JAPAN INTERNATIONAL COOPERATION AGENCY

STATE SECRETARIAT OF PLANNING, SCIENCE AND TECHNOLOGY
THE STATE OF SERGIPE, THE FEDERATIVE REPUBLIC OF BRAZIL

THE STUDY
ON
WATER RESOURCES DEVELOPMENT
IN THE STATE OF SERGIPE
IN
THE FEDERATIVE REPUBLIC OF BRAZIL

FINAL REPORT
SUPPORTING
(VOLUME II)
FEASIBILITY STUDY

[E] DAM PLAN

MARCH 2000

YACHIYO ENGINEERING CO., LTD. (YEC)

THE STUDY ON WATER RESOURCES DEVELOPMENT
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SUPPORTING REPORT (E)
DAM PLAN

Table of Contents

List of Tables

List of Figures

CHAPTER 1 AIMS OF THE PROJECT

1.1 Necessity of the Project

(1) Necessity of New Water Resources Development in Vaza Barris River

Itabaiana and Lagarto Water Supply areas (Agreste and Piauitinga Integrated Pipeline Systems) are located in Agreste/Semi-arid areas and are the second and third largest populated areas in Sergipe State. These areas are of poor ground/surface water resources potential and have been suffering from water shortage. In order to cope with present water shortage and increasing water demand, groundwater is not enough in both quantity and quality and it costs too much if surface water is conveyed from other river basins affluent in water resources, Sao Francisco River for example. Vaza Barris River has large water resources potential and is located between the large water-consumed cities of Itabaiana and Lagarto. Therefore, it has been expected to develop the river water not only by benefiting municipalities but also by the state of Sergipe.

(2) Refreshing High chlorine River Water by a Proposed Dam System

Vaza Barris River, who is the second largest river in the State with the total basin area of 16,229 km^2 including a part of the Bahia State, flows down between the second and third largest populated cities of Itabaiana and Lagarto. River water of the main stream has large potential of water resources but has not been able to be utilized as potable and irrigation water due to high chlorine concentration of flow from upstream. In fact, a dam plan had been progressed on the main stream of the river in the Sergipe State in 1980's and was abandoned due to unavailable water quality of the river. After elaborate investigation of river water quality, however, it was found that river flow has high chlorine concentration only during low flow condition from upstream and has less chlorine concentration during flood time and in the downstream. Considering such condition of the water quality in Vaza Barris River, the following reservoir operation plan is being established:

1) Low flow from the upstream with high chlorine concentration is bypassed around the dam to the downstream.
2) River water with less chlorine concentration during flood and in the downstream is stored in the dam reservoir.

Then,

3) Dam reservoir water is kept clean (low chlorine concentration) and could be utilized as portable and irrigation water.

Introducing such reservoir operation plan with a new system for river water quality, river water that could not be utilized before becomes clean and comes to be utilized as potable and irrigation water.

(3) Insufficient Water Supply

In the State of Sergipe in 2020, necessary supply water amount is estimated to be totally 830,000 m^3/day including 547,000 m^3/day of supply water shortage. Of this water shortage, Aracaju Capital Area is short of 175,000 m^3/day (equivalent to 32% of total supply water shortage in the State) of water, and Itabaiana and Lagarto Water Supply areas (Agreste and Piauitinga Integrated Pipeline System) are short of 129,000 m^3/day (equivalent to 24%) of water.

The population of these areas is 259,000 inhabitants in 1996 and is estimated to be 540,000 in 2020, which represents almost two thirds of population in Aracaju Capital Area at the same year (875,000 inhabitants). The lack of adequate water supply is a serious obstacle to the development of the so mentioned regions and creates a migratory pressure towards the State capital, worsening even more problems in Aracaju.

Therefore, it's mandatory to try to stabilize the water supply for high-concentrated population, indispensable to boost its social-economical development and to improve the quality of life. The positive consequences will spread all over the State, contributing to reduce the regional differences and alleviating the population and social-economic pressures towards the area of Aracaju.

(4) Irrigation Development

Fertile land suitable for irrigation extends around the right side of the planned dam site. This area is located between the three largest cities in the State of Sergipe, such as Aracaju, Itabaiana and Lagarto. It means the area has an advantage being near to large consumer cities. This area, which is presently utilized as orchards and pastures, could be irrigated and the agricultural production is to be supplied to the city areas. This irrigation project promotes improvement of agricultural productivity and activation of regional economy.

The planned dam is implemented as a multi-purpose dam for the development of domestic/industrial water and irrigation water. It results in decrease of the both project costs.

1.2 Objectives of the Project

"The Project of Water Resources Development and Supply in Vaza Barris River –Sergipe (PROVABASE)" is proposed for securing stable life of the state people through sustainable water resources development. The objectives of the project are set as follows:

- to improve river water quality and to develop potable water resources.
- to supply clean and enough water for the people through public water supply.
- to supply irrigation water to agriculturally potential land for the achievement of high productivity.
- to develop maintenance water of the river for riparian environment.

1.3 Project Components and Location

The project components of the facilities are summarized in Table-1.1. The target facilities of the project components in this feasibility study are:

- Facilities of Vaza Barris Multipurpose Dam
- Water conveyance pipelines of domestic/industrial water supply facilities

The project location is shown in Figure-1.1.

Table-1.1 Project Component and Facilities

Project Components	Facilities	
(1) Vaza Barris Multipurpose Dam		
Dam facilities	- Main Dam	- Spillway
	- Check Dam (Bypass Intake	- Low Flow Bypass
(2) Domestic/Industrial Water Supply Facilities: <Itabaiana Water Supply Area>		
Water conveyance pipeline	- Intake and raw water pump station	- Pipeline
Treatment and distribution facilities	*- Water treatment station*	*- Distribution pipeline network*
(3) Domestic/Industrial Water Supply Facilities: <Lagarto Water Supply Area>		
Water conveyance pipeline	- Intake and raw water pump station	- Pipeline
Treatment and distribution facilities	*- Water treatment station*	*- Distribution pipeline network*
(4) Irrigation Water Supply Facilities		
Water Conveyance Pipeline	*- Intake and raw water pump station*	*- Pipeline*
Irrigation Facilities	*- Farmland development*	*- Irrigation channel*

Note: Italic parts show the facilities not including in this feasibility study.

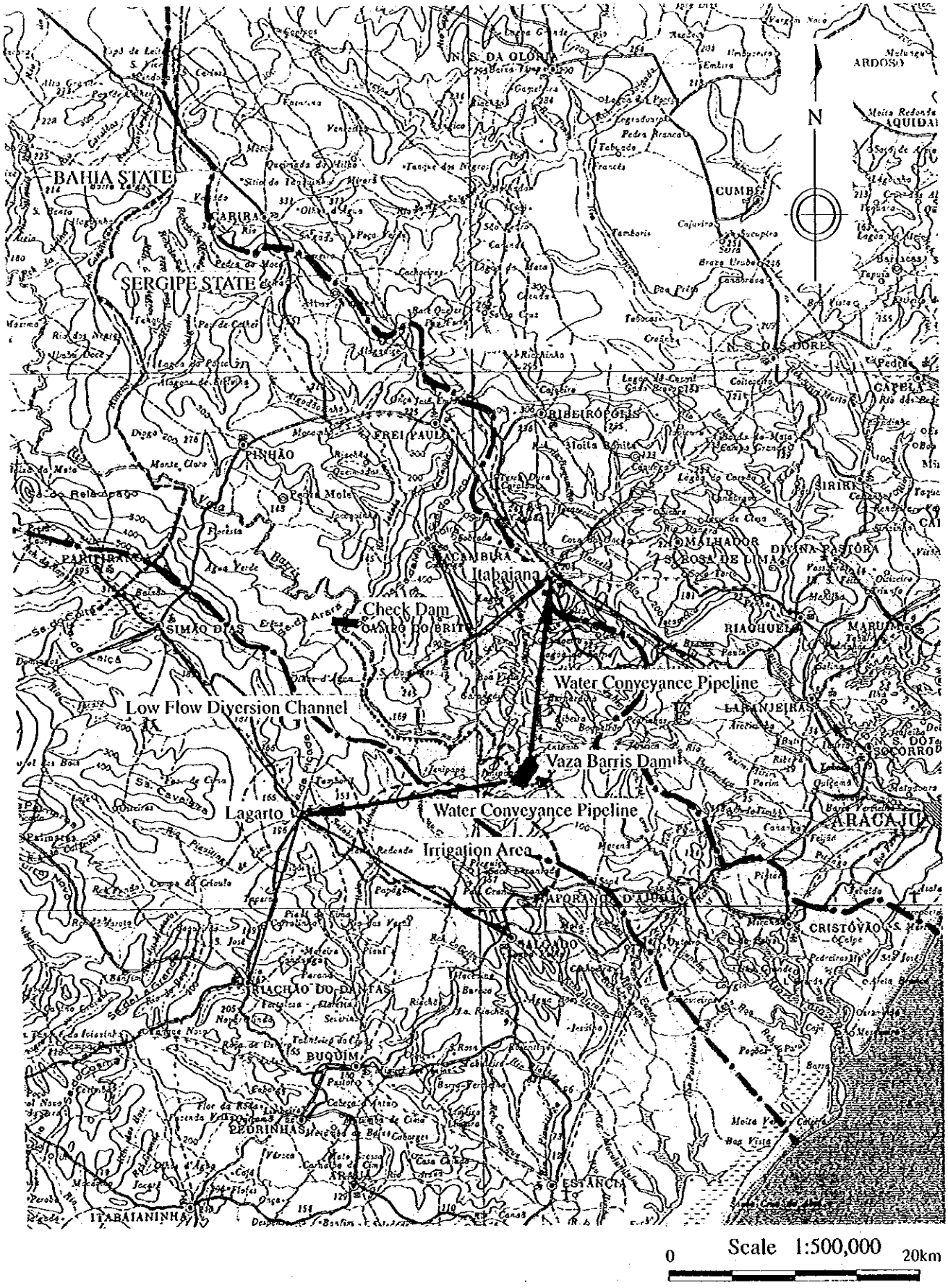

Figure-1.1 Location of the project

CHAPTER 2 CRITERIA FOR DAM PLAN

2.1 Compensation Discharge

The "compensation discharge" could be defined as the discharge necessary to maintain the normal function of a river, and consists of maintenance discharge and water-use discharge. Maintenance discharge has been stipulated to maintain river function even at the times of low flow, with overall consideration to the following: 1) fisheries, 2) scenery, 3) sea water intrusion, 4) preservation of fauna and flora, 5) preservation of cleanliness of river flow, 6) prevention of river-mouth clogging, 7) protection of river works, 8) groundwater level maintenance, 9) boat transportation. Water-use discharge is the flow necessary for the exclusive use of the river water at all points downstream.

After the following consideration, compensation discharge was set as the 100% of the 10-year return period 7-day flow (Q7, 10).

Maintenance Discharge: In this Study, the 100% of the 10-year return period 7-day flow (Q7, 10) is applied as maintenance discharge for dam planning. The (Q7, 10) refers to the mean annual minimum 7-day flow with 10-year return period, and this is secure as compensation discharge to the downstream when developing new water resources of river flow. As this value is not an absolute one, the above items to be considered should be studied in detailed environmental study.

Water Use Discharge: Based on the survey of current water use in Vaza Barris River in the Sergipe State, exclusive water use in the main stream could not be found. Consequently, no water use discharge is taken into account for compensation discharge. Refer to the following investigation of current water use.

< Current Water Use in Vaza Barris River in Sergipe State >

Current water use in Vaza Barris River concerning with water intake on domestic/industrial/irrigation water is summarized in Table-2.1 and their locations are pointed in Figure-2.1. All the intakes in the upstream and downstream are located in the tributaries of Vaza Barris River, not in the main stream.

Table-2.1 Current Water Use in Vaza Barris River

Station No.	River	Location	Objective	Operation Organization	Water Use Amount (m^3/day)
	Downstream				
02	Rio Chinduba	Antiga Intake	Industry	-	-
06	Riacho Taboca	Pov. Sape	Municipal	DESO	1,356
08	Riacho Pedras	Faz. Riacho Doce	Irrigation	COHIDRO	-
11	Rio Comprido	Intake SAAE	Municipal	DESO	2,400
12	Riacho Pindoba	Fonte Indaiá	Industry	-	-
	Upstream				
10	Riacho Taboca	Pov. Genipapo	Municipal	DESO	1,199
18-1	Rio Trairas	Riacho Ribeira	Municipal	DESO	346
18-3	Rio Trairas	Rio Trairas	Municipal/Irrigation	DESO/COHIDRO	12,216

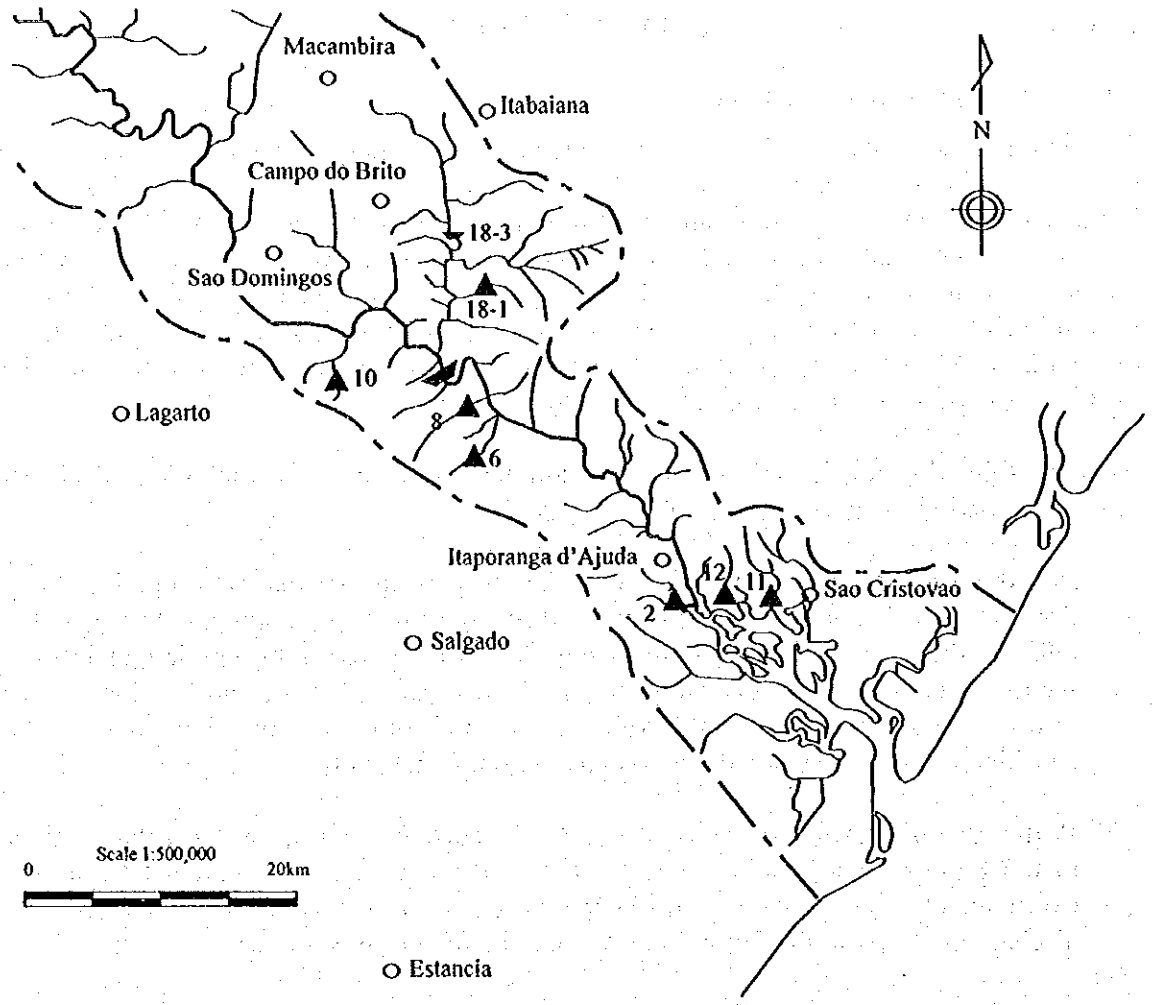

Figure-2.1 Location of Current Water Use in Vaza Barris River

2.2 Reservoir Reliability (Security Level of Water Supply)

Low flow security in the plan has been set to ensure the intake of newly developed discharge even in the worst drought in ten years for domestic/industrial and irrigation water supply.

2.3 Design Discharges

(1) Design Flood Discharge of Dam

A spillway is the safety valve for a dam. It must have the capacity to discharge major floods without damage to the dam or any appurtenant structures, at the same time keeping the reservoir level below some predetermined maximum level.

The selection of design flood discharge is related to the degree of protection that ought to be provided to the dam that depends on the type of dam, its location, and consequences of failure of the dam. A high dam storing a large volume of water located upstream of an inhabited area should have a much higher degree of protection than a low dam storing a small quantity of water whose downstream reach is uninhabited. The probable maximum flood is commonly used for the former while a smaller flood based on frequency analysis is suitable for the latter.

According to the "Criteria of Civil Design of Hydro-electric Plant, 1994 May, Preliminary Edition, CEMIG (Energy Company of Minas Gerais)", design flood discharge of a dam is set as follows:

1) For dams whose collapse involves loss risk of human lives (if permanent habitation exists downstream), the design flood discharge should be the Probable Maximum Flood.
2) For dams with the height less than 30 m or with the volume less than 50,000,000 m^3, and when there is not loss risk of human lives (if permanent habitation exists downstream), the design flood discharge should be defined as minimum return period of 1,000 years by probability analysis.

The proposed dam is categorized in 1) above and the design flood discharge should be the probable maximum discharge.

(2) Other Design Discharges

As the scale of the following design discharges are not defined in the criteria mentioned above, the Japanese standard for dam design are referred.

- Design discharge for energy dissipater of a dam
- Design discharge of diversion channel during construction
- Design discharge for spillway of a check dam

2.4 Design Criteria

Dam should be carefully designed to minimize the construction cost, holding necessary functions of each facility. Dam and related structures are designed according to reasonable balance between construction cost and safety level. A large safety level of structures requires a large scale of structures and complicated construction procedures. Consequently, the construction costs increase. Design Criteria are standards to decide a balance between construction cost and safety level.

Dam Design Criteria used in Brazil are as follows:

1) For almost existing dams, dam design standards of USBR (US, Dep. of the Interior, Bureau of Reclamation) and USCE (US Army, Corps of Engineer) were employed.
2) Sao Francisco Electricity Corporation (CHESF) recently uses the standard "Civil Works Criteria for Hydropower Generation" which was complied by Mina Gerais Power Company (CEMIG) on the basis of standards of USBR and USCE. This standard was used in designing Xingo Dam.

Considering the above situation, CEMIG standard is employed basically in design of Vaza Barris Dam and other related structures. If necessary, the Japanese Standards for dam design of "Manual of River Works, Ministry of Public Works, Japan" are referred.

CHAPTER 3 PLANNING CONDITION OF DAM

3.1 Required Development Water Amount

Vaza Barris Dam is planned for development of domestic/industrial water and irrigation water. Domestic and industrial water is to be supplied for the area covered by Agreste and Piauitinga Integrated Pipeline Systems, as shown in Table-3.1. Irrigation water is to be supplied for Vaza Barris Irrigation Project, of which monthly water requirement is shown in Table-3.2.

Table-3.1 Domestic and Industrial Water Supply in Agreste and Piauitinga Areas

Area Covered	Agreste Integrated Pipeline System		Piauitinga Integrated Pipeline System		Total Supply	
	(m^3/day)	(m^3/s)	(m^3/day)	(m^3/s)	(m^3/day)	(m^3/s)
Water amount necessary to be supplied in 2020	74,286	0.860	79,664	0.922	153,950	1.782
Present Capacity	12,810	0.148	12,130	0.141	24,940	0.289
Expansion project to be proposed to PROAGUA	22,200	0.257	30,200	0.349	52,400	0.606
Required development water amount in this project	39,276	0.455	37,334	0.432	76,610	0.887

Table-3.2 Monthly Water Requirement for Vaza Barris Irrigation Project

Item	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Average
Source Water Requirement (m^3/s)	2.912	2.469	2.022	0.717	0.273	0.000	0.033	0.542	1.344	2.484	2.463	2.820	1.507

Note: Efficiency of Water Conveyance: 0.9, Efficiency of Water Distribution: 0.9
Irrigation Area: 4,553 ha, Maximum Water Requirement: 2.912 m^3/s on January

Required water amount developed by Vaza Barris Dam is set as follows:

- Municipal and industrial water supply: 0.887 m^3/s
- Irrigation water supply: 2.912 m^3/s in maximum
  1.507 m^3/s in average
- Total required development amount: 3.799 m^3/s in maximum
  2.394 m^3/s in average

3.2 Reservoir Sedimentation

There are two dams existing in the Vaza Barris River basin, namely Cocorobo Dam (C.A.=3,600 km^2) and Cajaiba Dam (C.A.=195 km^2). As the hydrographic basin of the Vaza Barris dam is 15,560 km^2, sediment catchment area becomes 11,765km^2, subtracting the catchment areas of the existing dams.

Planned sediment capacities of the dams with the catchment area of over 100 km^2 in Sergipe State are shown in Table-3.3. The specific sediment capacities (m^3/km^2/year) of the above dams with the catchment area between 118-1,350 km^2 are ranged from 14-53 m^3/km^2/year. Taking into account of the catchment area size and the deference of basin elevation, the specific sediment capacity of the Vaza Barris Dam is set at 10 m^3/km^2/year. Then, securing 100-year sediment for the dam reservoir, the sediment capacity for the Vaza Barris Dam is necessary to be 12,000,000 m^3.

Table-3.3 Sediment Capacities of Dams in Sergipe State

Dam Name	River Name	Catch-ment Area (km^2)	Deference of Basin Elevation (m)	Reservoir Area (ha)	Reservoir Volume Vt (m^2)	Sediment Capacity Vs (m^3)	Specific Sediment Capacity ($m^3/km^2/year$)	Vs/Vt (%)
Vaza Barris	Vaza Barris	15,560 (11,765)	600	948	93,000,000	12,000,000	10	12.9%
Piaiu	Piaui	1,350	400	367	15,000,000	2,000,000	15	13.3%
Jabiberi	Jabiberi/Real	118	200	61	4,300,000	540,000	46	12.6%
Jacarecica I	Jac./Sergipe	221	200	115	4,700,000	300,000	14	6.4%
Cajaiba	Trairas/V.B.	195	200	250	16,500,000	1,032,000	53	6.3%

Note: ( ) shows sediment catchment area

3.3 Determination of Design Discharges

3.3.1 Design Flood Discharge of Dam

According to the planning criteria, the design flood discharge should be the probable maximum discharge, which is assumed to be the discharge with 10,000-year return period in this report. Based on annual maximum daily discharge for the 24-year data series (1971-1995) at Fazenda Belem (C.A.=15,740km^2), the discharge with 10,000-year return period was calculated to be 3,588 m^3/s. Taking into account of the ration of the catchment areas at Fazenda Belem and that at the dam site, the design flood discharge was set at 3,600 m^3/s.

- Design Flood Discharge: 3,600 m^3/s
- Specific Discharge: 0.23 m^3/s/km^2 (Dam C.A.= 15,560 km^2)

Figure-3.1 illustrates the range of specific discharge of the maximum flow for rivers in the world. As the design flood discharge of Vaza Barris Dam is pointed in the figure, it corresponds to the maximum discharge of the upper range in a plain area or the lower range in a mountain area. It seems to be reasonable studying the topographical condition of this basin.

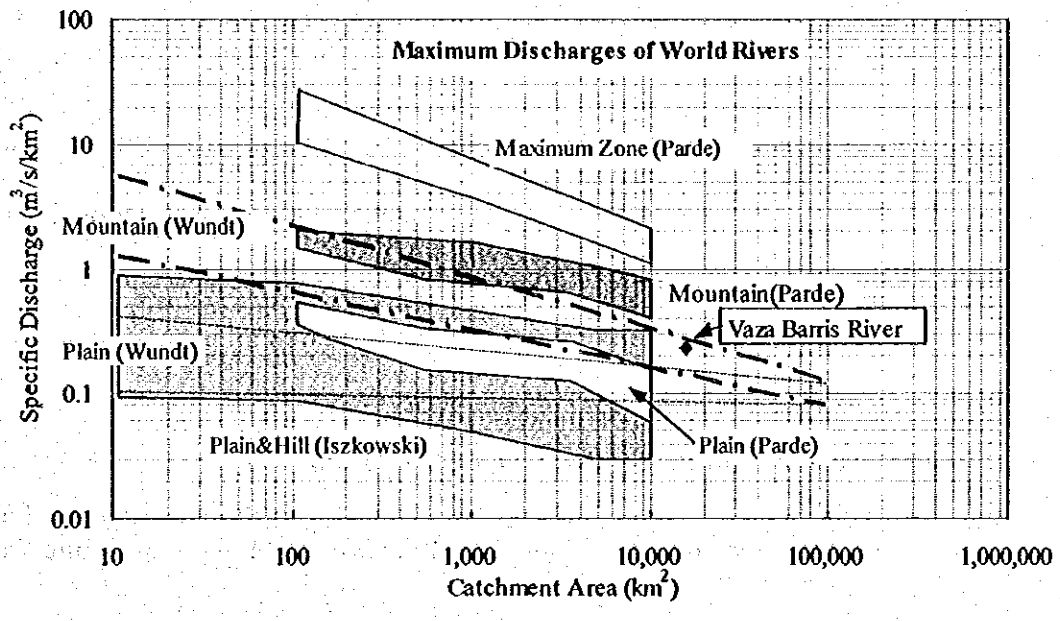

Figure-3.1 Specific Maximum Discharges of Rivers in the World

Table-3.4 Design Flood Discharge of Existing Dams in Sergipe State

Dam Name	Piaui	Jabiberi	Jacarecica I	Cajaiba	Xingo
River Name	Piaui	Jabiberi	Jacarecica	Trairas	S. Francisco
Responsible Organization	COHIDRO	COHIDRO	COHIDRO	COHIDRO	CODEVASF
Purpose	Irrigation	Irrigation	Irrigation	Irrigation/ Domestic	Power generation
Catchment Area (km^2)	1,350	118	221	195	633,000
Spillway Design Discharge (m^3/s) (Probable year)	2,238 (1,000)	331.5 (1,000)	738 (1,000)	1,621 (10,000)	33,000
Specific Discharge ($m^3/s/km^2$)	1.66	2.81	3.34	8.31	0.052

3.3.2 Design Discharge for Energy Dissipater of Dam

The design discharge of the dam energy dissipater is adopted as the discharge with 100-year return period, according to "Manual of River Works, Ministry of Public Works, Japan".

The probable discharge with 100-year return period at Fazenda Belem (C.A.=15,740km^2) is 1,211 m^3/s, and that at the dam site is 1,197 m^3/s taking into account of the basin areas' ratio. Then the design discharge of 1,200 m^3/s was taken for the design discharge.

3.3.3 Design Discharge of Diversion Channel during Construction

A design discharge of a diversion channel is defined, as the frequency resulting from probable analysis, comparing the expected value of losses cost resultant from respective floods. According to the degree of flood damage by a dam type, in this report, a design discharge of a diversion channel during construction is set as the following criteria:

- Concrete dam: Discharge with 2-year return period
- Earth-fill dam or Rock-fill dam: Discharge with 20-year return period

Therefore, the design discharge of the diversion channel during construction is set as follows:

- Concrete dam: 200 m^3/s (0.013 $m^3/s/km^2$)
- Earth-fill or rock-fill dam: 720 m^3/s (0.048 $m^3/s/km^2$)

3.3.4 Design Discharge for the Spillway of a Check Dam

The larger one of the following two discharges is adopted as a design discharge for the spillway of a check dam, according to "Manual of River Works, Ministry of Public Works, Japan".

- Discharge with 100-year return period
- Experienced maximum discharge

The probable discharge with 100-year return period at Fazenda Belem (C.A.=15,740km^2) is 1,211 m^3/s, and that at the dam site is 1,170 m^3/s taking into account of the basin areas' ratio. On the other hand, the experienced maximum discharge at Fazenda Belem is 647 m^3/s in 1975 during 25-year data series from 1971 to 1995. Then the design discharge with 100-year return period should be taken for the design discharge.

However, the proposed check dam has the function not only as a sand sedimentation facility but also as an intake facility. For the dam reservoir operation, this intake is inevitable for improvement of reservoir water quality. Therefore, adding 20 % of discharge due to its importance, the design discharge of the spillway for the check dam is set at 1,400 m^3/s.

- Design discharge: 1,400 m^3/s
- Specific Discharge: 0.09 m^3/s/km^2 (Check Dam C.A.= 15,030 km^2)

CHAPTER 4 RESERVOIR OPERATION PLAN

4.1 Main Function of the Proposed Dam

The proposed dam, Vaza Barris Dam has the functions of not only "Storing Water" but also "Improvement of Reservoir Water" as follows:

1) Storing water in the dam reservoir for the purpose of:
   - Maintenance of river and riparian environment
   - Domestic/Industrial water supply
   - Irrigation water supply
   - Dilution of water quality
2) Improvement of reservoir water with the new system of low flow bypass:
   - High saline concentration water is bypassed around the dam reservoir.
   - Clean or low saline concentration water is stored in the dam reservoir.

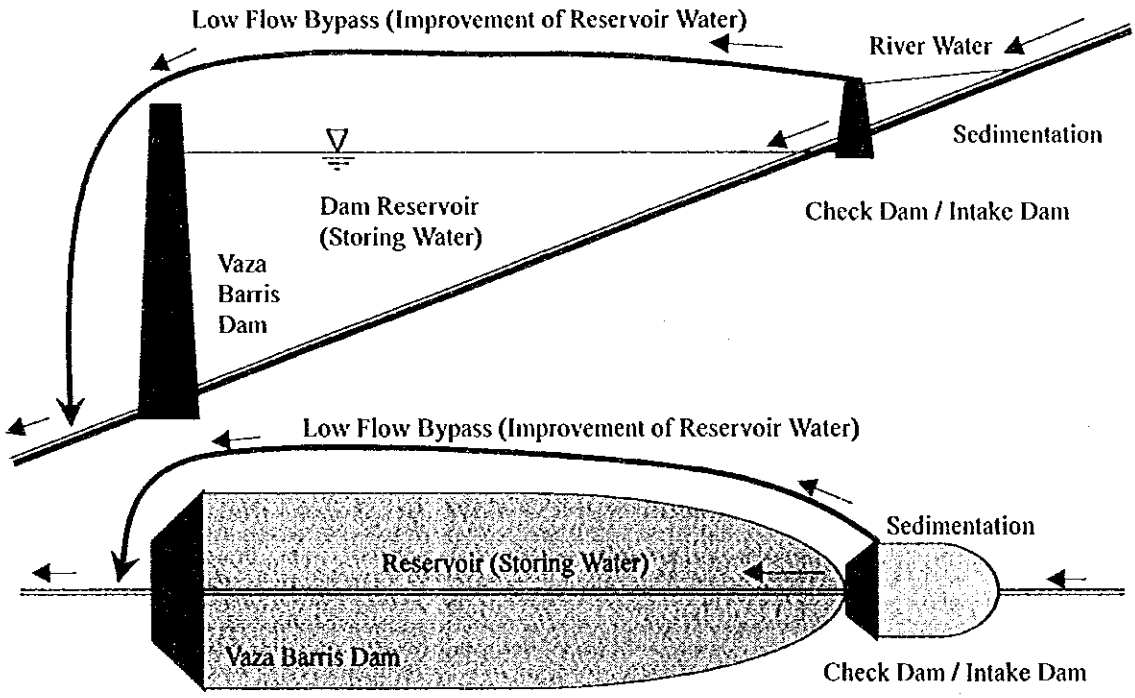

Figure-4.1 Main Function of Vaza Barris Dam

River water of the main stream has large potential of water resources but has not been able to be utilized as potable and irrigation water due to high concentration of chloride. After elaborate investigation of water quality, however, it was found that river flow has high chlorine concentration only during low flow condition but not during flood. Considering such condition of the water quality in Vaza Barris River, the following dam plan with the new system of a low flow bypass was established:

1) Low flow with high concentration of chloride from the upstream is bypassed around the dam reservoir to the downstream.
2) River water during flood is stored in the dam reservoir.

Then,

3) Dam reservoir water is kept clean and is utilized as potable and irrigation water.

4.2 Reservoir Operation Model

4.2.1 Concept of Reservoir Operation Model

The concept of the reservoir operation model is shown as a flow chart in Figure-4.3. The reference points for the reservoir operation plan were set at the check dam site and the main dam site, because the hydrological reference point in the most downstream, Fazenda Belem is located in the very near downstream of the dam. The both catchment area of the dam site (C.A.=15,560 km^2) is close to that of Fazenda Belem (C.A.=15,740 km^2). Given conditions in this model are listed as follows:

- Daily Discharge (Refer to Hydrological Model)
  1) Main dam: Q_D, 2) Check dam: Q_{CD}, 3) Reservoir inflow: Q_D - Q_{CD}
- Development Water Amount at the Dam Site
  1) Compensation (Maintenance) discharge: 0.44 m^3/s
  2) Domestic/Industrial water use: 0.887 m^3/s
  3) Irrigation water use: Maximum 2.912 m^3/s
     Average 1.507 m^3/s
- Physical Precondition
  1) H-V curve of the dam reservoir: Refer to Figure-4.2.
  2) Capacity of the check dam reservoir: 10,000,000 m^3 at the level of EL. 63.0 m
  3) Bypass discharge: 0 – 2.0 m^3/s (Assumption cases in trial calculation)
- Initial Condition
  1) Initial reservoir volume: Full of capacity
  2) Reservoir water quality:
     Average water quality according to the average flow during 1986-1995
     : Main Dam Reservoir: Cl=200 mg/l, EC= 0.90 dS/m, Q_{AVE}=8.74 m^3/s
     : Check Dam Reservoir: Cl=330 mg/l, EC= 1.40 dS/m, Q_{AVE}=4.65 m^3/s

Elevation (EL.m)	Area (m^2)	Volume (m^3)	Accumulated Volume (m^3)
20	0	0	0
25	113,241	283,103	283,103
30	844,695	2,394,840	2,677,943
35	2,980,121	9,562,040	12,239,983
40	5,884,332	22,161,133	34,401,115
45	8,295,946	35,450,695	69,851,810
50	10,665,979	47,404,813	117,256,623
55	13,049,208	59,287,968	176,544,590
60	15,527,046	71,440,635	247,985,225

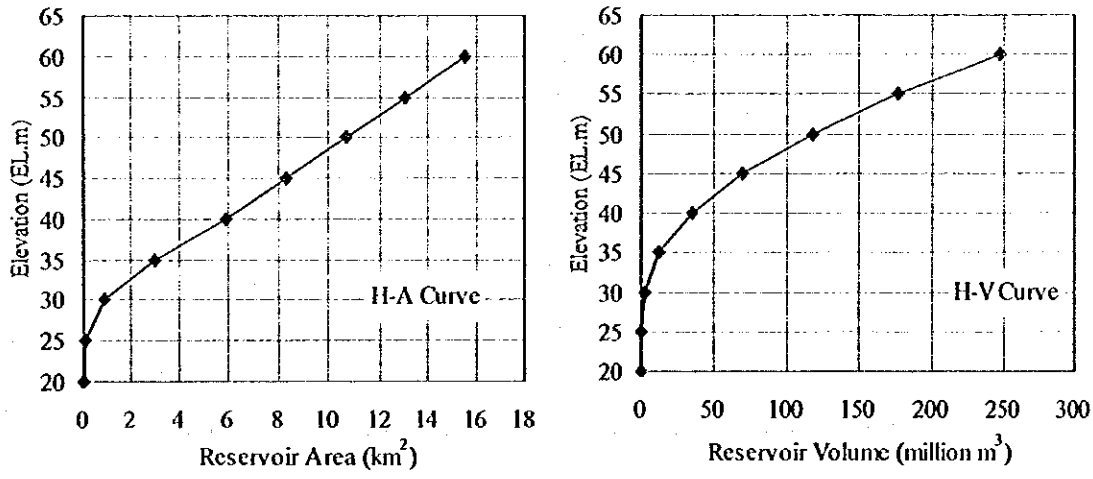

Figure-4.2 H-A and H-V Curve of Main Dam Reservoir

Based on the given conditions above, following water volume/flow and water quality are simulated:

1) Check Dam Reservoir
   - Inflow: Natural Flow
   - Outflow to Dam Reservoir and Low Flow Bypass
2) Main Dam Reservoir
   - Inflow: Natural Flow to Reservoir, Flow from Check Dam
   - Outflow: Dam Draw-off, Spillway Discharge, Evaporation
   - Intake Flow: Municipal/Industry Water, Irrigation Water

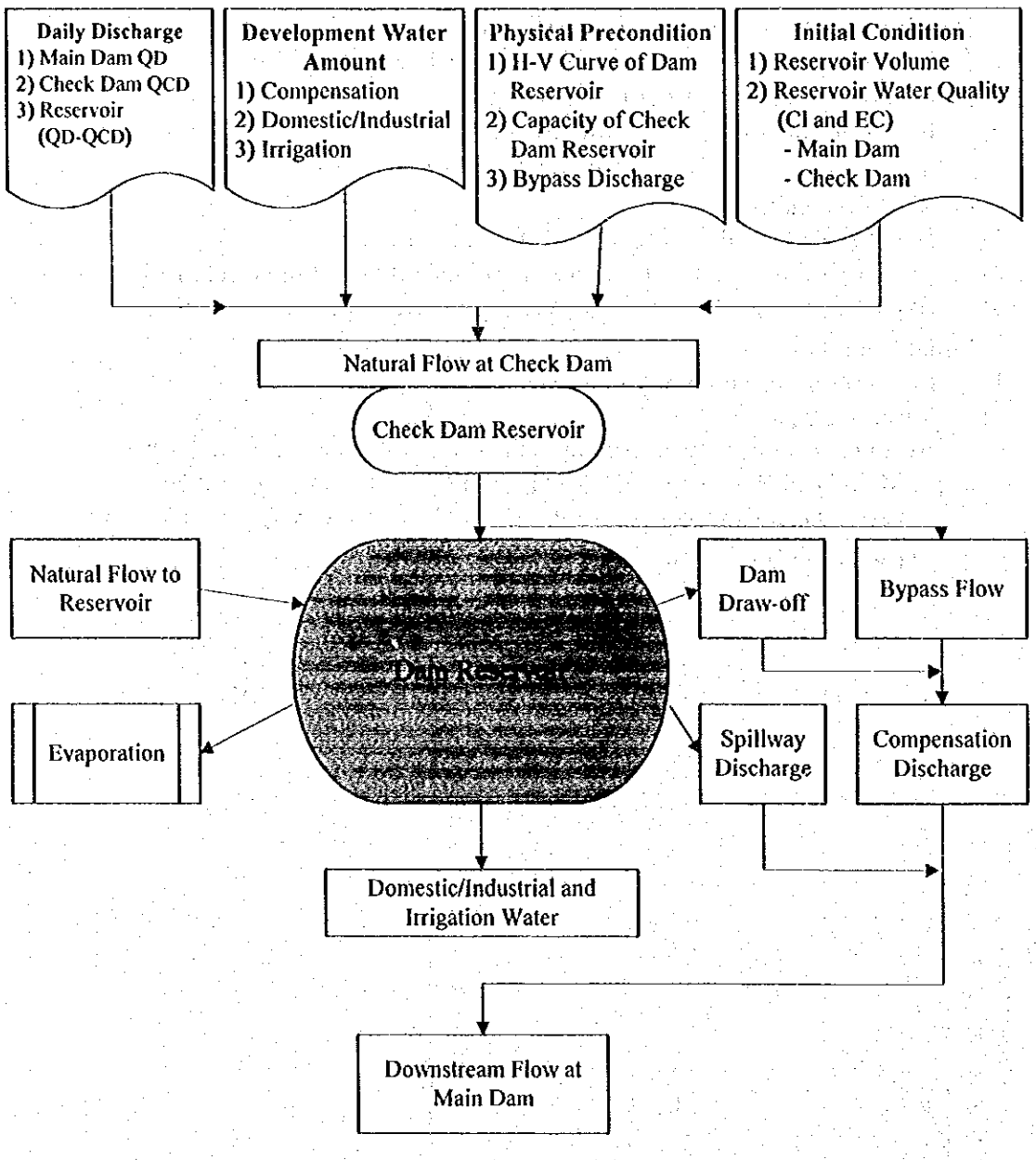

Figure-4.3 Concept of Reservoir Operation Model

Based on the given conditions above, following water volume/flow and water quality are simulated:

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   - Intake Flow: Municipal/Industry Water, Irrigation Water

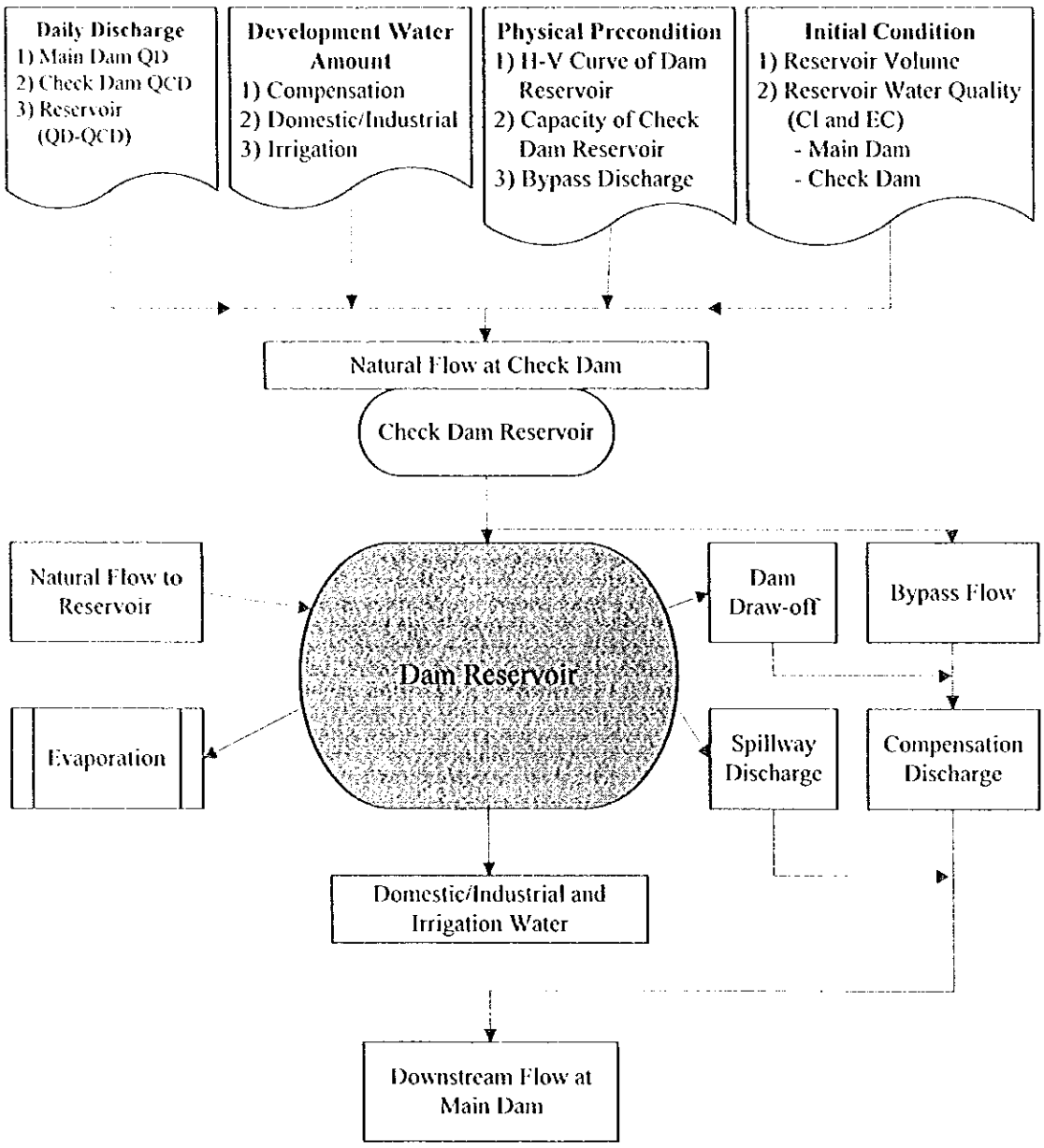


Figure-4.3 Concept of Reservoir Operation Model

4.2.2 Runoff Model

Runoff model of reservoir operation simulation is presented in Figure-4.4 and Table-4.1. Hydrological reference points were set at the check dam and the main dam, of which daily discharge is calculated based on the discharge at Ponte SE-302 (Q_{PSE}) and Fazenda Belem (Q_{FB}). The daily discharge from the basin B, C and D is calculated introducing "Runoff Contribution Factors", which were estimated as the following equations, based on the basin areas, the basin mean rainfalls and the runoff rates of the basins of B, C and D.

[Runoff Contribution Factors] = [Basin Ratio] * [Basin Rainfall Ratio] * [Runoff Rate Ratio]

Then, the runoff contribution factors were obtained as 0.24, 0.54 and 0.22 for the basins of B, C and D respectively, and discharges from these basins could be calculated by following equations:

$$Q_B = 0.24 \cdot (Q_{FB} - Q_{PSE}) \quad Q_C = 0.54 \cdot (Q_{FB} - Q_{PSE}) \quad Q_D = 0.22 \cdot (Q_{FB} - Q_{PSE})$$

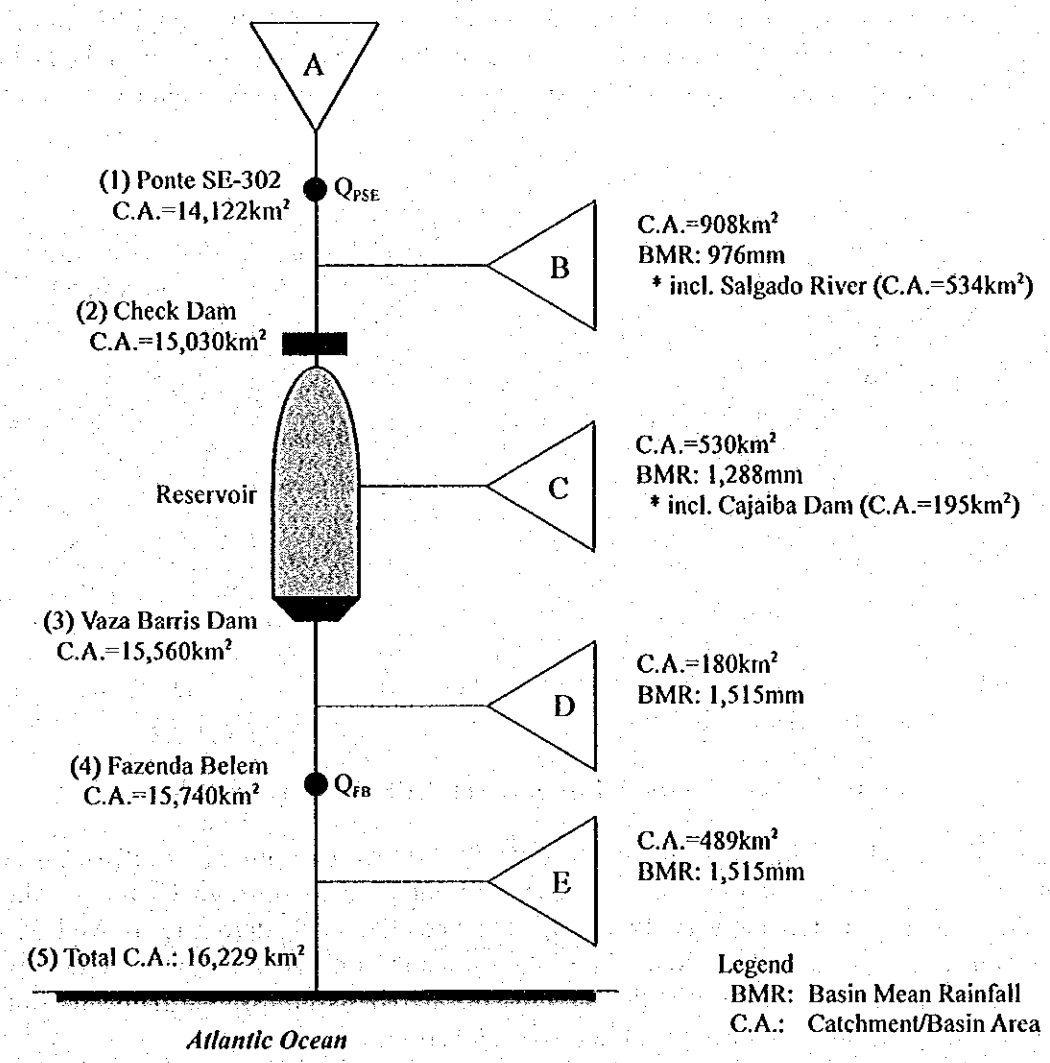


Figure-4.4 Runoff Model for Reservoir Operation Simulation

Table-4.1 Basin Division

River Basin					Reference Point		
No.	Basin Name	Basin Area. (km^2)	Basin Mean Rainfall (mm)	Runoff Rate	No.	Point Name	Basin Area (km^2)
A	Upstream (Bahia)	14,122	500-800	-	(1)	Ponte SE-302	14,122
B	Ponte – Check Dam	908	976	0.07	(2)	Check Dam	15,030
C	Dam Reservoir	530	1,288	0.21	(3)	Vaza Barris dam	15,560
D	Dam – F. Belem	180	1,515	0.21	(4)	Fazenda Belem	15,740
E	Most Downstream	489	1,515	-	(5)	River Mouth	16,229
Total/Average of B, C & D		1,618	1,138	0.14			

4.2.3 Water Quality Model

In Vaza Barris River of Sergipe State, the Cl loads that is drained from the basin can be basically distinguished in two origins: one came from the main river (river itself) and other from the tributaries (rain water), carrying the salts retained on the soil surface. According to available water quality data, it can be found a good relationship between Cl/EC and river flow, as can be seen in Figure-4.5 established in Ponte SE-302 and Fazenda Belem, showing feasibility of the study of Cl/EC behavior according to mass balance concept in these basins.

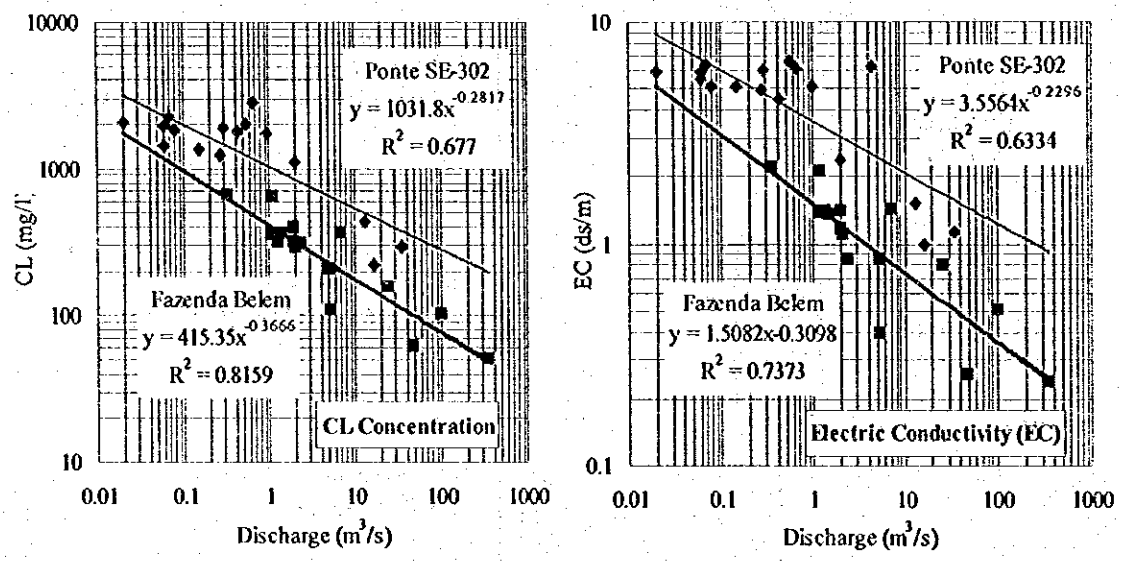


Figure-4.5 Relationship between Cl/EC and River Flow

All the loads produced in downstream of Ponte SE-302 were assumed as diffuse or non-point sources. The calculation of these loads might be held through Cl mass balance procedures in the stretch between Ponte SE-302 and the check dam site as well as the check dam site and Fazenda Belem. The data obtained on Sao Domingos station were applied for estimation of diffuse loads for this first stretch, assuming that the same station represents water characteristics in the check dam location.

Based on observation data at Fazenda Belem and Ponte SE-302, tributaries basin loads (Cl) between both stations was estimated as shown in Figure-4.6. Assuming that 80% of these loads be produced from the basin between Ponte SE-302 and the check dam site, and 20 % be produced from the check dam site and Fazenda Belem, water quality equations of both

basin loads is estimated as follows:

Between Ponte SE-302 and the check dam site: $[Cl]_{PSE\text{-}CH} = 590.43*Q^{-0.3557}$
Between the check dam site and Fazenda Belem: $[Cl]_{CD\text{-}FB} = 78.059*Q^{-0.3557}$

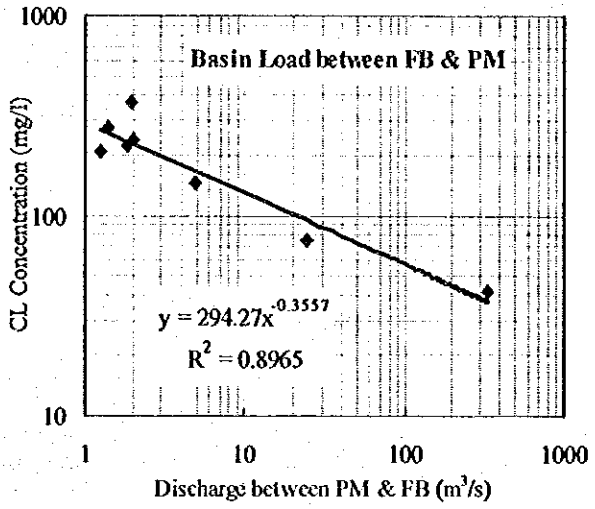


Figure-4.6 Tributaries Basin Load (Cl) between Fazenda Belem and Ponte SE-302 (Relationship between Cl and Discharge)

On the other hand, in Trairas River located in the basin between the check dam site and the main dam site, river water quality was observed to be about 80 mg/l of chloride concentration (Cl) and 0.4 ds/m of electric conductivity (EC) when river flow is about 1 m^3/s. When the same indexes of the equation at Fazenda Belem is employed as the trend of decreasing of Cl and EC according to the discharge decreasing, tributaries basin load of Trairas River could be estimated as following equations:

$$[Cl]_{TRAIRAS} = 80*Q^{-0.37}$$
$$[EC]_{TRAIRAS} = 0.4*Q^{-0.31}$$

This equation for Cl concentration is very similar with the equation estimated before.

Based on the above equations and main river equation at Fazenda Belem (Figure-4.5), the water quality (Cl and EC) at the main dam and the check dam could be calculated using the following equations:

$$[\text{Water Quality Cl and EC}]_{MD} = ([Load]_{FB} - [Load]_{MD\text{-}FB}) / Q_{MD}$$
$$[\text{Water Quality Cl and EC}]_{CD} = ([Load]_{MD} - [Load]_{CD\text{-}MD}) / Q_{CD}$$

Applying the hydrological model to the year of 1985 – 1995, the relationships between Cl/EC and discharge are obtained as shown in Figure-4.7 and Figure-4.8. As the dam site is very near from Fazenda Belem, daily calculation results of water quality are concentrated to the one line (Figure-4.7). However the water quality results at the check dam (Figure-4.8) are scattered with in the specific range due to daily deference of the water contribution from the basin between the check dam and Fazenda Belem. As Figure-4.7 is presented with the equation at Sao Domingos located 5 km upstream of the check dam site, Cl concentration by this equation is almost within the range and locates in upper or average part of the rage.

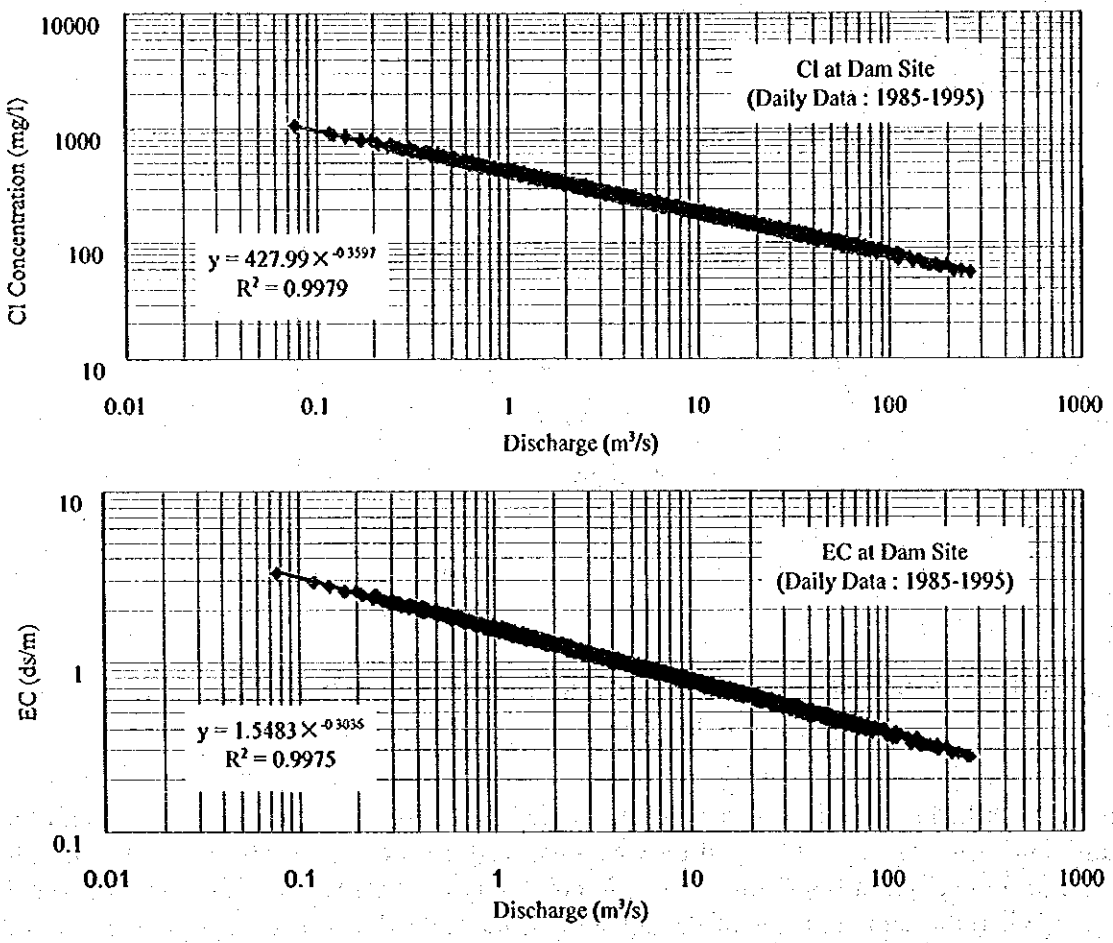


Figure-4.7 Estimation of Daily Water Quality (Cl and EC) at the Main Dam Site

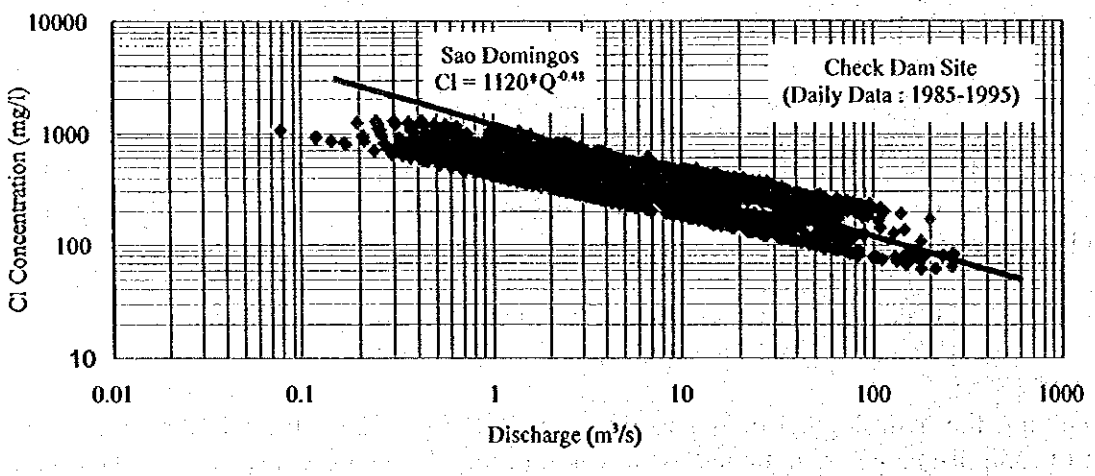


Figure-4.8 Estimation of Cl concentration at the check dam Site

Based on the above study, Water Quality Model of reservoir operation simulation is formulated as shown in Figure-4.9 and Table-4.2. River water quality of Sodium (Na), Magnesium (Mg) and Calcium (Ca) could be estimated applying the correlation equations to EC, of which equations were formulated based on the water quality observation data at Ponte SE-302 and Fazenda Belem.

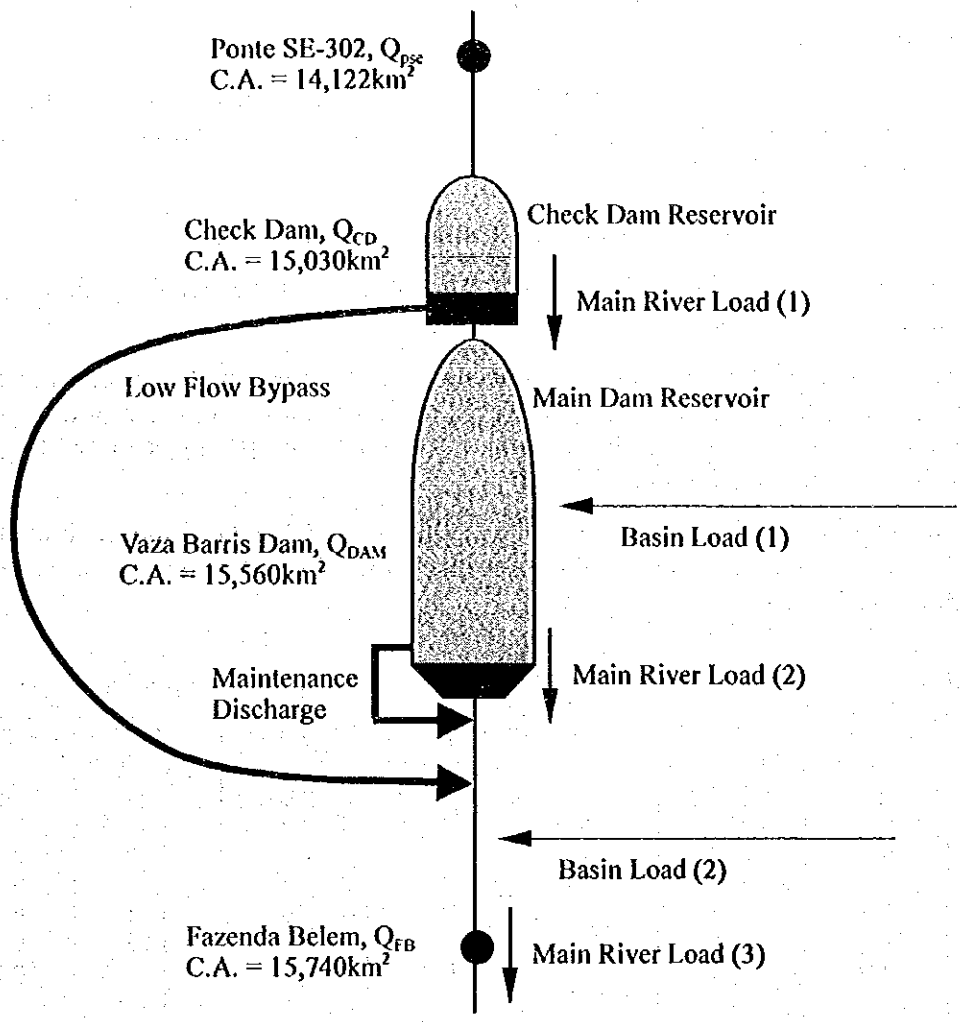


Figure-4.9 Water Quality Model for Reservoir Operation Simulation

Table-4.2 Water Quality Estimation Equation

Load	Point/Basin	Cl (mg/l)	EC (ds/m)
Main	(1) Check Dam (CD)	$[Cl \text{ or } EC]_{CD} = ([Load]_{MD} - [Load]_{CD\text{-}MD}) / Q_{CD}$	
River	(2) Main Dam (MD)	$[Cl]=427.99*Q^{-0.3597}$	$[EC]=1.5483*Q^{-0.3035}$
Load	(3) Fazenda Belem (FB)	$[Cl]=415.35*Q^{-0.3666}$	$[EC]=1.5082*Q^{-0.3098}$
Tributary	(1) Between CD and MD	$[Cl]=80*Q^{-0.37}$	$[EC]=0.4*Q^{-0.31}$
Basin Load	(2) Between MD and FB	$[Cl]=80*Q^{-0.37}$	$[EC]=0.4*Q^{-0.31}$
Water Quality Items		Correlation equation with EC (ds/m)	
Na (mg/l)		[Na] = 104.53*[EC]	
Mg (mg/l)		[Mg] = 35.514*[EC]	
Ca (mg/l)		[Ca] = 56.833*[EC]	

4.3 Simulation Result of Reservoir Operation

4.3.1 Trial Simulation

Reservoir operation simulation was carried out changing bypass discharge from 0.0 m^3/s to 10.0 m^3/s. The simulation was implemented in the two cases of "without water quality capacity for dilution" and "with that".

(1) Simulation without Water Quality Capacity for Dilution

Table-4.3 and Figure-4.10 shows the simulation result of "without water quality capacity for dilution" case, which is summarized as follows:

1) The maximum and average Cl concentration become smaller according to the increase of bypass discharge. However the maximum Cl concentration is higher than 250 mg/L (upper limit for drinking water by WHO and CONAMA), until bypass discharge would be set as 10 m^3/s.
2) The number of the days more than 250mg/L of Cl concentration dramatically decreases from 0 to 0.8 m^3/s of bypass discharge, but almost no decrease (19 to 17 days in case from 0.8 m^3/s to 2.0 m^3/s.

Table-4.3 Simulation Result of "without water quality capacity" Case

Case	Bypass Discharge (m^3/s)	Reservoir Volume * (Million m^3)	Reservoir Cl Concentration (mg/l)		No. of Days > 250 mg/l (day)	Reservoir Electric Conductivity (dS/m)	
			Max.	Ave		Max.	Ave
1	0.0	52.4	387	204	969	1.43	0.83
2	0.2	52.4	377	197	770	1.39	0.81
3	0.4	52.4	364	190	366	1.35	0.78
4	0.6	58.9	351	182	76	1.31	0.75
5	0.8	65.0	335	175	19	1.25	0.73
6	1.0	68.8	324	168	18	1.22	0.71
7	1.2	71.8	312	163	18	1.18	0.69
8	1.4	74.2	302	157	18	1.14	0.67
9	1.6	75.9	294	152	18	1.12	0.65
10	1.8	77.2	288	148	17	1.10	0.63
11	2.0	78.3	285	144	17	1.08	0.62
12	3.0	82.1	279	127	11	1.06	0.56
13	10.0	87.5	248	94	0	0.95	0.44

Note: * Includes sediment volume of 2,000,000 m^3.

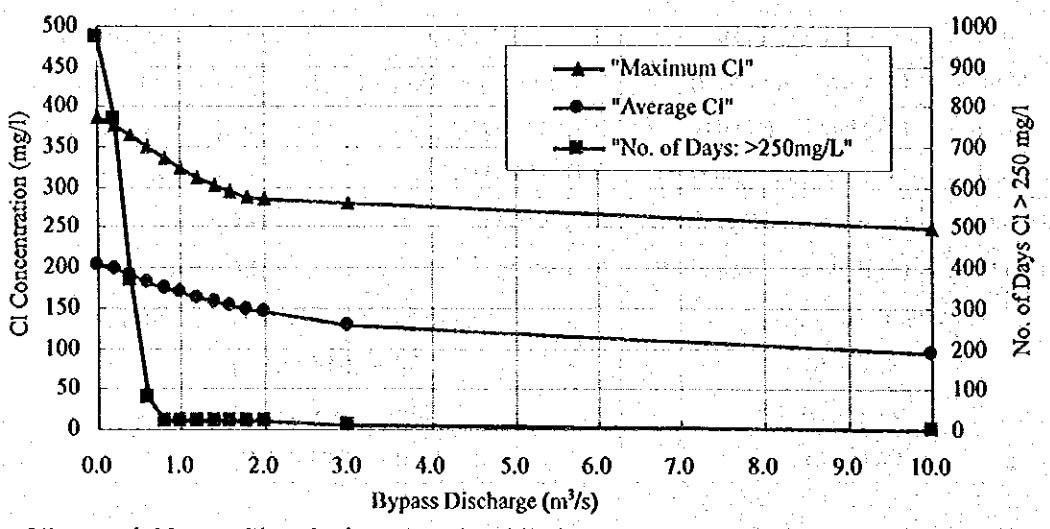


Figure-4.10 Simulation Result of "without water quality capacity" Case

(2) Simulation with Water Quality Capacity for Dilution

It is found that no dilution water volume causes these phenomena mentioned above when the reservoir is almost vacant of water. Then water quality capacity for dilution was introduced to this reservoir planning. Total necessary reservoir volume was calculated so as to be less than 250 mg/l of chloride concentration. The simulation result of "with water quality capacity for dilution" is shown in Table-4.4 and Figure-4.11 and is summarized as follows:

1) In case of no low flow bypass, total reservoir volume 634 million m^3 is needed.
2) Total reservoir volume is dramatically decreased according to the increase of bypass discharge from 0 to 0.7 m^3/s, but very small change in case from 0.7 to 3.0 m^3/s of bypass discharge.
3) The average Cl concentration is gradually decreased according to the increase of bypass discharge.
4) Total reservoir volume which is necessary to make reservoir water less than 250mg/l, is the smallest (84.9 million m^3) when bypass discharge is 1.3-1.5m^3/s, and increases when bypass discharge is lower and higher than that.

Table-4.4 Simulation Result of "with water quality capacity for dilution" Case

Case	Bypass Discharge	Reservoir Volume (Mm^3)			Reservoir Cl Concentration (mg/l)		Reservoir Electric Conductivity (dS/m)	
		Total Volume*	Dilution Volume	Other Volume	Max.	Ave.	Max.	Ave.
1	0.0	634.0	519.0	115.0	250	198	1.07	0.84
2	0.2	514.0	409.0	105.0	250	192	1.07	0.82
3	0.4	384.0	291.4	92.6	250	186	1.06	0.79
4	0.6	236.0	153.1	82.9	250	179	1.05	0.76
5	0.65	180.0	102.2	77.8	250	177	1.04	0.75
6	0.7	95.2	27.5	67.7	250	176	1.01	0.74
7	0.75	92.3	23.8	68.5	250	175	1.00	0.73
8	0.8	90.2	21.0	69.2	250	173	0.99	0.73
9	0.9	87.6	17.1	70.5	250	171	0.98	0.72
10	1.0	86.2	14.5	71.7	250	168	0.98	0.71
11	1.1	85.4	12.5	72.9	250	165	0.98	0.70
12	1.2	85.0	11.0	74.0	250	163	0.98	0.69
13	1.3	84.9	9.8	75.1	250	160	0.97	0.68
14	1.4	84.9	8.9	76.0	250	157	0.97	0.67
15	1.5	84.9	8.1	76.8	250	155	0.97	0.66
16	1.6	85.0	7.5	77.5	250	153	0.97	0.65
17	1.8	85.2	6.6	78.6	250	148	0.97	0.64
18	2.0	85.4	5.9	79.5	250	144	0.97	0.62
19	3.0	86.7	3.8	82.9	250	128	0.97	0.56

Note: * Includes sediment volume of 2,000,000 m^3.

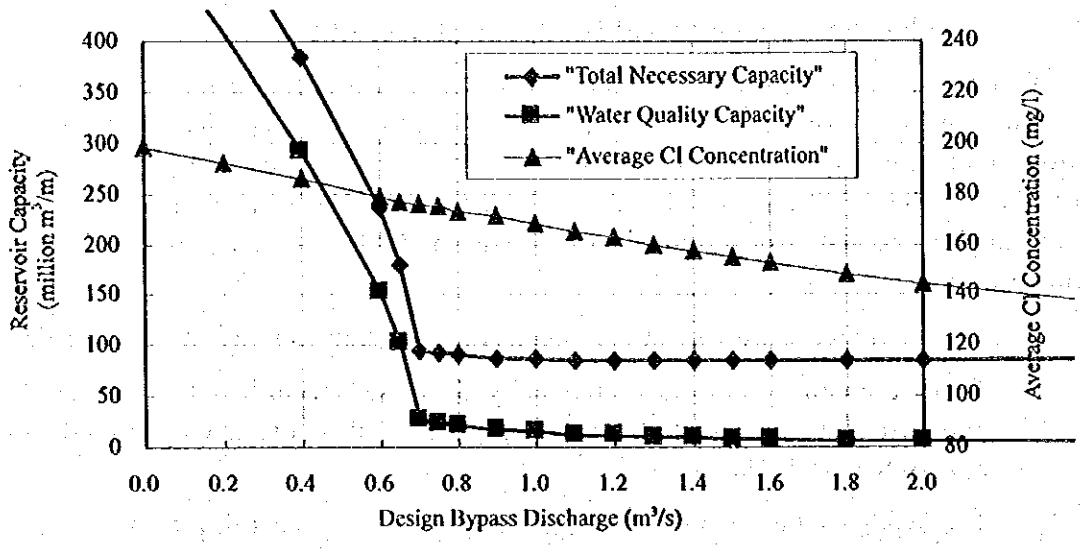


Figure-4.11 Simulation Result of "with water quality capacity for dilution" Case

4.3.2 Decision of Bypass Discharge and Reservoir Volume

The result of the above trial simulations of reservoir operation indicates feasibility to use the reservoir water plenty for domestic and irrigation supply. Based on the result, the following sets of bypass discharge and total reservoir volume are proposed as alternatives of Vaza Barris Dam plan.

Design Bypass Discharge: Q_{BP} =0.7-1.3 m^3/s
Total Reservoir Volume: V_{RES} =85-96 million m^3

It is apparent that the larger scale of bypass discharge and total reservoir volume are the higher cost of the low flow bypass and the dam. Table-4.5 shows rough cost comparison with the alternative scales of the low flow bypass and the dam. In the comparison, based on the study of design and cost estimate, it was assumed that the type of low flow bypass is a pipeline and the dam type is a concrete dam.

Table-4.5 Comparison with the Alternatives of Low Flow Bypass and dam

Bypass Discharge (m^3/s)	Total Reservoir Volume (M m^3)	Reservoir N.W.L (EL.m)	Low Flow Bypass Pipeline		Concrete Dam		Total Cost (M R$)
			Dimension of Channel (mm)	Cost *1 (M R$)	Volume (m^3)	Cost (M R$)	
0.70	96	47.8	1,050 x 1,050	31.6	222,000	29.1	60.7
0.75	93	47.5	1,050 x 1,050	31.6	219,000	28.7	60.3
0.80	91	47.2	1,100 x 1,100	36.6	216,000	28.3	64.9
0.90	88	46.9	1,150 x 1,150	37.7	213,000	27.9	65.6
1.00	87	46.8	1,200 x 1,200	38.7	212,000	27.8	66.5
1.10	86	46.7	1,200 x 1,200	38.7	211,000	27.6	66.3
1.20	85	46.6	1,250 x 1,250	44.3	210,000	27.5	71.8
1.30	85	46.6	1,300 x 1,300	45.4	210,000	27.5	72.9

Consequently, the most economical alternative was adopted and the design discharge of the low flow bypass and the total reservoir capacity are set as follows:

Design Discharge for Low Flow Bypass: 0.75 m^3/s
Total Reservoir Volume (N.W.L): 93000,000 m^3 (EL. 47.50 m)

4.3.3 Reservoir Operation Plan

The simulation result of the reservoir operation for Vaza Barris Dam is shown in Figure-4.12 and Figure-4.13. These figures show variation of reservoir water volume, inflow, chlorine concentration (Cl) and electric conductivity (EC). To identify the effect of low flow bypass, the simulation result in the case of no bypass is also presented in the same figures. Table-4.6 shows the maximum and the average of reservoir water quality during the calculation period of 1986 to 1995.

On the other hand, river flow and its water quality at the downstream of the dam shall be changed after construction of the dam. Figure-4.14 shows these changes on them.

Table-4.6 Summary of Water Quality in Vaza Barris Dam Reservoir

Parameter \ Low Flow Bypass Discharge		Q_{BP} = 0.75 m³/s		Q_{BP} = 0.0 m³/s	
		Maximum	Average	Maximum	Average
Chloride: Cl	(mg/l)	250	175	312	199
	(me/l)	7.1	4.9	8.8	5.6
Electric Conductivity: EC	(dS/m)	1.00	0.73	1.20	0.82
Sodium: Na	(mg/l)	104	77	126	85
	(me/l)	4.5	3.3	5.5	3.7
Calcium: Ca	(mg/l)	57	42	68	46
	(me/l)	2.8	2.1	3.4	2.3
Magnesium: Mg	(mg/l)	35	26	43	29
	(me/l)	2.9	2.1	3.5	2.4
Sodium Absorption Rate: SAR		2.7	2.3	2.9	2.4
Carbonic Acid: HCO_3 *	(mg/l)	50 – 180			
	(me/l)	0.8 - 3.0			
pH *		6.9 - 8.3			

Note: * HCO_3 and pH are not dependent on discharge.

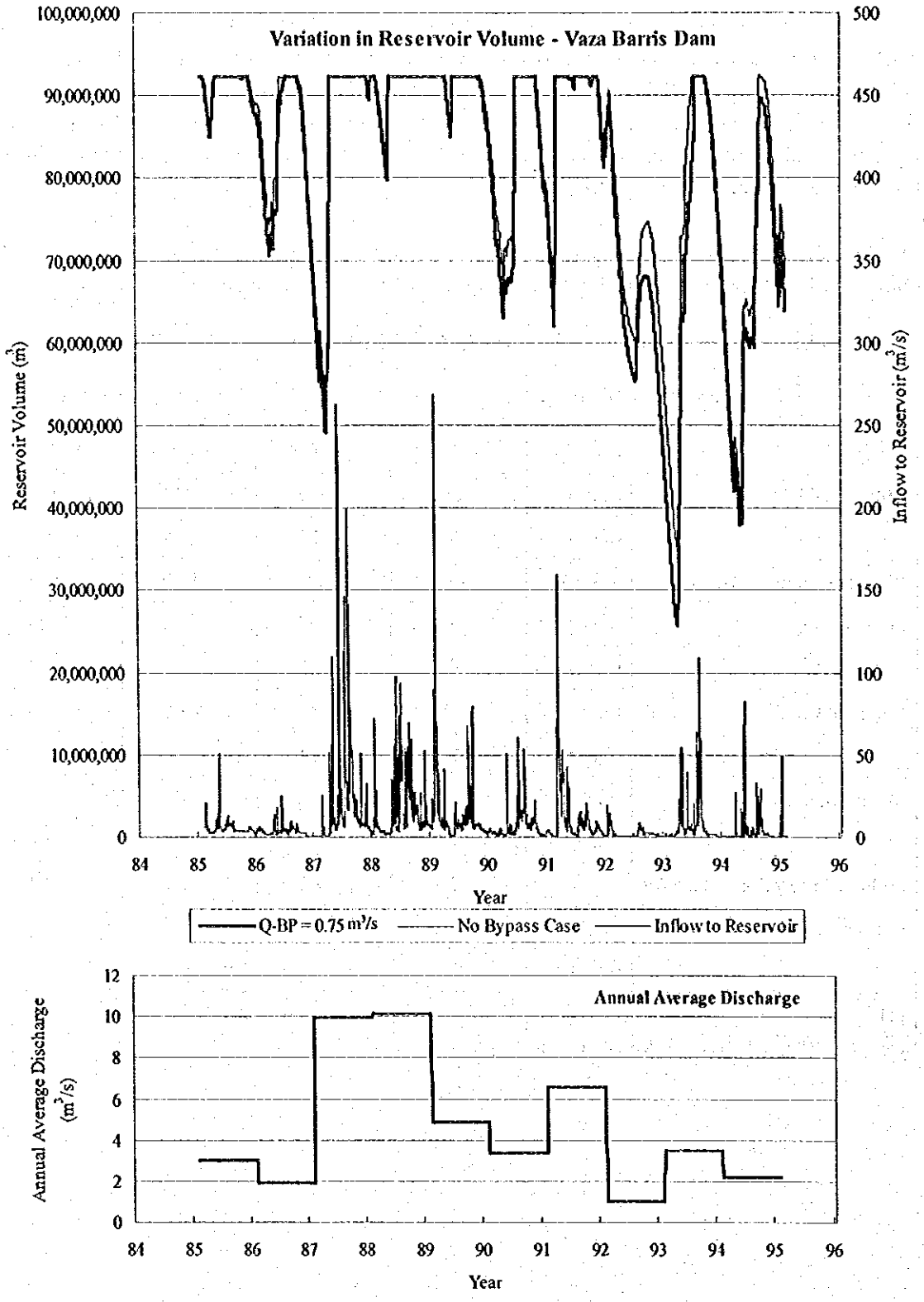
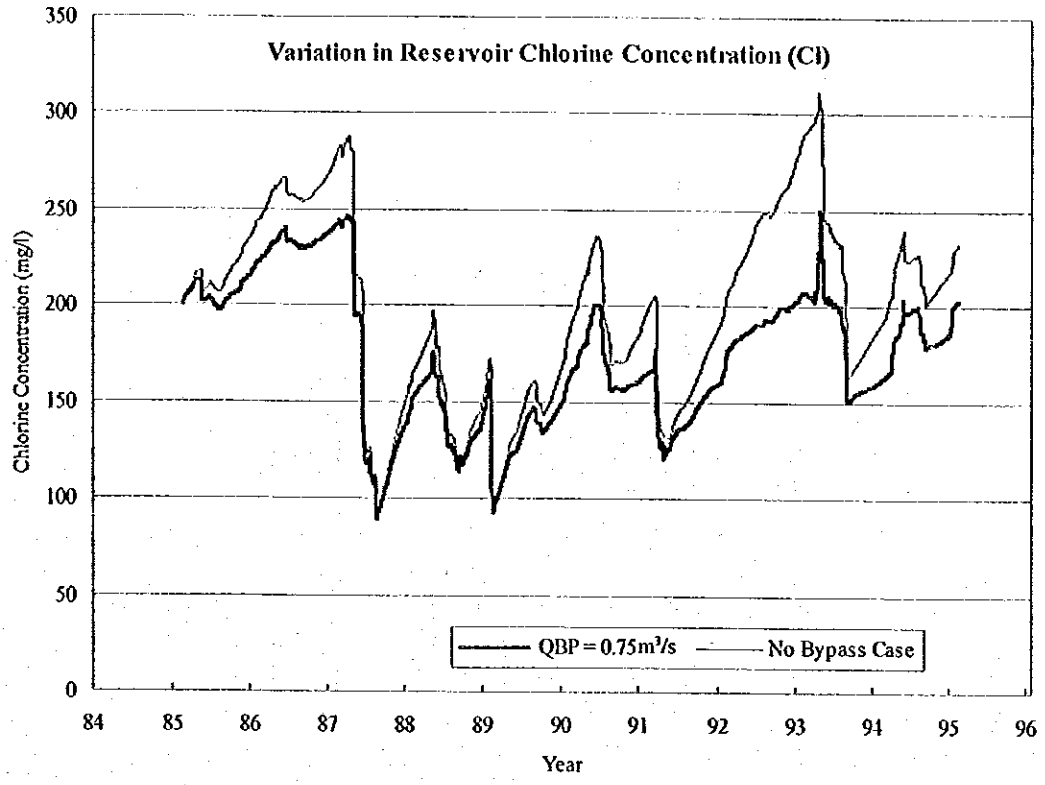


Figure-4.12 Variation of Reservoir Volume and Inflow



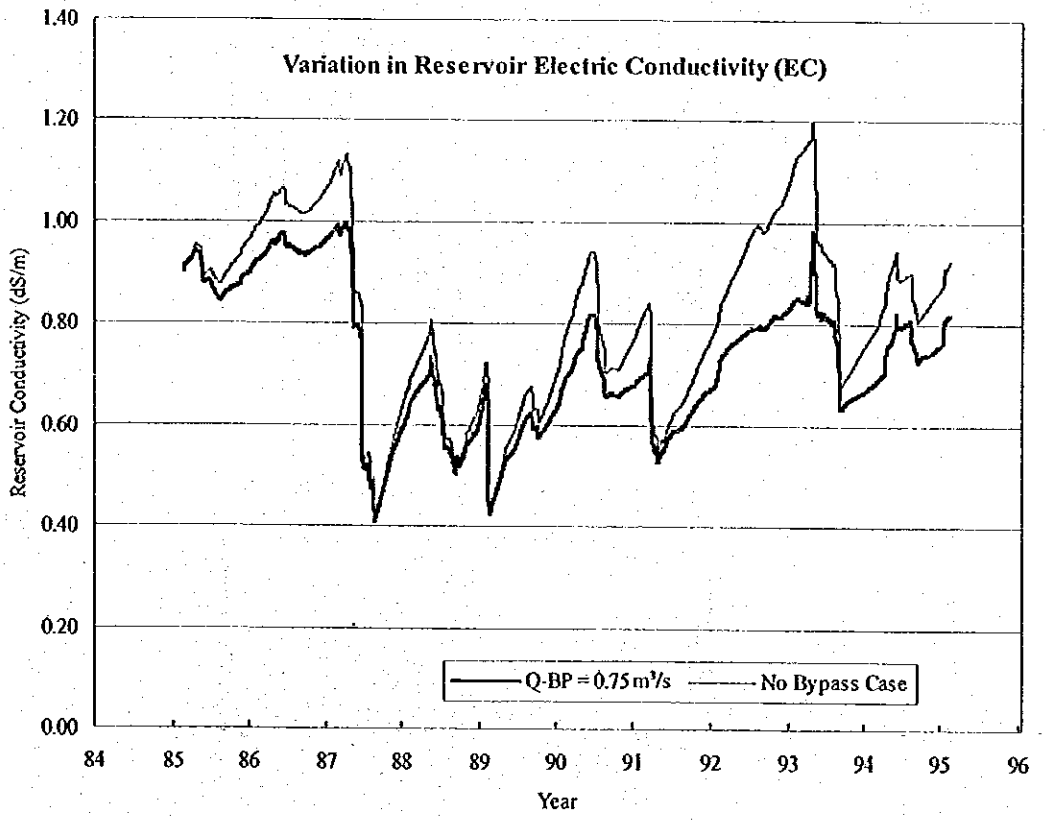


Figure-4.13 Variation of Chlorine Concentration and Electric Conductivity in the Vaza Barris Dam Reservoir

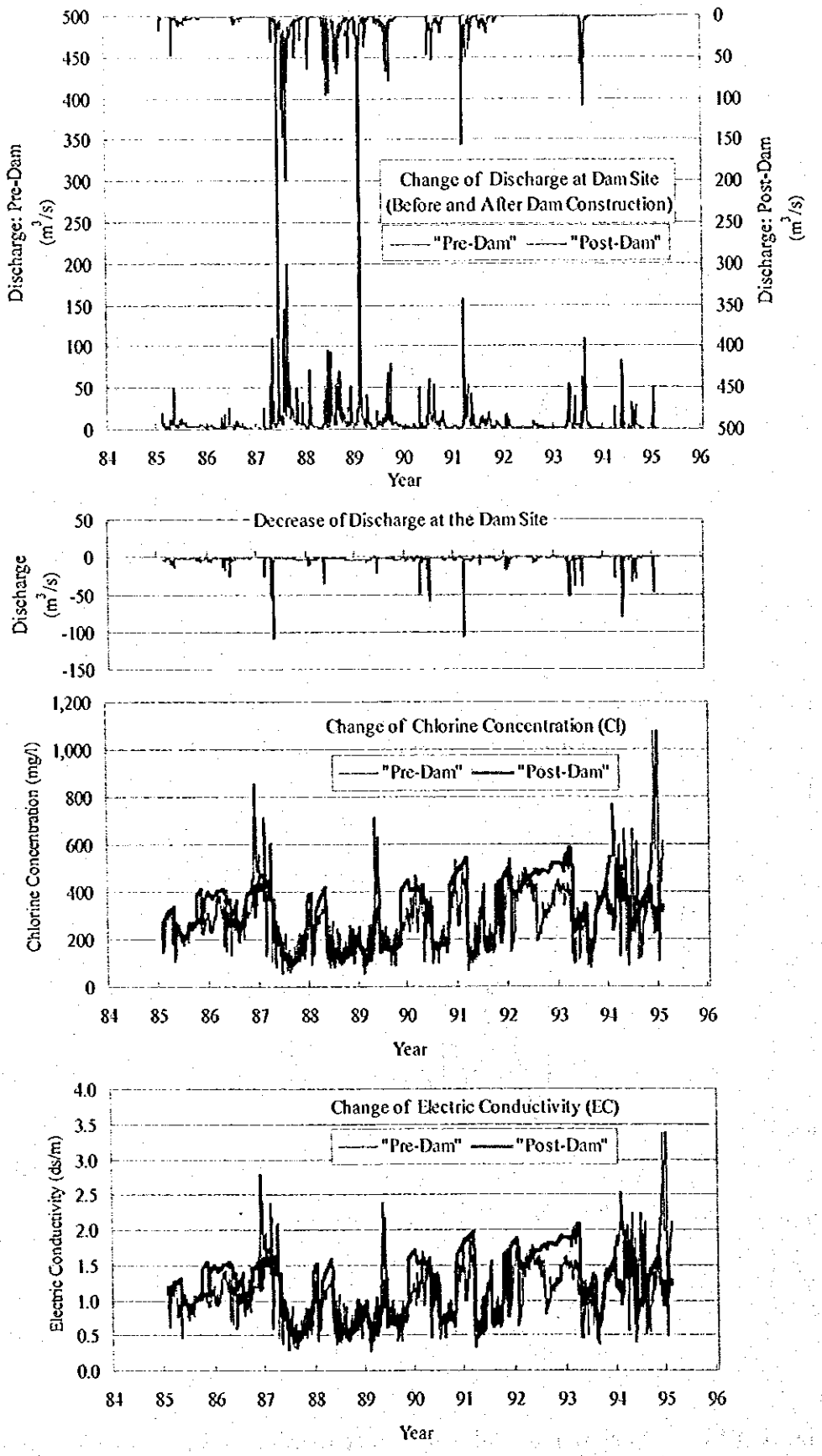


Figure-4.14 Changes on River Flow and Water Quality at the Dam Site Before and After Construction of the Dam

4.4 Specifications of the Plan of Vaza Barris Dam

Based on reservoir operation simulation, reservoir capacity distribution was set as shown in Figure-4.15.

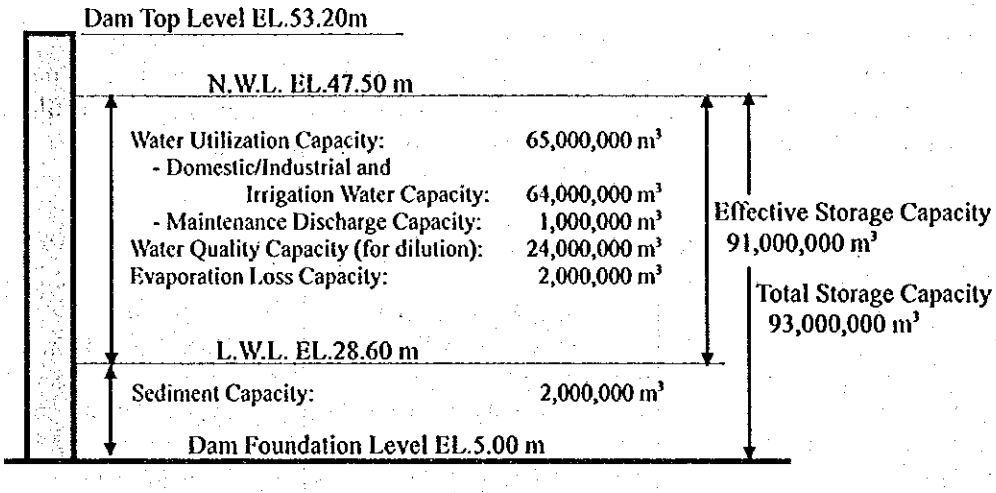


Figure-4.15 Schematic Description of Capacity and Planning Water Level

Planned specification of Vaza Barris Dam on development discharge, dam reservoir allocation, Dam/Spillway, check dam, low flow bypass is summarized in Table-4.7.

Table-4.7 Planed Specification of Vaza Barris Dam

Items		Unit	Specification	Remarks
Development Discharge	Domestic and Industrial Water	m^3/s	0.887	
	Irrigation Water (Max. /Ave.)	m^3/s	2.912 / 1.507	Vaza Barris Irrigation Project
	Total (Max. /Ave.)	m^3/s	3.799 / 2.394	
Dam Reservoir	Catchment Area	km^2	15,560	
	Reservoir Area (N.W.L.)	ha	950	
	Total Storage Capacity	m^3	93,000,000	
	Effective Storage Capacity	m^3	91,000,000	
	Water Utilization Capacity	m^3	65,000,000	
	- Domestic/Industrial and Irrigation Water Capacity	m^3	64,000,000	
	- Maintenance Discharge Capacity	m^3	1,000,000	Maintenance Discharge: 0.44 m^3/s
	Water Quality Capacity (for dilution)	m^3	24,000,000	
	Evaporation Loss Capacity	m^3	2,000,000	
	Sediment Capacity	m^3	2,000,000	10 m^3/km^2/year, 100 years (10Mm^3 into Check Dam)
	Normal Water Level (N.W.L.)	EL.m	47.50	
	Low Water Level (L.W.L.)	EL.m	28.60	Sediment Level
Dam and Spillway	Design Flood Discharge	m^3/s	3,600	Probable Maximum (10,000-yr. Return Period)
	Design Discharge of Energy Dissipater	m^3/s	1,200	100-year return period
	Design Discharge of Diversion Channel during Construction	m^3/s	Concrete Dam: 200 Fill Dam: 720	2-year return period 20-year return period
Check Dam	Dam Type	-	Concrete Dam	
	Dam Top Level	EL.m	63.00	
	Design Discharge of Spillway	m^3/s	1,400	1.2 times of 100-year return period
	Sediment Capacity	m^3	10,000,000	Level at EL.63.0m
Low Flow Bypass	Design Discharge	m^3/s	0.75	

4.5 Single Purpose Dam

Vaza Barris Dam was designed as a multi-purpose dam in terms of domestic/industrial and irrigation water supply. In this section, two single dams for the each purpose are planned for estimating the cost of the single-purpose dam.

Assuming that bypass discharge is same as 0.75 m^3/s of Vaza Barris Dam, reservoir operation simulation was carried out and necessary reservoir volume was determined so that chloride concentration of reservoir water becomes less than 250 mg/l. Table-4.8 shows the reservoir capacity distribution plan for each single-purpose dam.

Table-4.8 Specification of Single-purpose Dam

Items	Domestic/Industrial Water Supply Dam	Irrigation Water Supply Dam
Total Storage Capacity	42,900,000 m^3	59,500,000 m^3
Effective Storage Capacity	40,900,000 m^3	57,500,000 m^3
Water Utilization Capacity	11,100,000 m^3	34,300,000 m^3
- Domestic/Industrial Water Capacity	10,100,000 m^3	-
- Irrigation Water Capacity	-	33,300,000 m^3
- Maintenance Discharge Capacity	1,000,000 m^3	1,000,000 m^3
Water Quality Capacity (for dilution)	28,600,000 m^3	21,700,000 m^3
Evaporation Loss Capacity	1,200,000 m^3	1,500,000 m^3
Sediment Capacity	2,000,000 m^3	2,000,000 m^3
Normal Water Level (N.W.L.)	EL. 41.20 m	EL. 43.60 m
Low Water Level (L.W.L.)	EL. 28.60 m	EL. 28.60 m
Dam Top Level	EL. 46.90 m	EL. 49.30 m
Dam Height	41.90 m	44.30 m

JAPAN INTERNATIONAL COOPERATION AGENCY

STATE SECRETARIAT OF PLANNING, SCIENCE AND TECHNOLOGY
THE STATE OF SERGIPE, THE FEDERATIVE REPUBLIC OF BRAZIL

THE STUDY
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FINAL REPORT
SUPPORTING
(VOLUME II)
FEASIBILITY STUDY

[F] DAM DESIGN

MARCH 2000

YACHIYO ENGINEERING CO., LTD. (YEC)

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Table of Contents

List of Tables

List of Figures

CHAPTER 1 INTRODUCTION

1.1 Contents of Examination

On the basis of the study results: Water Resources Development Plan; Reservoir Operation Plan; Topographical Survey; Geological Survey, Design of Vaza Barris Dam is examined on the following aspects:

1) Dam basic dimensions: Level of dam top and foundation, dam height, overflow depth
2) Type of dam and type of spillway
3) Design of dam and hydraulic structures
4) Design of check dam and low flow bypass
5) Construction Plan

CEMIG standard "Civil Works Criteria for Hydropower Generation" is employed basically in design of Vaza Barris Dam and other related structures. If necessary, the Japanese Standards for dam design are referred.

1.2 Main Structures

Main structures of Vaza Barris Dam are:

1) Dam body to store water of river: Highest elevation of dam top is EL.60m. Dam is put on base rock. Dam suffices design safety factor.
2) Spillway to safely spill design flood discharge: Design flood discharge = 3,600 m^3/s (1/10,000-year).
3) Low Flow Outlet to discharge maintenance flow: Between N.W.L and L.W.L, outlet discharge of 0.44 m^3/s.

1.3 Flowchart of Dam Design

Flowchart of dam design is shown in Figure-1.1.

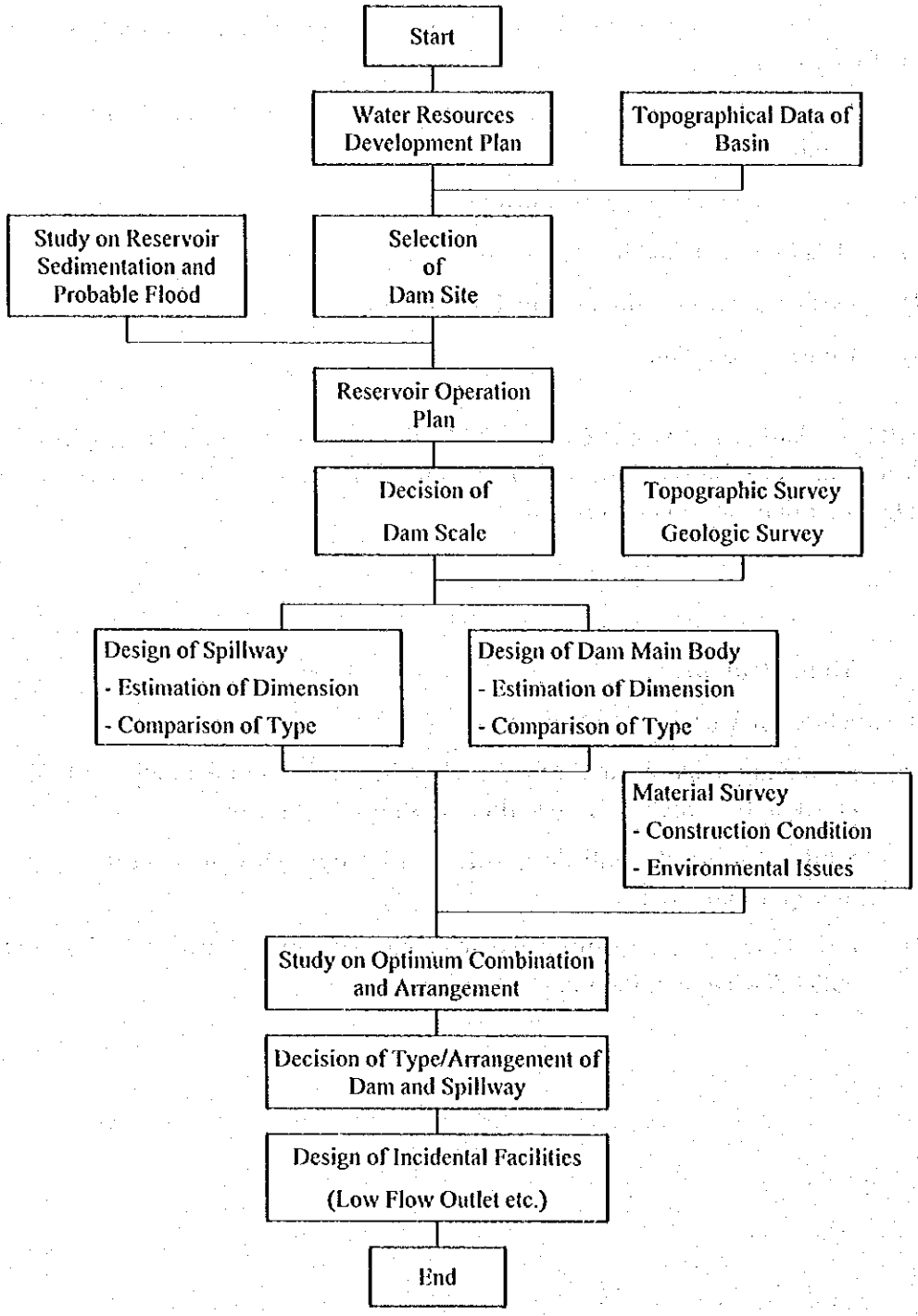


Figure-1.1 Flowchart of Dam Design

1.4 Result of Examination

Based on the result of the examination, design items are set as listed below as to Vaza Barris Dam, Check Dam and Low Flow Bypass.

Dam Body

Item	Dimension
Dam Type	: Gravity Concrete Dam
Dam Top Level	: EL. 53.2 m
Dam Foundation Level	: EL. 5.0 m
Height of Dam	: 48.2 m
Crest Length of Dam	: 280.0 m
Width of Dam Top	: 5.0 m
Volume of Dam	: 216,100 m^3
Upstream Slope	: Vertical
Downstream Slope	: 1:0.88

Spillway

Item	Dimension
Design Flood Discharge	: 3,600 m^3/s
Design Discharge for Energy Dissipater	: 1,200 m^3/s
Spillway Type	: Free Overflow Crest
Width of Overflow Crest	: 150 m
Dissipater Type	: Hydraulic jump
Size of Basin	: W:150 m × H:12 m

Low Flow Outlet

Item	Dimension
Design Discharge	: 0.44 m^3/s
Diameter of Pipe	: 800 mm
Diameter of Gate	: 250 mm

Check Dam

Item	Dimension
Dam Top Level	: EL. 63.0 m
Height of Dam	: 20.0 m
Width of Waterway	: 70.0 m
Concrete Volume	: 28,400 m^3/s

Low Flow Bypass

Item	Dimension
Bypass Type	: Box Culvert
Design Discharge	: 0.75 m^3/s
Width and Height	: 1.05 m × 1.05 m
Diameter of Gate	: 400 mm
Auxiliary Facility	: Spillway, Sedimentation basin

Construction Plan

Method of Diversion Work ; Diversion Flowing in Half of a River Section
Construction Term ; 2,004 - 2,006

CHAPTER 2 DESIGN CRITERIA

2.1 Basic Design Principle

Dam should be carefully designed to minimize the construction cost, holding necessary functions of each facility. Dam and related structures are designed according to reasonable balance between construction cost and safety level. A large safety level of structures requires a large scale of structures and complicated construction procedures. Consequently, the construction costs increase. Design Criteria are standards to decide a balance between construction cost and safety level.

Dam Design Criteria used in Brazil are as follows:

1) For almost existing dams, dam design standards of USBR (US, Dep. of the Interior, Bureau of Reclamation) and USCE (US Army, Corps of Engineer) were employed.
2) Sao Francisco Electricity Corporation (CHESF) recently uses the standard Civil Works Criteria for Hydropower Generation which was complied by Mina Gerais Power Company (CEMIG) on the basis of standards of USBR and USCE. This standard was used in designing Xingo Dam.

Considering the above situation, CEMIG standard is employed basically in design of Vaza Barris Dam and other related structures. If necessary, the Japanese Standards for dam design are referred.

2.2 Freeboard of Dam

According to CEIG standard, dam freeboards are as follows:

Table-2.1 Criteria of Freeboard

Items	Freeboard (in Normal)	Freeboard (in Flood)
Concrete Dam	N.W.L. + Min. 1.5 m	H.W.L. + Min. 0.5 m
Rock & Earth Fill Dam	N.W.L. + Min. 3.0 m	H.W.L. + Min. 1.0 m
Coffer Dam	----	W.L. + Min. 1.0 m
Spillway Chute	----	$0.6 + 0.037 \times V \times h^{(1/3)}$ Where: V = Runoff Speed (m/s) h = Runoff depth (m)

2.3 Discharge Capacity

The Discharge Capacity of the sill operating as overflow spillway will be calculated by the formula

$$Q = C \times L \times H^{1.5}$$

where

C : discharge coefficient
L : effective width of the spillway (m)
H : head upon the crest (m)

The effective width of the spillway will be defined by the formula

$$L = L' - 2\,(Ka + nKp)\,H$$

where

L	:	effective width (m)
L'	:	usable geometric width (m)
Ka	:	contraction coefficient of the abutment
Kp	:	contraction coefficient of the piers
n	:	number of piers
H	:	head upon the spillway (m)

The coefficients Ka and Kp can be based in the indications of the Figure-2.1 and Figure-2.2.

2.4 Design Flood Water Level

Design flood water level is defined as the highest water level of the reservoir at just behind the dam in case of the design flood discharge flowing down through spillway. It is generally formulated as shown below;

$$H.W.L. = N.W.L. + h$$

where

N.W.L.	:	Normal Water Level
h	:	overflow depth of spillway

Design flood water level of the Vaza Barris dam is set as follows;

- The elevations of the lowest saddles of both bank are EL. 62 m on the right bank ridge and 67 m on the left bank ridge. Therefore, the maximum reservoir water level is set as EL. 55 m considering some allowance.
- The river width including fluvial terrace at the dam site is about 100 m. The width of water flow is 150 – 200m in case of the design flood discharge. The width of spillway at overflow section is set as 150m.
- Normal water level of reservoir is EL. 47.5 m according to the reservoir operation plan.

Relationship between overflow depth (h), overflow width (L) and overflow discharge (Q) is expressed as the following equation:

$$Q = C \times L \times h^{1.5}$$

Giving Q = 3,600 m^3/s, coefficient C = 2.0 and L = 150 m to the above equation, overflow depth h = 5.2 m is obtained. The design flood water level, normal water level = EL. 47.5 m + overflow depth(5.2 m), is set as EL. 52.7 m.

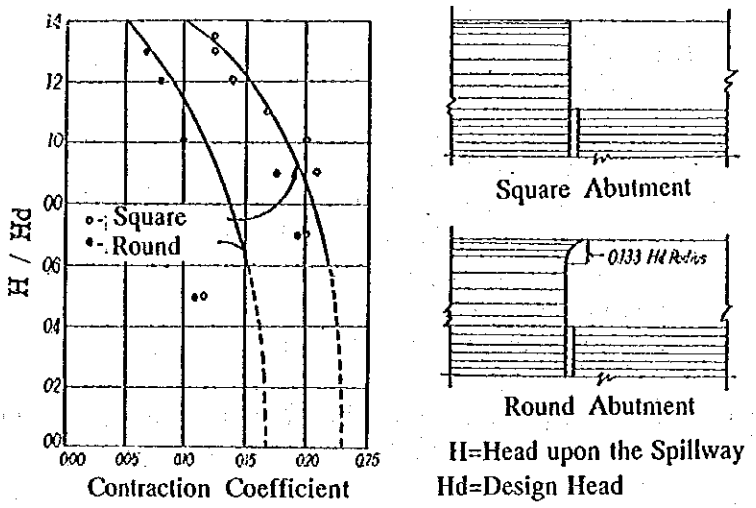
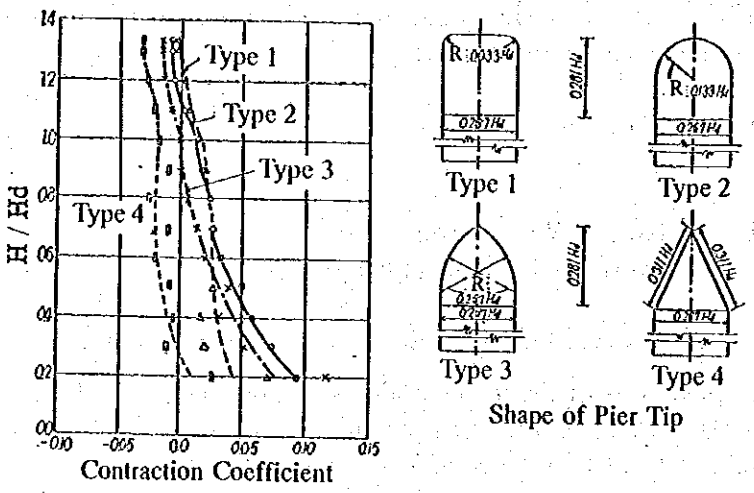


Figure-2.1 Contraction Coefficient of the Abutment



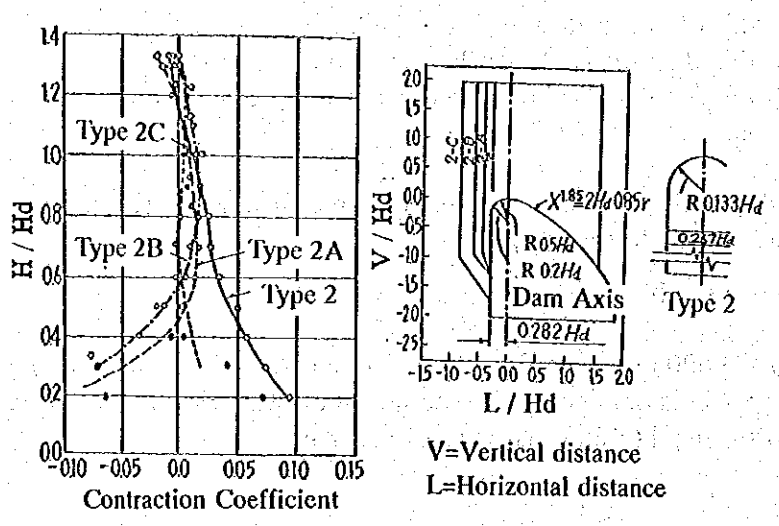


Figure-2.2 Contraction Coefficient of the Piers

2.5 Design River Water Level

Water level must be set at the right below the dam. The water level is decided calculating uniform flow calculation as to narrow pass of river. The uniform flow formula is as follows;

$$Q = (1/n) \times A \times R^{(2/3)} \times i^{0.5}$$

where

n	:	coefficient of roughness(natural channel = 0.1)
A	:	flow area
R	:	hydraulic mean depth
i	:	bed slope

Section to be calculated is shown in Figure-2.3, relationship between discharge and water level were obtained under the assumption that the shape of river floor is approximated with trapezoid. Figure-2.4 shows H-Q curve of the downstream. Water level of the downstream corresponding to each design discharge is shown below;

Table-2.2 River Water Level

Probable Year	Discharge (m^3/s)	River Water Level (m)	Remark
2	200	24.2	Design Discharge of Concrete Dam Diversion Channel
20	720	28.3	Design Discharge of Fill Dam Diversion Channel
100	1,200	31.2	Design Discharge of Energy Dissipater
10,000	3,600	38.2	Design Flood Discharge

2.6 Design Load

The load to the dam body is designed for stability analysis as shown below based on the CEMIG design criteria. The road to the dam body is listed in Figure-2.5.

(1) Own Weight

Item	Unit Weight
Mass Concrete	: 2.3 t/m^3
Water	: 1.0 t/m^3

(2) Hydrostatic Pressure

Static pressure acts at right angle against both side of the dam body. The static pressure is calculated by depth.

$$Pw = \gamma w \times H$$

where

Pw	:	hydrostatic pressure at the interest point (t/m^2)
γw	:	unit weight of water ($1.0 t/m^3$)
H	:	piezometric height at the interest point (m)

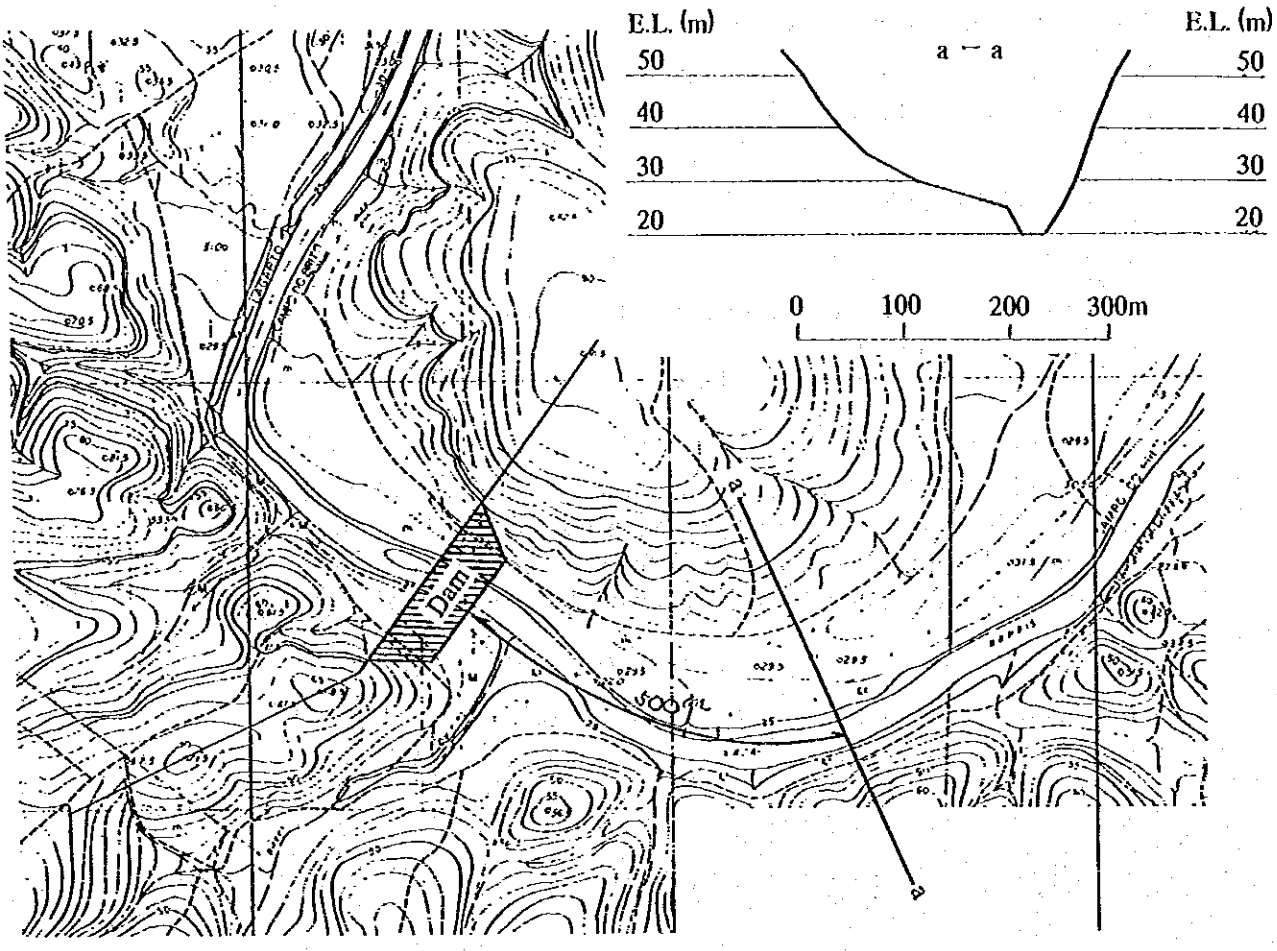


Figure-2.3 Section for Calculation

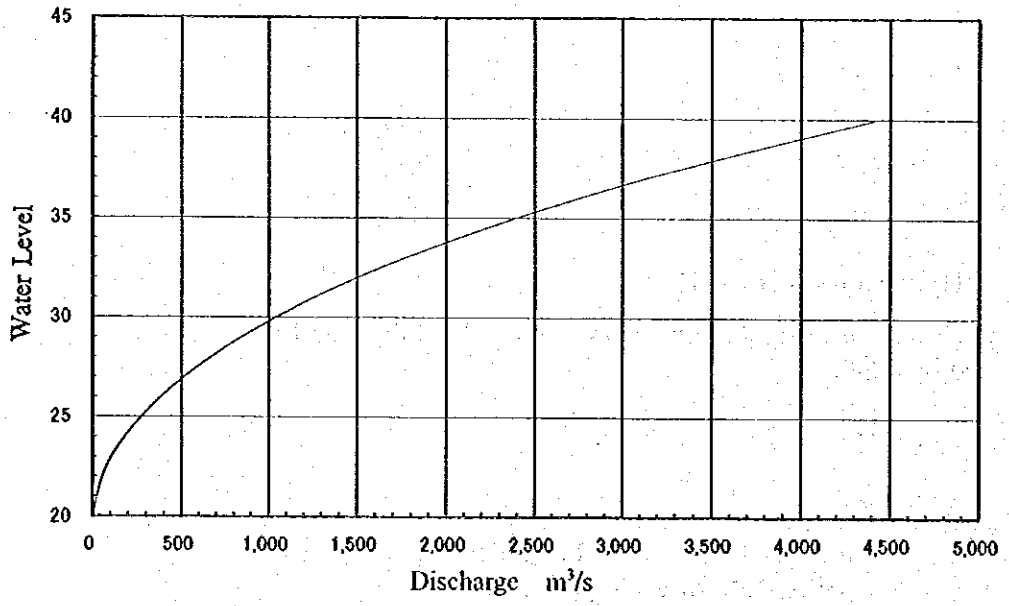


Figure-2.4 H-Q Curve at Station a-a

(3) Pressures due to Sedimentation

Pressure due to sedimentation is supposed as acting on the dam body from the up-stream side. Unit weight of the sediments under the water is designed applying the special value of 0.35 t/m^3.

$$Ps = \gamma s \times H$$

where

Ps	:	pressures due to sedimentation at the interest point (t/m^2)
γs	:	unit weight of sedimentation ($0.35 t/m^3$)
H	:	depth from design sedimentation level (m)

(4) Seismic Inertia Force

Seismic Inertia Force is designed as follows;

- Horizontal Fh = 0.05P
- Vertical Fv = 0.03P

where P is dead load.

(5) Dynamic Water Pressure during Earthquake

Dynamic Water Pressure during Earthquake is calculated using the formula below;

$$Pd = C \times \gamma w \times \lambda \times H$$

where

Pd	:	dynamic water pressure during earthquake
C	:	$(Cm/2)\,(H'(2-H') + (H'(2-H'))^{0.5})$
H'	:	Z/H
Cm	:	C in case of maximum Pd (see Figure-2.6)
γw	:	unit weight of water ($1.0 t/m^3$)
λ	:	(maximum horizontal speed of the foundation) / (gravity speed) = 0.05
H	:	maximum depth of the reservoir
Z	:	vertical distance from the surface of the reservoir until the section being study

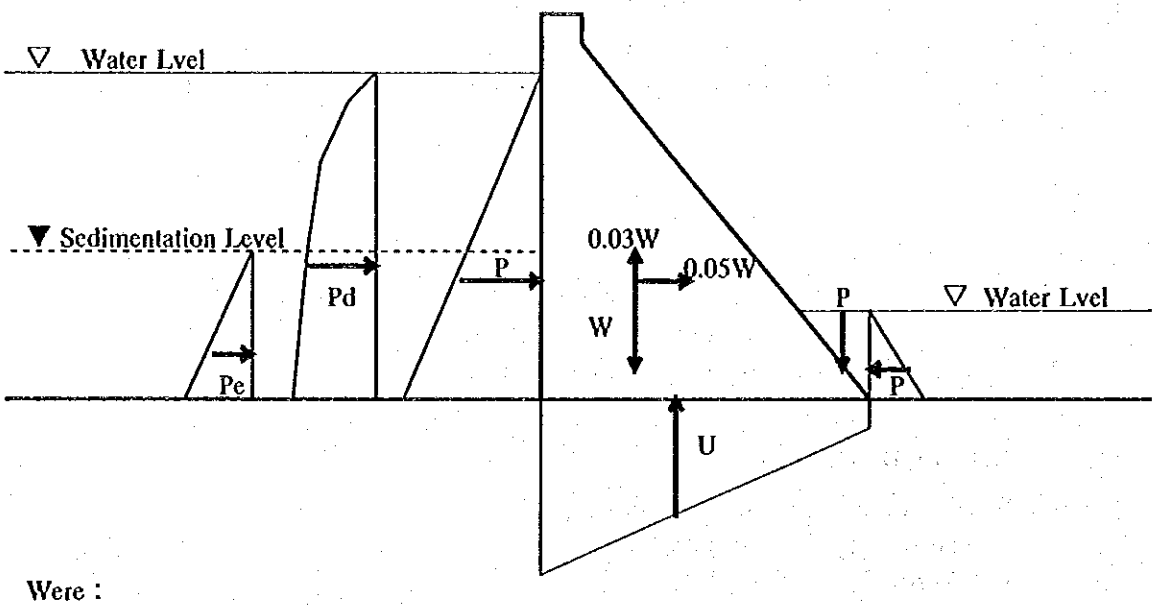
The resultant value and the moment in depth "Z" can be calculated by the expressions indicated below;

$$Ht = 0.726Pd \times Z$$
$$M = 0.299Pd \times Z^2$$

(6) Uplift

Uplift is assumed to act vertically upward on the dam base. The uplift distribution is assumed for the following three (3) types as shown in Figure-2.7.

- Drain is effective and the water level in the down stream is higher than the elevation of drained water.
- Drain is effective and the water level in the downstream is lower than the elevation of drained water.
- Drain is not effective.



Were :

W = Own Weight of Dam Body

P = Hydrostatic Pressure Force

Pe= Forces of the Pressures due to Sedimentasion

Pd= Dynamic Water Pressure during Earthquake

U = Uplift

Figure-2.5 Load Acting on Dam

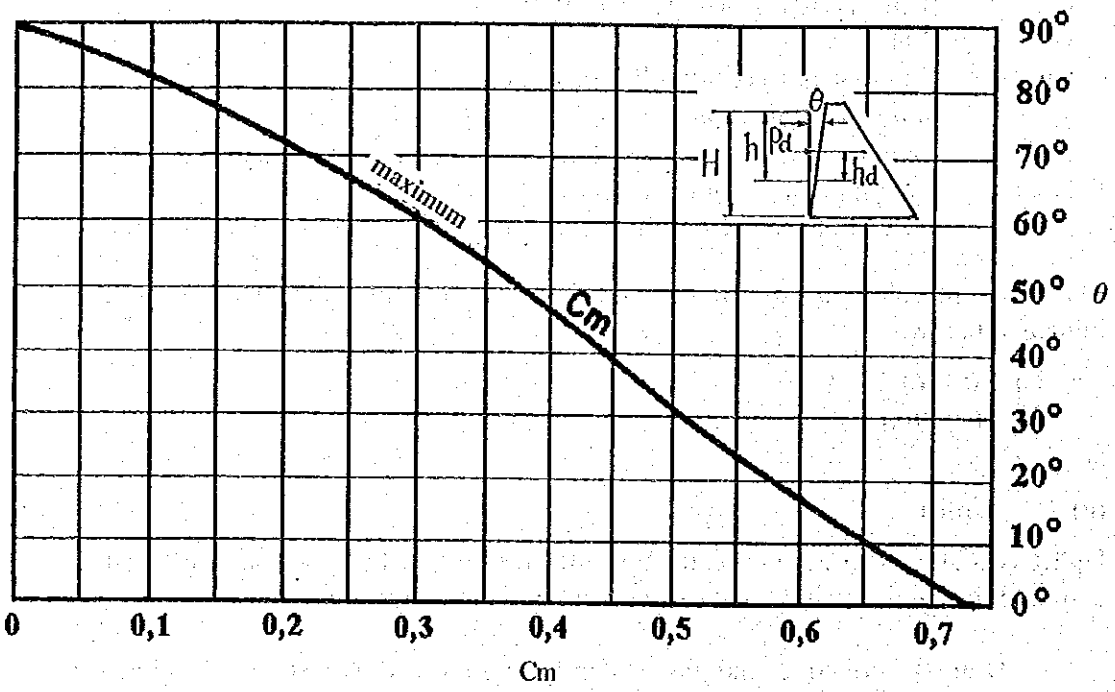
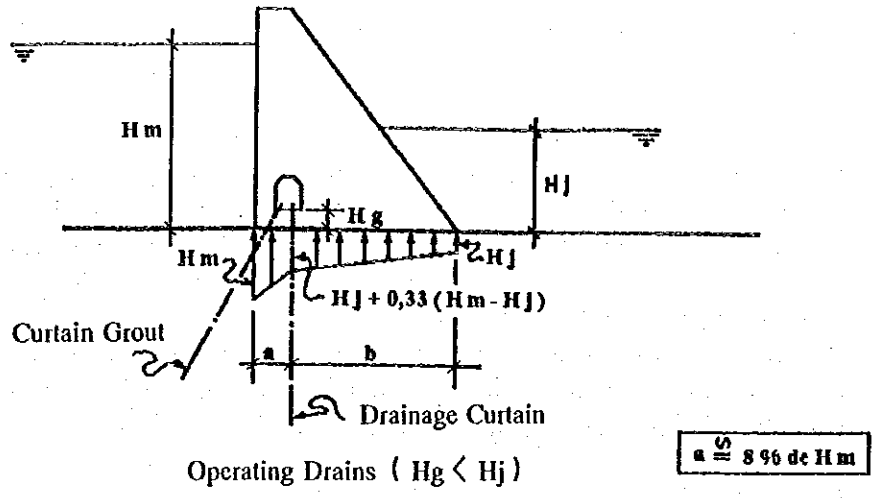
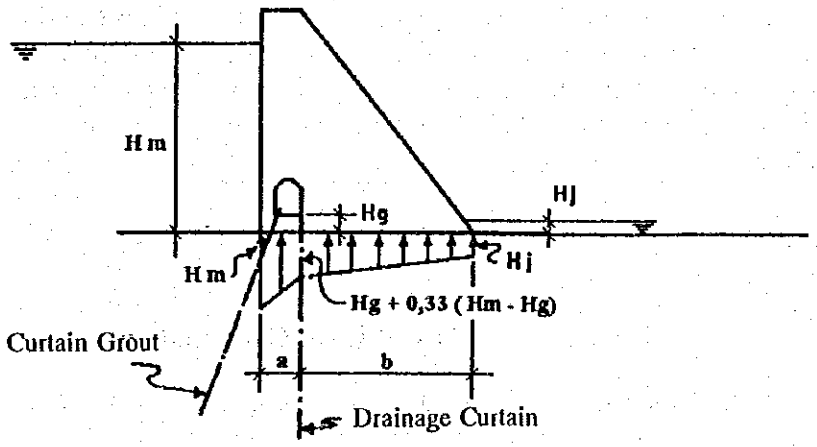


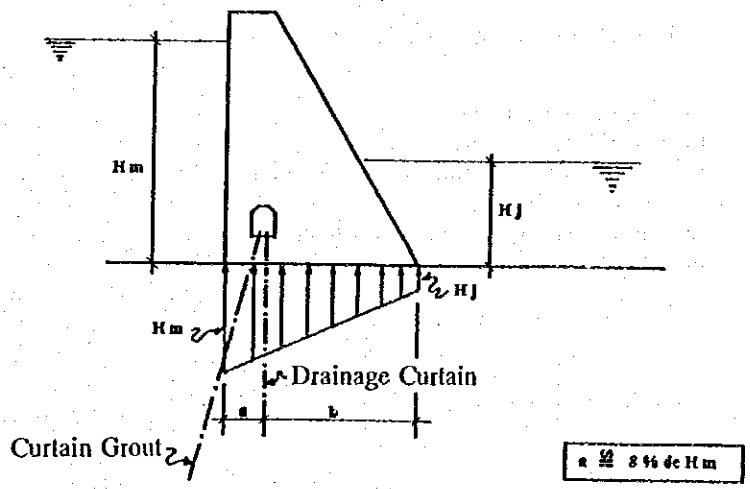
Figure-2.6 Pressure Coefficient Cm



Operating Drains (Hg < Hj)



Operating Drains (Hg > Hj)



Nonoperating Drains (Hg < Hj)

Figure-2.7 Uplift Criteria