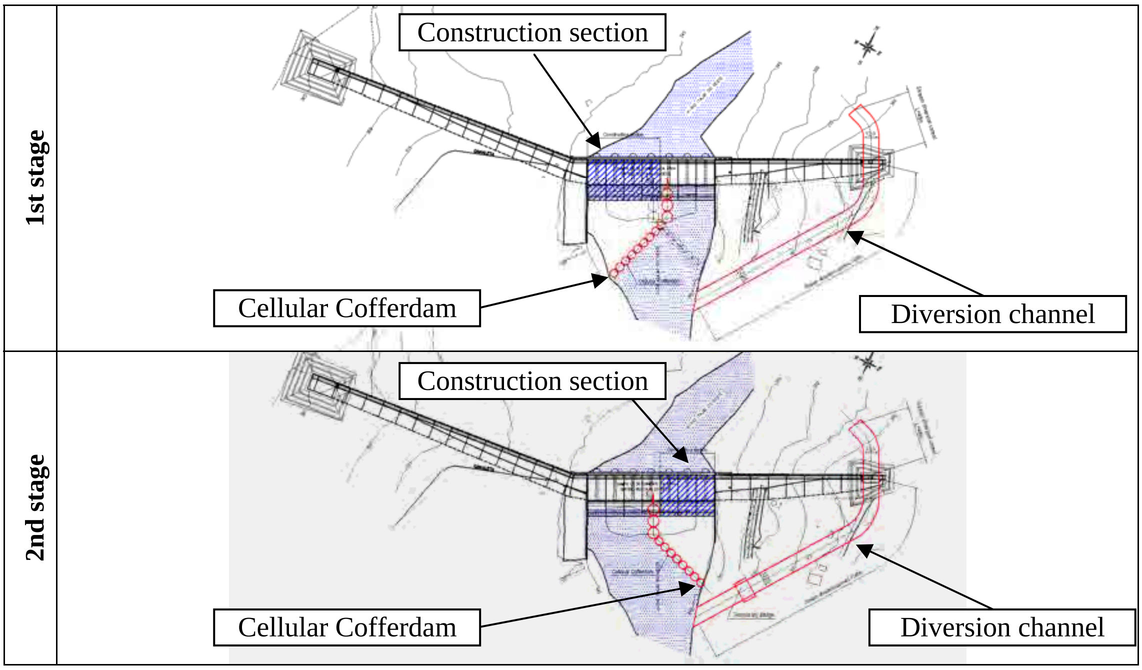
5) Design of cofferdam

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

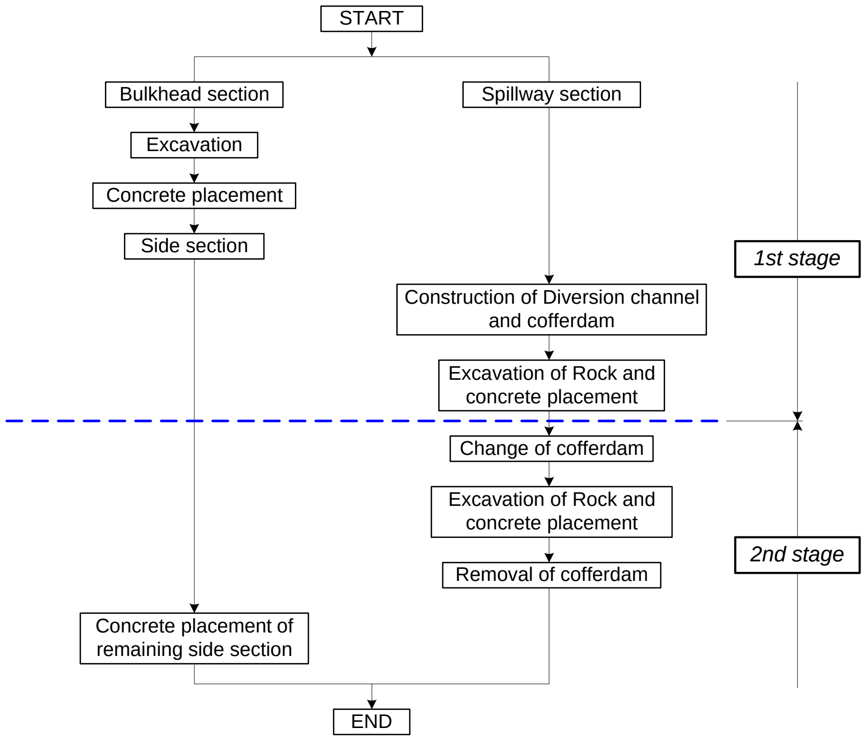


Source: JICA survey team

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

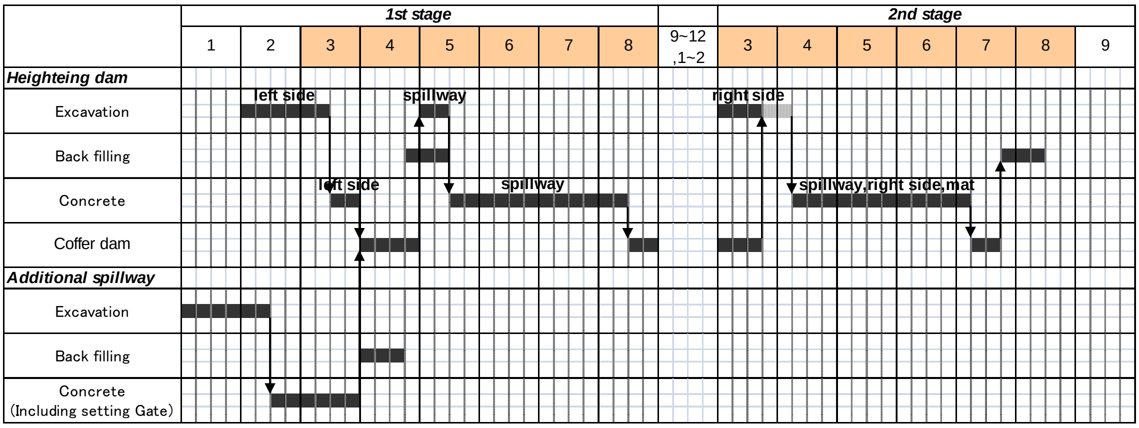
Figure 7.2.8 Scope of Construction Work

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.

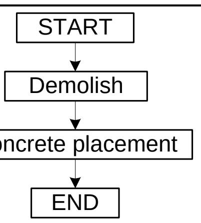
- Compared with the Oeste dam, the capacity of conduit discharge is not changed.
- The construction term is short and the only concrete material is need to be done. Thus there is little risk of flood.

1) Procedure and Area of Construction

The procedure is shown as follows.

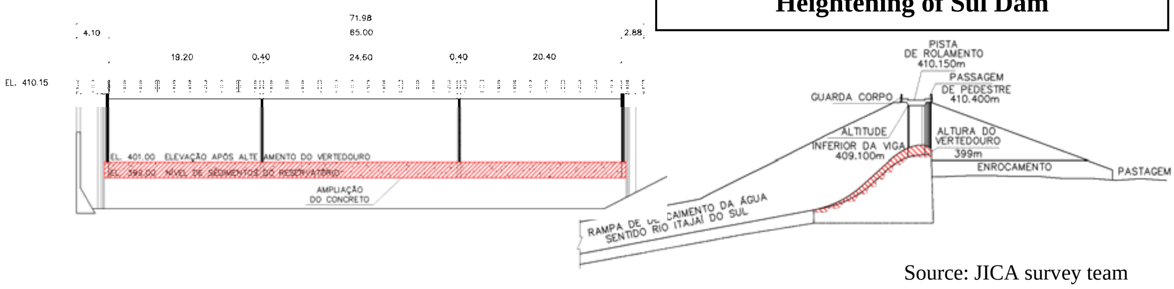
The reinforcing area is shown below.

- 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



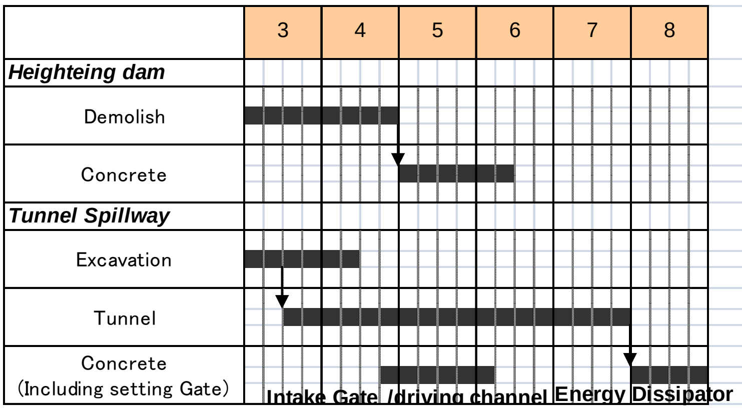
Source: JICA survey team

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.

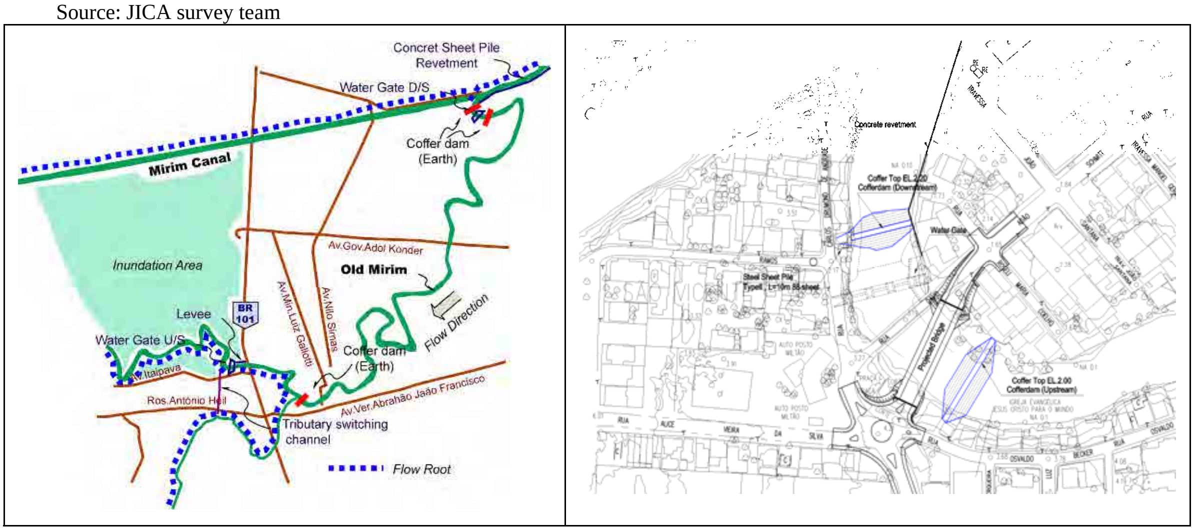
- The elevation of cofferdam height is set at the water level, which is less than 10-year flood at the Itajai River.
- The drainage of the runoff of original catchment area is turned to the upstream gate side.
- The tributary of the Old Mirim River is turned to the upstream gate side through a tunnel.

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

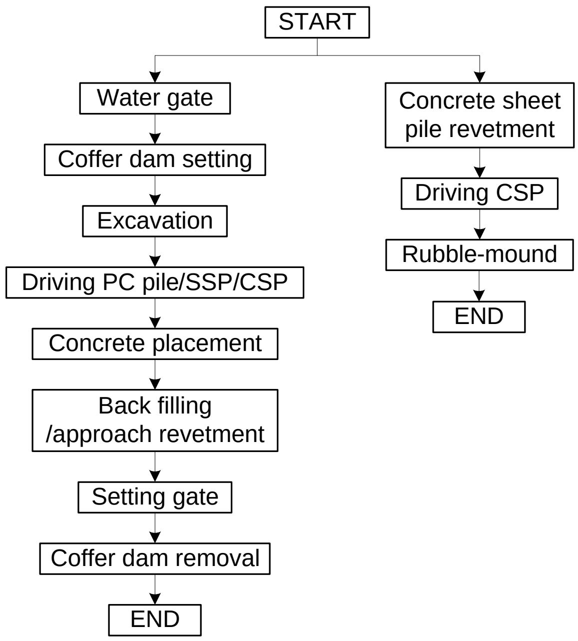


Source: JICA survey team

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

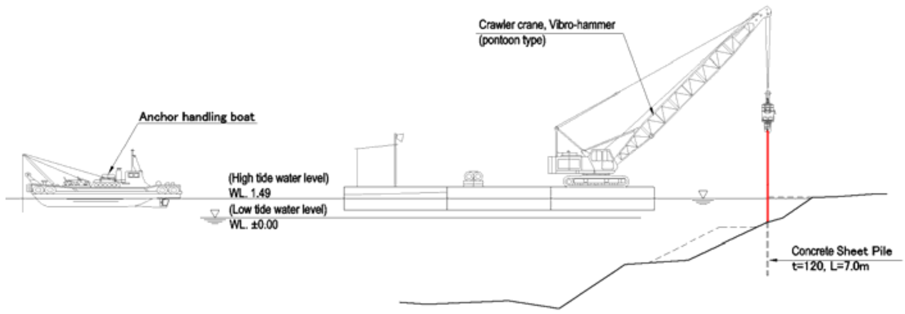
Source: JICA survey team

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.

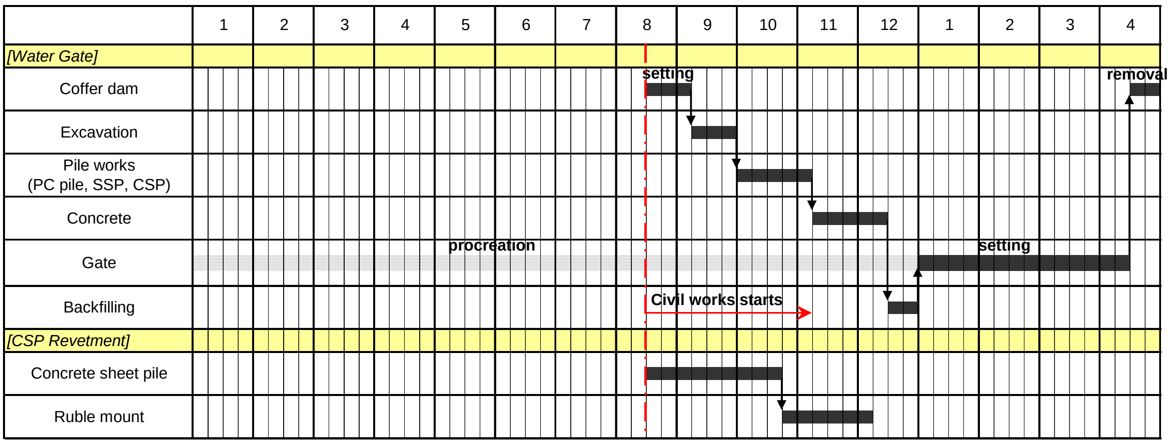


Source: JICA survey team

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

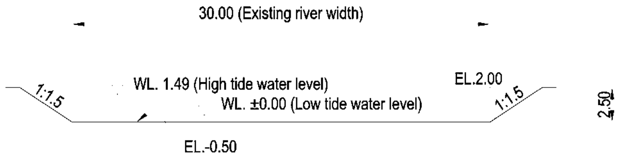
2) Setting of diversion cannel / coffer dam

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

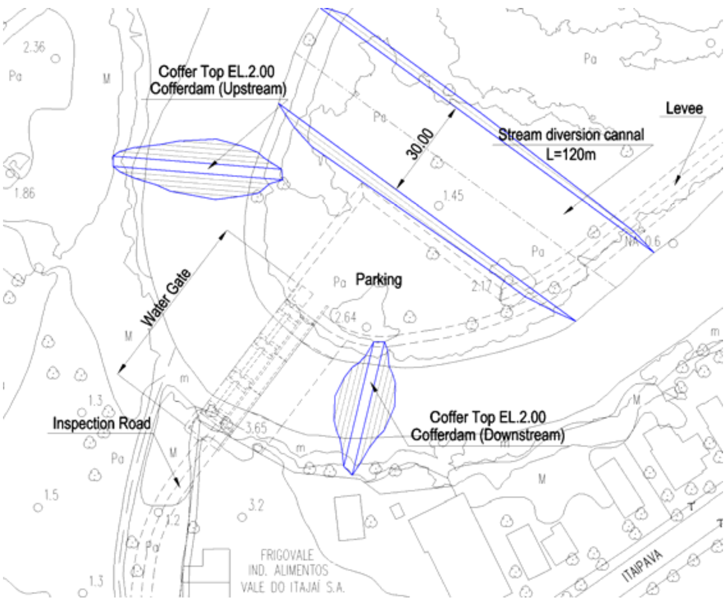
Diversion Channel		**Remarks**
Bottom Elevation	EL.-0.5 m	Low tide water level -0.50
Top Elevation	EL.2.0 m	Present ground elevation
Diversion Channel Width	30.0m	Present river width
Cofferdam		**Remarks**
Elevation of Top	EL.2.0 m	Minimum ground elevation of surrounding land

Source: JICA survey team



Source: JICA survey team

Figure 7.2.17 Section of Diversion Channel

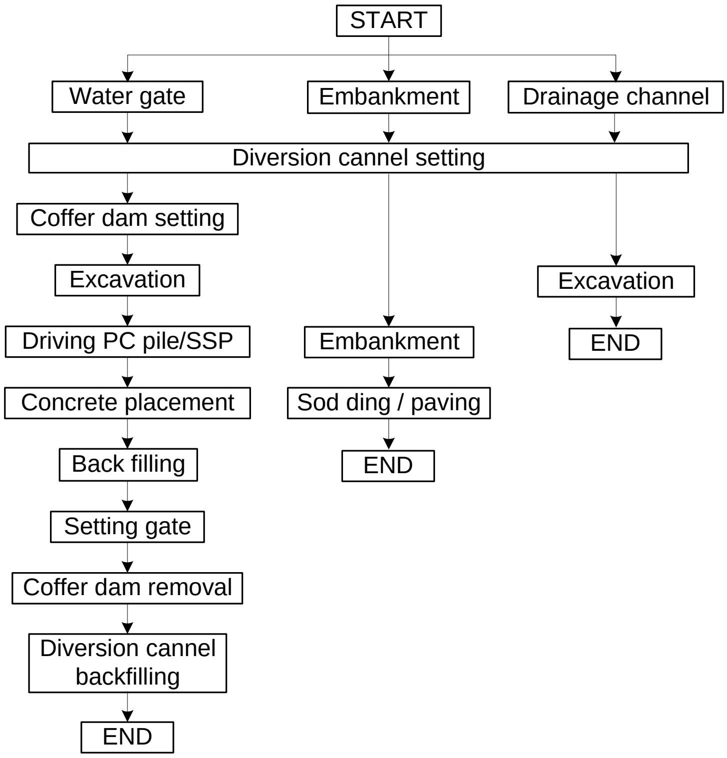


Source: JICA survey team

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

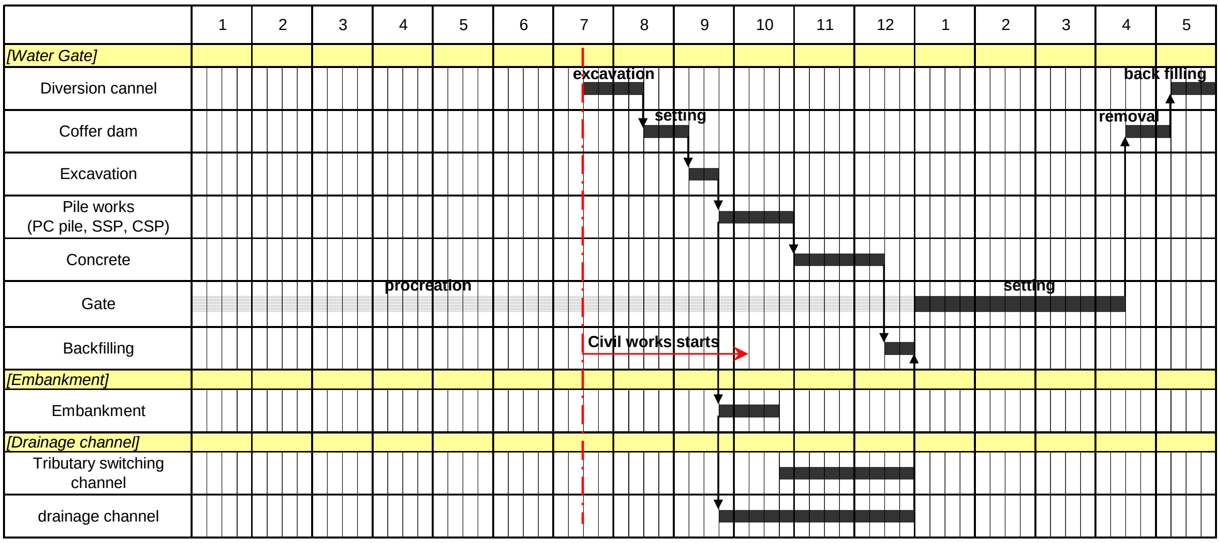
Figure 7.2.19 Construction Flow of Upstream Floodgate

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

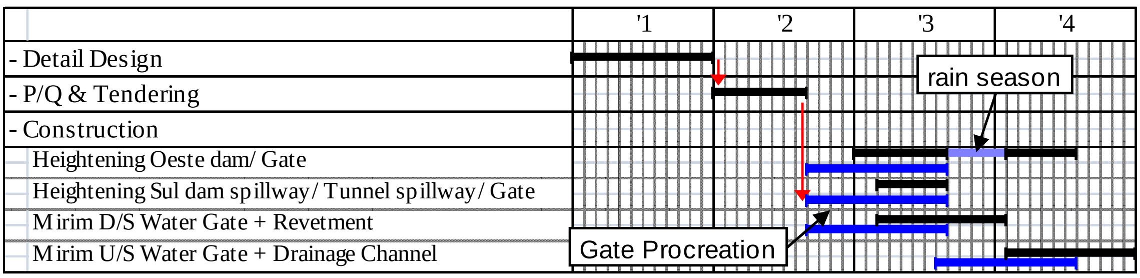
Table 7.2.9 Operation Capability

		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)

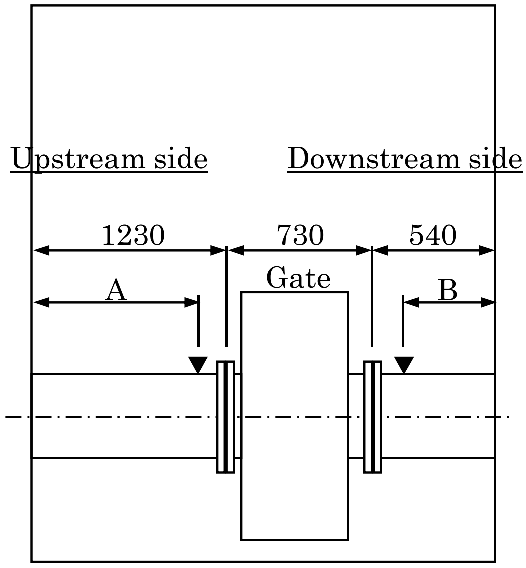
(unit:R$)

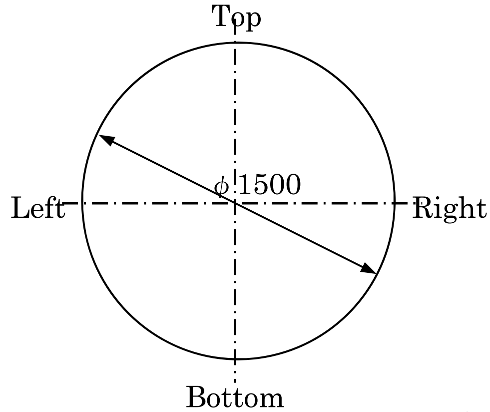
		Unit	Oeste dam		Sul dam spillway		Water Gate U/S		Water Gate D/S		Revetment		Remarks
			Quantity	Amount	Quantity	Amount	Quantity	Amount	Quantity	Amount	Quantity	Amount	
Earth Works													
Excavation (Sand)	(DMT up to 5km)	m3	59,000	885,000	4,400	66,000	4,800	72,000	3,600	54,000	---	---	
Excavation (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	

APPENDIX-1 :

Result of measure thickness

Dam Oeste Dam Date 12 May, 2011

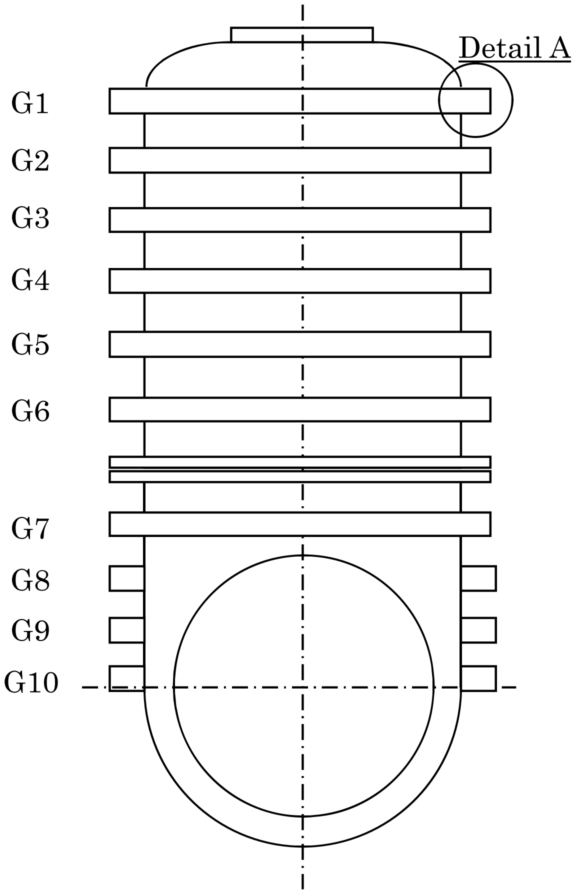




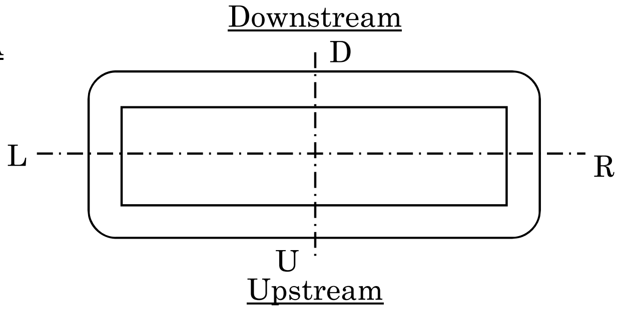
Unit : mm

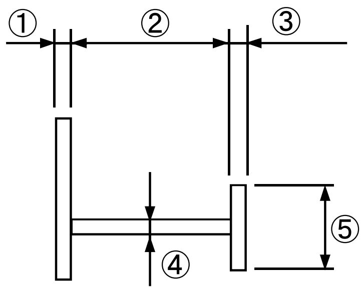
Measurement Location		Position	Result of Measurement						Evaluated thickness
			No.1	No.2	No.3	No.4	No.5	Average	
No.1 unit upstream side	A 1100	Top	5.4	5.4	6.2	6.3	5.7	5.80	5.93
		Left	5.4	5.4	5.4	5.8	6.0	5.60	
		Bottom	5.9	6.5	6.5	6.8	6.2	6.38	
		Right	—	—	—	—	—	—	
No.1 unit Downstream side	B 450	Top	6.4	6.4	7.3	6.2	6.7	6.60	6.51
		Left	6.1	6.2	6.8	6.7	6.2	6.40	
		Bottom	7.1	6.1	6.8	6.8	5.8	6.52	
		Right	—	—	—	—	—	—	

Dam	Oeste Dam	Date	12 May, 2011
Unit No.	No.1 Unit		



Front View





Detail A

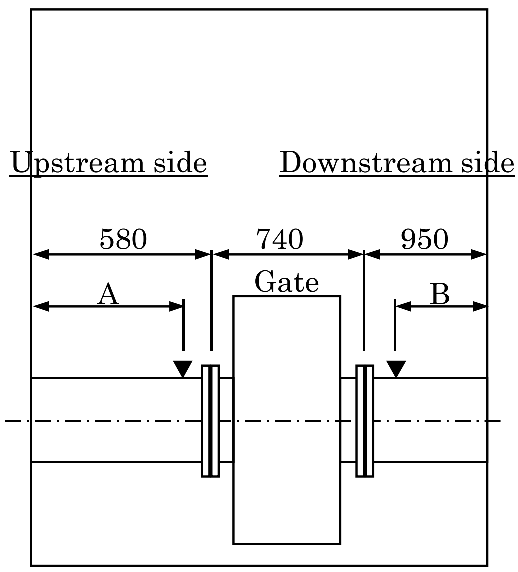
Unit : mm

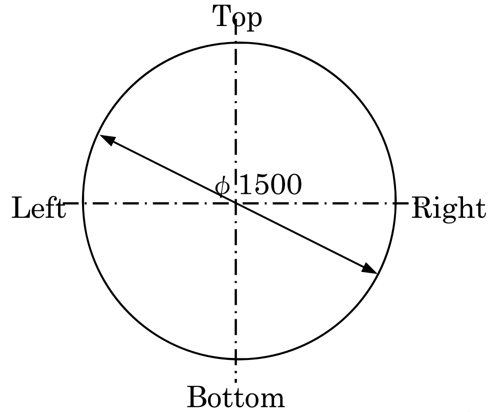
Measurement Location			Result of Measurement						
			No.1	No.2	No.3	No.4	No.5	Average	Design
G1	Right	②	–	–	–	–	–	–	100
		③	20.0	–	–	–	–	20.0	20
		⑤	65.0	–	–	–	–	65.0	60
G2	Right	②	105.0	–	–	–	–	105.0	100
		③	20.0	–	–	–	–	20.0	20
		⑤	65.0	–	–	–	–	65.0	60
G3	Right	①	12.3	12.5	12.2	13.1	12.6	12.5	15
		②	100.0	–	–	–	–	100.0	100
		③	20.0	–	–	–	–	20.0	20
		④	12.7	12.6	13.2	12.7	12.8	12.8	15
		⑤	65.0	–	–	–	–	65.0	60
G4	Right	①	13.1	11.8	11.9	13.1	12.9	12.6	15
		②	100.0	–	–	–	–	100.0	100
		③	20.0	–	–	–	–	20.0	20
		④	12.3	12.8	12.3	13.2	12.8	12.7	15
		⑤	65.0	–	–	–	–	65.0	60
G5	Right	②	100.0	–	–	–	–	100.0	100
		③	20.0	–	–	–	–	20.0	20
		⑤	65.0	–	–	–	–	65.0	60

Unit : mm

Measurement Location			Result of Measurement						
			No.1	No.2	No.3	No.4	No.5	Average	Design
G6	Right	②	100.0	ー	ー	ー	ー	100.0	100
		③	20.0	ー	ー	ー	ー	20.0	20
		⑤	65.0	ー	ー	ー	ー	65.0	60
G7	Right	①	10.5	10.6	10.2	10.1	10.6	10.4	15
		②	95.0	ー	ー	ー	ー	95.0	100
		③	20.0	ー	ー	ー	ー	20.0	20
		④	13.8	13.5	13.5	13.6	13.5	13.6	15
		⑤	65.0	ー	ー	ー	ー	65.0	60
G8	Right	②	100.0	ー	ー	ー	ー	100.0	100
		③	20.0	ー	ー	ー	ー	20.0	20
		⑤	65.0	ー	ー	ー	ー	65.0	60
G9	Right	②	100.0	ー	ー	ー	ー	100.0	100
		③	20.0	ー	ー	ー	ー	20.0	20
		⑤	65.0	ー	ー	ー	ー	65.0	60
G10	Right	②	100.0	ー	ー	ー	ー	100.0	100
		③	20.0	ー	ー	ー	ー	20.0	20
		⑤	65.0	ー	ー	ー	ー	65.0	60

Dam Sul Dam Date 12 May, 2011

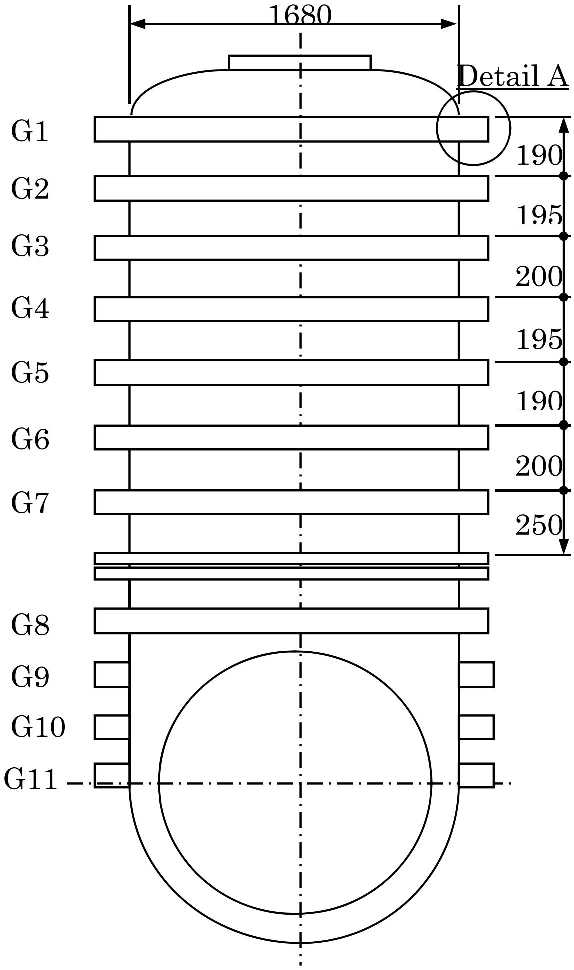




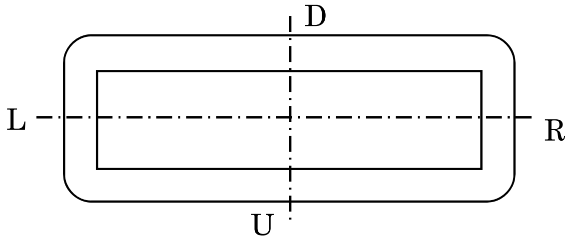
Unit : mm

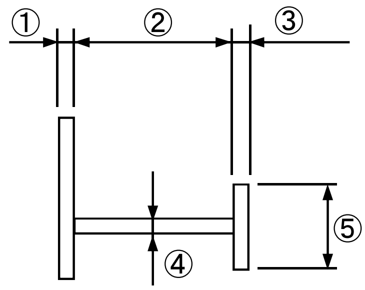
Measurement Location		Position	Result of Measurement						Evaluated thickness
			No.1	No.2	No.3	No.4	No.5	Average	
No. 2 Unit upstream side	A 250	Top	9.4	9.3	9.3	9.2	9.1	9.26	9.17
		Left	9.2	9.2	9.5	9.2	9.2	9.26	
		Bottom	9.2	9.2	8.7	9.2	8.7	9.00	
		Right	—	—	—	—	—	—	
No. 2 Unit Downstream side	B 100	Top	8.6	8.6	8.5	8.8	8.8	8.66	8.66
		Left	—	—	—	—	—	—	
		Bottom	—	—	—	—	—	—	
		Right	—	—	—	—	—	—	

Dam	Sul Dam	Date	12 May, 2011
Unit No.	No.2 Unit		



Front View





Detail A

Unit : mm

Measurement Location			Result of Measurement						
			No.1	No.2	No.3	No.4	No.5	Average	Design
G1	Upstream	①	12.5	12.5	12.6	12.6	12.7	12.58	12.7
		②	122.0	–	–	–	–	122.00	123
		③	26.0	–	–	–	–	26.00	25.4
		④	16.0	16.0	16.5	16.5	16.0	16.20	16
		⑤	100.0	–	–	–	–	100.00	100

APPENDIX-2 :

Structural calculation for control gates (After heightning)

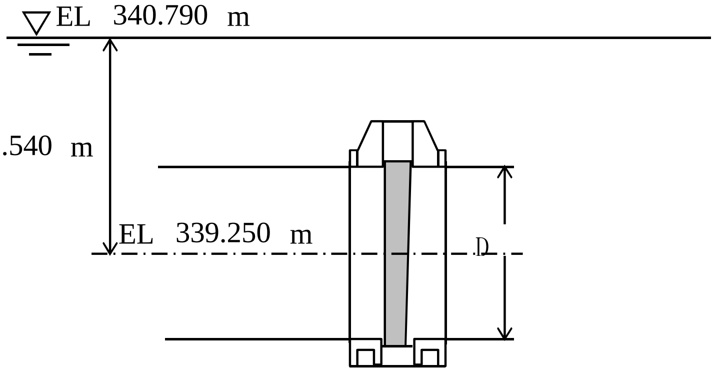
1. Strength Calculation for Control Gate in Oeste Dam (After heightning)

1.1 Design conditions

(1)	Type	Slide gate			
(2)	Quantity	7	sets		
(3)	Gate center elevation	EL.	339.25	m	
(4)	Max. water level	EL.	364.65	m	(heightning 2.0 m)
(5)	Flood water level	EL.	362.30	m	
(6)	Normal water level	EL.	340.79	m	
(7)	Diameter	φ	1.50	m	
(8)	Seismic intensity		0.05		
(9)	Sealing system	Metal seal at both side of gate leaf			
(10)	Foundation rock elevation	EL.	337.60	m	
(11)	Operation device	Hydraulic cylinder			
(12)	Lifting height		1.57	m	
(13)	Operating system	Local			
(14)	Allowable stress	ABNT NBR 8883			

1.2 Design load

(1) CCN (Normal water level Only)



D：Diameter = φ1.50 m

Load of normal water level only

Ps= γo×H×A

= 9.81 × 1.54 × 1.77

= 26.69 kN

Where, Ps ：Hydrostatic load

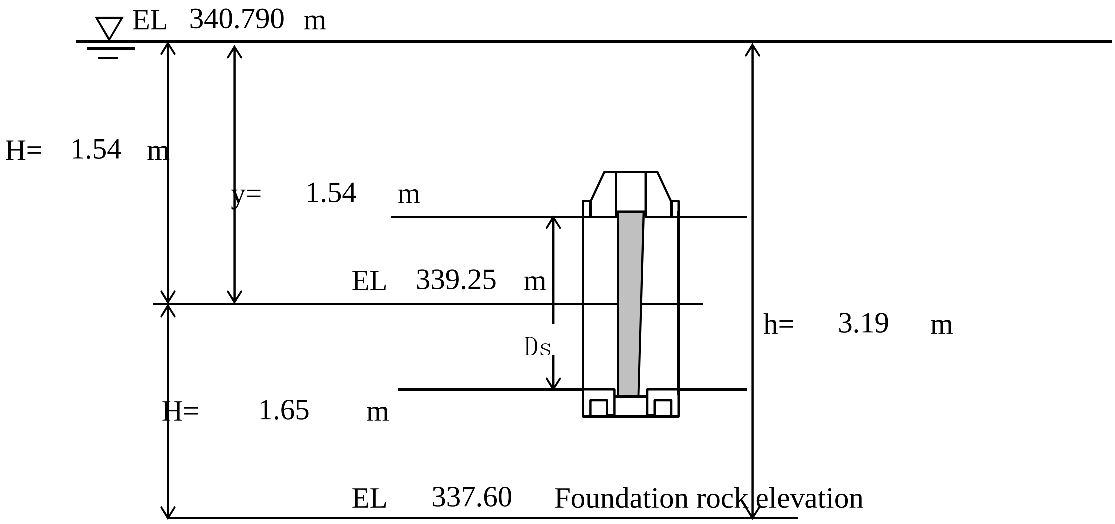
γo ：Specific gravity of water = 9.81 kN/m^3

H ：Design head = 1.54 m

A ：Receiving pressure area = $\pi \cdot Ds^2/4$ = π × 1.50 ^{2} /4

= 1.77 m^2

(2) CCE1(Normal water level + Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \times H \times A$

$= 9.81 \times 1.54 \times 1.77$

$= 26.69$ kN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 3.19 \times 1.54^{1/2} \times 1.77$

$= 1.68$ kN

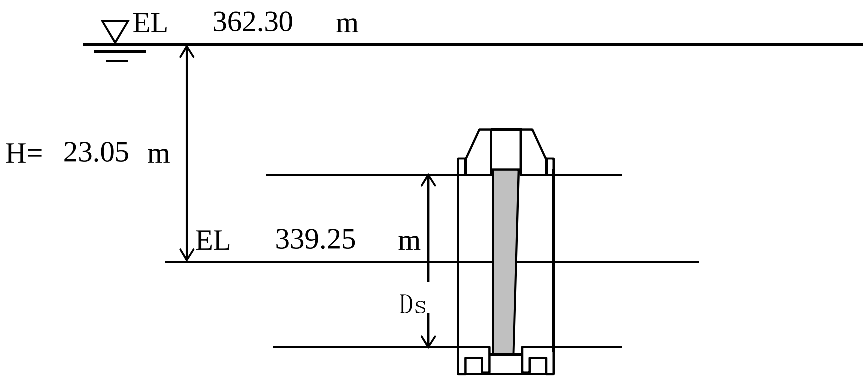
c) Total load

$Pw = Ps + P_d$

$= 26.69 + 1.68$

$= 28.38$ kN

(3) CCE2(Flood water level only)



D：Diameter = φ1.50 m

$Ps = \gamma o \times H \times A$

$= 9.81 \times 23.05 \times 1.77$

$= 399.55$ kN

Where, Ps : Hydrostatic load

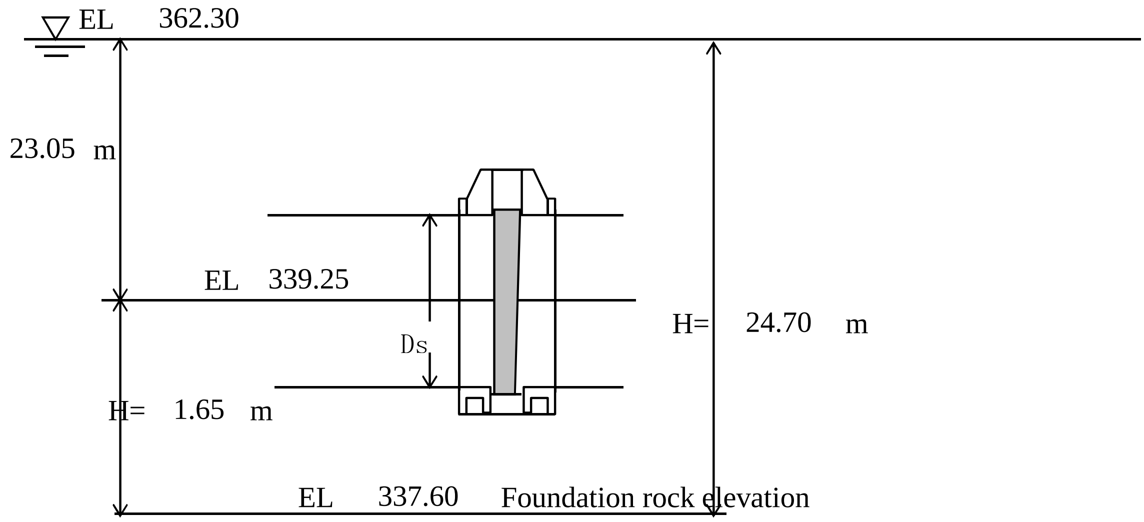
γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 23.05 m

A : Receiving pressure area = $\pi \cdot Ds^2/4$ = $\pi \times 1.50^2/4$

= 1.77 m^2

(4) CCL(Flood water level+ Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \cdot H \cdot A$

$= 9.81 \times 23.05 \times 1.77$

$= 399.55$ KN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 24.70 \times 23.05^{1/2} \times 1.77$

$= 18.10$ kN

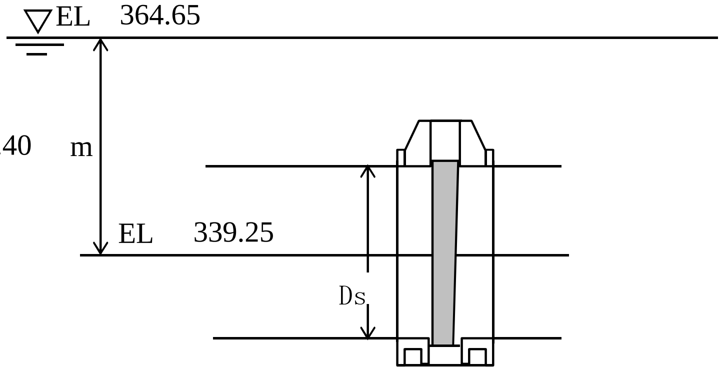
c) Total load

$Pw = Ps + P_d$

$= 399.55 + 18.10$

$= 417.65$ kN

(5) Max. water level



D : Diameter = φ1.50 m

Ps= γo×H×A

= 9.81 × 25.40 × 1.77

= 440.29 kN

Where, Ps : Hydrostatic load

γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 25.40 m

A : Receiving pressure area = $\pi \cdot Ds^2/4$ = π × $1.50^2/4$

= 1.77 m^2

(5) Comparison of loads

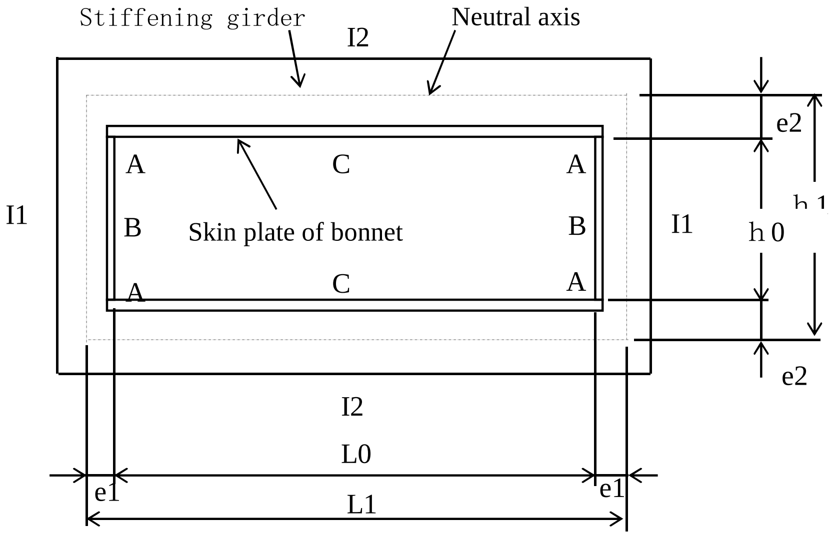
unit : kN

Case / Water level	Coefficient		Hydrostatic load only		Dynamic water pressure	
	Hydrostatic load only	Dynamic water pressure	Actual load	Converted load	Actual load	Converted load
Normal water level	0.50	0.90	26.69	53.39	28.38	31.53
			CCN		CCE1	
Flood water level	0.63	0.90	399.55	634.21	417.65	464.06
			CCE2		CCL	
Max. water level	0.80	—	440.29	550.36	—	—

The strength calculation is made for CCE2 since the maximum converted load acts on the bonnet at CCE2.

1.3 Strength calculation of bonnet

The bonnet is calculated as a box ramen as shown in the model figure below.



Where, L0 : Width of bonnet = 1575 mm

h0 : Depth of bonnet = 315 mm

(1) Internal pressure

pi= γo×H

= 9.81 × 23.05 = 226.121 kN/m^2

= 0.226 N/mm^2

pi : Internal pressure (N/mm^2)

γ0 : Specific gravity of water = 9.81 kN/m^3

H : Design head = 23.05 m

(2) Effective width of skin plate

The effective width is calculated so that the flange of stiffening girder may support the load together with the skin plate.

a) Point A

l/L≦0.02

λ= l

0.02<l/L<0.3

$\lambda=\{1.06-3.2(l/L)+4.5(l/L)^2\}\,l$

0.3≦l/L

λ= 0.15L

b) Point B and C

l/L≦0.05

λ= l

0.05<l/L<0.3

$\lambda=\{1.1-2(l/L)\}\,l$

0.3≦l/L

λ= 0.15L

Where, λ : Effective width of one side of skin plate mm

l : Half of supporting length of skin plate = 315 / 2 = 158 mm

L : Equivalent supporting length

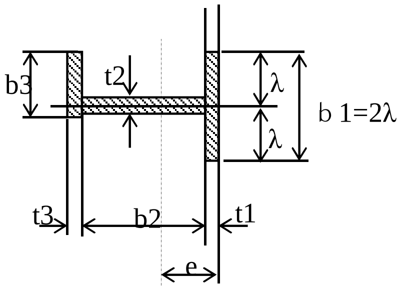
Point A = 0.2 (l0+h0) = 0.2 × (1575 + 315)= 378 mm

Point B = 0.6 h0 = 0.6 × 315 = 189 mm

Point C = 0.6 L0 = 0.6 × 1575 = 945 mm

Position	Effective width of skin plate				
	l mm	Lmm	l/L	λmm	2λmm
Point A	158	378	0.42	57	114
Point B	158	189	0.83	28	56
Point C	158	945	0.17	121	242

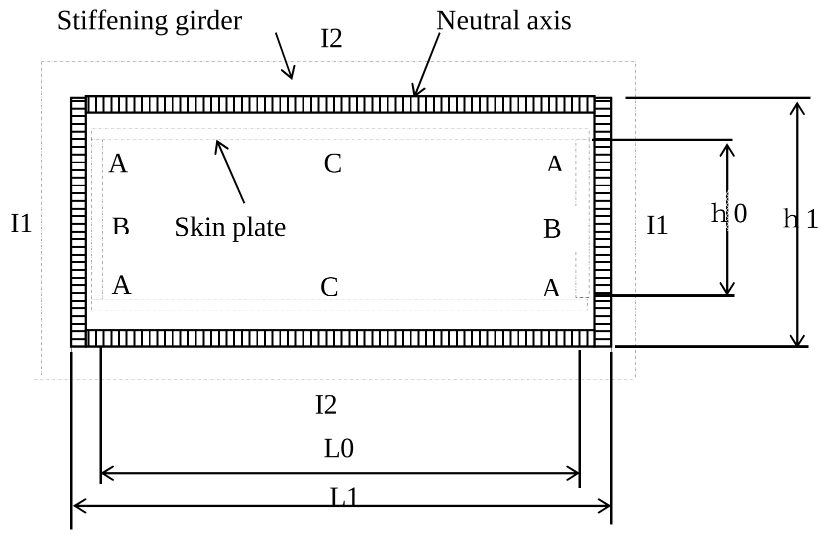
(3) Section properties of stiffening girder



t1 : Thickness of skin plate mm
t2 : Thickness of web mm
t3 : Thickness of flange mm
b1 : Effective width mm
b2 : Width of web mm
b3 : Width of flange mm

Position	Skin plate		Web		Flange		Section properties					
	t1	b1	t2	b2	t3	b3	I (mm^4)	Zi (mm^3)	Zo (mm^3)	A (mm^2)	Aw(mm^2)	e (mm)
A	12.5	114	12.8	100	20	65	10297124	166083	146058	4005	1280	62
B	12.5	56	12.8	100	20	65	7547377	101580	129680	3280	1280	74
C	12.5	242	12.8	100	20	65	13863875	300735	160462	5605	1280	46

(4) Sectional force



1) Acting load

It is assumed that the internal design pressure between the stiffeners acts as the distributed load. The acting load converts into the design load which is calculated by the ratio of an acting axis and a neutral axis.

W = pi・b・(2h0+L0)/(2h1+L1)

= 0.226 × 315 × (2 × 315 + 1575) / (2 × 407 ＋ 1724 ）

= 62 N/mm

Where, W : Converted acting load N/mm

ps : Design internal pressure = 0.226 N/mm^2

b : Width of receiving pressure = 315 mm

h0 : Depth of bonnet = 315 mm

h1 : Length of neutral axis = h0+e = 315 + 2 × 46 = 407 mm

L0 : Width of bonnet = 1575 mm

L1 : Length of neutral axis = L0+2e 1575 + 2 × 74 = 1724 mm

2) Acting load on each part

[Stiffness ratio]

k = (I2・h1)/(I1・L1)

= (13863875 × 407)/ (7547377 × 1724) = 0.434

n = h1/L1

= 407 / 1724 = 0.236

[Bending moment]

$MA=W \cdot L1^2/12 \cdot \{(1+n^2 \cdot k)/(1+k)\}$

$MB=MA-W \cdot h^2/8$

$MC=MA-W \cdot L1^2/8$

[Axial force]

Section A-B NAB=W・L1/2 (Tensile force)

Section B-C NBC=W・h1/2 (Tensile force)

[Shearing force]

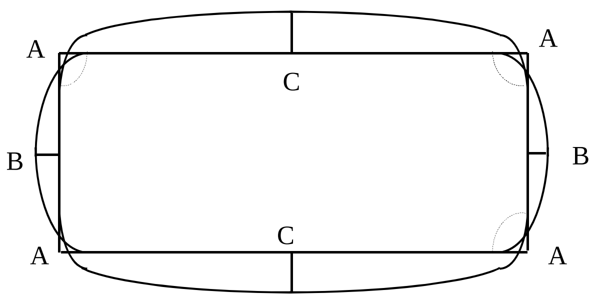
Section A-B SAB=W・h1/2

Section B-C SAC=W・l1/2

[Result of calculation]

MA= 10942410 N-mm MB= 9659805 N-mm

MC= -12037612 N-mm



VA= 0 N

NAB= 53330 N NBC= 12599 N

SAB= 12599 N SBC= 53330 N

(5) Stress of bonnet

1) Stress at "A"

Bending stress

[Bending stress(Inside)]

σAi = MA/Zi+NAB/A

= 10942410 / 166083 + 53330 / 4005

= 65.9 + 13.3 = 79.2 N/mm^2 $<$ σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MA/Zo+NAB/A

= -10942410 / 146058 + 53330 / 4005

= -74.9 + 13.3 = -61.6 N/mm^2 $<$ σa = 157.5 N/mm^2

Shearing stress

τA = SBC/Aw

= 53330 / 1280

= 41.7 N/mm^2 $<$ τa= 90.9 N/mm^2

2) Stress at "B"

Bending stress

[Bending stress(Inside)]

σBi = MB/Zi+NAB/A

= 9659805 / 101580 + 53330 / 3280

= 95.1 + 16.3 = 111.4 N/mm^2 $<$ σa = 157.5 N/mm^2

[Bending stress(Outside)]

σBo = -MB/Zo+NAB/A

= -9659805 / 129680 + 53330 / 3280

= -74.5 + 16.3 = -58.2 N/mm^2 $<$ σa = 157.5 N/mm^2

Shearing stress

τB = SAB/Aw

= 12599 / 1280

= 9.8 N/mm^2 $<$ τa= 90.9 N/mm^2

3) Stress at "C"

Bending stress

[Bending stress(Inside)]

σCi = MC/Zi+NBC/A

= -12037612 / 300735 + 12599 / 5605

= -40.0 + 2.2 = -37.8 N/mm^2 $<$ σa = 157.5 N/mm^2

[Bending stress(Outside)]

σCo = -MC/Zo+NBC/A

= 12037612 / 160462 + 12599 / 5605

= 75.0 + 2.2 = 77.3 N/mm^2 $<$ σa = 157.5 N/mm^2

Shearing stress

τC = SBC/Aw

= 53330 / 1280

= 41.7 N/mm^2 $<$ τa= 90.9 N/mm^2

(6) Allowable stresses

Allowable bending stress

Outside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Inside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Coefficient : 0.63

Allowable shearing stress

Outside

$\tau a = 90.9$ N/mm^2 Material : A36(ASTM)

1.4 Operating load

The operating load is summed up the following loads.

(1) Self weight

Gate leaf	G1	=	1.6 × 9.81 = 15.7 kN
Rod of cylinder	G2	=	0.77 kN
Total load	G	=	16.47 kN

(2) Friction force of seal plate

$F2 = \mu_2 \cdot P$

$= 0.4 \times 440.290 = 176.12$ kN

Where, μ_2 : Frictional coefficient of metal seal = 0.4

P : Hydrostatic pressure at operation = 440.29 kN

(3) Buoyancy

$F3 = \gamma 0 / W0 \cdot G1$

$= 9.81 / 77.0 \times 15.70 = 2.00$ kN

ここに、 $\gamma 0$: Specific gravity of water = 9.81 kN/m3

W0 : Specific gravity of steel material = 77.01 kN/m3

(4) Friction force of seal in cylinder

$F4 = d \cdot \pi \cdot b \cdot n \cdot \mu_2 \cdot P$

$= 0.090 \times \pi \times 0.006 \times 1 \times 0.7 \times 440.290 = 0.523$ kN

Where, d : Outside diameter of rod = 0.090 m

b : Contact width of V-packing = 0.006 m

n : Quantity of V-packing = 1 piece

μ_2 : Frictional coefficient of V-packing = 0.7

P : Pressure on V-packing = 440.290 kN

(5) Total operating load

(Unit : kN)

Load		Raising		Lowering	
Self weight	G	↓	16.47	↓	16.47
Friction force of seal plate	F2	↓	176.12	↑	176.12
Buoyancy	F3	↑	2.00	↑	2.00
Friction force of seal in cylinder	F4	↓	0.52	↑	0.52
Total load		↓	191.11	↑	162.17

Raising load	Fu	=	191.11	kN	→	200.00	kN
Lowering load	Fd	=	162.17	kN	→	170.00	kN

1.5 Capacity of cylinder

(1) Design conditions

Type of cylinder	Fixed cylinder				
Rated pressure	Raising（Setting pressure of relief valve）	P1	=	21.0	MPa
	Lowering（Setting pressure of relief valve）	P2	=	12.6	MPa
Working pressure	Raising（Effective operating pressure）	P1'	=	18.9	MPa
	Lowering（Effective operating pressure）	P2'	=	11.3	MPa
Operating speed	0.1 m/min				
Operating load	Raising	Wu	=	200.00	kN
	Lowering	Wd	=	170.00	kN
Cylinder	Inside diameter of tube	D	=	160	mm
	Outside diameter of rod	d	=	90	mm
	Cylinder stroke	S	=	1570	mm

(2) Pulling and pushing forces of cylinder

1）Rated pressure

Pulling force (Raising)

$$F_u = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (160^2 - 90^2) \times \frac{21.0}{1000}$$

$$= 288.6 \text{ kN}$$

Pushing force (Lowering)

$$F_d = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 160^2 \times \frac{12.6}{1000}$$

$$= 253.3 \text{ kN}$$

2）Working pressure

Pulling force (Raising)

$$F_u' = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (160^2 - 90^2) \times \frac{18.9}{1000}$$

$$= \underline{259.8 \text{ kN} > W_u = 200 \text{ kN}}$$

Pushing force (Lowering)

$$F_d' = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 160^2 \times \frac{11.3}{1000}$$

$$= \underline{228.0 \text{ kN} > W_d = 170.00 \text{ kN}}$$

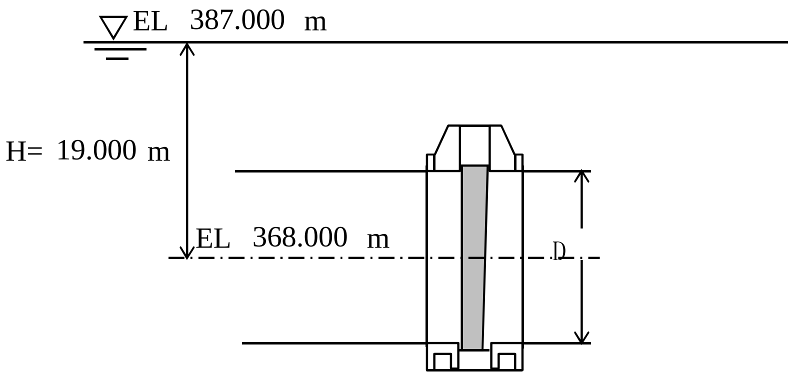
2. Strength Calculation for control gate in Oeste dam (After heightning)

2.1 Design conditions

(1)	Type	Slide gate			
(2)	Quantity	5	sets		
(3)	Gate center elevation	EL.	368.00	m	
(4)	Max. water level	EL.	408.00	m	(heightning 2.0 m)
(5)	Flood water level	EL.	401.00	m	
(6)	Normal water level	EL.	387.00	m	
(7)	Diameter	φ	1.50	m	
(8)	Seismic intensity		0.05		
(9)	Sealing system	Metal seal at both side of gate leaf			
(10)	Basic grand level	EL.	357.50	m	
(11)	Operation device	Hydraulic cylinder			
(12)	Lifting height		1.57	m	
(13)	Operating system	Local			
(14)	Allowable stress	ABNT NBR 8883			

2.2 Design head

(1) CCN (Nomal water level Only)



D : Diameter = φ1.50 m

Load of normal water level only

$Ps = \gamma o \times H \times A$

$= 9.81 \times 19.00 \times 1.77$

$= 329.35$ kN

Where, Ps : Hydrostatic load

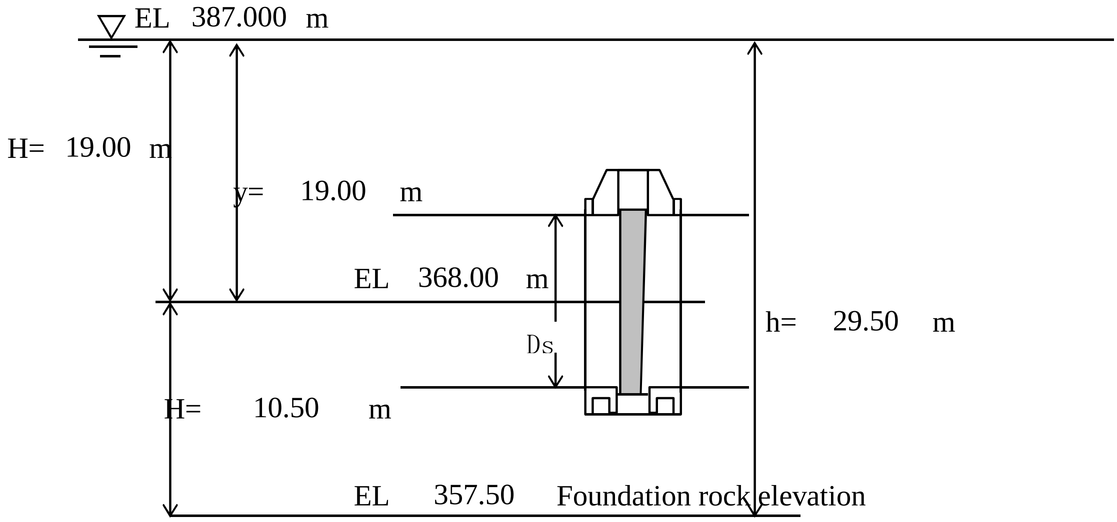
γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 19.00 m

A : Receiving pressure area = $\pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77\ m^2$

(2) CCE1(Normal water level + Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \times H \times A$

$= 9.81 \times 19.00 \times 1.77$

$= 329.35$ kN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 29.50 \times 19.00^{1/2} \times 1.77$

$= 17.95$ kN

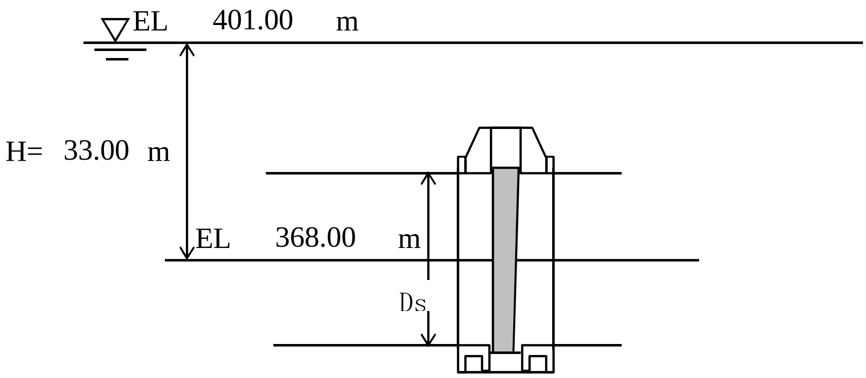
c) Total load

$Pw = Ps + P_d$

$= 329.35 + 17.95$

$= 347.31$ kN

(3) CCE2(Flood water level only)



D：Caliber = φ1.50 m

$Ps = \gamma o \times H \times A$

$= 9.81 \times 33.00 \times 1.77$

$= 572.03$ kN

Where, Ps : Hydrostatic load

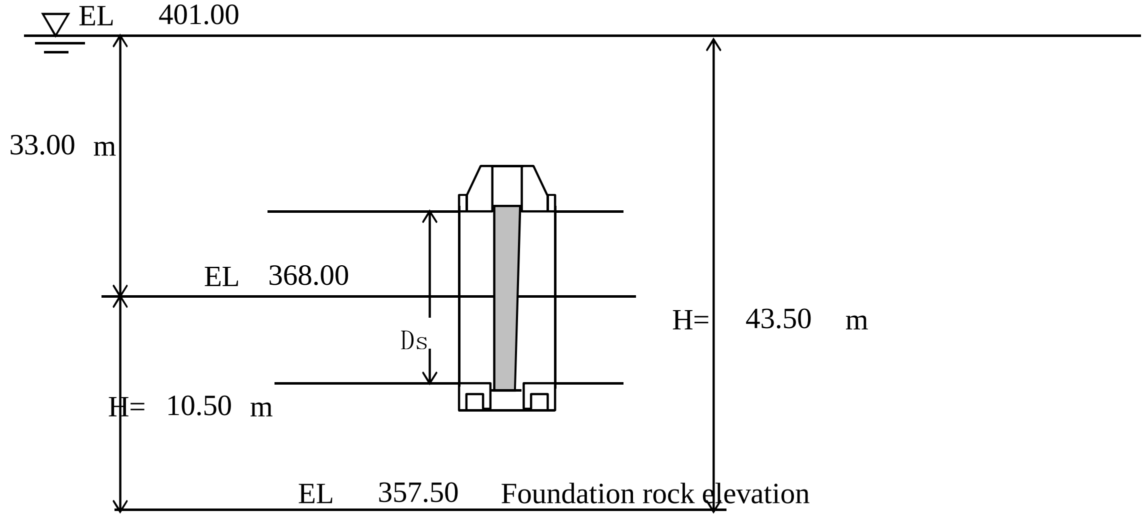
γo : Specific gravity of water $= 9.81$ kN/m^3

H : Design head $= 33.00$ m

A : Receiving pressure area $= \pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77$ m^2

(4) CCL(Flood water level+ Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \cdot H \cdot A$

$= 9.81 \times 33.00 \times 1.77$

$= 572.03$ KN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 43.50 \times 33.00^{1/2} \times 1.77$

$= 28.73$ kN

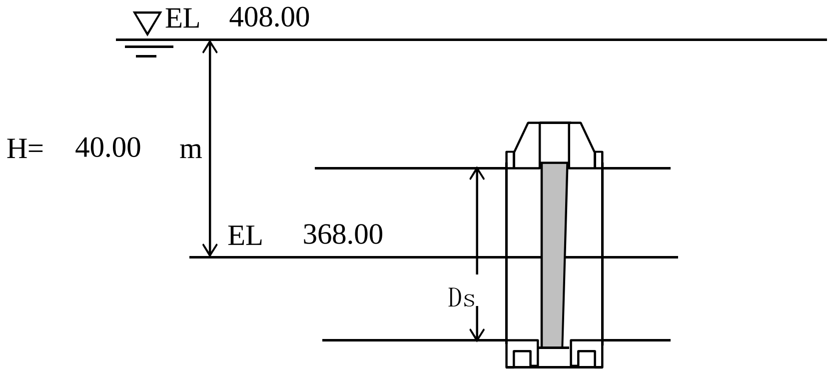
c) Total load

$Pw = Ps + P_d$

$= 572.03 + 28.73$

$= 600.76$ kN

(5) Max. water level



D : Diameter = φ1.50 m

$Ps = \gamma o \times H \times A$

$= 9.81 \times 40.00 \times 1.77$

$= 693.37$ kN

Where, Ps : Hydrostatic load

γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 40.00 m

A : Receiving pressure area $= \pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77$ m^2

(5) Comparison of loads

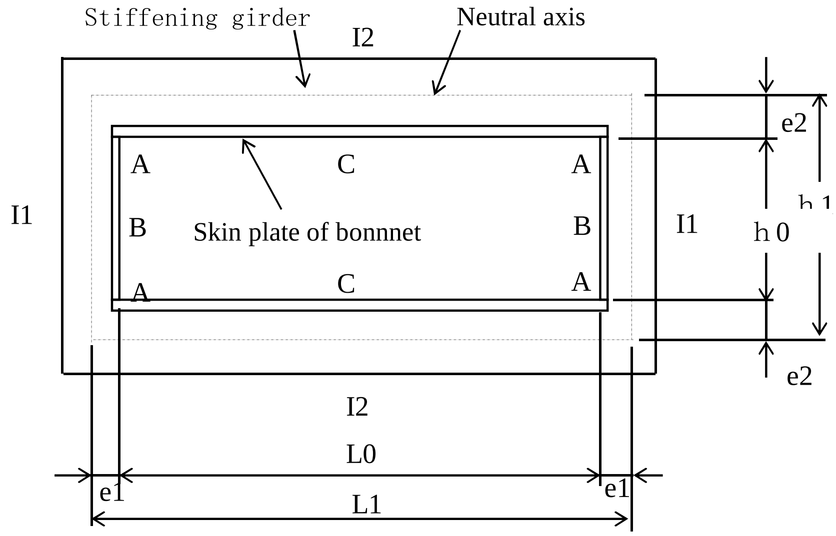
unit : kN

Case / 水　位	Coefficient		Hydrostatic load only		Dynamic water pressure	
	Hydrostatic load only	Dynamic water pressure	Actual load	Converted load	Actual load	Converted load
Normal water level	0.50	0.90	329.35	658.70	347.31	385.90
			CCN		CCE1	
Flood water level	0.63	0.90	572.03	907.99	600.76	667.52
			CCE2		CCL	
Max. water level	0.80	—	693.37	866.71	—	—

Because the load of "CCE2" becomes the maximum, strength of the load of "CCE2" is checked.

2.3 Strength calculation of bonnet

The bonnet is calculated as a box ramen as shown in the model figure below.



Where, L0 : Width of bonnet = 1650 mm

h0 : Depth of bonnet = 315 mm

(1) Internal pressure

pi= γo×H

= 9.81 × 33.00 = 323.73 kN/m^2

= 0.324 N/mm^2

pi : Internal pressure (N/mm^2)

γ0 : Specific gravity of water = 9.81 kN/m^3

H : Design head = 33.00 m

(2) Effective width of skin plate

The effective width is calculated so that the flange of stiffening girder may support the load together with the skin plate.

a) Point of A

$l/L \leqq 0.02$

$\lambda = l$

$0.02 < l/L < 0.3$

$\lambda = \{1.06 - 3.2(l/L) + 4.5(l/L)^2\}\, l$

$0.3 \leqq l/L$

$\lambda = 0.15L$

b) Point of B and C

$l/L \leqq 0.05$

$\lambda = l$

$0.05 < l/L < 0.3$

$\lambda = \{1.1 - 2(l/L)\}\, l$

$0.3 \leqq l/L$

$\lambda = 0.15L$

Where, λ : Working width in one side of skinplate mm

l : Half of skin plate at support intervals = 315 / 2 = 158 mm

L : Equivalent support inter

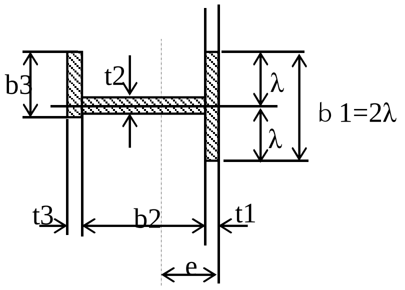
Point A = 0.2 (l0+h0) = 0.2 × (1650 + 315)= 393 mm

Point B = 0.6 h0 = 0.6 × 315 = 189 mm

Point C = 0.6 L0 = 0.6 × 1650 = 990 mm

Position	Effective width of skin plate				
	l mm	Lmm	l/L	λmm	2λmm
Point A	158	393	0.40	59	118
Point B	158	189	0.83	28	56
Point C	825	990	0.83	149	298

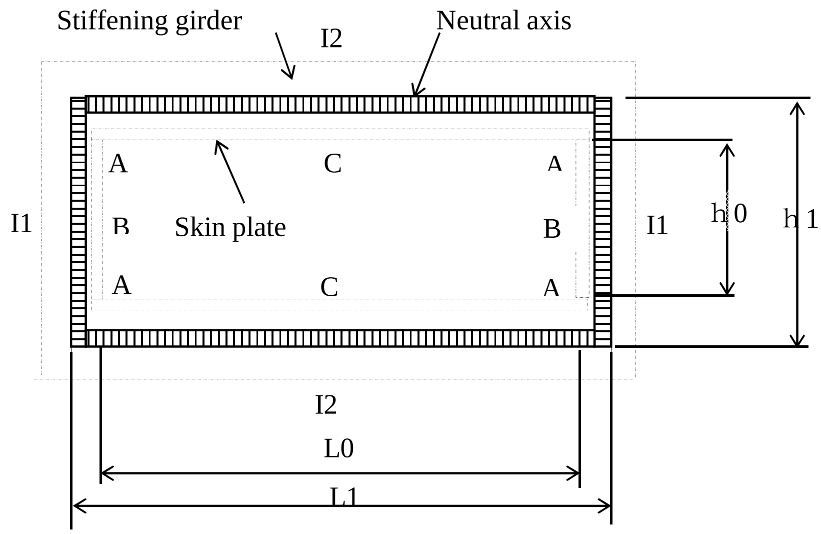
(3) Section properties of stiffening girder



t1 : Thickness of skin plate mm
t2 : Thickness of web mm
t3 : Thickness of flange mm
b1 : Effective width mm
b2 : Width of web mm
b3 : Width of flange mm

Position	Skin plate		Web		Flange		Section properties					
	t1	b1	t2	b2	t3	b3	I (mm^4)	Zi (mm^3)	Zo (mm^3)	A (mm^2)	Aw(mm^2)	e (mm)
A	12.6	118	16.2	122	26	100	22214599	250164	309396	6063	1976.4	89
B	12.6	56	16.2	122	26	100	16102576	159431	270177	5282	1976.4	101
C	12.6	298	16.2	122	26	100	33413694	503218	354710	8331	1976.4	66

(4) Sectional force



1) Acting load

It is assumed that the internal design pressure between the stiffeners acts as the distributed load. The acting load converts into the design load which is calculated by the ratio of an acting axis and a neutral axis.

$W = pi \cdot b \cdot (2h0+L0)/(2h1+L1)$

$= 0.324 \times 315 \times (2 \times 315 + 1650) / (2 \times 448 + 1852)$

$= 85$ N/mm

Where, W : Converted acting load N/mm

ps : Design internal pressure = 0.324 N/mm^2

b : Width of receiving pressure = 315 mm

h0 : Depth of bonnet = 315 mm

h1 : Length of neutral axis = h0+e = 315 + 2 × 66 = 448 mm

L0 : Width of bonnet = 1650 mm

L1 : Length of neutral axis = L0+2e : 1650 + 2 × 101 = 1852 mm

2) Acting load on each part

[Stiffness ratio]

$k = (I2 \cdot h1)/(I1 \cdot L1)$

$= (33413694 \times 448)/(16102576 \times 1852) = 0.502$

$n = h1/L1$

$= 448 / 1852 = 0.242$

[Bending moment]

$MA = W \cdot L1^2/12 \cdot \{(1+n^2 \cdot k)/(1+k)\}$

$MB = MA - W \cdot h^2/8$

$MC = MA - W \cdot L1^2/8$

[Axial force]

Section A-B NAB=W・L1/2（Tensile force）

Section B-C NBC=W・h1/2（Tensile force）

[Shearing force]

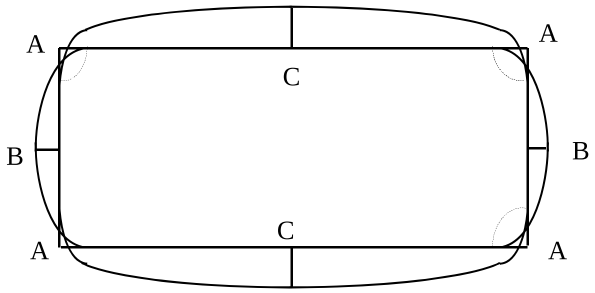
Section A-B SAB=W・h1/2

Section B-C SAC=W・l1/2

[Result of calculation]

MA= 16578259 N-mm MB= 14457199 N-mm

MC= -19701702 N-mm



VA= 0 N

NAB= 78358 N NBC= 18946 N

SAB= 18946 N SBC= 78358 N

(5) Stress of bonnet

1) Stress at "A"

Bending stress

[Bending stress(Inside)]

σAi = MA/Zi+NAB/A

= 16578259 / 250164 + 78358 / 6063

= 66.3 + 12.9 = 79.2 N/mm^2 ＜σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MA/Zo+NAB/A

= -16578259 / 309396 + 78358 / 6063

= -53.6 + 12.9 = -40.7 N/mm^2 ＜σa = 157.5 N/mm^2

Shearing stress

τA = SBC/Aw

= 78358 / 1976

= 39.6 N/mm^2 ＜τa= 90.9 N/mm^2

2) Stress of "B"

Bending stress

[Bending stress(Inside)]

σAi = MB/Zi+NAB/A

= 14457199 / 159431 + 78358 / 5282

= 90.7 + 14.8 = 105.5 N/mm^2 ＜σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MB/Zo+NAB/A

= -14457199 / 270177 + 78358 / 5282

= -53.5 + 14.8 = -38.7 N/mm^2 ＜σa = 157.5 N/mm^2

Shearing stress

τA = SAB/Aw

= 18946 / 1976

= 9.6 N/mm^2 ＜τa= 90.9 N/mm^2

3) Stress of "C"

Bending stress

[Bending stress(Inside)]

σAi = MC/Zi+NBC/A

= -19701702 / 503218 + 18946 / 8331

= -39.2 + 2.3 = -36.9 N/mm^2 ＜σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MC/Zo+NBC/A

= 19701702 / 354710 + 18946 / 8331

= 55.5 + 2.3 = 57.8 N/mm^2 ＜σa = 157.5 N/mm^2

Shearing stress

τA = SBC/Aw

= 78358 / 1976

= 39.6 N/mm^2 ＜τa= 90.9 N/mm^2

(6) Allowable stresses

Allowable bending stress

Outside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Inside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Coefficient : 0.63

Allowable shearing stress

Outside

$\tau a = 90.9$ N/mm^2 Material : A36(ASTM)

2.4 Operating load

The operating load is summed up the following loads.

(1) Self weight

Gate leaf G1 = 2.5 × 9.81 = 24.53 kN

Rod of cylinder G2 = 0.77 kN

Total load G = 25.30 kN

(2) Seal friction

$F2 = \mu_2 \cdot P$

= 0.4 × 693.371 = 277.35 kN

Where, μ_2 : Frictional coefficient of metal seal = 0.4

P : Hydrostatic pressure at operation = 693.37 kN

(3) Buoyancy

$F3 = \gamma 0 / W0 \cdot G1$

= 9.81 / 77.0 × 24.53 = 3.12 kN

ここに、 $\gamma 0$: Specific gravity of water = 9.81 kN/m3

W0 : Specific gravity of steel material = 77.01 kN/m3

(4) Friction force of seal in cylinder

$F4 = d \cdot \pi \cdot b \cdot n \cdot \mu_2 \cdot P$

= 0.090 × π × 0.006 × 1 × 0.7 × 693.371 = 0.823 kN

Where, d : Rod outside diameter = 0.090 m

b : Width of contact of V-packing = 0.006 m

n : Quantity of V-paccking = 1 piece

μ_2 : Frictional coefficient of V-packing = 0.7

P : Pressure on V-packing = 693.371 kN

(5) Total operating load

(Unit : kN)

Load		Raising	Lowering
Self weight	G	↓ 25.30	↓ 25.30
Seal friction	F2	↓ 277.35	↑ 277.35
Buoyancy	F3	↑ 3.12	↑ 3.12
Friction force of seal in cylinder	F4	↓ 0.82	↑ 0.82
Total load		↓ 300.34	↑ 256.00

Raising load	Fu	=	300.34	kN	→	310.00	kN
Lowerring load	Fd	=	256.00	kN	→	260.00	kN

2.5 Capacity of cylinder

(1) Design conditions

Type of hoist	Fixed cylinder				
Rated pressure	Raising（Setting pressure of relief valve）	P1	=	16.0	MPa
	Lowering（Setting pressure of relief valve）	P2	=	9.6	MPa
Working pressure	Raising（Effective operating pressure）	P1'	=	14.4	MPa
	Lowering（Effective operating pressure）	P2'	=	8.6	MPa
Operating speed	0.1 m/min				
Operating load	Raising	Wu	=	310.00	kN
	Lowerring	Wd	=	260.00	kN
Cylinder	Inside diameter of tube	D	=	200	mm
	Outside diameter of rod	d	=	100	mm
	Cylinder stroke	S	=	1570	mm

(2) Power to push and power to pull

1）Rated pressure

Pulling force (Raising)

$$F_u = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (200^2 - 100^2) \times \frac{16.0}{1000}$$

$$= 377 \quad kN$$

Pushing force (Lowering)

$$F_d = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 200^2 \times \frac{9.6}{1000}$$

$$= 301.6 \quad kN$$

2）Working pressure

Pulling force (Raising)

$$F_u' = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (200^2 - 100^2) \times \frac{14.4}{1000}$$

$$= \underline{339.3 \quad kN \quad > \quad W_u = 310 \quad kN}$$

Pushing force (Lowering)

$$F_d' = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 200^2 \times \frac{8.6}{1000}$$

$$= \underline{271.4 \quad kN \quad > \quad W_d = 260.00 \quad kN}$$

APPENDIX-3 :

Structural calculation for control gates (Before heightning)

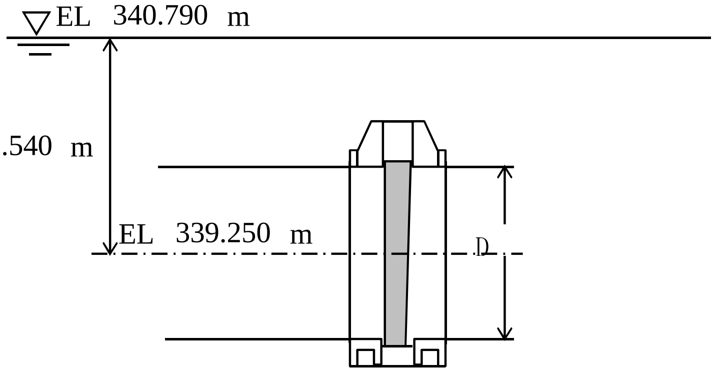
1. Strength Calculation for Control Gate in Oeste Dam (Before heightning)

1.1 Design conditions

(1)	Type	Slide gate
(2)	Quantity	7 sets
(3)	Gate center elevation	EL. 339.25 m
(4)	Max. water level	EL. 362.65 m (heightning 0.0 m)
(5)	Flood water level	EL. 360.30 m
(6)	Normal water level	EL. 340.79 m
(7)	Diameter	φ 1.50 m
(8)	Seismic intensity	0.05
(9)	Sealing system	Metal seal at both side of gate leaf
(10)	Foundation rock elevation	EL. 337.60 m
(11)	Operation device	Hydraulic cylinder
(12)	Lifting height	1.57 m
(13)	Operating system	Local
(14)	Allowable stress	ABNT NBR 8883

1.2 Design load

(1) CCN (Normal water level Only)



D：Diameter = φ1.50 m

Load of normal water level only

$Ps = \gamma o \times H \times A$

$= 9.81 \times 1.54 \times 1.77$

$= 26.69$ kN

Where, Ps : Hydrostatic load

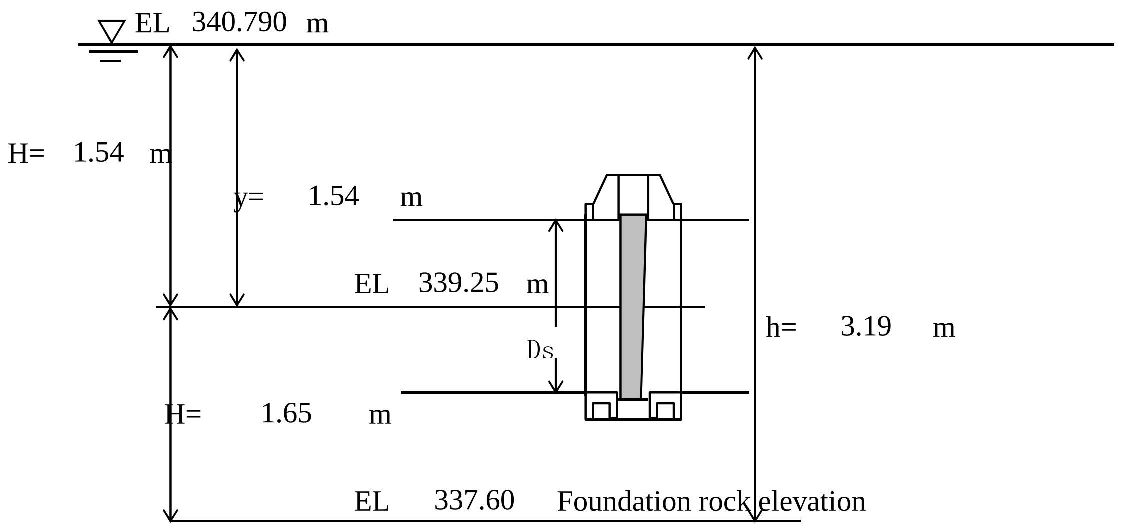
γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 1.54 m

A : Receiving pressure area = $\pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77\ m^2$

(2) CCE1(Normal water level + Dynamic water pressure during earthquake)



a) Hydrostatic load

Ps= γo×H×A

= 9.81 × 1.54 × 1.77

= 26.69 kN

b) Dynamic pressure load during earthquake

P_d= γo・7/8・k・(h・y)$^{1/2}$・A

= 9.81 × 7/8 × 0.05 × 3.19 × 1.54 $^{1/2}$ × 1.77

= 1.68 kN

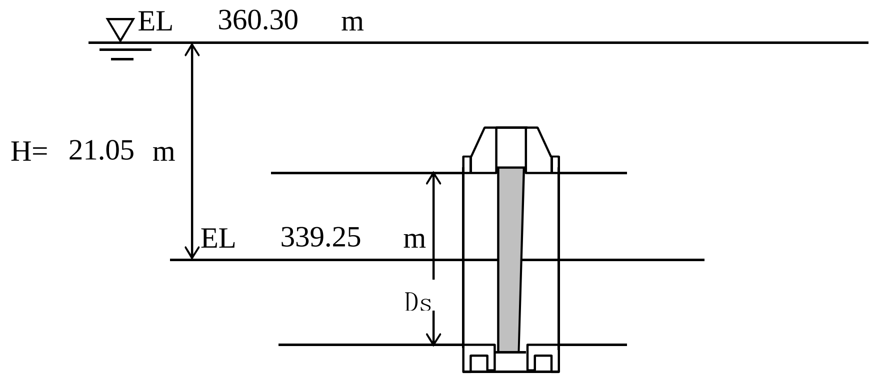
c) Total load

Pw= Ps+P_d

= 26.69 ＋ 1.68

= 28.38 kN

(3) CCE2(Flood water level only)



D：Diameter = φ1.50 m

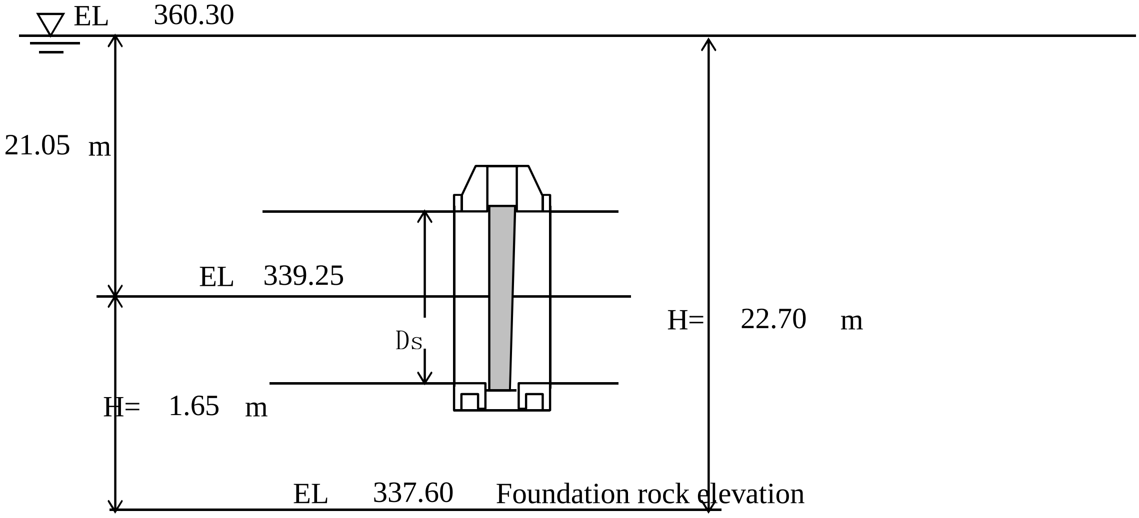
$Ps = \gamma o \times H \times A$

$= 9.81 \times 21.05 \times 1.77$

$= 364.89$ kN

Where,
- Ps : Hydrostatic load
- γo : Specific gravity of water = 9.81 kN/m^3
- H : Design head = 21.05 m
- A : Receiving pressure area $= \pi \cdot Ds^2/4 = \pi \times 1.50^2/4$ $= 1.77$ m^2

(4) CCL(Flood water level+ Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \cdot H \cdot A$

$= 9.81 \times 21.05 \times 1.77$

$= 364.89$ KN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 22.70 \times 21.05^{1/2} \times 1.77$

$= 16.58$ kN

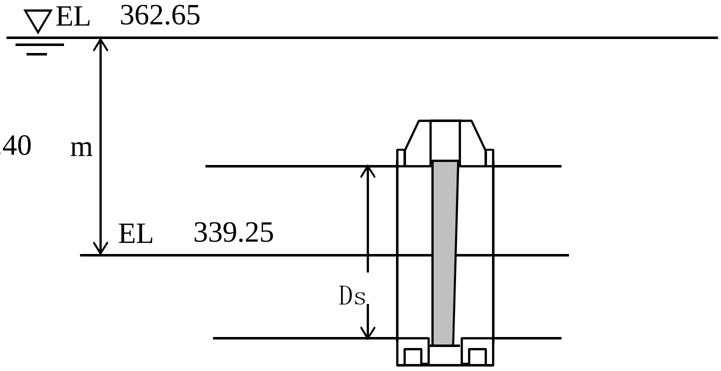
c) Total load

$Pw = Ps + P_d$

$= 364.89 + 16.58$

$= 381.46$ kN

(5) Max. water level



D : Diameter = φ1.50 m

$Ps = \gamma o \times H \times A$

$= 9.81 \times 23.40 \times 1.77$

$= 405.62$ kN

Where, Ps : Hydrostatic load

γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 23.40 m

A : Receiving pressure area $= \pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77$ m^2

(5) Comparison of loads

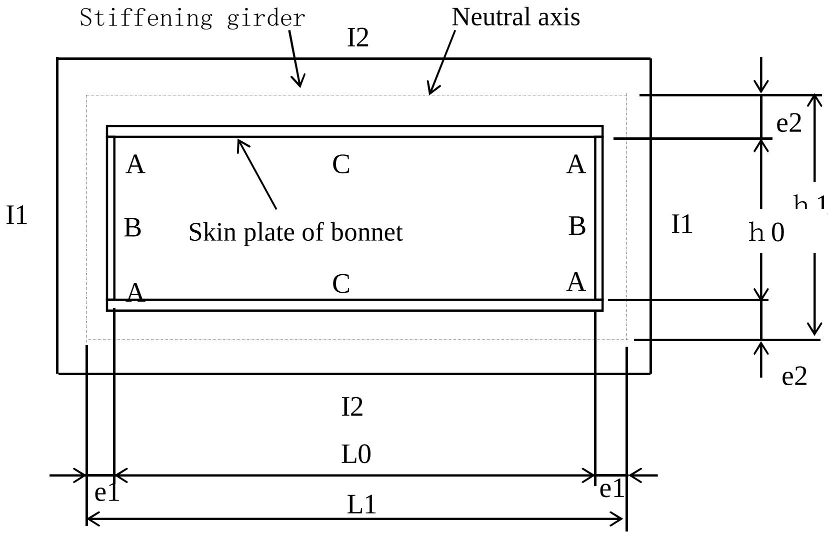
unit : kN

Case / Water level	Coefficient		Hydrostatic load only		Dynamic water pressure	
	Hydrostatic load only	Dynamic water pressure	Actual load	Converted load	Actual load	Converted load
Normal water level	0.50	0.90	26.69	53.39	28.38	31.53
			CCN		CCE1	
Flood water level	0.63	0.90	364.89	579.18	381.46	423.85
			CCE2		CCL	
Max. water level	0.80	—	405.62	507.03	—	—

The strength calculation is made for CCE2 since the maximum converted load acts on the bonnet at CCE2.

1.3 Strength calculation of bonnet

The bonnet is calculated as a box ramen as shown in the model figure below.



Where, L0 : Width of bonnet = 1575 mm

h0 : Depth of bonnet = 315 mm

(1) Internal pressure

pi= γo×H

= 9.81 × 21.05 = 206.501 kN/m^2

= 0.207 N/mm^2

pi : Internal pressure (N/mm^2)

γ0 : Specific gravity of water = 9.81 kN/m^3

H : Design head = 21.05 m

(2) Effective width of skin plate

The effective width is calculated so that the flange of stiffening girder may support the load together with the skin plate.

a) Point A

$l/L \leqq 0.02$

$\lambda = l$

$0.02 < l/L < 0.3$

$\lambda = \{1.06 - 3.2(l/L) + 4.5(l/L)^2\}\ l$

$0.3 \leqq l/L$

$\lambda = 0.15L$

b) Point B and C

$l/L \leqq 0.05$

$\lambda = l$

$0.05 < l/L < 0.3$

$\lambda = \{1.1 - 2(l/L)\}\ l$

$0.3 \leqq l/L$

$\lambda = 0.15L$

Where, λ : Effective width of one side of skin plate mm

l : Half of supporting length of skin plate = 315 / 2 = 158 mm

L : Equivalent supporting length

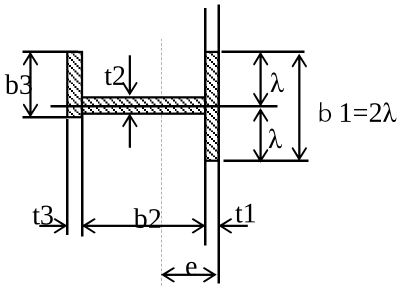
Point A = 0.2 (l0+h0) = 0.2 × (1575 + 315)= 378 mm

Point B = 0.6 h0 = 0.6 × 315 = 189 mm

Point C = 0.6 L0 = 0.6 × 1575 = 945 mm

Position	Effective width of skin plate				
	l mm	Lmm	l/L	λmm	2λmm
Point A	158	378	0.42	57	114
Point B	158	189	0.83	28	56
Point C	158	945	0.17	121	242

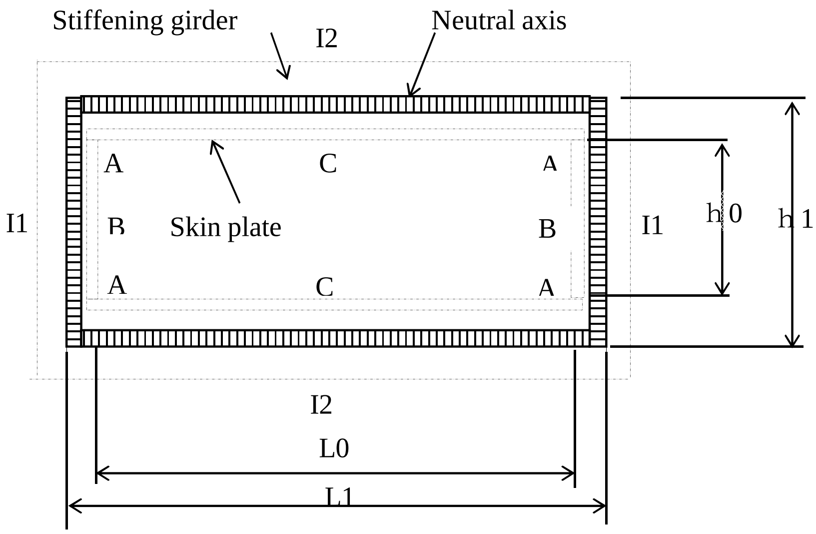
(3) Section properties of stiffening girder



t1 : Thickness of skin plate mm
t2 : Thickness of web mm
t3 : Thickness of flange mm
b1 : Effective width mm
b2 : Width of web mm
b3 : Width of flange mm

Position	Skin plate		Web		Flange		Section properties					
	t1	b1	t2	b2	t3	b3	I (mm^4)	Zi (mm^3)	Zo (mm^3)	A (mm^2)	Aw(mm^2)	e (mm)
A	12.5	114	12.8	100	20	65	10297124	166083	146058	4005	1280	62
B	12.5	56	12.8	100	20	65	7547377	101580	129680	3280	1280	74
C	12.5	242	12.8	100	20	65	13863875	300735	160462	5605	1280	46

(4) Sectional force



1) Acting load

It is assumed that the internal design pressure between the stiffeners acts as the distributed load. The acting load converts into the design load which is calculated by the ratio of an acting axis and a neutral axis.

$$W = pi \cdot b \cdot (2h0+L0)/(2h1+L1)$$

$$= 0.207 \times 315 \times (2 \times 315 + 1575) / (2 \times 407 + 1724)$$

$$= 57 \quad N/mm$$

Where, W : Converted acting load N/mm

ps : Design internal pressure = 0.207 N/mm^2

b : Width of receiving pressure = 315 mm

h0 : Depth of bonnet = 315 mm

h1 : Length of neutral axis = h0+e = 315 + 2 × 46 = 407 mm

L0 : Width of bonnet = 1575 mm

L1 : Length of neutral axis = L0+2e = 1575 + 2 × 74 = 1724 mm

2) Acting load on each part

[Stiffness ratio]

$k = (I2 \cdot h1)/(I1 \cdot L1)$

$= (13863875 \times 407)/(7547377 \times 1724) = 0.434$

$n = h1/L1$

$= 407 / 1724 = 0.236$

[Bending moment]

$MA=W \cdot L1^2/12 \cdot \{(1+n^2 \cdot k)/(1+k)\}$

$MB=MA-W \cdot h^2/8$

$MC=MA-W \cdot L1^2/8$

[Axial force]

Section A-B NAB=W・L1/2（Tensile force）

Section B-C NBC=W・h1/2（Tensile force）

[Shearing force]

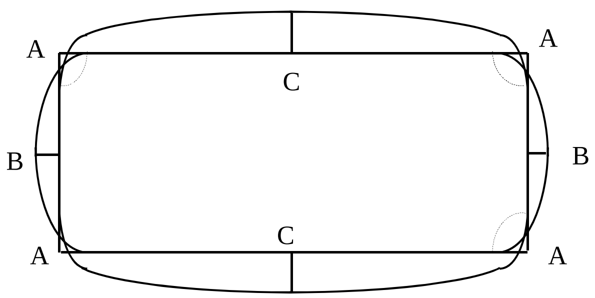
Section A-B SAB=W・h1/2

Section B-C SAC=W・l1/2

[Result of calculation]

MA= 9992960 N-mm MB= 8821644 N-mm

MC= -10993134 N-mm



VA= 0 N

NAB= 48703 N NBC= 11506 N

SAB= 11506 N SBC= 48703 N

(5) Stress of bonnet

1) Stress at "A"

Bending stress

[Bending stress(Inside)]

σAi = MA/Zi+NAB/A

= 9992960 / 166083 + 48703 / 4005

= 60.2 + 12.2 = 72.3 N/mm^2 < σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MA/Zo+NAB/A

= -9992960 / 146058 + 48703 / 4005

= -68.4 + 12.2 = -56.3 N/mm^2 < σa = 157.5 N/mm^2

Shearing stress

τA = SBC/Aw

= 48703 / 1280

= 38.0 N/mm^2 < τa= 90.9 N/mm^2

2) Stress at "B"

Bending stress

[Bending stress(Inside)]

σBi = MB/Zi+NAB/A

= 8821644 / 101580 + 48703 / 3280

= 86.8 + 14.8 = 101.7 N/mm^2 < σa = 157.5 N/mm^2

[Bending stress(Outside)]

σBo = -MB/Zo+NAB/A

= -8821644 / 129680 + 48703 / 3280

= -68.0 + 14.8 = -53.2 N/mm^2 < σa = 157.5 N/mm^2

Shearing stress

τB = SAB/Aw

= 11506 / 1280

= 9.0 N/mm^2 < τa= 90.9 N/mm^2

3) Stress at "C"

Bending stress

[Bending stress(Inside)]

σCi = MC/Zi+NBC/A

= -10993134 / 300735 + 11506 / 5605

= -36.6 + 2.1 = -34.5 N/mm^2 < σa = 157.5 N/mm^2

[Bending stress(Outside)]

σCo = -MC/Zo+NBC/A

= 10993134 / 160462 + 11506 / 5605

= 68.5 + 2.1 = 70.6 N/mm^2 < σa = 157.5 N/mm^2

Shearing stress

τC = SBC/Aw

= 48703 / 1280

= 38.0 N/mm^2 < τa= 90.9 N/mm^2

(6) Allowable stresses

Allowable bending stress

Outside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Inside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Coefficient : 0.63

Allowable shearing stress

Outside

$\tau a = 90.9$ N/mm^2 Material : A36(ASTM)

1.4 Operating load

The operating load is summed up the following loads.

(1) Self weight

Gate leaf G1 = 1.5 × 9.81 = 14.72 kN

Rod of cylinder G2 = 0.77 kN

Total load G = 15.49 kN

(2) Friction force of seal plate

$F2 = \mu_2 \cdot P$

$= 0.4 \times 405.622 = 162.25$ kN

Where, μ_2 : Frictional coefficient of metal seal = 0.4

P : Hydrostatic pressure at operation = 405.62 kN

(3) Buoyancy

$F3 = \gamma 0 / W0 \cdot G1$

$= 9.81 / 77.0 \times 14.72 = 1.87$ kN

ここに、 $\gamma 0$: Specific gravity of water = 9.81 kN/m3

W0 : Specific gravity of steel material = 77.01 kN/m3

(4) Friction force of seal in cylinder

$F4 = d \cdot \pi \cdot b \cdot n \cdot \mu_2 \cdot P$

$= 0.090 \times \pi \times 0.006 \times 1 \times 0.7 \times 405.622 = 0.482$ kN

Where, d : Outside diameter of rod = 0.090 m

b : Contact width of V-packing = 0.006 m

n : Quantity of V-packing = 1 piece

μ_2 : Frictional coefficient of V-packing = 0.7

P : Pressure on V-packing = 405.622 kN

(5) Total operating load

(Unit : kN)

Load		Raising		Lowering	
Self weight	G	↓	15.49	↓	15.49
Friction force of seal plate	F2	↓	162.25	↑	162.25
Buoyancy	F3	↑	1.87	↑	1.87
Friction force of seal in cylinder	F4	↓	0.48	↑	0.48
Total load		↓	176.34	↑	149.12

Raising load	Fu	=	176.34	kN	→	180.00	kN
Lowering load	Fd	=	149.12	kN	→	150.00	kN

1.5 Capacity of cylinder

(1) Design conditions

Type of cylinder	Fixed cylinder				
Rated pressure	Raising（Setting pressure of relief valve）	P1	=	21.0	MPa
	Lowering（Setting pressure of relief valve）	P2	=	12.6	MPa
Working pressure	Raising（Effective operating pressure）	P1'	=	18.9	MPa
	Lowering（Effective operating pressure）	P2'	=	11.3	MPa
Operating speed	0.1 m/min				
Operating load	Raising	Wu	=	180.00	kN
	Lowering	Wd	=	150.00	kN
Cylinder	Inside diameter of tube	D	=	160	mm
	Outside diameter of rod	d	=	90	mm
	Cylinder stroke	S	=	1570	mm

(2) Pulling and pushing forces of cylinder

1）Rated pressure

Pulling force (Raising)

$$F_u = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (160^2 - 90^2) \times \frac{21.0}{1000}$$

$$= 288.6 \text{ kN}$$

Pushing force (Lowering)

$$F_d = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 160^2 \times \frac{12.6}{1000}$$

$$= 253.3 \text{ kN}$$

2）Working pressure

Pulling force (Raising)

$$F_u' = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (160^2 - 90^2) \times \frac{18.9}{1000}$$

$$= \underline{259.8 \text{ kN} > W_u = 180 \text{ kN}}$$

Pushing force (Lowering)

$$F_d' = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 160^2 \times \frac{11.3}{1000}$$

$$= \underline{228.0 \text{ kN} > W_d = 150.00 \text{ kN}}$$

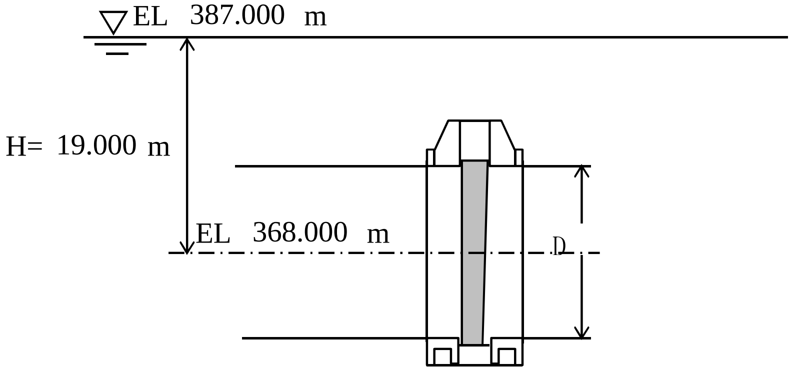
2. Strength Calculation for control gate in Oeste dam (Before heightning)

2.1 Design conditions

(1)	Type	Slide gate			
(2)	Quantity	5	sets		
(3)	Gate center elevation	EL.	368.00	m	
(4)	Max. water level	EL.	408.00	m	(heightning 0.0 m)
(5)	Flood water level	EL.	399.00	m	
(6)	Normal water level	EL.	387.00	m	
(7)	Diameter	φ	1.50	m	
(8)	Seismic intensity		0.05		
(9)	Sealing system	Metal seal at both side of gate leaf			
(10)	Basic grand level	EL.	357.50	m	
(11)	Operation device	Hydraulic cylinder			
(12)	Lifting height		1.57	m	
(13)	Operating system	Local			
(14)	Allowable stress	ABNT NBR 8883			

2.2 Design head

(1) CCN (Nomal water level Only)



D : Diameter = φ1.50 m

Load of normal water level only

$Ps = \gamma o \times H \times A$

$= 9.81 \times 19.00 \times 1.77$

$= 329.35$ kN

Where, Ps : Hydrostatic load

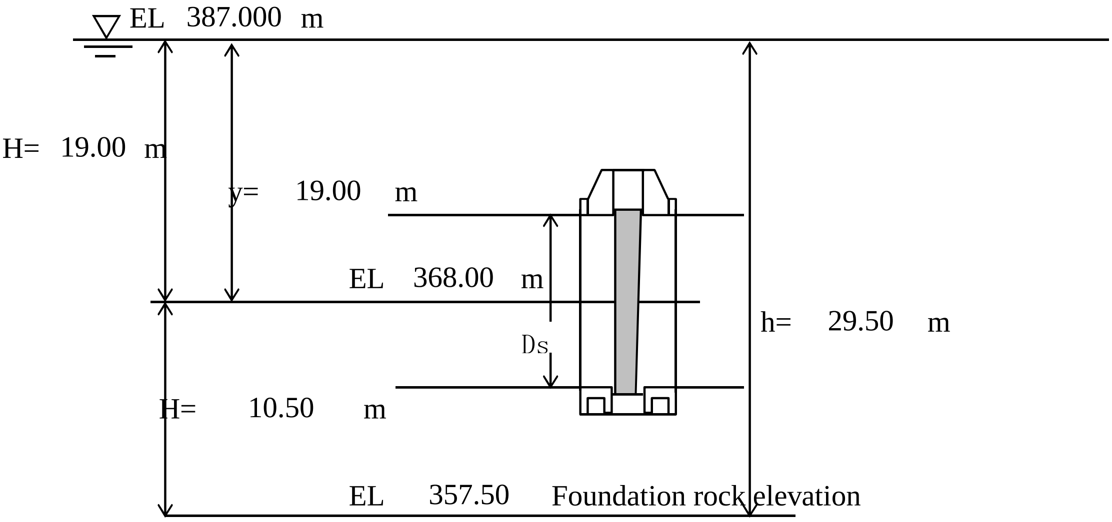
γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 19.00 m

A : Receiving pressure area = $\pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77\ m^2$

(2) CCE1(Normal water level + Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \times H \times A$

$= 9.81 \times 19.00 \times 1.77$

$= 329.35$ kN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 29.50 \times 19.00^{1/2} \times 1.77$

$= 17.95$ kN

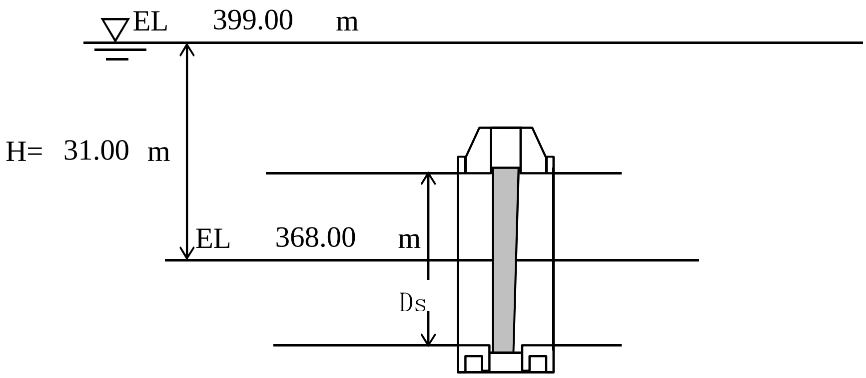
c) Total load

$Pw = Ps + P_d$

$= 329.35 + 17.95$

$= 347.31$ kN

(3) CCE2(Flood water level only)



D：Caliber = φ1.50 m

$Ps = \gamma o \times H \times A$

$= 9.81 \times 31.00 \times 1.77$

$= 537.36$ kN

Where, Ps : Hydrostatic load

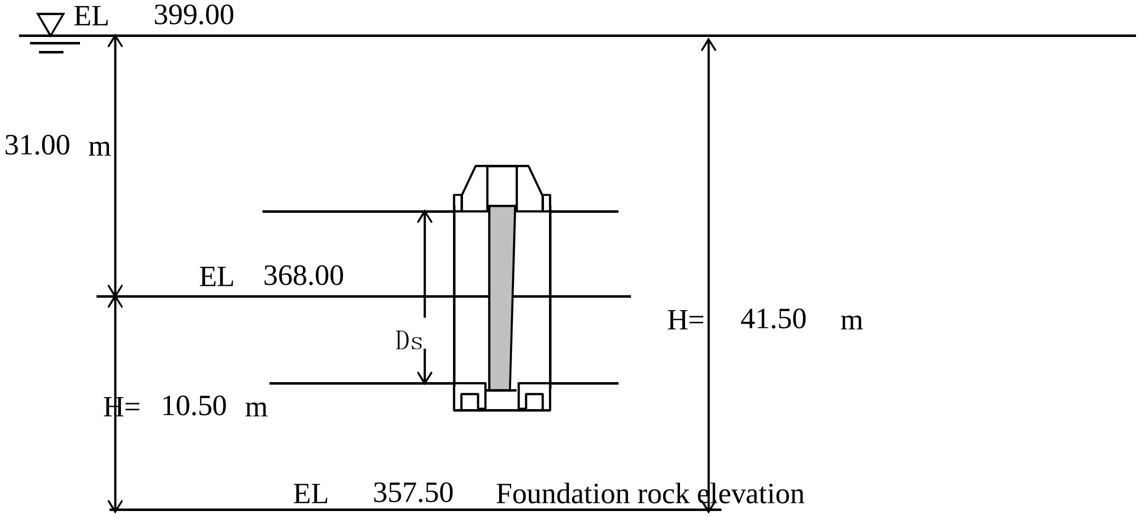
γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 31.00 m

A : Receiving pressure area $= \pi \cdot Ds^2/4 = \pi \times 1.50^2/4$

$= 1.77$ m^2

(4) CCL(Flood water level+ Dynamic water pressure during earthquake)



a) Hydrostatic load

$Ps = \gamma o \cdot H \cdot A$

$= 9.81 \times 31.00 \times 1.77$

$= 537.36$ KN

b) Dynamic pressure load during earthquake

$P_d = \gamma o \cdot 7/8 \cdot k \cdot (h \cdot y)^{1/2} \cdot A$

$= 9.81 \times 7/8 \times 0.05 \times 41.50 \times 31.00^{1/2} \times 1.77$

$= 27.20$ kN

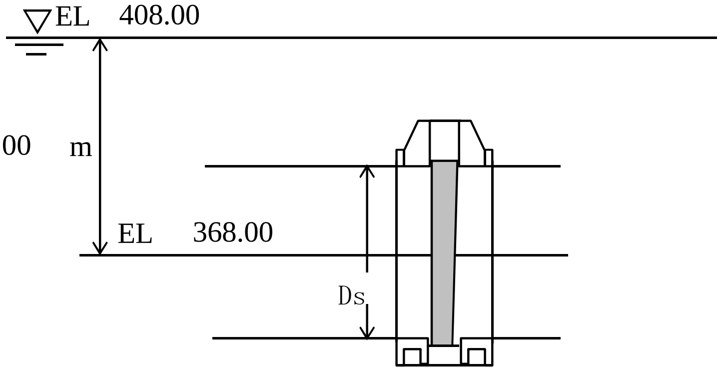
c) Total load

$Pw = Ps + P_d$

$= 537.36 + 27.20$

$= 564.56$ kN

(5) Max. water level



D : Diameter = φ1.50 m

Ps= γo×H×A

= 9.81 × 40.00 × 1.77

= 693.37 kN

Where, Ps : Hydrostatic load

γo : Specific gravity of water = 9.81 kN/m^3

H : Design head = 40.00 m

A : Receiving pressure area = $\pi \cdot Ds^2/4$ = π × 1.50 ^{2} /4

= 1.77 m^2

(5) Comparison of loads

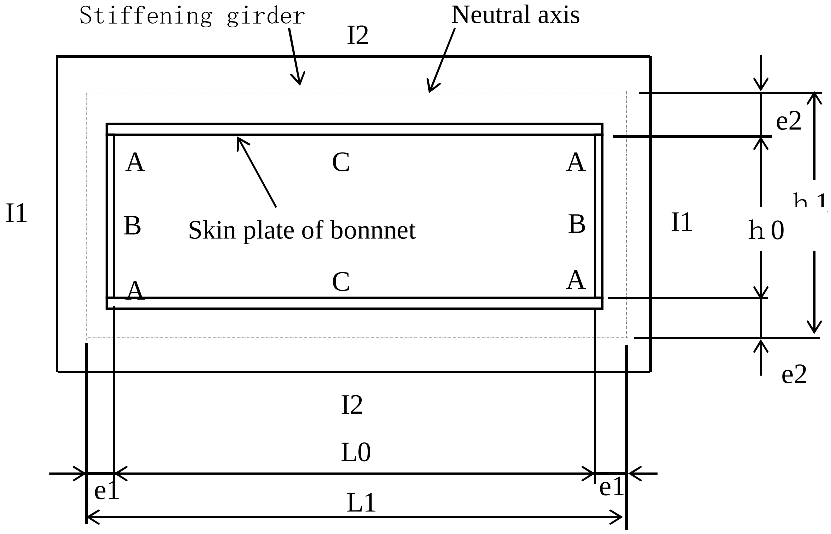
unit : kN

Case / 水　位	Coefficient		Hydrostatic load only		Dynamic water pressure	
	Hydrostatic load only	Dynamic water pressure	Actual load	Converted load	Actual load	Converted load
Normal water level	0.50	0.90	329.35	658.70	347.31	385.90
			CCN		CCE1	
Flood water level	0.63	0.90	537.36	852.96	564.56	627.29
			CCE2		CCL	
Max. water level	0.80	—	693.37	866.71	—	—

Because the load of "CCE2" becomes the maximum, strength of the load of "CCE2" is checked.

2.3 Strength calculation of bonnet

The bonnet is calculated as a box ramen as shown in the model figure below.



Where, L0 : Width of bonnet = 1650 mm

h0 : Depth of bonnet = 315 mm

(1) Internal pressure

pi= γo×H

= 9.81 × 31.00 = 304.11 kN/m^2

= 0.304 N/mm^2

pi : Internal pressure (N/mm^2)

γ0 : Specific gravity of water = 9.81 kN/m^3

H : Design head = 31.00 m

(2) Effective width of skin plate

The effective width is calculated so that the flange of stiffening girder may support the load together with the skin plate.

a) Point of A

$l/L \leqq 0.02$

$\lambda = l$

$0.02 < l/L < 0.3$

$\lambda = \{1.06 - 3.2(l/L) + 4.5(l/L)^2\}\, l$

$0.3 \leqq l/L$

$\lambda = 0.15L$

b) Point of B and C

$l/L \leqq 0.05$

$\lambda = l$

$0.05 < l/L < 0.3$

$\lambda = \{1.1 - 2(l/L)\}\, l$

$0.3 \leqq l/L$

$\lambda = 0.15L$

Where, λ : Working width in one side of skinplate mm

l : Half of skin plate at support intervals = 315 / 2 = 158 mm

L : Equivalent support inter

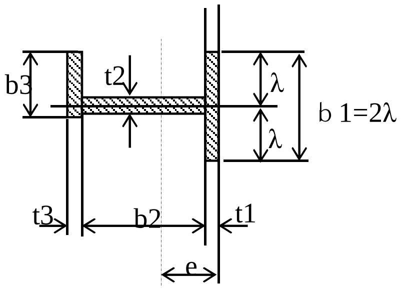
Point A = 0.2 (l0+h0) = 0.2 × (1650 + 315)= 393 mm

Point B = 0.6 h0 = 0.6 × 315 = 189 mm

Point C = 0.6 L0 = 0.6 × 1650 = 990 mm

Position	Effective width of skin plate				
	l mm	Lmm	l/L	λmm	2λmm
Point A	158	393	0.40	59	118
Point B	158	189	0.83	28	56
Point C	825	990	0.83	149	298

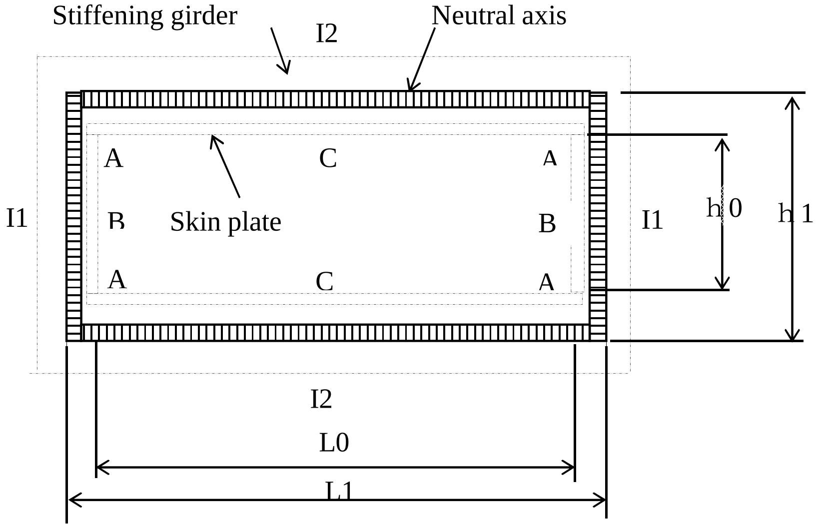
(3) Section properties of stiffening girder



t1 : Thickness of skin plate mm
t2 : Thickness of web mm
t3 : Thickness of flange mm
b1 : Effective width mm
b2 : Width of web mm
b3 : Width of flange mm

Position	Skin plate		Web		Flange		Section properties					
	t1	b1	t2	b2	t3	b3	I (mm^4)	Zi (mm^3)	Zo (mm^3)	A (mm^2)	Aw(mm^2)	e (mm)
A	12.6	118	16.2	122	26	100	22214599	250164	309396	6063	1976.4	89
B	12.6	56	16.2	122	26	100	16102576	159431	270177	5282	1976.4	101
C	12.6	298	16.2	122	26	100	33413694	503218	354710	8331	1976.4	66

(4) Sectional force



1) Acting load

It is assumed that the internal design pressure between the stiffeners acts as the distributed load. The acting load converts into the design load which is calculated by the ratio of an acting axis and a neutral axis.

W = pi・b・(2h0+L0)/(2h1+L1)

= 0.304 × 315 × (2 × 315 + 1650) / (2 × 448 ＋ 1852 ）

= 79 N/mm

Where, W : Converted acting load N/mm

ps : Design internal pressure = 0.304 N/mm^2

b : Width of receiving pressure = 315 mm

h0 : Depth of bonnet = 315 mm

h1 : Length of neutral axis = h0+e = 315 + 2 × 66 = 448 mm

L0 : Width of bonnet = 1650 mm

L1 : Length of neutral axis = L0+2e : 1650 + 2 × 101 = 1852 mm

2) Acting load on each part

[Stiffness ratio]

k = (I2・h1)/(I1・L1)

= (33413694 × 448)/ (16102576 × 1852) = 0.502

n = h1/L1

= 448 / 1852 = 0.242

[Bending moment]

$MA=W \cdot L1^2/12 \cdot \{(1+n^2 \cdot k)/(1+k)\}$

$MB=MA-W \cdot h^2/8$

$MC=MA-W \cdot L1^2/8$

[Axial force]

Section A-B NAB=W・L1/2（Tensile force）

Section B-C NBC=W・h1/2（Tensile force）

[Shearing force]

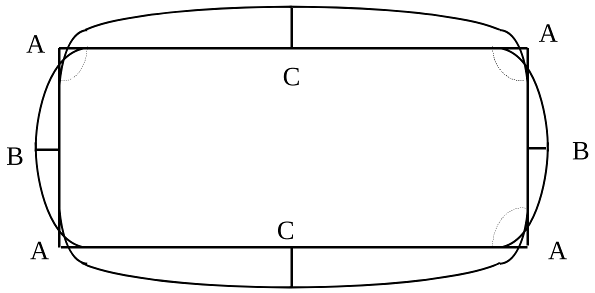
Section A-B SAB=W・h1/2

Section B-C SAC=W・l1/2

[Result of calculation]

MA= 15573516 N-mm MB= 13581005 N-mm

MC= -18507660 N-mm



VA= 0 N

NAB= 73609 N NBC= 17798 N

SAB= 17798 N SBC= 73609 N

(5) Stress of bonnet

1) Stress at "A"

Bending stress

[Bending stress(Inside)]

σAi = MA/Zi+NAB/A

= 15573516 / 250164 + 73609 / 6063

= 62.3 + 12.1 = 74.4 N/mm^2 ＜ σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MA/Zo+NAB/A

= -15573516 / 309396 + 73609 / 6063

= -50.3 + 12.1 = -38.2 N/mm^2 ＜ σa = 157.5 N/mm^2

Shearing stress

τA = SBC/Aw

= 73609 / 1976

= 37.2 N/mm^2 ＜ τa= 90.9 N/mm^2

2) Stress of "B"

Bending stress

[Bending stress(Inside)]

σAi = MB/Zi+NAB/A

= 13581005 / 159431 + 73609 / 5282

= 85.2 + 13.9 = 99.1 N/mm^2 ＜ σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MB/Zo+NAB/A

= -13581005 / 270177 + 73609 / 5282

= -50.3 + 13.9 = -36.3 N/mm^2 ＜ σa = 157.5 N/mm^2

Shearing stress

τA = SAB/Aw

= 17798 / 1976

= 9.0 N/mm^2 ＜ τa= 90.9 N/mm^2

3) Stress of "C"

Bending stress

[Bending stress(Inside)]

σAi = MC/Zi+NBC/A

= -18507660 / 503218 + 17798 / 8331

= -36.8 + 2.1 = -34.6 N/mm^2 ＜ σa = 157.5 N/mm^2

[Bending stress(Outside)]

σAo = -MC/Zo+NBC/A

= 18507660 / 354710 + 17798 / 8331

= 52.2 + 2.1 = 54.3 N/mm^2 ＜ σa = 157.5 N/mm^2

Shearing stress

τA = SBC/Aw

= 73609 / 1976

= 37.2 N/mm^2 ＜ τa= 90.9 N/mm^2

(6) Allowable stresses

Allowable bending stress

Outside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Inside

$\sigma a = 250 \times 0.63 = 157.5$ N/mm^2 Material : A36(ASTM)

Coefficient : 0.63

Allowable shearing stress

Outside

$\tau a = 90.9$ N/mm^2 Material : A36(ASTM)

2.4 Operating load

The operating load is summed up the following loads.

(1) Self weight

Gate leaf G1 = 2.5 × 9.81 = 24.53 kN

Rod of cylinder G2 = 0.77 kN

Total load G = 25.30 kN

(2) Seal friction

$F2 = \mu_2 \cdot P$

$= 0.4 \times 693.371 = 277.35$ kN

Where, μ_2 : Frictional coefficient of metal seal = 0.4

P : Hydrostatic pressure at operation = 693.37 kN

(3) Buoyancy

$F3 = \gamma 0 / W0 \cdot G1$

$= 9.81 / 77.0 \times 24.53 = 3.12$ kN

ここに、 $\gamma 0$: Specific gravity of water = 9.81 kN/m3

W0 : Specific gravity of steel material = 77.01 kN/m3

(4) Friction force of seal in cylinder

$F4 = d \cdot \pi \cdot b \cdot n \cdot \mu_2 \cdot P$

$= 0.090 \times \pi \times 0.006 \times 1 \times 0.7 \times 693.371 = 0.823$ kN

Where, d : Rod outside diameter = 0.090 m

b : Width of contact of V-packing = 0.006 m

n : Quantity of V-paccking = 1 piece

μ_2 : Frictional coefficient of V-packing = 0.7

P : Pressure on V-packing = 693.371 kN

(5) Total operating load

(Unit : kN)

Load		Raising	Lowering
Self weight	G	↓ 25.30	↓ 25.30
Seal friction	F2	↓ 277.35	↑ 277.35
Buoyancy	F3	↑ 3.12	↑ 3.12
Friction force of seal in cylinder	F4	↓ 0.82	↑ 0.82
Total load		↓ 300.34	↑ 256.00

Raising load	Fu	=	300.34	kN	→	310.00	kN
Lowerring load	Fd	=	256.00	kN	→	260.00	kN

2.5 Capacity of cylinder

(1) Design conditions

Type of hoist	Fixed cylinder				
Rated pressure	Raising（Setting pressure of relief valve）	P1	=	16.0	MPa
	Lowering（Setting pressure of relief valve）	P2	=	9.6	MPa
Working pressure	Raising（Effective operating pressure）	P1'	=	14.4	MPa
	Lowering（Effective operating pressure）	P2'	=	8.6	MPa
Operating speed	0.1 m/min				
Operating load	Raising	Wu	=	310.00	kN
	Lowerring	Wd	=	260.00	kN
Cylinder	Inside diameter of tube	D	=	200	mm
	Outside diameter of rod	d	=	100	mm
	Cylinder stroke	S	=	1570	mm

(2) Power to push and power to pull

1) Rated pressure

Pulling force (Raising)

$$F_u = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (200^2 - 100^2) \times \frac{16.0}{1000}$$

$$= 377 \ \text{kN}$$

Pushing force (Lowering)

$$F_d = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 200^2 \times \frac{9.6}{1000}$$

$$= 301.6 \ \text{kN}$$

2) Working pressure

Pulling force (Raising)

$$F_u' = \frac{\pi}{4} \times (D^2 - d^2) \times p_1'$$

$$= \frac{\pi}{4} \times (200^2 - 100^2) \times \frac{14.4}{1000}$$

$$= \underline{339.3 \ \text{kN} > W_u = 310 \ \text{kN}}$$

Pushing force (Lowering)

$$F_d' = \frac{\pi}{4} \times D^2 \times p_2'$$

$$= \frac{\pi}{4} \times 200^2 \times \frac{8.6}{1000}$$

$$= \underline{271.4 \ \text{kN} > W_d = 260.00 \ \text{kN}}$$

APPENDIX-4 :

Structural calculation for conduit pipes (After heightning)

1. Strength Calculation for Conduit Pipe in Oeste Dam (After heightning)

1.1 Design Conditions

(1)	Type	Circular section embedded steel pipe (Exposed pipe at control gate chamber)
(2)	Quantity	7 lanes
(3)	Diameter	1500 mm
(4)	Pipe center elevation	EL. 339.25 m
(5)	Max. water level	EL. 364.65 m （heightning 2.0 m)
(6)	Flood water level	EL. 362.30 m
(7)	Normal water level	EL. 341.50 m
(8)	Material	ASTM A36(equivalent to SS400 of JIS G3101）
(9)	Allowable stress	ABNT NBR 8883:2008
(10)	Young's modulus	Es= 206 kN/mm^2

1.2 Allowable Stress

Material	Yield point σ_y (N/mm^2)	ABNT NBR 8883	Allowable stress CCN σa (N/mm^2)	CCE σa (N/mm^2)	CCL σa (N/mm^2)
A36	250	Safety factor	0.50	0.63	0.80
		Allowable stress	125.0	157.5	200.0

1.3 Strength Calculation for Conduit Pipe

$$\sigma_1 = \frac{P \times D}{2 \times t} \ (\text{N/mm}^2)$$

Where,

D : Internal diameter(mm)

P : Hydraulic pressure(MPa)

t : Shell thickness(mm)

Location	Case	D (mm)	t (mm)	H (m)	P (MPa)	σ_1 (N/mm^2)	Allowable stress (N/mm^2)
Upstream	Max. water level	1500.0	5.93	25.40	0.249	31.5	200.0
	Flood water level	1500.0	5.93	23.05	0.226	28.6	157.5
	Normal water level	1500.0	5.93	2.25	0.022	2.8	125.0
Downstream	Max. water level	1500.0	6.51	25.40	0.249	28.7	200.0
	Flood water level	1500.0	6.51	23.05	0.226	26.1	157.5
	Normal water level	1500.0	6.51	2.25	0.022	2.5	125.0

2. Strength Calculation for Conduit Pipe in Sul dam (After heightning)

2.1 design conditions

(1)	Type	Circular section embedded steel pipe (Exposed pipe at control gate chamber)
(2)	Quantity	5 lanes
(3)	Diameter	φ 1500 mm
(4)	Pipe center elevation	EL. 368.00 m
(5)	Max. water level	EL. 408.00 m （heightning 2.0 m ）
(6)	Flood water level	EL. 401.00 m
(7)	Normal water level	EL. 387.00 m
(8)	Material	ASTM A36(equivalent to SS400 of JIS G3101）
(9)	Allowable stress	ABNT NBR 8883:2008
(10)	Young's modulus	Es= 206 kN/mm^2

2.2 Allowable Stress

Material	Yield point σ_y (N/mm^2)	ABNT NBR 8883	Arrowed stress CCN σa (N/mm^2)	CCE σa (N/mm^2)	CCL σa (N/mm^2)
A36	250	Safety factor	0.50	0.63	0.80
		Allowable stress	125.0	157.5	200.0

2.3 Strength Calculation for Conduit Pipe

$$\sigma_1 = \frac{P \times D}{2 \times t} \ (N/mm^2)$$

Where,

D : Internal diameter(mm)
P : Hydraulic pressure(MPa)
t : Shell thickness(mm)

Location	Case	D (mm)	t (mm)	H (m)	P (MPa)	σ_1 (N/mm^2)	Allowable stress (N/mm^2)
Upstream	Max. water level	1500.0	9.17	40.00	0.392	32.1	200.0
	Flood water level	1500.0	9.17	33.00	0.324	26.5	157.5
	Normal water level	1500.0	9.17	19.00	0.186	15.2	125.0
Downstream	Max. water level	1500.0	8.66	40.00	0.392	34.0	200.0
	Flood water level	1500.0	8.66	33.00	0.324	28.0	157.5
	Normal water level	1500.0	8.66	19.00	0.186	16.1	125.0

APPENDIX-5 :

Structural calculation for conduit pipes (Before heightning)

1. Strength Calculation for Conduit Pipe in Oeste Dam (Before heightning)

1.1 Design Conditions

(1)	Type	Circular section embedded steel pipe			
		(Exposed pipe at control gate chamber)			
(2)	Quantity	7 lanes			
(3)	Diameter		1500	mm	
(4)	Pipe center elevation	EL.	339.25	m	
(5)	Max. water level	EL.	362.65	m	（heightning 0.0 m ）
(6)	Flood water level	EL.	360.30	m	
(7)	Normal water level	EL.	341.50	m	
(8)	Material	ASTM A36(equivalent to SS400 of JIS G3101）			
(9)	Allowable stress	ABNT NBR 8883:2008			
(10)	Young's modulus	Es=	206	kN/mm^2	

1.2 Allowable Stress

Material	Yield point σ_y (N/mm^2)	ABNT NBR 8883	Allowable stress		
			CCN σa (N/mm^2)	CCE σa (N/mm^2)	CCL σa (N/mm^2)
A36	250	Safety factor	0.50	0.63	0.80
		Allowable stress	125.0	157.5	200.0

1.3 Strength Calculation for Conduit Pipe

$$\sigma_1 = \frac{P \times D}{2 \times t} \ (N/mm^2)$$

Where,

D : Internal diameter(mm)

P : Hydraulic pressure(MPa)

t : Shell thickness(mm)

Location	Case	D (mm)	t (mm)	H (m)	P (MPa)	σ_1 (N/mm^2)	Allowable stress (N/mm^2)
Upstream	Max. water level	1500.0	5.93	23.40	0.230	29.0	200.0
	Flood water level	1500.0	5.93	21.05	0.207	26.1	157.5
	Normal water level	1500.0	5.93	2.25	0.022	2.8	125.0
Downstream	Max. water level	1500.0	6.51	23.40	0.230	26.4	200.0
	Flood water level	1500.0	6.51	21.05	0.207	23.8	157.5
	Normal water level	1500.0	6.51	2.25	0.022	2.5	125.0

2. Strength Calculation for Conduit Pipe in Sul dam (Before heightning)

2.1 design conditions

(1)	Type	Circular section embedded steel pipe						
		(Exposed pipe at control gate chamber)						
(2)	Quantity	5 lanes						
(3)	Diameter	φ	1500	mm				
(4)	Pipe center elevation	EL.	368.00	m				
(5)	Max. water level	EL.	408.00	m	（heightning	0.0	m	)
(6)	Flood water level	EL.	399.00	m				
(7)	Normal water level	EL.	387.00	m				
(8)	Material	ASTM A36(equivalent to SS400 of JIS G3101）						
(9)	Allowable stress	ABNT NBR 8883:2008						
(10)	Young's modulus	Es=	206	kN/mm^2				

2.2 Allowable Stress

Material	Yield point σ_y (N/mm^2)	ABNT NBR 8883	Arrowed stress		
			CCN σa (N/mm^2)	CCE σa (N/mm^2)	CCL σa (N/mm^2)
A36	250	Safety factor	0.50	0.63	0.80
		Allowable stress	125.0	157.5	200.0

2.3 Strength Calculation for Conduit Pipe

$$\sigma_1 = \frac{P \times D}{2 \times t} \ (N/mm^2)$$

Where,

D : Internal diameter(mm)

P : Hydraulic pressure(MPa)

t : Shell thickness(mm)

Location	Case	D (mm)	t (mm)	H (m)	P (MPa)	σ_1 (N/mm^2)	Allowable stress (N/mm^2)
Upstream	Max. water level	1500.0	9.17	40.00	0.392	32.1	200.0
	Flood water level	1500.0	9.17	31.00	0.304	24.9	157.5
	Normal water level	1500.0	9.17	19.00	0.186	15.2	125.0
Downstream	Max. water level	1500.0	8.66	40.00	0.392	34.0	200.0
	Flood water level	1500.0	8.66	31.00	0.304	26.3	157.5
	Normal water level	1500.0	8.66	19.00	0.186	16.1	125.0

APPENDIX-6 :

Stability Analysis of Oeste dam

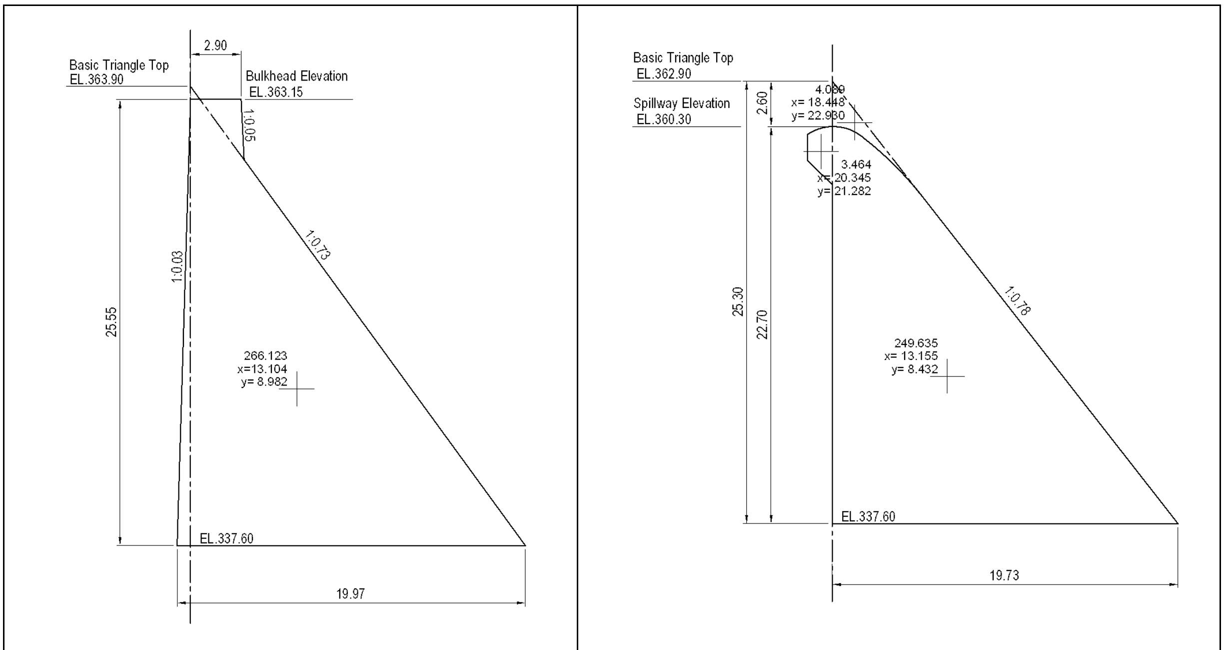
(1) Existing

1) Design Condition

Design condition of Dam stability analysis is considered as shown in the table 1 below.

Table 1 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	←



Bulkhead section	Spillway section

Fig 1 Typical section of Existing

2) Stability Analysis of Existing dam

[Bulkhead section]

- CCN: Normal water

Resume of Acting Force and Moment

[CCN : Normal water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,251.14		13.103		81,905.56		
W/O Dead Load							
Seismic							
W/O Seismic							
U/S Water weight	1.53		19.934		30.50		
D/S Water weight	22.63		0.605		13.70		
U/S Water Pressure		50.88		1.063		54.09	
D/S Water Pressure		-31.00		0.830		-25.73	
Dynamic Water Pressure							
Earth Pressure	0.10		19.957		2.00		
Soil weight		1.38		0.300		0.41	
Uplift	-520.43		10.132		-5,272.78		
Total	5,754.97	21.26			76,678.98	28.77	

Control of Stability [CCN]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{76,650.21}{5,754.97} \quad =13.319 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} \quad - 13.319 \quad =|-3.336 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,275.40}{520.43} \quad =12.058 \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{76,678.98}{28.77} \quad =2,665.241 \quad > 1.50 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	5,754.97	kN	FSD-φ	1.50
H=	21.26	kN	FSD-c	3.00
L=	19.966	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5,754.97*0.78}{1.50} \quad =2,992.58$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.966}{3.00} \quad =6,655.33$$

$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{2,992.58+6,655.33}{21.26} \quad =453.806 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5,754.97}{19.966} \quad \times(1.0\pm \quad \frac{6\times3.336}{19.966})$$

vertical stress of upstream = 577.220 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = -0.729 kN/㎡ ＜ 0 kN/㎡ (Tensile force occur) but downstream side -OK-

- CCE: Maximum Flood water

Resume of Acting Force and Moment

[CCE : Maximum Flood water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,251.14		13.103		81,905.56		
W/O Dead Load							
Seismic							→
W/O Seismic							→
U/S Water weight	94.13		19.716		1,855.82		
D/S Water weight	375.29		2.467		926.03		
U/S Water Pressure		3,137.51		8.350		26,198.21	
D/S Water Pressure		-514.10		3.380		-1,737.66	
Dynamic Water Pressure							
Earth Pressure	0.10		19.957		2.00		
Soil weight		1.38		0.300		0.41	
Uplift	-2,520.64		10.638		-26,813.31		
Total	4,200.02	2,624.79			57,876.10	24,460.96	

Control of Stability [CCE]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{33,415.14}{4,200.02}$$ =7.956 m

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2}$$ - 7.956 =|2.027 m|

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,720.66}{2,520.64}$$ =2.666 > 1.10 … -OK-

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{57,876.10}{24,460.96}$$ =2.366 > 1.20 … -OK-

- Safety factor due to sliding

V=	4,200.02	kN	FSD-φ	1.10
H=	2,624.79	kN	FSD-c	1.50
L=	19.966	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4,200.02*0.78}{1.10}$$ =2,978.20

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.966}{1.50}$$ =13,310.67

$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{2,978.20+13,310.67}{2,624.79}$$ =6.206 > 1.0 … -OK-

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4,200.02}{19.966} \times (1.0 \pm \frac{6 \times 2.027}{19.966})$$

vertical stress of upstream = 82.221 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 338.507 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCL: Flood water + Seismic

Resume of Acting Force and Moment

[CCL : Flood water + Seismic]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,251.14		13.103		81,905.56		
W/O Dead Load							
Seismic	-187.53	312.56	13.103	8.977	-2,457.17	2,805.86	
W/O Seismic							
U/S Water weight	77.29		19.739		1,525.59		
D/S Water weight	69.07		1.059		73.18		
U/S Water Pressure		2,576.45		7.567		19,496.00	
D/S Water Pressure		-94.61		1.450		-137.18	
Dynamic Water Pressure		150.29		9.080		1,364.63	
Earth Pressure	0.10		19.957		2.00		
Soil weight		1.38		0.300		0.41	
Uplift	-1,479.11		11.357		-16,797.51		
Total	4,730.96	2,946.07			64,251.65	23,529.72	

Control of Stability [CCL]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{40{,}721.93}{4{,}730.96} = 8.608 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} - 8.608 = |1.375 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}210.07}{1{,}479.11} = 4.199 \quad > 1.10 \text{ … -OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{64{,}251.65}{23{,}529.72} = 2.731 \quad > 1.10 \text{ … -OK-}$$

- Safety factor due to sliding

V=	4,730.96 kN		FSD-φ	1.10
H=	2,946.07 kN		FSD-c	1.30
L=	19.966 m		tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4{,}730.96 * 0.78}{1.10} = 3{,}354.68$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0 * 19.966}{1.30} = 15{,}358.46$$

$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}354.68 + 15{,}358.46}{2{,}946.07} = 6.352 \quad > 1.0 \text{ … -OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4{,}730.96}{19.966} \times \left(1.0 \pm \frac{6 \times 1.375}{19.966}\right)$$

vertical stress of upstream = 139.043 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 334.870 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCC: Construction

Resume of Acting Force and Moment

[CCC : Construction]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,251.14		13.103		81,905.56		
W/O Dead Load							
Seismic							
W/O Seismic							
U/S Water weight							
D/S Water weight							
U/S Water Pressure							
D/S Water Pressure							
Dynamic Water Pressure							
Earth Pressure							
Soil weight							
Uplift							
Total	6,251.14				81,905.56		

Control of Stability [CCC]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{81,905.56}{6,251.14} \quad =13.102 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} \quad - 13.102 \quad =|-3.119 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,251.14}{0.00} \quad = \infty \quad > 1.20 \dots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{81,905.56}{0.00} \quad = \infty \quad > 1.30 \dots \text{-OK-}$$

- Safety factor due to sliding

V=	6,251.14	kN	FSD-φ	1.30
H=	0.00	kN	FSD-c	2.00
L=	19.966	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{6,251.14*0.78}{1.30} \quad =3,750.68$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.966}{2.00} \quad =9,983.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,750.68+9,983.00}{0.00} \quad = \infty \quad > 1.0 \dots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{6,251.14}{19.966} \times (1.0 \pm \frac{6 \times 3.119}{19.966})$$

vertical stress of upstream = 606.568 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 19.626 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

[Spillway section]

- CCN: Normal water

Resume of Acting Force and Moment

[CCN : Normal water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,866.42		13.156		77,178.62		
W/O Dead Load	-14.69		7.934		-116.55		
Seismic							
W/O Seismic							
U/S Water weight							
D/S Water weight	24.18		0.647		15.64		
U/S Water Pressure		50.88		1.063		54.09	
D/S Water Pressure		-31.00		0.830		-25.73	
Dynamic Water Pressure							
Earth Pressure							
Soil weight		1.38		0.300		0.41	
Uplift	-514.40		10.014		-5,151.20		
Total	5,361.51	21.26			71,926.51	28.77	

Control of Stability [CCN]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{71,897.74}{5,361.51} \quad =13.410 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.734}{2} \quad - 13.410 \quad =|-3.543 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5,875.91}{514.40} \quad =11.423$$

> 1.30 … -OK-

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{71,926.51}{28.77} \quad =2,500.052$$

> 1.50 … -OK-

- Safety factor due to sliding

					48
V=	5,361.51	kN		FSD-φ	1.50
H=	21.26	kN		FSD-c	3.00
L=	19.734	m		tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5,361.51*0.78}{1.50} \quad =2,787.99$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.734}{3.00} \quad =6,578.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{2,787.99+6,578.00}{21.26} \quad =440.545$$

> 1.0 … -OK-

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5,361.51}{19.734} \times (1.0 \pm \frac{6 \times 3.543}{19.734})$$

vertical stress of upstream = 564.360 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = -20.982 kN/㎡ ＜ 0 kN/㎡ (Tensile force occur) but downstream side -OK-

- CCE: Maximum flood water

Resume of Acting Force and Moment

[CCE : Maximum Flood water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,866.42		13.156		77,178.62		
W/O Dead Load	-14.69		7.934		-116.55		
Seismic							
W/O Seismic							
U/S Water weight							
D/S Water weight	401.00		2.637		1,057.44		
U/S Water Pressure		3,137.51		8.350		26,198.21	
D/S Water Pressure		-514.10		3.380		-1,737.66	
Dynamic Water Pressure							
Earth Pressure							
Soil weight		1.38		0.300		0.41	
Uplift	-2,491.42		10.514		-26,194.79		
Total	3,761.31	2,624.79			51,924.72	24,460.96	

Control of Stability [CCE]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{27{,}463.76}{3{,}761.31} \quad = 7.302\ \text{m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.734}{2} \quad - 7.302 \qquad = |2.565\ \text{m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}252.73}{2{,}491.42} \quad = 2.510 \qquad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{51{,}924.72}{24{,}460.96} \quad = 2.123 \qquad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to sliding

				49
V=	3,761.31	kN	FSD-φ	1.10
H=	2,624.79	kN	FSD-c	1.50
L=	19.734	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{3{,}761.31 * 0.78}{1.10} \quad = 2{,}667.11$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0 * 19.734}{1.50} \quad = 13{,}156.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{2{,}667.11 + 13{,}156.00}{2{,}624.79} \quad = 6.028 \qquad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{3{,}761.31}{19.734} \quad \times (1.0 \pm \quad \frac{6 \times 2.565}{19.734})$$

vertical stress of upstream = 41.956 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 339.245 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCL: Flood water + Seismic

Resume of Acting Force and Moment

[CCL : Flood water + Seismic]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,866.42		13.156		77,178.62		
W/O Dead Load	-14.69		7.934		-116.55		
Seismic	-175.99	293.32	13.156	8.433	-2,315.36	2,473.67	
W/O Seismic	0.44	-0.73	7.934	32.047	3.50	-23.54	
U/S Water weight							
D/S Water weight	73.80		1.132		83.54		
U/S Water Pressure		2,576.45		7.567		19,496.00	
D/S Water Pressure		-94.61		1.450		-137.18	
Dynamic Water Pressure		150.29		9.080		1,364.63	
Earth Pressure							
Soil weight		1.38		0.300		0.41	
Uplift	-1,461.96		11.225		-16,410.50		
Total	4,288.02	2,926.10			58,423.25	23,173.99	

Control of Stability [CCL]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{35,249.26}{4,288.02} \quad = 8.220 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.734}{2} \quad - 8.220 \quad = |1.647 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5,749.98}{1,461.96} \quad = 3.933$$

> 1.10 … -OK-

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{58,423.25}{23,173.99} \quad = 2.521$$

> 1.10 … -OK-

- Safety factor due to sliding

				50
V=	4,288.02	kN	FSD-φ	1.10
H=	2,926.10	kN	FSD-c	1.30
L=	19.734	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4,288.02*0.78}{1.10} \quad = 3,040.60$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.734}{1.30} \quad = 15,180.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,040.60+15,180.00}{2,926.10} \quad = 6.227$$

> 1.0 … -OK-

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4,288.02}{19.734} \quad \times (1.0 \pm \frac{6\times1.647}{19.734})$$

vertical stress of upstream = 108.480 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 326.101 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCC: Construction

Resume of Acting Force and Moment

[CCC : Construction]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,866.42		13.156		77,178.62		
W/O Dead Load	-14.69		7.934		-116.55		
Seismic							
W/O Seismic							
U/S Water weight							
D/S Water weight							
U/S Water Pressure							
D/S Water Pressure							
Dynamic Water Pressure							
Earth Pressure							
Soil weight							
Uplift							
Total	5,851.73				77,062.07		

Control of Stability [CCC]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{77{,}062.07}{5{,}851.73} \quad =13.169 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.734}{2} \quad - 13.169 \quad =|-3.302 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5{,}851.73}{0.00} \quad = \infty \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{77{,}062.07}{0.00} \quad = \infty \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to sliding

				51
V=	5,851.73	kN	FSD-φ	1.30
H=	0.00	kN	FSD-c	2.00
L=	19.734	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5{,}851.73*0.78}{1.30} \quad =3{,}511.04$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*19.734}{2.00} \quad =9{,}867.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}511.04+9{,}867.00}{0.00} \quad = \infty \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5{,}851.73}{19.734} \quad \times(1.0\pm \quad \frac{6\times 3.302}{19.734})$$

vertical stress of upstream = 594.233 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = -1.172 kN/㎡ ＜ 0 kN/㎡ (Tensile force occur) but downstream side -OK-

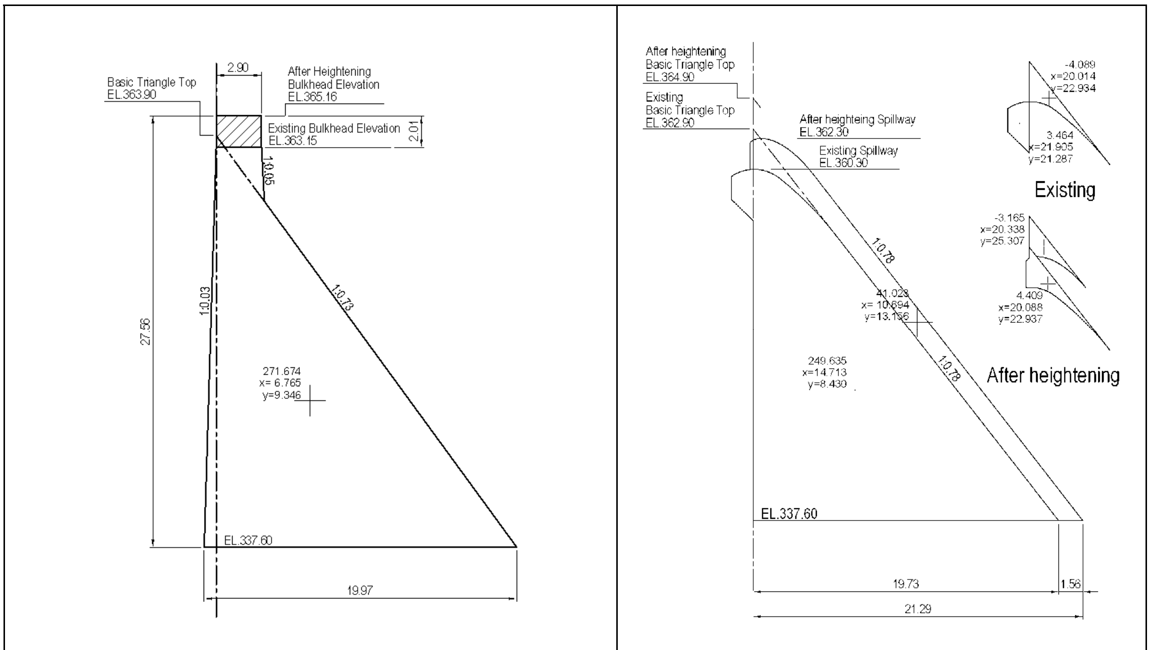
(2) After heightening

1) Design Condition

Design condition of Dam stability analysis is considered as shown in the table 2 below.

Table 2 Design condition of After heightening

		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	←



Determinatimg Bulkhead Section	Determinatimg Spillway section

Fig 2 After heightening Bulkhead section

2) Stability Analysis of after heightening

[Bulkhead section]

- CCN: Normal water

Resume of Acting Force and Moment

[CCN : Normal water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,384.34		13.201		84,276.48		
Mat section							
W/O Dead Load							
Seismic							
Seismic of mat							
W/O Seismic							
U/S Water weight	1.53		19.934		30.50		
D/S Water weight	22.63		19.360		438.12		
U/S Water Pressure		50.88		1.063		54.09	
D/S Water Pressure		-31.00		0.830		-25.73	
Dynamic Water Pressure							
Earth Pressure	0.10		19.957		2.00		
Soil weight		1.38		0.300		0.41	
Uplift	-520.43		10.132		-5,272.78		
Total	5,888.17	21.26			79,474.32	28.77	

Control of Stability [CCN]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{79{,}445.55}{5{,}888.17} \quad =13.492 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} \quad - 13.492 \quad =|-3.509 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}408.60}{520.43} \quad =12.314 \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{79{,}474.32}{28.77} \quad =2{,}762.403 \quad > 1.50 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	5,888.17	kN	FSD-φ	1.50
H=	21.26	kN	FSD-c	3.00
L=	19.966	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5{,}888.17*0.78}{1.50} \quad =3{,}061.85$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*19.966}{3.00} \quad =6{,}655.33$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}061.85+6{,}655.33}{21.26} \quad =457.064 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5{,}888.17}{19.966} \times (1.0 \pm \frac{6 \times 3.509}{19.966})$$

vertical stress of upstream = 605.913 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = -16.079 kN/㎡ ＜ 0 kN/㎡ (Tensile force occur) but downstream side -OK-

- CCE: Maximum flood water

Resume of Acting Force and Moment

[CCE : Maximum Flood water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,384.34		13.201		84,276.48		
Mat section							
W/O Dead Load							
Seismic							
Seismic of mat							
W/O Seismic							
U/S Water weight	109.50		19.697		2,156.77		
D/S Water weight	375.29		17.498		6,566.82		
U/S Water Pressure		3,661.22		9.020		33,024.20	
D/S Water Pressure		-514.10		3.380		-1,737.66	
Dynamic Water Pressure							
Earth Pressure	0.10		19.957		2.00		
Soil weight		1.38		0.300		0.41	
Uplift	-2,587.53		10.707		-27,703.39		
Total	4,281.70	3,148.50			65,298.68	31,286.95	

Control of Stability [CCE]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{34{,}011.73}{4{,}281.70} = 7.944 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} - 7.944 = |2.039 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}869.23}{2{,}587.53} = 2.655 \quad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{65{,}298.68}{31{,}286.95} = 2.087 \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	4,281.70	kN	FSD-φ	1.10
H=	3,148.50	kN	FSD-c	1.50
L=	19.966	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4{,}281.70 * 0.78}{1.10} = 3{,}036.11$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0 * 19.966}{1.50} = 13{,}310.67$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}036.11 + 13{,}310.67}{3{,}148.50} = 5.192 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4{,}281.70}{19.966} \times (1.0 \pm \frac{6 \times 2.039}{19.966})$$

vertical stress of upstream = 83.046 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 345.864 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCL: Flood water + Seismic

Resume of Acting Force and Moment

[CCL : Flood water + Seismic]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,384.34		13.201		84,276.48		
Mat section							
W/O Dead Load							
Seismic	-191.53	319.22	13.201	9.346	-2,528.29	2,983.40	→
Seismic of mat							→
W/O Seismic							→
U/S Water weight	91.51		19.719		1,804.44		
D/S Water weight	72.60		18.881		1,370.76		
U/S Water Pressure		3,050.45		8.233		25,114.35	
D/S Water Pressure		-99.46		1.487		-147.90	
Dynamic Water Pressure		177.94		9.880		1,758.05	
Earth Pressure	0.10		19.957		2.00		
Soil weight		1.38		0.300		0.41	
Uplift	-1,563.96		11.416		-17,853.39		
Total	4,793.06	3,449.53			67,072.00	29,708.31	

Control of Stability [CCL]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{37,363.69}{4,793.06} \quad =7.795 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} \quad - 7.795 \quad =|2.188 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,357.02}{1,563.96} \quad =4.065 \quad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{67,072.00}{29,708.31} \quad =2.258 \quad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	4,793.06	kN	FSD-φ	1.10
H=	3,449.53	kN	FSD-c	1.30
L=	19.966	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4,793.06*0.78}{1.10} \quad =3,398.72$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.966}{1.30} \quad =15,358.46$$

$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,398.72+15,358.46}{3,449.53} \quad =5.438 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4,793.06}{19.966} \times (1.0 \pm \frac{6 \times 2.188}{19.966})$$

vertical stress of upstream = 82.215 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 397.919 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCC: Construction

Resume of Acting Force and Moment

[CCC : Construction]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,384.34		13.201		84,276.48		
Mat section							
W/O Dead Load							
Seismic							
Seismic of mat							
W/O Seismic							
U/S Water weight							
D/S Water weight							
U/S Water Pressure							
D/S Water Pressure							
Dynamic Water Pressure							
Earth Pressure							
Soil weight							
Uplift							
Total	6,384.34				84,276.48		

Control of Stability [CCC]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{84,276.48}{6,384.34} \quad =13.200 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.966}{2} \quad - 13.200 \quad =|-3.217 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,384.34}{0.00} \quad = \infty \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{84,276.48}{0.00} \quad = \infty \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	6,384.34 kN		FSD-φ	1.30
H=	0.00 kN		FSD-c	2.00
L=	19.966 m		tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{6,384.34*0.78}{1.30} \quad =3,830.60$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*19.966}{2.00} \quad =9,983.00$$

$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,830.60+9,983.00}{0.00} \quad = \infty \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{6,384.34}{19.966} \quad \times (1.0 \pm \frac{6 \times 3.217}{19.966})$$

vertical stress of upstream = 628.911 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 10.627 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

[Spillway section]

- CCC: Construction

Resume of Acting Force and Moment

[CCC : Construction]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,851.74		13.169		77,061.56		
Seismic					0.00	0.00	
U/S Water pressure,weight	0.00	45.00	19.734	1.000	0.00	45.00	
D/S Water pressure,weight					0.00	0.00	
Dynamic Water Pressure					0.00	0.00	
Earth Pressure	0.00		19.734		0.00	0.00	
Soil weight		1.38		0.300	0.00	0.41	
Uplift	-98.67	0.00	13.156	0.000	-1,298.10	0.00	
Total	5,753.07	46.38			75,763.46	45.41	

Control of Stability [CCC]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{75{,}718.05}{5{,}753.07} \quad =13.161 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{19.734}{2} - 13.161 \quad =|-3.294 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5{,}851.74}{98.67} \quad =59.306 \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{75{,}763.46}{45.41} \quad =1{,}668.431 \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	5,753.07	kN	FSD-φ	1.30
H=	46.38	kN	FSD-c	2.00
L=	19.734	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5{,}753.07 * 0.78}{1.30} \quad =3{,}451.84$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0 * 19.734}{2.00} \quad =9{,}867.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}451.84+9{,}867.00}{46.38} \quad =287.168 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5{,}753.07}{19.734} \times (1.0 \pm \frac{6 \times 3.294}{19.734})$$

vertical stress of upstream = 583.505 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = -0.443 kN/㎡ ＜ 0 kN/㎡ (Tensile force occur)

but downstream side -OK-

- CCN: Normal water

Resume of Acting Force and Moment

[CCN : Normal water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	993.39		11.287		11,212.39		
Seismic							
U/S Water pressure,weight	0.00	31.05	21.294	1.730	0.00	53.72	
D/S Water pressure,weight	24.96	32.00	0.658	0.843	16.42	26.99	
Dynamic Water Pressure							
Earth Pressure							
Soil weight							
Uplift	-488.69	0.00	10.179	0.000	-4,974.15	0.00	
Total	529.66	63.05			6,254.66	80.71	

	V(kN)	U(kN)	H(kN)	Me(kN.m)	Mt(kN.m)
[CCC]	5,851.74	-98.67	46.38	75,763.46	45.41
[CCN]	1,018.35	-488.69	63.05	6,254.66	80.71
	6,870.09	-587.36	109.43	82,018.12	126.12

Control of Stability [CCN]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{6{,}173.95}{529.66} = 11.656 \text{ m}$$

- Safety factor due to Lifting

$$e = \frac{B}{2} - x = \frac{21.294}{2} - 11.656 \quad = |-1.009 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}870.09}{587.36} = 11.697 \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{82{,}018.12}{126.12} = 650.318 \quad > 1.50 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	6,282.73	kN	FSD-φ	1.50
H=	109.43	kN	FSD-c	3.00
L=	21.294	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{6{,}282.73*0.78}{1.50} = 3{,}267.02$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*21.294}{3.00} = 7{,}098.00$$

$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}267.02+7{,}098.00}{109.43} = 94.714 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{529.66}{21.294} \times \left(1.0 \pm \frac{6\times1.009}{21.294}\right)$$

(Stress during to construction)

vertical stress of upstream =	31.95 kN/㎡ +	583.51 kN/㎡	= 615.46	kN/㎡	≧ 0 kN/㎡
Existing dam downstream part (-)=	18.83 kN/㎡ +	-0.44 kN/㎡	= 18.39	kN/㎡	≧ 0 kN/㎡
Existing dam downstream part (+)=	(17.79-31.95)×19.734/21.294+31.95		= 18.83	kN/㎡	≧ 0 kN/㎡
vertical stress of downstream =			= 17.79	kN/㎡	≧ 0 kN/㎡

-OK-

- CCE: Maximum flood water

Resume of Acting Force and Moment

[CCE : Maximum Flood water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	993.39		11.287		11,212.39		
Seismic							
U/S Water pressure,weight	0.00	3,616.22	21.294	9.120	0.00	32,979.93	
D/S Water pressure,weight	401.00	514.10	2.636	3.380	1,057.19	1,737.65	
Dynamic Water Pressure							
Earth Pressure							
Soil weight							
Uplift	-2,661.03	0.00	11.297	0.000	-30,061.67	0.00	
Total	-1,266.65	4,130.32			-17,792.09	34,717.58	

	V(kN)	U(kN)	H(kN)	Me(kN.m)	Mt(kN.m)
[CCC]	5,851.74	-98.67	46.38	75,763.46	45.41
[CCE]	1,394.39	-2,661.03	4,130.32	-17,792.09	34,717.58
	7,246.13	-2,759.70	4,176.70	57,971.37	34,762.99

Control of Stability [CCE]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{-52{,}509.67}{-1{,}266.65} = 41.456 \text{ m}$$

41.45555

- Safety factor due to Lifting

$$e = \frac{B}{2} - x = \frac{21.294}{2} - 41.456 = |-30.809 \text{ m}|$$

-30.8085481

1.1

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{7{,}246.13}{2{,}759.70} = 2.626 \quad > 1.10 \ldots \text{-OK-}$$

1.2

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{57{,}971.37}{34{,}762.99} = 1.668 \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to sliding

53

V=	4,486.42	kN	FSD-φ	1.10
H=	4,176.70	kN	FSD-c	1.50
L=	21.294	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4{,}486.42*0.78}{1.10} = 3{,}181.28$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*21.294}{1.50} = 14{,}196.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}181.28+14{,}196.00}{4{,}176.70} = 4.161 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{-1{,}266.65}{21.294} \times \left(1.0 \pm \frac{6\times30.809}{21.294}\right)$$

(Stress during construction)

vertical stress of upstream = -575.88 kN/㎡ + 583.51 kN/㎡ = 7.63 kN/㎡ ≧ 0 kN/㎡

Existing dam downstream part (-)= 381.25 kN/㎡ + -0.44 kN/㎡ = 380.81 kN/㎡ ≧ 0 kN/㎡

Existing dam downstream part (+)= (456.91-575.88)×19.734/21.294-575.88 = 381.25 kN/㎡ ≧ 0 kN/㎡

vertical stress of downstream = = 456.91 kN/㎡ ≧ 0 kN/㎡

-OK-

- CCL: Flood water + Seismic

Resume of Acting Force and Moment

[CCL : Flood water + Seismic]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	993.39		11.287		11,212.39		
Seismic	-205.35	342.26	14.229	9.084	-2,921.93	3,109.09	→
U/S Water pressure,weight	0.00	3,005.45	21.294	8.340	0.00	25,065.45	
D/S Water pressure,weight	84.33	108.11	1.209	1.550	101.95	167.57	
Dynamic Water Pressure	0.00	177.94	21.294	9.880	0.00	1,758.05	
Earth Pressure							
Soil weight							
Uplift	-1,603.08	0.00	11.972	0.000	-19,191.84	0.00	
Total	-730.71	3,633.76			-10,799.43	30,100.16	

	V(kN)	U(kN)	H(kN)	Me(kN.m)	Mt(kN.m)
[CCC]	5,851.74	-98.67	46.38	75,763.46	45.41
[CCL]	872.37	-1,603.08	3,633.76	-10,799.43	30,100.16
	6,724.11	-1,701.75	3,680.14	64,964.03	30,145.57

Control of Stability [CCL]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{-40,899.59}{730.7} \quad = -55.972 \text{ m}$$

- Safety factor due to Lifting

$$e = \frac{B}{2} - x = \frac{21.294}{2} \quad - 55.972 \qquad = |-45.325 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,724.11}{1,701.75} \quad = 3.951 \qquad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{64,964.03}{30,145.57} \quad = 2.155 \qquad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	5,022.36 kN		FSD-φ	1.10
H=	3,680.14 kN		FSD-c	1.30
L=	21.294 m		tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5,022.36*0.78}{1.10} \quad = 3,561.31$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*21.294}{1.30} \quad = 16,380.00$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,561.31+16,380.00}{3,680.14} \quad = 5.419 \qquad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{-730.71}{21.294} \times \left(1.0 \pm \frac{6 \times 45.325}{21.294} \right)$$

(Stress during to construction)

vertical stress of upstream =	-472.51	kN/㎡ +	583.51 kN/㎡	= 111.00	kN/㎡ ≧ 0 kN/㎡
Existing dam downstream part (-)=	339.68	kN/㎡ +	-0.44 kN/㎡	= 339.24	kN/㎡ ≧ 0 kN/㎡
Existing dam downstream part (+)=	(403.88-472.51)×19.734/21.294-472.51			= 339.68	kN/㎡ ≧ 0 kN/㎡
vertical stress of downstream =				= 403.88	kN/㎡ ≧ 0 kN/㎡

-OK-

APPENDIX-7 :

Stability Analysis of Sul dam

(1) Design condition

Design condition of Dam Spillway stability analysis is considered as shown in the table 1 below.

Table 1 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	←

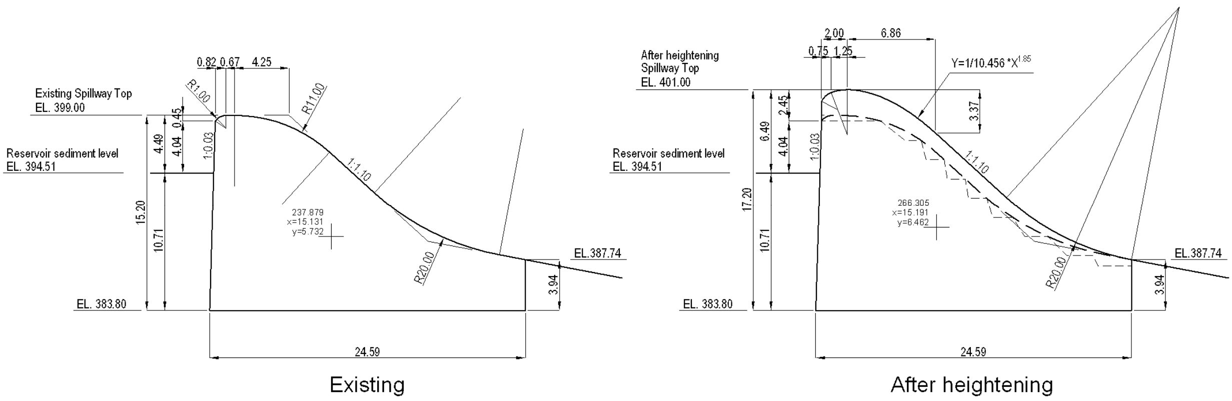


Fig 1 Typical section of Existing

(2) Stability Analysis

1) Existing dam

- CCN: Normal water

Resume of Acting Force and Moment

[CCN : Normal water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,590.16		15.131		84,584.66		
Seismic							
U/S Water Pressure							
Dynamic Water Pressure							
Earth Pressure		401.46		3.570		1,433.23	
Uplift							
Total	5,590.16	401.46			84,584.66	1,433.23	

Control of Stability [CCN]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{83{,}151.43}{5{,}590.16} = 14.875 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{24.590}{2} - 14.875 = |-2.580 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5{,}590.16}{0.00} = \infty \quad > 1.30 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{84{,}584.66}{1{,}433.23} = 59.017 \quad > 1.50 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	5,590.16	kN	FSD-φ	1.50
H=	401.46	kN	FSD-c	3.00
L=	24.590	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5{,}590.16*0.78}{1.50} = 2{,}906.88$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*24.590}{3.00} = 8{,}196.67$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{2{,}906.88+8{,}196.67}{401.46} = 27.658 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5{,}590.16}{24.590} \times \left(1.0 \pm \frac{6 \times 2.580}{24.590}\right)$$

vertical stress of upstream = 370.447 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 84.222 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCE: Maximum flood water

Resume of Acting Force and Moment

[CCE : Maximum Flood water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,590.16		15.131		84,584.66		
Seismic							
U/S Water Pressure		2,219.20		6.281		13,939.41	
Dynamic Water Pressure							
Earth Pressure		195.00		3.570		696.14	
Uplift	-909.83		16.393		-14,915.15		
Total	4,680.33	2,414.20			69,669.51	14,635.55	

Control of Stability [CCE]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{55,033.96}{4,680.33} = 11.759 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{24.590}{2} - 11.759 = |0.536 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5,590.16}{909.83} = 6.144 \quad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{69,669.51}{14,635.55} = 4.760 \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	4,680.33	kN	FSD-φ	1.10
H=	2,414.20	kN	FSD-c	1.50
L=	24.590	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4,680.33*0.78}{1.10} = 3,318.78$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*24.590}{1.50} = 16,393.33$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,318.78+16,393.33}{2,414.20} = 8.165 \quad > 1.0 \ldots \text{-OK-}$$

- Safety factor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4,680.33}{24.590} \times (1.0 \pm \frac{6 \times 0.536}{24.590})$$

vertical stress of upstream = 165.442 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 215.227 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCL: Flood water + Seismic

Resume of Acting Force and Moment

[CCL : Flood water + Seismic]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	5,590.16		15.131		84,584.66		
Seismic	-167.70	279.51	15.131	5.732	-2,537.54	1,602.14	
U/S Water Pressure		1,155.20		5.067		5,853.01	
Dynamic Water Pressure		67.39		6.080		409.71	
Earth Pressure		195.00		3.570		696.14	
Uplift	-622.95		16.393		-10,212.17		
Total	4,799.51	1,697.09			71,834.95	8,561.00	

Control of Stability [CCL]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{63,273.95}{4,799.51} \quad =13.183 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{24.590}{2} \quad - 13.183 \quad =|-0.888 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{5,422.45}{622.95} \quad =8.705$$

> 1.10 … -OK-

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{71,834.95}{8,561.00} \quad =8.391$$

> 1.10 … -OK-

- Safety factor due to sliding

V=	4,799.51	kN	FSD-φ	1.10
H=	1,697.09	kN	FSD-c	1.30
L=	24.590	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{4,799.51*0.78}{1.10} \quad =3,403.29$$

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*24.590}{1.30} \quad =18,915.38$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,403.29+18,915.38}{1,697.09} \quad =13.151$$

> 1.0 … -OK-

ctor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{4,799.51}{24.590} \times (1.0 \pm \frac{6\times0.888}{24.590})$$

vertical stress of upstream = 237.472 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 152.891 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

2) After heightening of dam

- CCN: Normal water

Resume of Acting Force and Moment

[CCN : Normal water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,258.17		15.192		95,074.08		
Seismic							
U/S Water Pressure							
Dynamic Water Pressure							
Earth Pressure		401.46		3.570		1,433.23	
Uplift							
Total	6,258.17	401.46			95,074.08	1,433.23	

Control of Stability [CCN]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{93{,}640.85}{6{,}258.17}$$ =14.963 m

- Excentricity

$$e = \frac{B}{2} - x = \frac{24.590}{2}$$ - 14.963 =|-2.668 m|

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}258.17}{0.00}$$ = ∞ > 1.30 … -OK-

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{95{,}074.08}{1{,}433.23}$$ =66.336 > 1.50 … -OK-

- Safety factor due to sliding

V=	6,258.17	kN	FSD-φ	1.50
H=	401.46	kN	FSD-c	3.00
L=	24.590	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{6{,}258.17*0.78}{1.50}$$ =3,254.25

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*24.590}{3.00}$$ =8,196.67

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}254.25+8{,}196.67}{401.46}$$ =28.523 > 1.0 … -OK-

:tor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{6{,}258.17}{24.590} \times (1.0 \pm \frac{6\times2.668}{24.590})$$

vertical stress of upstream = 420.179 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 88.822 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCE: Maximum flood water

Resume of Acting Force and Moment

[CCE : Maximum Flood water]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,258.17		15.192		95,074.08		
Seismic							
U/S Water Pressure		2,683.20		7.020		18,835.15	
Dynamic Water Pressure							
Earth Pressure		195.00		3.570		696.14	
Uplift	-991.80		16.393		-16,258.85		
Total	5,266.37	2,878.20			78,815.23	19,531.29	

Control of Stability [CCE]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{59{,}283.94}{5{,}266.37} \quad = 11.257 \text{ m}$$

- Excentricity

$$e = \frac{B}{2} - x = \frac{24.590}{2} - 11.257 \quad = |1.038 \text{ m}|$$

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6{,}258.17}{991.80} \quad = 6.310 \quad > 1.10 \ldots \text{-OK-}$$

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{78{,}815.23}{19{,}531.29} \quad = 4.035 \quad > 1.20 \ldots \text{-OK-}$$

- Safety factor due to sliding

V=	5,266.37	kN	FSD-φ	1.10
H=	2,878.20	kN	FSD-c	1.50
L=	24.590	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5{,}266.37*0.78}{1.10} \quad = 3{,}734.34$$

$$\frac{c \cdot l}{FSD_c} = \frac{1{,}000.0*24.590}{1.50} \quad = 16{,}393.33$$

$$FSD = \frac{\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \frac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3{,}734.34+16{,}393.33}{2{,}878.20} \quad = 6.993 \quad > 1.0 \ldots \text{-OK-}$$

ctor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5{,}266.37}{24.590} \times (1.0 \pm \frac{6 \times 1.038}{24.590})$$

vertical stress of upstream = 159.924 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 268.410 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

- CCL: Flood water + Seismic

Resume of Acting Force and Moment

[CCL : Flood water + Seismic]

---	V(kN)	H(kN)	X(m)	Y(m)	Me(kN.m)	Mt(kN.m)	Remark
Dead load	6,258.17		15.192		95,074.08		
Seismic	-187.75	312.91	15.192	6.462	-2,852.22	2,022.01	
U/S Water Pressure		1,479.20		5.733		8,480.75	
Dynamic Water Pressure		86.29		6.880		593.65	
Earth Pressure		195.00		3.570		696.14	
Uplift	-704.91		16.393		-11,555.88		
Total	5,365.51	2,073.39			80,665.98	11,792.55	

Control of Stability [CCL]

- Barycentric position

$$x = \frac{Mx + My}{V} = \frac{68,873.43}{5,365.51}$$ =12.836 m

- Excentricity

$$e = \frac{B}{2} - x = \frac{24.590}{2}$$ - 12.836 =|-0.541 m|

- Safety factor due to Lifting

$$FSF = \frac{\Sigma V}{\Sigma U} = \frac{6,070.42}{704.91}$$ =8.612 > 1.10 … -OK-

- Safety factor due to overturning

$$FST = \frac{\Sigma Me}{\Sigma Mt} = \frac{80,665.98}{11,792.55}$$ =6.840 > 1.10 … -OK-

- Safety factor due to sliding

V=	5,365.51	kN	FSD-φ	1.10
H=	2,073.39	kN	FSD-c	1.30
L=	24.590	m	tanφ	0.78

$$\frac{\Sigma V \cdot \tan\phi}{FSD_\phi} = \frac{5,365.51*0.78}{1.10}$$ =3,804.63

$$\frac{c \cdot l}{FSD_c} = \frac{1,000.0*24.590}{1.30}$$ =18,915.38

F$$FSD = \frac{\dfrac{\Sigma V \cdot \tan\phi}{FSD_\phi} + \dfrac{c \cdot l}{FSD_c}}{\Sigma H} = \frac{3,804.63+18,915.38}{2,073.39}$$ =10.958 > 1.0 … -OK-

:tor due to bearing power

$$q = \frac{V}{B} \times \left(1 \pm \frac{6 \times e}{B}\right) = \frac{5,365.51}{24.590} \times (1.0 \pm \frac{6 \times 0.541}{24.590})$$

vertical stress of upstream = 247.002 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur)

vertical stress of downstream = 189.396 kN/㎡ ≧ 0 kN/㎡ (Tensile force not occur) -OK-

Supporting Report (H)
Economic Evaluation

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

FINAL REPORT

VOLUME III : SUPPORTING REPORT
ANNEX H: ECONOMIC EVALUATION

Table of Contents

<u>**Table**</u>

<u>Figure</u>

CHAPTER 1 METHODOLOGY OF ECONOMIC EVALUATION

1.1 Evaluation Criteria

In economic and financial evaluations, the followings criteria were applied;

– Applied price for the cost and benefit estimation is of a base of year 2010.

– The evaluation will be made for whole program for each return period of 5, 10, 25 and 50 years.

– The evaluation period is of 50 years.

– The evaluations will be carried out as a total program of the mitigation measure for disasters of flood, flashflood and prevention / alert / alarm.

– The evaluation will be carried out the financial and economical point of view. In the financial evaluation, the market price will be applied and for the economical evaluation, the discounted price excluded the taxes and the compensations fees, will be applied.

– In an evaluation, the concept of the Net Present Value (NPV), Internal Rate of Return (IRR), and Benefit/Cost Ratio (B/C) will be used.

– As a discount rate for the estimation of NPV and B/C, the commonly used rate of 12%, rate calculated from the Certificate of Inter banking Deposit’s Rate and the rate of the long term Interest (TJLP) in last 9 years will be utilized.

– The estimated benefit values for each safety level will be calculated by statistic method, on the basis of the registered disaster’s damages value published by the State Government. The medium annual benefit will be considered multiplying the probabilities of each inundation and the damages caused by each safety level. Besides this, also, the benefit from land valorizations with improvement of safety level will be possible. However, this kind of benefit, in this evaluation, will not be considered.

– The values used as bases of damages estimation for each safety level were of flood damages registered at October, 2001 and November, 2008.

– The flood in October, 2001 was evaluated to the 7 – year flood, and the flood in November, 2008 was considered as the 50 – year flood.

1.2 Tax

The taxes included in a cost in Brazil are as followings items;

Table 1.2.1 Rate of Taxes in Brazil

Tax	Tax Objective	Rate
Federal Tax		
Physical Person Income Tax IRPF	Percentage for each salary	7.5%、15%、22%、27.5%
Judicial Person Income Tax IRPJ	Companies profit	15% / 25%
Industries Product Tax (IPI)	Charged for the industrialized products, national and foreigners. The field of incidence of the tax includes all of the products with index allocation, although, it reduce to zero, observed the dispositions contained in the respective complementally notes, excluded those that corresponds the (no-taxed) notation "NT."	Related in the Table of Incidence of IPI (TIPI)

Tax	Tax Objective	Rate
Import Tax (II)	Imported product	Goods, import origin, volumes
Financial Operation Tax (IOF)	Tax on operations of credit, exchange and security, or relative to titles and real estate values	
State Tax		
Tax for Circulated Good and Services (ICMS)	Tax about relative operations to the circulation of goods and services rendered of interstate transport, inter municipal and of communication.	17% or 25%
Tax for Properties of Vehicles Terrestrial (IPVA)	On the property of vehicle	Type of vehicles
Municipal Tax		
Tax for Services (ISS)	Rendered service (cleaning of properties, safety, building site, labor supply)	3% or 5%
Social Contribution		
Contribution for the Social Security Finance(COFINS)		3% or 7.6%
Social Integration Program (PIS PASEP)	Totality of the incomes gained by the legal entity	065 - 1.65%
Social Contribution over net Profit (CSLL)	Conceited profit will correspond the: 12% of the gross revenue in the activities commercial, industrial, services hospitalizes and of transport	9%
Others Contribution		
National Institution of Social Security (INSS)	Executed by discount in the payroll, before the employee of the company to receive the total value of salary.	Salaried；11% Employer；20%
Grantee Fond for Working period (FGTS)	Executed by discount in the payroll, before the employee of the company to receive the total value of salary.	2% or 8% In the rescission of the labor agreement ; 40%

Source: JICA Survey Team, http://www.receita.fazenda.gov.br/

1.3 Conversion Rate (for Economic Evaluation)

The applied price for the economic evaluation is considered the economic price using a conversion rate. In this study, the conversion rate for estimation of economic price, a conversion value of 0.5 is used. Detailed information is shown below.

Table 1.3.1 Conversion Rate between Tax Rate and Construction Works

Item	Rate	Total Tax	Conversion Rate	Weighted Value	Considered Tax
Salary	15%+11%+20%+8.8% =54.8%	93.7%	0.52	30%	IRPF, INSS, FGTS
Materials	20%	50.2%	0.67	20%	ICMS
Fuel	107%	159.0%	0.39	20%	ICMS, PIS, COFINS, IRPJ, CSLL
Machineries	47%+20%+3%=70%	112.7%	0.47	20%	IPI, ICMS, IPVA
Imported Machineries	47%+30%+20%+3%=100%	150.3%	0.40	10%	IPI, II, ICMS, IPVA
Administration	1.5%+5%+7.6%+1.65%+9% +0.38% =25.13%				IRPJ, ISS, COFINS, CSLL, PIS
Weighted			0.50		

Source: JICA Study Team

In this estimation, proportion of each item to the total construction cost is assumed to respective percents as the weighted value in the table above. Further, a contractor is subject to administration tax of 25.13%.

1.4 Discount Rate

The discount rate applied for the financial evaluation is considered the rate of Certificate of Inerbanking Deposit (CDI) and for the economical evaluation, the Tax of Interest the Long term was considered. The annual medium taxes of the considered each years are the following ones:

Table 1.4.1 Tax of CDI & TJLP

Year	CDI	TJLP
2009	9.88%	6.00%
2008	12.38%	6.00%
2007	11.81%	6.50%
2006	15.04%	9.00% - 6.85%
2005	19.00%	9.75%
2004	16.16%	10.00% - 9.75%
2003	23.26%	11.00%- 12.00%
2002	19.10%	9.50% - 10.00%
2001	17.27%	9.25% - 10.00%

Source: Dados de BACEN http://www.portalbrasil.net/indices_cdi.htm

On the base of the indicated rate above, the discount rate is the following ones

Table 1.4.2 Discount Rate

	Financial Evaluation	Economic Evaluation
Discount rate (1)	10.0 %	6.0%
Discount rate (2)	23.0 %	10.0 %
Referred Discount Rate	12.0 %	12.0 %

Source: JICA Study Team

The discount rate (1) is the value when the economy of Brazil is stable. The discount rate (2) is the value for the economy of Brazil is in situation of high interest rate.

CHAPTER 2 ECONOMIC & FINANCIAL EVALUATION FOR MASTER PLAN

2.1 Cost

The required cost for the mitigations of the disasters is the following ones:

Table 2.1.1 Cost for Each Return Period

(Unit: R$ thousand)

		5 year	10 year	25 year	50 year
Flood Mitigation Measure	Direct Cost	99,000	155,000	399,000	831,000
	Land Compensation	72,000	296,000	435,000	779,000
	Engineering	7,000	12,000	37,000	80,000
	Administration	3,000	10,000	20,000	41,000
	Physical Contingency	14,000	43,000	86,000	170,000
	Price Escalation	8,000	24,000	47,000	94,000
	Subtotal	202,000	541,000	1,025,000	1,996,000
Land Slide Mitigations Measures	Direct Cost	42,000	42,000	42,000	42,000
	Engineering	4,200	4,200	4,200	4,200
	Administration	1,500	1,500	1,500	1,500
	Physical Contingency	4,200	4,200	4,200	4,200
	Price Escalation	2,100	2,100	2,100	2,100
	Subtotal	54,000	54,000	54,000	54,000
Flood Prevention and Alert	Equipment	2,400	2,400	2,400	2,400
	Inventory Study	900	900	900	900
	Training	300	300	300	300
	Engendering	400	400	400	400
	Subtotal	4,000	4,000	4,000	4,000
Land Slide Preventi on and	Installation and Equipments	2,300	2,300	2,300	2.300
	Program	1,700	1,700	1,700	1.700
	Subtotal	4,000	4,000	4,000	4.000
Total		264.000	603,000	1,087,000	2,058,000

Source: JICA Survey Team

2.1.1 Cost in a Market Price

(1) Cost for safety level

The annual cost and maintenance cost for each safety level are considered the following ones;

Table 2.1.2 Annual Cost for Each Return Period

(Unit: R$ thousand)

Safety Level	Total cost	1st year	2nd year	3rd year	4th year	5th year
5 year	264,000	88,000	88,000	88,000		
10 year	603,000	201,000	201,000	201,000		
25 year	1,087,000	271,750	271,750	271,750	271,750	
50 year	2,058,000	411,600	411,600	411,600	411,600	411,600

Source: JICA Survey Team

(2) Maintenance Cost

The maintenance cost is considered 5% of the total construction cost;

Table 2.1.3 Maintenance Cost (R$ thousand)

Safety Level	Total Cost	Maintenance Cost
5 year	264,000	13,200
10 year	603,000	30,200
25 year	1,087,000	54,400
50 year	2,058,000	102,900

Source: JICA Study Team

2.1.2 Economic Cost – Economic Values

(1) Economic cost for each safety level

The economic cost to be applied for the economic evaluation is estimated by applying the conversion rate. The schedule of cost application is shown in the following table.

Table 2.2.4 Application of Annual Cost in Economic Price (R$ thousand)

Safety Level	Total Cost	1st year	2nd year	3rd year	4th year	5th year
5 year	132,000	44,000	44,000	44,000		
10 year	302,000	100,667	100,667	100,667		
25 year	544,000	136,000	136,000	136,000	136,000	
50 year	1,029,000	205,800	205,800	205,800	205,800	205,800

Source: JICA Survey Team

(2) Maintenance cost

The maintenance cost is considered 5% of the total construction cost;

Table 2.1.5 Estimated Maintenance Cost (R$ thousand)

Safety Level	Total Cost	Maintenance Cost
5 year	132,000	6,600
10 year	302,000	15,100
25 year	544,000	27,200
50 year	1,029,000	51,500

Source: JICA Survey Team

2.2 Benefit

2.2.1 Accounting Method of Benefit

In this Study, as a benefit, the estimated damages that will be caused by disasters for each safety level as the effect of the adopted measures are considered. The damages caused by the disasters are considered the following ones:

The mentioned losses will be minimized by the implementation of the measures for inundations. With this concept, the benefits of measures were considered and classified as:

– Emergency expenses

– Cost of reconstruction works

– Losses in the economic activities (agricultural, trade, industry and transport sectors)

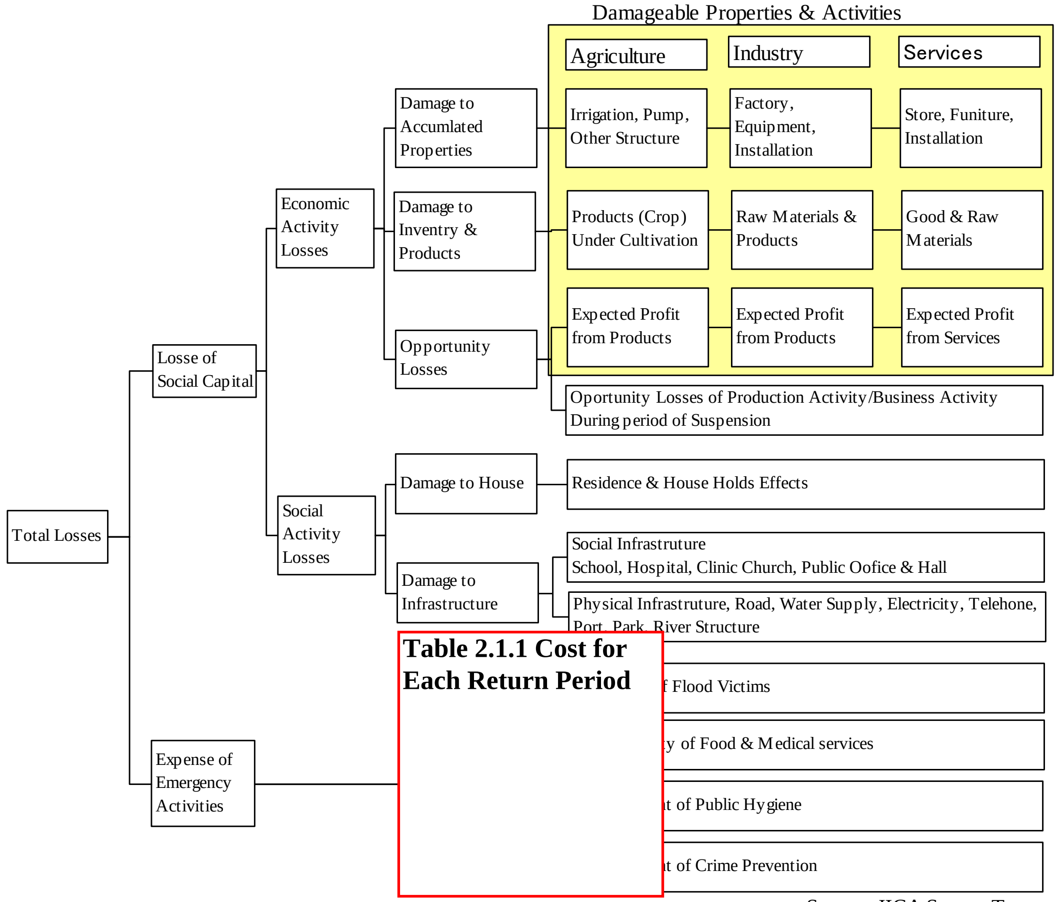
Besides the listed benefits, the possibility of land valorization exists with the improvement of safety's degree, however, this valorization was not considered.

The human damages by death and wounded, were not considered as a benefit, due to the accountancy difficulties.

The emergency expenses are those applied in the public calamities, rescue, expenses with shelters, health, feeding, etc.

The expenses of the reconstructions are those expenses with the works of reconstructions in the affected areas for the catastrophe, as ports, highways, electrification, sanitation, school, hospital, etc.

The economical losses were estimated by the difference between annual economic productions of years with and without disaster. The items considered to estimate the economical loss were of agricultural production, service and transport. The economical losses in the agricultural production were estimated for lost cereals for the disaster. The economical losses in the industry, transport and services were estimated with base in the data of ICMS.



Source: JICA Survey Team

Figure 2.2.1 Concept of Loss in a Disaster

2.2.2 Benefit at Market Price

(1) Emergencies expenses and reconstruction cost

The expenses and the costs of reconstructions in the inundations of October, 2001 and November, 2008 were the following ones:

Table2.2.1 Emergencies Expenses and Reconstruction Costs (R$ million)

	Flood in 2001	Flood in 2008
Emergency Expenses	12.6	656.5
Reconstruction Cost		2,065.8
2001/2008 Conversion Rate	2.78	1
Values at year 2008	34.9	2,065.8

Source:
(1) Plano de Recursos Hídricos da Bacia Hidrográfica do Rio Itajaí 2010, elaborado pela JICA Study Team
(2) Relatório “Reconstrução das Áreas Afetadas Catástrofe Novembro/2008” Gov. SC novembro de 2009.

Based on the data of production of rice, the estimated value of the agricultural section was calculated. The estimated values are in time of normality and time of disaster:

Table 2.2.2 Estimated Economic Loss Values in a Agricultural Sector (R$ thousand)

	Flood in 2001			Flood in 2008		
	2000	2001	2002	2008	2009	Difference
Blumenau	67	68	67	140	140	0
Brusque	95	202	293	630	630	0
Gaspar	6,912	7,168	7,654	13,940	8,500	(5,440)
Ilhota	3,640	3,120	6,119	13,312	5,857	(7,455)
Itajaí	3,360	4,742	5,824	9,660	6,048	(3,612)
Subtotal of 5 municipalities	14,074	15,300	19,957	37,682	21,175	(16,507)
Medium of 2000 & 2002		17,016				
Estimated loss		(1,716)				(16,507)
Values in 2009		(4,770)				(16,507)

Source: JICA Survey Team

(3) Economical loss in the services sector

The economical losses in the services sector were estimated using the data of variations of ICMS as follows:

Table 2.2.3 Estimated Economic Loss in ICMS (R$ thousand)

	Flood in 2001			Flood in 2008		
	2000	2001	2002	Real	Without flood	
Blumenau	178,604	173,034	185,664	292,980	451,285	
Brusque	44,489	42,867	44,276	90,124	140,728	
Ilhota	313	424	442	476	1,132	
Itajaí	62,180	76,397	164,634	366,299	575,301	
Subtotal of 4 cities	301,955	309,209	410,748	749,880	1,168,447	
Mediums of 2000 & 2002		356,352				
Economic Loss		(23,789)			(418,067)	
Price in 2009		(66,135)				

Source: JICA Survey Team

The total amount of ICMS in the State occupies around 5% of the State's GDP. The services sector contributes around 50% of the State's GDP. Thus the economic loss in the services sector is estimated by 10 times of the decrease of ICMS.

(4) Estimated Total Economic Loss

The economical losses considered by the flood inundations in October of 2001 (return period of 7 years) and in November of 2008 (return period of 50 years) can be estimated as follows:

Table2.2.4 Economic Losses by Flood (Unit; R$ million)

Safety Level	Emergency Expenses & Reconstruction	Agriculture	ICMS	Services	Total
7 year	34.9	4.4	66.1	661.3	700.7
50 year	2,722.3	19.5	418.0	4,180.0	6,921.8

Source: JICA Survey Team
Remarks: Total does not include ICMS.

(5) Estimated Probable Disaster Damage (Economic Loss)

The estimates of probable disaster damage for each return period are indicated in the following table. It is noted that agricultural loss is not considered because the agricultural area is expected as a flood retarding area without protection.

Table 2.2.5 Estimated Probable Disaster Damage (R$ million)

Safety Level	Emergency Expenses & Reconstruction	Services	Total	GDP in Basin	Rate to GDP (%)
2 year	2.2	204.1	206.3	34,110.0	0.6%
5 year	16.6	482.1	498.6	34,110.0	1.5%

Safety Level	Emergency Expenses & Reconstruction	Services	Total	GDP in Basin	Rate to GDP (%)
7 year	34.9	661.3	695.9	34,110.0	2.1%
10 year	76.9	923.7	1,000.6	34,110.0	3.0%
25 year	585.9	2,181.8	2,767.7	34,110.0	8.1%
50 year	2,721.8	4,180.0	6,902.1	34,110.0	20.3%

Source: JICA Survey Team

(6) Expected Annual Average Disaster Prevention Benefit

The expected annual average disaster prevention benefit for each return period was estimated by the following equation.

Expected annual average disaster prevention benefit = Σ[(Pn - Pm) x （Dn＋Dm/2]

where, Pn, Pm: Probability of occurrence of the return period of n-year and m-year

Dn, Dm: Probable disaster damage of the return period of n-year and m-year

The estimated annual average disaster prevention benefit is as follows:

Table2.2.6 Expected Annual Average Disaster Prevention Benefit by Return Period (R$ million)

Safety Level	Probable Disaster Damage	5 year	10 year	25 year	50 year
2 year	206.3	51.6	51.6	51.6	51.6
5 year	498.6	105.7	105.7	105.7	105.7
10 year	1,000.6		75.0	75.0	75.0
25year	2,767.7			113.0	113.0
50 year	6,902.1				96.7
Expected Annual Disaster Prevention Benefit		157.3	232.3	345.3	442.0

Source: JICA Survey Team

2.2.3 Economic Benefit

(1) Estimated Probable Disaster Damage in Economic Value

The estimation of probable disaster damage in economic value for each safety level was converted using the conversion factor of 0.5 as described in subsection 1.3.1. The estimated probable disaster damage in economic value is as follows:

Table 2.2.7 Estimated Probable Disaster Damage in Economic Value (R$ millions)

Safety Level	Emergency Expenses & Reconstruction	Service	Total
2 year	1.1	102.0	103.1
5 year	8.3	241.0	249.3
10 year	38.5	461.8	500.3
25 year	293.0	1,090.9	1,383.9
50 year	1,360.9	2,090.2	3,451.1

Source: JICA Survey Team

(2) Expected annual value of the mitigations of the economical losses for the interventions for each Return period (Price without tax and without compensation)

The estimated annual average disaster prevention benefit for each safety level in economic value is summarized below.

Table 2.2.8 Expected Annual Average Disaster Prevention Benefit by Return Period in Economic Value

(Unit: R$ million)

Safety Level	Probable Disaster Damage	5 year	10 year	25 year	50 year
2 year	103.1	25.8	25.8	25.8	25.8
5 year	249.3	52.9	52.9	52.9	52.9
10 year	500.3		37.5	37.5	37.5
25 year	1,383.9			56.5	56.5
50 year	3,451.1				48.3
Annual Expected Values of the mitigation of Economic Loss		78.7	116.1	172.7	221.0

Source: JICA Survey Team

2.3 Financial and Economical Evaluations

2.3.1 Financial Evaluation

(1) Cash Flow for 5 year safety level

The cash flow of the Master Plan for 5 year safety level is as follow;

Table 2.3.1 Financial Cost and Benefit Flow (5 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	88.0	0.0	88.0	0.0	-88.0
2	88.0	0.0	88.0	0.0	-88.0
3	88.0	0.0	88.0	0.0	-88.0
4		13.2	13.2	157.3	144.1
5		13.2	13.2	157.3	144.1
6		13.2	13.2	157.3	144.1
7		13.2	13.2	157.3	144.1
8		13.2	13.2	157.3	144.1
9		13.2	13.2	157.3	144.1
10		13.2	13.2	157.3	144.1
11		13.2	13.2	157.3	144.1
12		13.2	13.2	157.3	144.1
13		13.2	13.2	157.3	144.1
14		13.2	13.2	157.3	144.1
15		13.2	13.2	157.3	144.1
16		13.2	13.2	157.3	144.1
17		13.2	13.2	157.3	144.1
18		13.2	13.2	157.3	144.1
19		13.2	13.2	157.3	144.1
20		13.2	13.2	157.3	144.1
21		13.2	13.2	157.3	144.1
22		13.2	13.2	157.3	144.1
23		13.2	13.2	157.3	144.1
24		13.2	13.2	157.3	144.1
25		13.2	13.2	157.3	144.1
26		13.2	13.2	157.3	144.1
27		13.2	13.2	157.3	144.1
28		13.2	13.2	157.3	144.1
29		13.2	13.2	157.3	144.1
30		13.2	13.2	157.3	144.1
31		13.2	13.2	157.3	144.1
32		13.2	13.2	157.3	144.1
33		13.2	13.2	157.3	144.1
34		13.2	13.2	157.3	144.1
35		13.2	13.2	157.3	144.1
36		13.2	13.2	157.3	144.1
37		13.2	13.2	157.3	144.1
38		13.2	13.2	157.3	144.1
39		13.2	13.2	157.3	144.1
40		13.2	13.2	157.3	144.1
41		13.2	13.2	157.3	144.1
42		13.2	13.2	157.3	144.1
43		13.2	13.2	157.3	144.1
44		13.2	13.2	157.3	144.1
45		13.2	13.2	157.3	144.1
46		13.2	13.2	157.3	144.1
47		13.2	13.2	157.3	144.1
48		13.2	13.2	157.3	144.1
49		13.2	13.2	157.3	144.1
50		13.2	13.2	157.3	144.1

Source: JICA Survey Team

(2) Cash Flow for 10 year safety level

The cash flow of the Master Plan for 10 year safety level is as follow;

Table 2.3.2 Financial Cost and Benefit Flow (10 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	201.0	0.0	201.0	0.0	-201.0
2	201.0	0.0	201.0	0.0	-201.0
3	201.0	0.0	201.0	0.0	-201.0
4		30.2	30.2	232.3	202.1
5		30.2	30.2	232.3	202.1
6		30.2	30.2	232.3	202.1
7		30.2	30.2	232.3	202.1
8		30.2	30.2	232.3	202.1
9		30.2	30.2	232.3	202.1
10		30.2	30.2	232.3	202.1
11		30.2	30.2	232.3	202.1
12		30.2	30.2	232.3	202.1
13		30.2	30.2	232.3	202.1
14		30.2	30.2	232.3	202.1
15		30.2	30.2	232.3	202.1
16		30.2	30.2	232.3	202.1
17		30.2	30.2	232.3	202.1
18		30.2	30.2	232.3	202.1
19		30.2	30.2	232.3	202.1
20		30.2	30.2	232.3	202.1
21		30.2	30.2	232.3	202.1
22		30.2	30.2	232.3	202.1
23		30.2	30.2	232.3	202.1
24		30.2	30.2	232.3	202.1
25		30.2	30.2	232.3	202.1
26		30.2	30.2	232.3	202.1
27		30.2	30.2	232.3	202.1
28		30.2	30.2	232.3	202.1
29		30.2	30.2	232.3	202.1
30		30.2	30.2	232.3	202.1
31		30.2	30.2	232.3	202.1
32		30.2	30.2	232.3	202.1
33		30.2	30.2	232.3	202.1
34		30.2	30.2	232.3	202.1
35		30.2	30.2	232.3	202.1
36		30.2	30.2	232.3	202.1
37		30.2	30.2	232.3	202.1
38		30.2	30.2	232.3	202.1
39		30.2	30.2	232.3	202.1
40		30.2	30.2	232.3	202.1
41		30.2	30.2	232.3	202.1
42		30.2	30.2	232.3	202.1
43		30.2	30.2	232.3	202.1
44		30.2	30.2	232.3	202.1
45		30.2	30.2	232.3	202.1
46		30.2	30.2	232.3	202.1
47		30.2	30.2	232.3	202.1
48		30.2	30.2	232.3	202.1
49		30.2	30.2	232.3	202.1
50		30.2	30.2	232.3	202.1

Source: JICA Survey Team

(3) Cash Flow for 25 year safety level

The cash flow of the Master Plan for 25 year safety level is as follow;

Table 2.3.3 Financial Cost and Benefit Flow (25 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	271.8	0.0	271.8	0.0	-271.8
2	271.8	0.0	271.8	0.0	-271.8
3	271.8	0.0	271.8	0.0	-271.8
4	271.8	0.0	271.8	0.0	-271.8
5		54.4	54.4	345.3	291.0
6		54.4	54.4	345.3	291.0
7		54.4	54.4	345.3	291.0
8		54.4	54.4	345.3	291.0
9		54.4	54.4	345.3	291.0
10		54.4	54.4	345.3	291.0
11		54.4	54.4	345.3	291.0
12		54.4	54.4	345.3	291.0
13		54.4	54.4	345.3	291.0
14		54.4	54.4	345.3	291.0
15		54.4	54.4	345.3	291.0
16		54.4	54.4	345.3	291.0
17		54.4	54.4	345.3	291.0
18		54.4	54.4	345.3	291.0
19		54.4	54.4	345.3	291.0
20		54.4	54.4	345.3	291.0
21		54.4	54.4	345.3	291.0
22		54.4	54.4	345.3	291.0
23		54.4	54.4	345.3	291.0
24		54.4	54.4	345.3	291.0
25		54.4	54.4	345.3	291.0
26		54.4	54.4	345.3	291.0
27		54.4	54.4	345.3	291.0
28		54.4	54.4	345.3	291.0
29		54.4	54.4	345.3	291.0
30		54.4	54.4	345.3	291.0
31		54.4	54.4	345.3	291.0
32		54.4	54.4	345.3	291.0
33		54.4	54.4	345.3	291.0
34		54.4	54.4	345.3	291.0
35		54.4	54.4	345.3	291.0
36		54.4	54.4	345.3	291.0
37		54.4	54.4	345.3	291.0
38		54.4	54.4	345.3	291.0
39		54.4	54.4	345.3	291.0
40		54.4	54.4	345.3	291.0
41		54.4	54.4	345.3	291.0
42		54.4	54.4	345.3	291.0
43		54.4	54.4	345.3	291.0
44		54.4	54.4	345.3	291.0
45		54.4	54.4	345.3	291.0
46		54.4	54.4	345.3	291.0
47		54.4	54.4	345.3	291.0
48		54.4	54.4	345.3	291.0
49		54.4	54.4	345.3	291.0
50		54.4	54.4	345.3	291.0

Source: JICA Survey Team

(4) Cash Flow for 50 year safety level

The cash flow of the Master Plan for 50 year safety level is as follow;

Table 2.3.4 Financial Cost and Benefit Flow (50 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	411.6	0.0	411.6	0.0	-411.6
2	411.6	0.0	411.6	0.0	-411.6
3	411.6	0.0	411.6	0.0	-411.6
4	411.6	0.0	411.6	0.0	-411.6
5	411.6	0.0	411.6	0.0	-411.6
6		102.9	102.9	442.0	339.1
7		102.9	102.9	442.0	339.1
8		102.9	102.9	442.0	339.1
9		102.9	102.9	442.0	339.1
10		102.9	102.9	442.0	339.1
11		102.9	102.9	442.0	339.1
12		102.9	102.9	442.0	339.1
13		102.9	102.9	442.0	339.1
14		102.9	102.9	442.0	339.1
15		102.9	102.9	442.0	339.1
16		102.9	102.9	442.0	339.1
17		102.9	102.9	442.0	339.1
18		102.9	102.9	442.0	339.1
19		102.9	102.9	442.0	339.1
20		102.9	102.9	442.0	339.1
21		102.9	102.9	442.0	339.1
22		102.9	102.9	442.0	339.1
23		102.9	102.9	442.0	339.1
24		102.9	102.9	442.0	339.1
25		102.9	102.9	442.0	339.1
26		102.9	102.9	442.0	339.1
27		102.9	102.9	442.0	339.1
28		102.9	102.9	442.0	339.1
29		102.9	102.9	442.0	339.1
30		102.9	102.9	442.0	339.1
31		102.9	102.9	442.0	339.1
32		102.9	102.9	442.0	339.1
33		102.9	102.9	442.0	339.1
34		102.9	102.9	442.0	339.1
35		102.9	102.9	442.0	339.1
36		102.9	102.9	442.0	339.1
37		102.9	102.9	442.0	339.1
38		102.9	102.9	442.0	339.1
39		102.9	102.9	442.0	339.1
40		102.9	102.9	442.0	339.1
41		102.9	102.9	442.0	339.1
42		102.9	102.9	442.0	339.1
43		102.9	102.9	442.0	339.1
44		102.9	102.9	442.0	339.1
45		102.9	102.9	442.0	339.1
46		102.9	102.9	442.0	339.1
47		102.9	102.9	442.0	339.1
48		102.9	102.9	442.0	339.1
49		102.9	102.9	442.0	339.1
50		102.9	102.9	442.0	339.1

Source: JICA Survey Team

(5) Results of financial evaluation

The results of the financial evaluation are the following ones:

Table 2.3.5 Results of Financial Evaluation

Evaluation Index		**5 year**	**10 year**	**25 year**	**50 year**
	FIRR	38.2%	26.1%	19.9%	12.7%
Discount Rate 10%	B/C	3.69	2.38	1.89	1.24
	FNPV(R$$10^6$)	851.5	1,001.4	1,101.1	516.4
Discount Rate 23%	B/C	1.77	1.14	0.85	0.52
	FNPV(R$$10^6$)	159.7	67.9	-112.7	-630.3
Discount Rate 12%	B/C	3.21	2.07	1.63	1.06
	FNPV(R$$10^6$)	639.2	710.2	707.1	110.0

Source: JICA Survey Team

The result of the evaluation for the index FIRR (Financial Internal Rate of Return), is indicated 26.1 % for safety level of the 10-year flood, and 12.7 % for safety level of the 50-year flood.

In the cost-benefit (B/C) ratio with the discount rate of 10 %/year, the index shows positive results. But, with the discount rate of 23 %/year, the index shows low profitability. However, the discount rate of 23 %/year is considered very high in a current economical scenery of Brazil.

In the relationship of Financial Net Present Value (FNPV), with the discount rate of 23 %/year, the result is shown negative. However, if taking in consideration the last tendencies of CDI, having varied among 10 %/year to 12 %/year, the possibility of the high rate to return is low. Considering these circumstances, it is considered viable the implementation of the measure presented in this report with the safety level of the 50-year flood. Besides, to be considered the valorizations of the lands with less disaster risk, the economical viability would be getting better abruptly.

2.3.2 Economic Evaluation

(1) Cash Flow for 5 year safety level

The cash flow of the Master Plan for 5 year safety level is as follow;

Table 2.3.6 Economic Cost and Benefit Flow (5 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	44.0	0.0	44.0	0.0	-44.0
2	44.0	0.0	44.0	0.0	-44.0
3	44.0	0.0	44.0	0.0	-44.0
4		6.6	6.6	78.7	72.1
5		6.6	6.6	78.7	72.1
6		6.6	6.6	78.7	72.1
7		6.6	6.6	78.7	72.1
8		6.6	6.6	78.7	72.1
9		6.6	6.6	78.7	72.1
10		6.6	6.6	78.7	72.1
11		6.6	6.6	78.7	72.1
12		6.6	6.6	78.7	72.1
13		6.6	6.6	78.7	72.1
14		6.6	6.6	78.7	72.1
15		6.6	6.6	78.7	72.1
16		6.6	6.6	78.7	72.1
17		6.6	6.6	78.7	72.1
18		6.6	6.6	78.7	72.1
19		6.6	6.6	78.7	72.1
20		6.6	6.6	78.7	72.1
21		6.6	6.6	78.7	72.1
22		6.6	6.6	78.7	72.1
23		6.6	6.6	78.7	72.1
24		6.6	6.6	78.7	72.1
25		6.6	6.6	78.7	72.1
26		6.6	6.6	78.7	72.1
27		6.6	6.6	78.7	72.1
28		6.6	6.6	78.7	72.1
29		6.6	6.6	78.7	72.1
30		6.6	6.6	78.7	72.1
31		6.6	6.6	78.7	72.1
32		6.6	6.6	78.7	72.1
33		6.6	6.6	78.7	72.1
34		6.6	6.6	78.7	72.1
35		6.6	6.6	78.7	72.1
36		6.6	6.6	78.7	72.1
37		6.6	6.6	78.7	72.1
38		6.6	6.6	78.7	72.1
39		6.6	6.6	78.7	72.1
40		6.6	6.6	78.7	72.1
41		6.6	6.6	78.7	72.1
42		6.6	6.6	78.7	72.1
43		6.6	6.6	78.7	72.1
44		6.6	6.6	78.7	72.1
45		6.6	6.6	78.7	72.1
46		6.6	6.6	78.7	72.1
47		6.6	6.6	78.7	72.1
48		6.6	6.6	78.7	72.1
49		6.6	6.6	78.7	72.1
50		6.6	6.6	78.7	72.1

Source: JICA Survey Team

(2) Cash Flow for 10 year safety level

The cash flow of the Master Plan for 10 year safety level is as follow;

Table 2.3.7 Economic Cost and Benefit Flow (10 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	100.7	0.0	100.7	0.0	-100.7
2	100.7	0.0	100.7	0.0	-100.7
3	100.7	0.0	100.7	0.0	-100.7
4		15.1	15.1	116.1	101.0
5		15.1	15.1	116.1	101.0
6		15.1	15.1	116.1	101.0
7		15.1	15.1	116.1	101.0
8		15.1	15.1	116.1	101.0
9		15.1	15.1	116.1	101.0
10		15.1	15.1	116.1	101.0
11		15.1	15.1	116.1	101.0
12		15.1	15.1	116.1	101.0
13		15.1	15.1	116.1	101.0
14		15.1	15.1	116.1	101.0
15		15.1	15.1	116.1	101.0
16		15.1	15.1	116.1	101.0
17		15.1	15.1	116.1	101.0
18		15.1	15.1	116.1	101.0
19		15.1	15.1	116.1	101.0
20		15.1	15.1	116.1	101.0
21		15.1	15.1	116.1	101.0
22		15.1	15.1	116.1	101.0
23		15.1	15.1	116.1	101.0
24		15.1	15.1	116.1	101.0
25		15.1	15.1	116.1	101.0
26		15.1	15.1	116.1	101.0
27		15.1	15.1	116.1	101.0
28		15.1	15.1	116.1	101.0
29		15.1	15.1	116.1	101.0
30		15.1	15.1	116.1	101.0
31		15.1	15.1	116.1	101.0
32		15.1	15.1	116.1	101.0
33		15.1	15.1	116.1	101.0
34		15.1	15.1	116.1	101.0
35		15.1	15.1	116.1	101.0
36		15.1	15.1	116.1	101.0
37		15.1	15.1	116.1	101.0
38		15.1	15.1	116.1	101.0
39		15.1	15.1	116.1	101.0
40		15.1	15.1	116.1	101.0
41		15.1	15.1	116.1	101.0
42		15.1	15.1	116.1	101.0
43		15.1	15.1	116.1	101.0
44		15.1	15.1	116.1	101.0
45		15.1	15.1	116.1	101.0
46		15.1	15.1	116.1	101.0
47		15.1	15.1	116.1	101.0
48		15.1	15.1	116.1	101.0
49		15.1	15.1	116.1	101.0
50		15.1	15.1	116.1	101.0

Source: JICA Survey Team

(3) Cash Flow for 25 year safety level

The cash flow of the Master Plan for 25 year safety level is as follow;

Table 2.3.8 Economic Cost and Benefit Flow (25 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	136.0	0.0	136.0	0.0	-136.0
2	136.0	0.0	136.0	0.0	-136.0
3	136.0	0.0	136.0	0.0	-136.0
4	136.0	0.0	136.0	0.0	-136.0
5		27.2	27.2	172.7	145.5
6		27.2	27.2	172.7	145.5
7		27.2	27.2	172.7	145.5
8		27.2	27.2	172.7	145.5
9		27.2	27.2	172.7	145.5
10		27.2	27.2	172.7	145.5
11		27.2	27.2	172.7	145.5
12		27.2	27.2	172.7	145.5
13		27.2	27.2	172.7	145.5
14		27.2	27.2	172.7	145.5
15		27.2	27.2	172.7	145.5
16		27.2	27.2	172.7	145.5
17		27.2	27.2	172.7	145.5
18		27.2	27.2	172.7	145.5
19		27.2	27.2	172.7	145.5
20		27.2	27.2	172.7	145.5
21		27.2	27.2	172.7	145.5
22		27.2	27.2	172.7	145.5
23		27.2	27.2	172.7	145.5
24		27.2	27.2	172.7	145.5
25		27.2	27.2	172.7	145.5
26		27.2	27.2	172.7	145.5
27		27.2	27.2	172.7	145.5
28		27.2	27.2	172.7	145.5
29		27.2	27.2	172.7	145.5
30		27.2	27.2	172.7	145.5
31		27.2	27.2	172.7	145.5
32		27.2	27.2	172.7	145.5
33		27.2	27.2	172.7	145.5
34		27.2	27.2	172.7	145.5
35		27.2	27.2	172.7	145.5
36		27.2	27.2	172.7	145.5
37		27.2	27.2	172.7	145.5
38		27.2	27.2	172.7	145.5
39		27.2	27.2	172.7	145.5
40		27.2	27.2	172.7	145.5
41		27.2	27.2	172.7	145.5
42		27.2	27.2	172.7	145.5
43		27.2	27.2	172.7	145.5
44		27.2	27.2	172.7	145.5
45		27.2	27.2	172.7	145.5
46		27.2	27.2	172.7	145.5
47		27.2	27.2	172.7	145.5
48		27.2	27.2	172.7	145.5
49		27.2	27.2	172.7	145.5
50		27.2	27.2	172.7	145.5

Source: JICA Survey Team

(4) Cash Flow for 50 year safety level

The cash flow of the Master Plan for 50 year safety level is as follow;

Table 2.3.9 Economic Cost and Benefit Flow (50 year Safety Level Plan)

Unit: (R$ million)

Year	Cost	O&M	Total Cost	Benefit	Balance
1	205.8	0.0	205.8	0.0	-205.8
2	205.8	0.0	205.8	0.0	-205.8
3	205.8	0.0	205.8	0.0	-205.8
4	205.8	0.0	205.8	0.0	-205.8
5	205.8	0.0	205.8	0.0	-205.8
6		51.5	51.5	221.0	169.5
7		51.5	51.5	221.0	169.5
8		51.5	51.5	221.0	169.5
9		51.5	51.5	221.0	169.5
10		51.5	51.5	221.0	169.5
11		51.5	51.5	221.0	169.5
12		51.5	51.5	221.0	169.5
13		51.5	51.5	221.0	169.5
14		51.5	51.5	221.0	169.5
15		51.5	51.5	221.0	169.5
16		51.5	51.5	221.0	169.5
17		51.5	51.5	221.0	169.5
18		51.5	51.5	221.0	169.5
19		51.5	51.5	221.0	169.5
20		51.5	51.5	221.0	169.5
21		51.5	51.5	221.0	169.5
22		51.5	51.5	221.0	169.5
23		51.5	51.5	221.0	169.5
24		51.5	51.5	221.0	169.5
25		51.5	51.5	221.0	169.5
26		51.5	51.5	221.0	169.5
27		51.5	51.5	221.0	169.5
28		51.5	51.5	221.0	169.5
29		51.5	51.5	221.0	169.5
30		51.5	51.5	221.0	169.5
31		51.5	51.5	221.0	169.5
32		51.5	51.5	221.0	169.5
33		51.5	51.5	221.0	169.5
34		51.5	51.5	221.0	169.5
35		51.5	51.5	221.0	169.5
36		51.5	51.5	221.0	169.5
37		51.5	51.5	221.0	169.5
38		51.5	51.5	221.0	169.5
39		51.5	51.5	221.0	169.5
40		51.5	51.5	221.0	169.5
41		51.5	51.5	221.0	169.5
42		51.5	51.5	221.0	169.5
43		51.5	51.5	221.0	169.5
44		51.5	51.5	221.0	169.5
45		51.5	51.5	221.0	169.5
46		51.5	51.5	221.0	169.5
47		51.5	51.5	221.0	169.5
48		51.5	51.5	221.0	169.5
49		51.5	51.5	221.0	169.5
50		51.5	51.5	221.0	169.5

Source: JICA Survey Team

The results of the economical evaluation are the following ones:

Table 2.3.10 Results of Economic Evaluation

Evaluation Index		**5 year**	**10 year**	**25 year**	**50 year**
	Economic IRR	38.2%	26.1%	19.9%	12.7%
Discount Rate 6%	B/C	5.05	3.26	2.64	1.75
	ENPV(R$$10^6$)	825.4	1,053.3	1,317.4	1,090.8
Discount Rate 10%	B/C	3.69	2.38	1.89	1.24
	ENPV(R$$10^6$)	425.8	500.1	550.0	257.9
Discount Rate 12%	B/C	3.21	2.07	1.63	1.06
	ENPV(R$$10^6$)	319.6	354.6	353.1	54.8

Source: JICA Study Team

The results of economic evaluation show the positive indicators in all of the aspects. These results indicate high economical viability of the implementations of the interventions presented in this report.

2.3.3 Total Evaluation

The Itajaí River basin shows a positive tendency of development, especially in the areas of mouth of the Itajaí River, with great attractiveness to new investments. Every year, the need to structure this area of strategic importance for the State is big, mainly in what refers to the prevention of disasters.

In the results of the evaluations high economical viability is shown, even with the implementations aiming at safety level of the 50-year flood.

CHAPTER 3 FEASIBILITY STUDY PROJECT EVALUATION

3.1 Methodology of Economic Evaluation

The economical evaluation in this Feasibility Study was carried out for the following projects;

Table 3.1.1 Project subject to Evaluation

Project	Outlook of Project
Water storage in paddy fields	Paddy fields ridge heightening (5,000ha)
Change of current dam operation method and heightening of the dam（Oeste）	Heightening of dam by 2 m
Change of current dam operation method and heightening of the dam（Sul）	Heightening of dam by 2 m
Utilization of the existing hydropower generation dam for flood control	2 dams
Installation of floodgate and improving Itajai Mirim River in Itajai City	2 floodgates
Strengthening the existing flood forecasting and warning system	1 Unit
Installation of early warning system for land slide and flush flood	1Unit

Source; JICA survey team

The evaluation period is of 50 years. Respective benefits are considered that the differences between the potential value of disaster that can be caused by the existent infrastructures and the potential value to be mitigated with the implantation of the project proposed as a mitigation measure. The benefit of structural measures for landslide disasters is separately estimated. The reaches of flood inundation were estimated through hydrological simulation by respective probable floods and it was transformed to damage values. The expected annual average flood mitigation benefit is estimated based on the probability of occurrence of probable flood and probable flood damages. Besides this, there is benefit of valorizations of the lands through improvement of safety. However, this benefit, in this evaluation, was not considered.

3.2 Cost and Benefit

3.2.1 Cost

The project cost proposed in this study is shown below. The details of project cost are indicated in the Main Report Part II Chapter 10.

Table3.2.1 Proposed Project Cost

(Unit: R$ 1,000)

Item		Direct Cost	Administration Expenses	Expropriation	Subtotal
I. Direct Cost of Measure					
(1) Basin Storage Measures	Water storage in paddy fields	18,000	3,600		21,600
	Heightening of dams（Oeste）	27,200	800	1,110	29,110
	Heightening of dams（Sul）	22,500	700		23,200
(2) River Improvement Measures	Floodgates in Itajaí Mirim River (Upper stream)	17,800	500	10	18,310
	Floodgates in Itajaí Mirim River (Lower stream)	14,000	400		14,400
(3) Structural Measures for Sediment Disaster Prevention		25,800	800	50	26,650
(4) Strengthening of the Existing Flood Forecasting and Warning System (FFWS)		4,000	120		4,120
(5) Installation of Early Warning System for Sediment Disaster and Flash Flood		4,000	120		4,120

Item	Direct Cost	Administration Expenses	Expropriation	Subtotal
II. Subtotal	133,300	7,040	1,170	141,510
III. Engineering Services	25,100	750		25,850
IV. II+III	158,400	7,790	1,170	167,360

Source; JICA survey team

3.2.2 Benefit

As the result of the implementations of the measure proposed in this Feasibility Study, it is foreseen to obtain the following benefits;

Table 3.2.2 Expected Impact of the Project

Item	Results of Measure
Water storage in paddy fields	Increase of rice production
Heightening of the dam（Oeste） Heightening of the dam（Sul）	Flood disaster mitigation in Taio city Flood disaster mitigation in Rio do Sul
Utilization of the existing hydropower generation dam for flood control	Flood disaster mitigation in Timbó city
Installation of floodgate and improving Itajai Mirim River in Itajai City	Flood disaster mitigation in Itajaí City Mitigation of Economic loss in Itajaí city
Strengthening the existing flood forecasting and warning system & Installation of Early Warning system for Landslide and Flush flood	Mitigation of scarified (Injured and death)

Source; JICA survey team

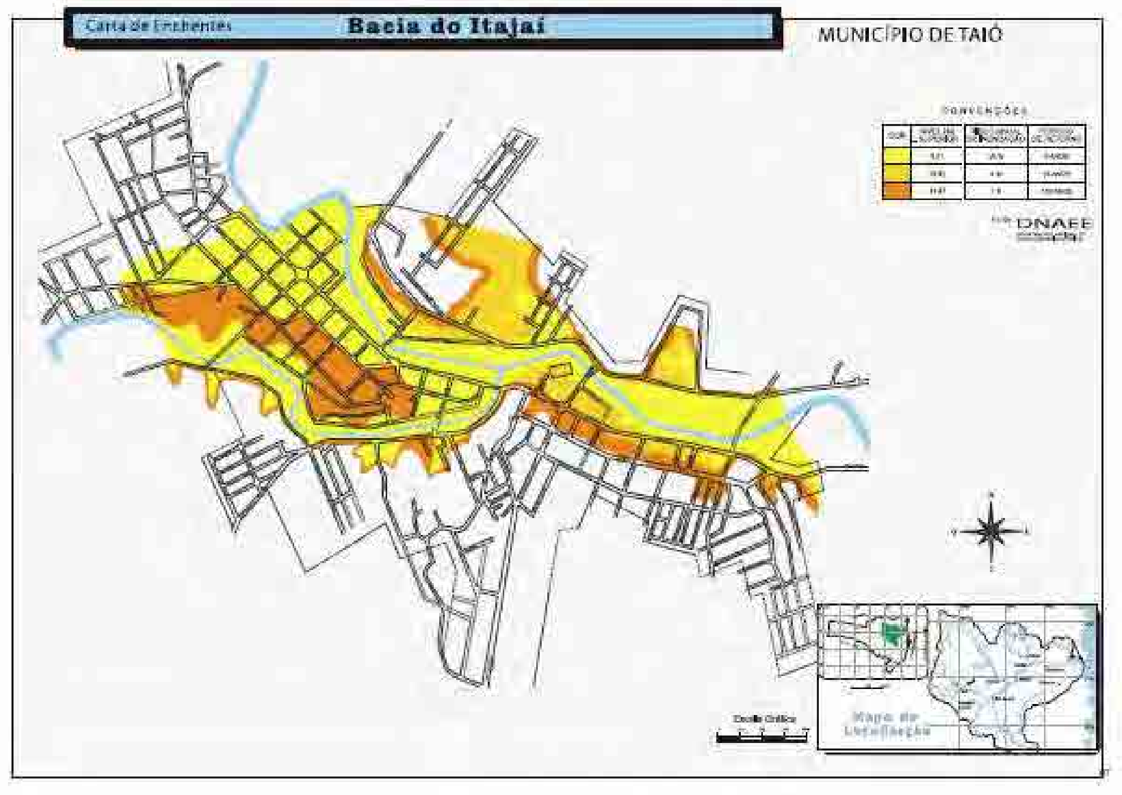
The benefits counted in the proposed project in this Feasibility Study were estimated in the following forms;

(1) Increase of rice production

The benefit of the Project “Water storage in paddy fields” will be expected by the increase of productivities and improvement of quality of the products through the improvement of the paddy fields infrastructures. The expected value of the benefit was estimated R$ 2.0 million/year (Project area x Increase of productivity x Rice price = 5,000 ha x 0.8 t/ha x R$ 500/t). The increase of productivity was estimated to be 10% of the average rice yield of 7.9 t/ha in the period of 2000-2008.

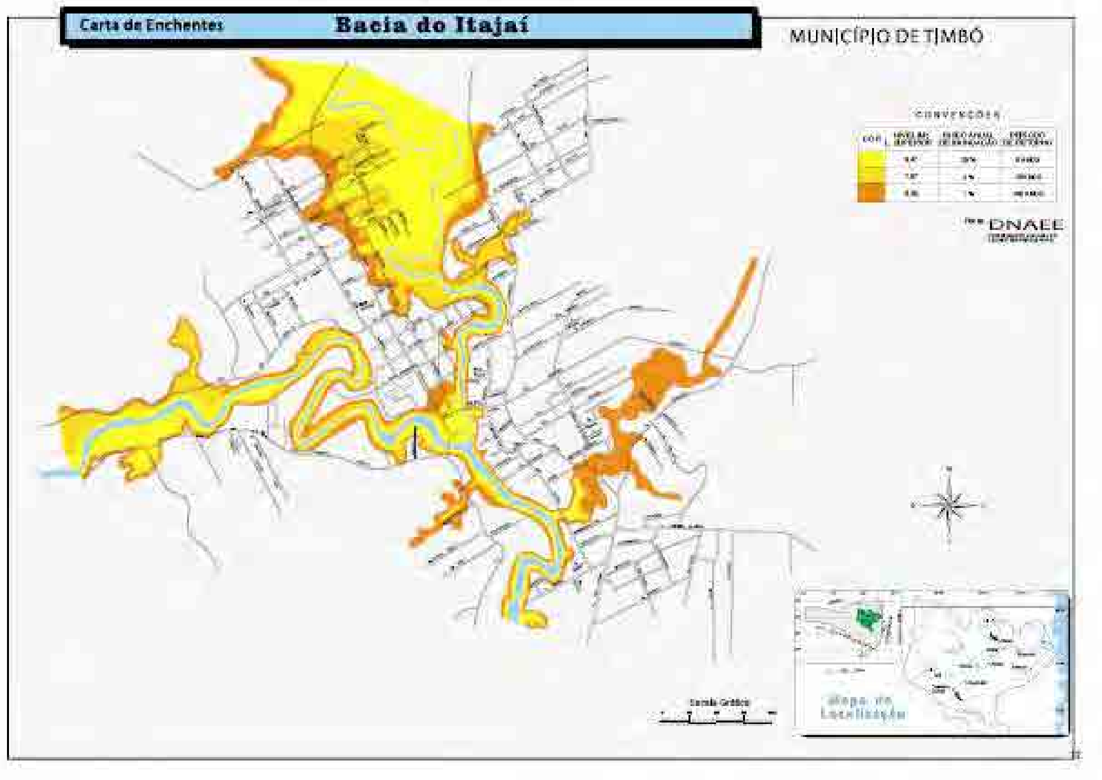
(2) Benefit of the Project “Change of current dam operation method and heightening of the dam”

The effect of flood damage mitigations due to heightening of two dams was roughly estimated based on the available probable flood inundation maps at major cities which wre prepared by FURB. Flood inundation maps at main cities are shown below.



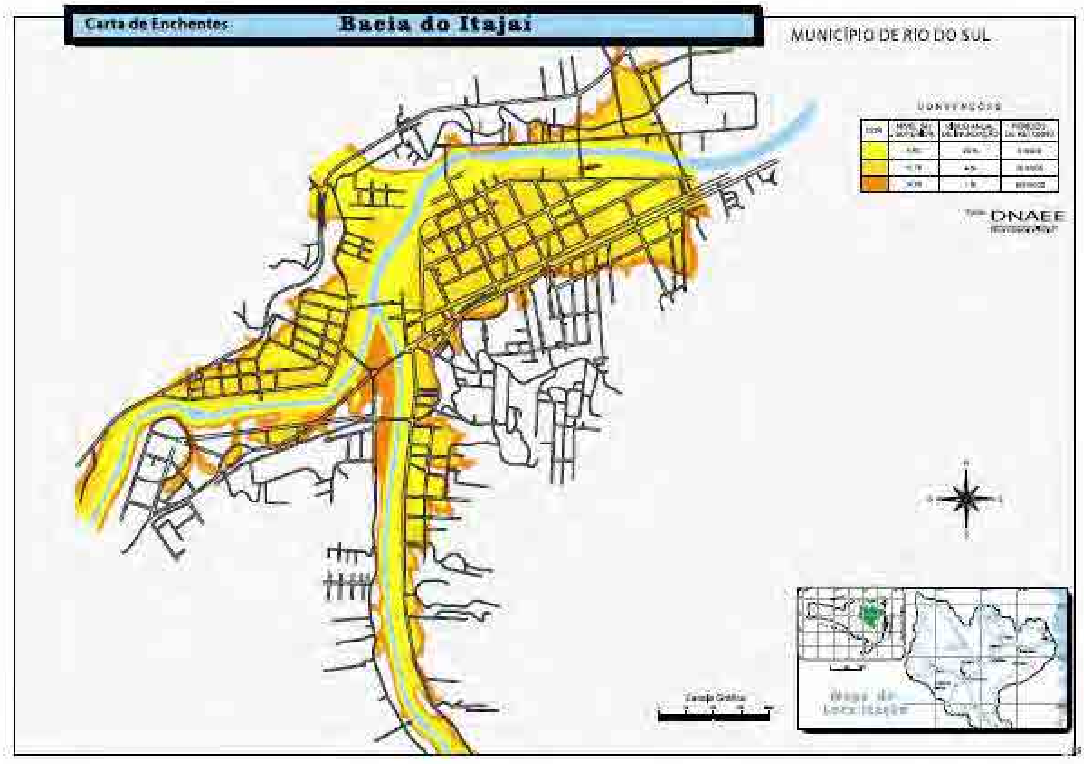
Source: FURB

Fig 3.2.1 Flood Inundation Map at Taio city



Source: FURB

Figure 3.2.2 Flood Inundation Map at Timbo City



Source: FURB

Figure 3.2.3 Flood Inundation Map at Rio do Sul City

The results of estimates were the following ones;

Table3.2.3 Estimation of Flood Damages Mitigation Benefit by Dam Heightening

City	Number of housing	Number of affected houses（Estimated）			
		5year	10 year	25 year	50 year
Taio （present）	2,541	250	300	400	500
Taio （with project）		-	-	400	500
Timbo (present)	8,297	150	200	250	300
Timbo （with project）		-	-	250	300
Rio do Sul	15,504	100	500	1,000	1,500
Rio do Sul (with project)		50	480	1,000	1,500
Total		500	1,000	1,650	2,300
With project		50	730	1,650	2,300
Effect of project		450	270	0	0
Flood damage（R$1,000）		9,000	10,400	0	0
Annual average damage（R$1,000）		3,600	970	0	0
Expected annual average flood mitigation benefit（R$1,000）		4,570			

Source; JICA Survey team

As seen above, the expected annual average flood mitigation benefit was estimated, presupposing of R$ 20,000 of asset for each housing affected by flood disaster. The number of affected houses by the flood was calculated for each safety level, using the existing reports/data in this theme. The expected annual average flood mitigation benefit was estimated R$ 4.6 million.

(3) Benefit by building damage mitigation by floodgates in the Mirim River

With the detailed digital topographical maps of 1/2,000, Flood Inundation area was evaluated as the following maps.

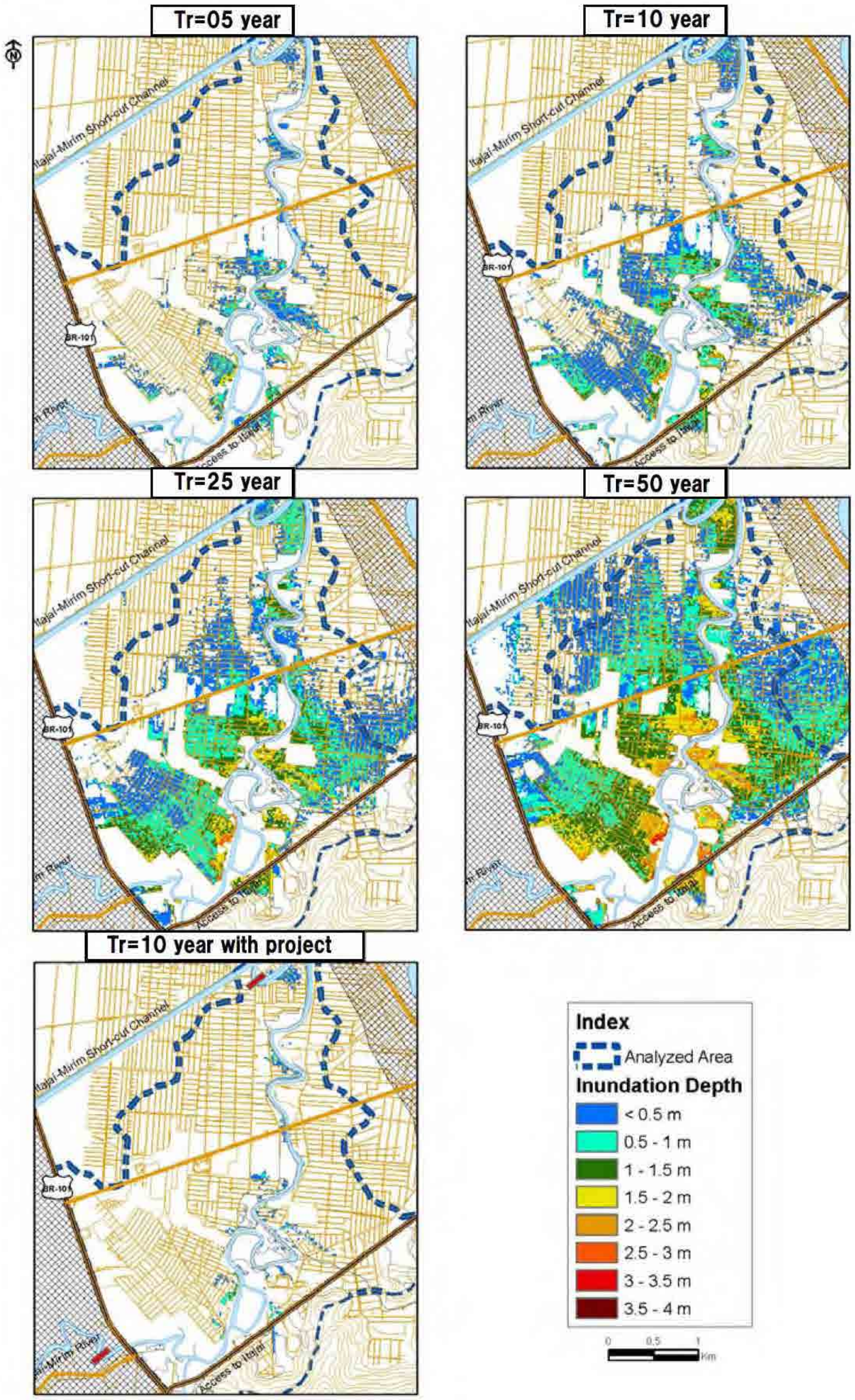


Figure 3.2.4 Present Situation of Flood Inundation in Mirim River at Itajai City

The expected annual average benefit was estimated assuming that the asset of house to be affected by flood was R$ 100,000 and damage rates due to flood were 20% with an inundation depth of more than 0.5m and 5% for less than 0.5m. Considering that an installation of two floodgates is expected to protect the 10-year flood, the expected annual average benefit of building damage mitigation was estimated R$ 4.6 million as follows:

Table 3.2.4 Estimation of Building Damage Mitigation Benefit by Floodgates in the Mirim

		5 year	10 year	25 year	50 year
Number of affected houses	Less than 0.5m	512	1,552	1,632	1,596
	More than 0.5m	232	940	2,637	3,911
	Sub-total	744	2,492	4,269	5,506
Flood damage (R$ 1,000)	Less than 0.5m	2,562	7,759	8,161	7,978
	More than 0.5m	4,633	18,795	52,732	78,214
	Sub-total	7,196	26,555	60,894	86,193
Annual average damage (R$ 1,000)		2,878	1,688	2,623	1,471
Expected annual average flood mitigation benefit (R$ 1,000)			4,566	7,189	8,660

Source; JICA Survey team

(4) Benefit by mitigation of economic loss by floodgates in the Mirim River

In addition to the above, the economic loss in the services sector due to flood disaster in the Itajai Mirim River was estimated based on the decrease of ICMS at Itajai City assuming that 20% of the total number of existing companies is located in a beneficiary area and to be protected by floodgate installation. The total amount of ICMS in the State occupies around 5% of the State's GDP. The services sector contributes around 50% of the State's GDP. Thus the economic loss in the services sector is estimated by 10 times of the decrease of ICMS. Considering that an installation of two floodgates is expected to protect the 10-year flood, the expected annual average benefit of economic loss mitigation was estimated R$ 42.0 million as follows:

Table 3.2.5 Benefit Estimation of Economic Loss Mitigation Benefit by Floodgates in the Mirim River

		5 year	10 year	25 year	50 year
ICMS (R$ million)	Decrease of ICMS	7.9	12.9	24.5	39.9
	Economic Loss	79.0	129.0	245.0	399.0
Annual average damage (R$ million)		31.6	10.4	11.2	6.4
Expected annual average benefit (R$ million)			42.0	53.2	59.7

Source; JICA survey team

(5) Benefit by Structure measure of landslide

The benefit originated by structure measure of landslide was estimated as follows;

Table3.2.6 Benefit of Structure Measure of Landslide

No. of priority order	Site	Potential annual loss (R$ x 10^3/year)	Total cost (direct and indirect) (R$ x 10^3)	Benefit: decrease in potential annual loss (R$ x 10^3/year)
1	Road SC 302 Taio-Passo Manso-5	1,255	551	1,062
2	Road SC470 Gaspar River Bank	1,095	2,810	581
3	Blumenau -Av Pres Castelo Branco	1,021	3,883	654
4	Road SC418 Blumenau - Pomerode	989	2,522	841
5	Road SC474 Blumenau-Massaranduba 2	907	5,077	641

No. of priority order	Site	Potential annual loss (R$ x 10^3/year)	Total cost (direct and indirect) (R$ x 10^3)	Benefit: decrease in potential annual loss (R$ x 10^3/year)
6	Road Gaspar - Luiz Alves, Gaspar 9	774	4,664	653
7	Road Gaspar - Luiz Alves, Luiz Alves 6	700	1,974	591
8	Road SC470 Gaspar Bypass	689	3,772	402
9	Road SC477 Benedito Novo - Doutor Pedrinho 1	680	1,399	575
10	Road SC418 Pomerode- Jaragua do Sul 1	651	1,187	553
11	Road Gaspar - Luiz Alves, Luiz Alves 4	629	5,078	532
12	Road SC474 Blumenau - Massaranduba 1	601	702	425
13	Road SC 302 Taio - Passo Manso 4	526	1,599	446
Total of the 13 risk sites		10,516	35,219	7,956

Source: JICA Survey Team

(6) Benefit of Alarm/alert system

The table below shows the victims by the flood disaster in November 2008. Although reduction of victims is expected by the strengthening of existing FFWS and installation of early warning system of landslide and flashflood, such benefit was not considered in this study due to difficulty of estimation in terms of monetary value.

Table 3.2.7 Victims by the Flood Disaster in November 2008

	Injured	Death
2008/11 Flood	4,637	89
With project	-	-

Source : AVADAMs enviados pelos munincipios á Defesa Civil de Santa Catarina, nos dias 24 e 25 de novembro de 2008.

3.3 Project Evaluation

3.3.1 Cost and Benefit Flow

The cost and benefit flow of the Project is as follow;

Table 3.3.1 Economic Cost and Benefit Flow of the Project

Unit: (R$ million)

Year	Cost	Maintenance Cost	Total Cost	Benefit	Balance
1	2.5	0.0	2.5	0.0	-2.5
2	14.5	0.0	14.5	0.0	-14.5
3	16.0	0.0	16.0	0.0	-16.0
4	34.8	0.0	34.8	0.0	-34.8
5	16.0	0.0	16.0	0.0	-16.0
6		4.2	4.2	30.5	26.4
7		4.2	4.2	30.5	26.4
8		4.2	4.2	30.5	26.4
9		4.2	4.2	30.5	26.4
10		4.2	4.2	30.5	26.4
11		4.2	4.2	30.5	26.4
12		4.2	4.2	30.5	26.4
13		4.2	4.2	30.5	26.4
14		4.2	4.2	30.5	26.4
15		4.2	4.2	30.5	26.4
16		4.2	4.2	30.5	26.4
17		4.2	4.2	30.5	26.4
18		4.2	4.2	30.5	26.4
19		4.2	4.2	30.5	26.4
20		4.2	4.2	30.5	26.4
21		4.2	4.2	30.5	26.4
22		4.2	4.2	30.5	26.4
23		4.2	4.2	30.5	26.4
24		4.2	4.2	30.5	26.4
25		4.2	4.2	30.5	26.4
26		4.2	4.2	30.5	26.4
27		4.2	4.2	30.5	26.4
28		4.2	4.2	30.5	26.4
29		4.2	4.2	30.5	26.4
30		4.2	4.2	30.5	26.4
31		4.2	4.2	30.5	26.4
32		4.2	4.2	30.5	26.4
33		4.2	4.2	30.5	26.4
34		4.2	4.2	30.5	26.4
35		4.2	4.2	30.5	26.4
36		4.2	4.2	30.5	26.4
37		4.2	4.2	30.5	26.4
38		4.2	4.2	30.5	26.4
39		4.2	4.2	30.5	26.4
40		4.2	4.2	30.5	26.4
41		4.2	4.2	30.5	26.4
42		4.2	4.2	30.5	26.4
43		4.2	4.2	30.5	26.4
44		4.2	4.2	30.5	26.4
45		4.2	4.2	30.5	26.4
46		4.2	4.2	30.5	26.4
47		4.2	4.2	30.5	26.4
48		4.2	4.2	30.5	26.4
49		4.2	4.2	30.5	26.4
50		4.2	4.2	30.5	26.4

Source: JICA Survey Team

3.3.2 Results of Economic Evaluation

The results of the economic evaluation are as follows;

Table 3.3.2 Results of Economic Evaluation

Evaluation Index		Indicator
EIRR		22.9 %
Discount rate（6%）	B/C	3.03
	ENPV(R$$10^6$)	236.4
Discount Rate（10%）	B/C	2.19
	ENPV(R$$10^6$)	101.6
Discount Rate（12%）	B/C	1.89
	ENPV(R$$10^6$)	67.6

Source; JICA survey team

The economic evaluation was conducted in terms of the EIRR (Economic Internal Rate of Return) on the basis of the economic cost and benefit. The EIRR is indicated 22.9 %. The project is considered to be highly economically feasible.

3.4 Total Evaluation

This Project, motivated by the extraordinary flood in November, 2008, with the consensus of taking the preventive measures for floods, formulated the Master plan and selected the priority projects for the Feasibility Study.

The economical importance in the basin is being more and more significant inside of the economical scenery of the State, with the tendencies of new investments, especially in the Itajai Port area. Further large quantity of investments is expected more and more inside of the basin, it needs to assure the protection of the installed goods, through the disasters mitigation measure. It is notable that the economical activity in the lower Itajaí River basin had 5 times of economical growth in the 8 years in the period from 1999 to 2008, being significant that the needs to protect the basin from disaster are more and more important.