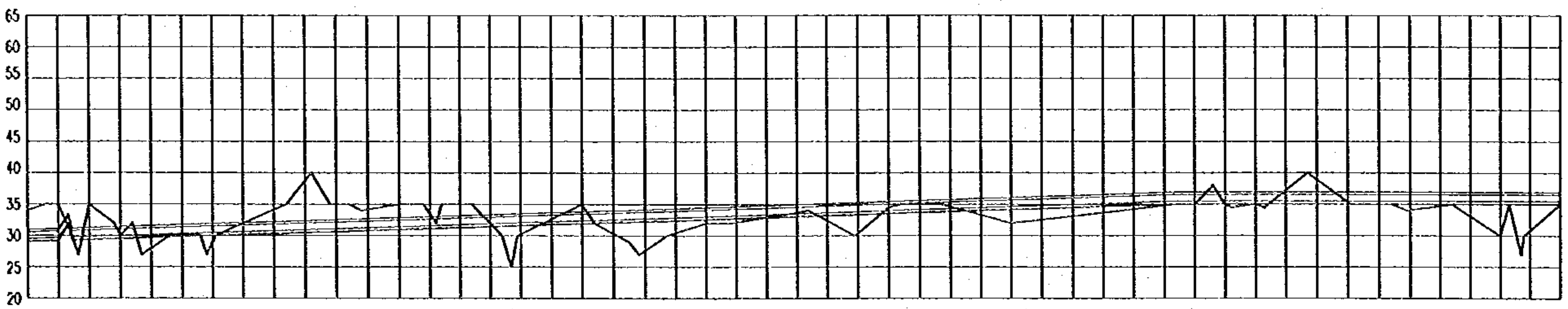
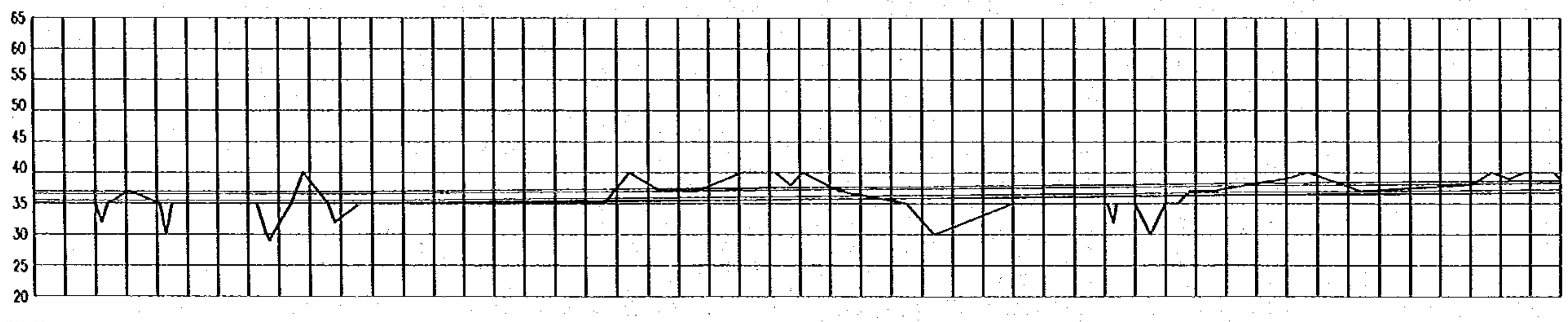
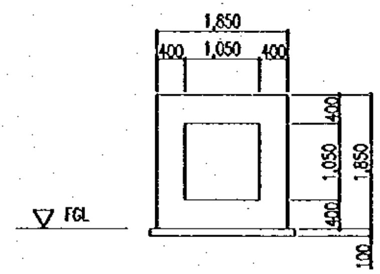



| STATION NO. | 0 | 187 | 250 |
|---|---|---|---|
| TOTAL DISTANCE (M) | 0 | 3740 | |
| PARTIAL DISTANCE (M) | 0 | 3740 | |
| GROUND ELEVATION (M) | 34.0 | 35.0 | |
| INVERT ELEVAION (M) | 29.0 | 35.4 | |

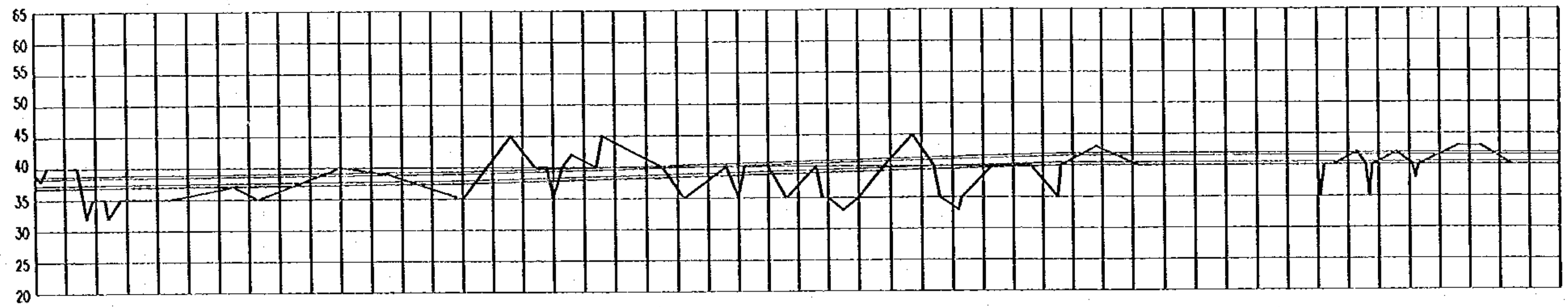



| STATION NO. | 250 | 303 | 500 |
|---|---|---|---|
| TOTAL DISTANCE (M) | | 6060 | |
| PARTIAL DISTANCE (M) | | 2320 | |
| GROUND ELEVATION (M) | | 35.0 | |
| INVERT ELEVAION (M) | | 35.4 | |

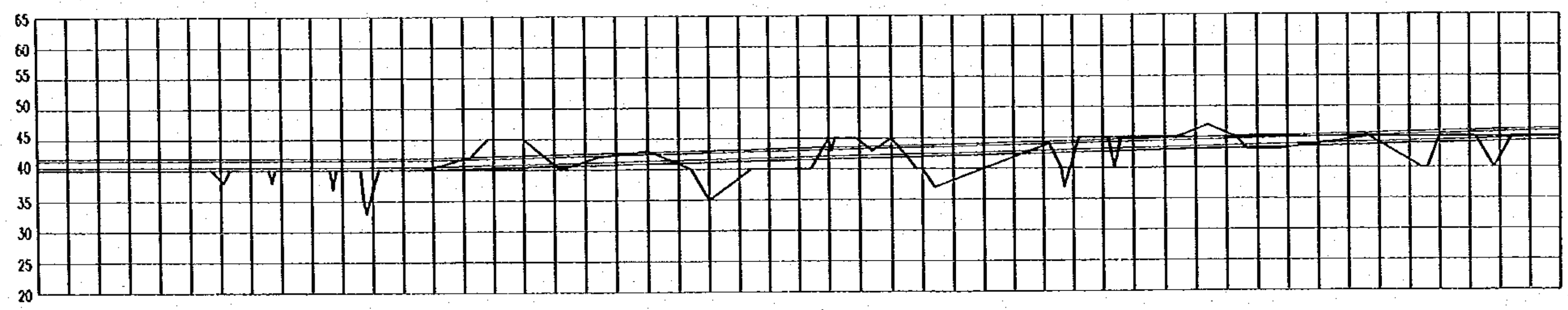



CLOSS SECTION S=1/50

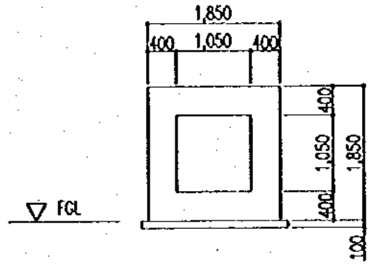
**Figure-5.2 (1/3)  Profile of Bypass Box Culvert**




| STATION NO. | 500 | 570 | 668 | 750 |
|---|---|---|---|---|
| TOTAL DISTANCE (M) | | 11400 | 13360 | |
| PARTIAL DISTANCE (M) | | 5340 | 1960 | |
| GROUND ELEVATION (M) | | 35.0 | 40.0 | |
| INVERT ELEVAION (M) | | 37.8 | 40.4 | |

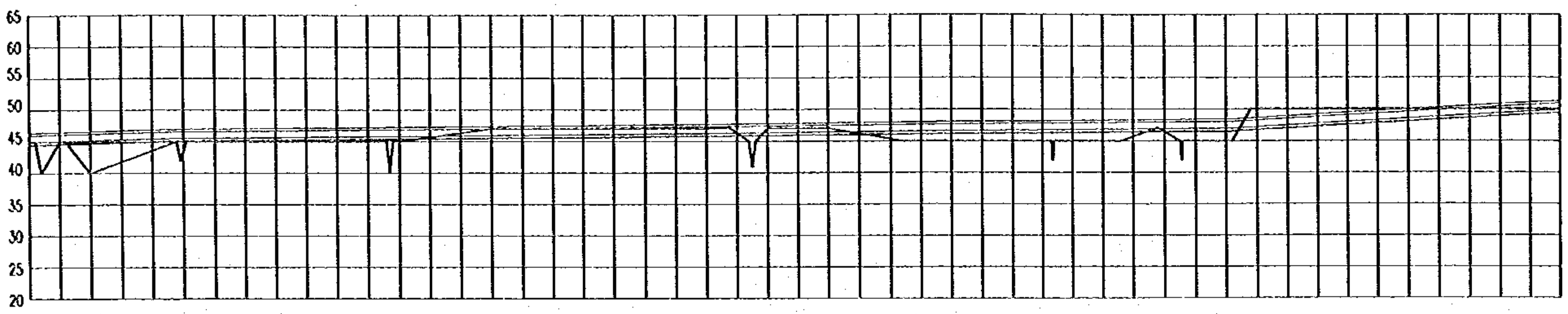



| STATION NO. | 750 | 813+10 | 1000 |
|---|---|---|---|
| TOTAL DISTANCE (M) | | 16270 | |
| PARTIAL DISTANCE (M) | | 2910 | |
| GROUND ELEVATION (M) | | 40.0 | |
| INVERT ELEVAION (M) | | 40.4 | |

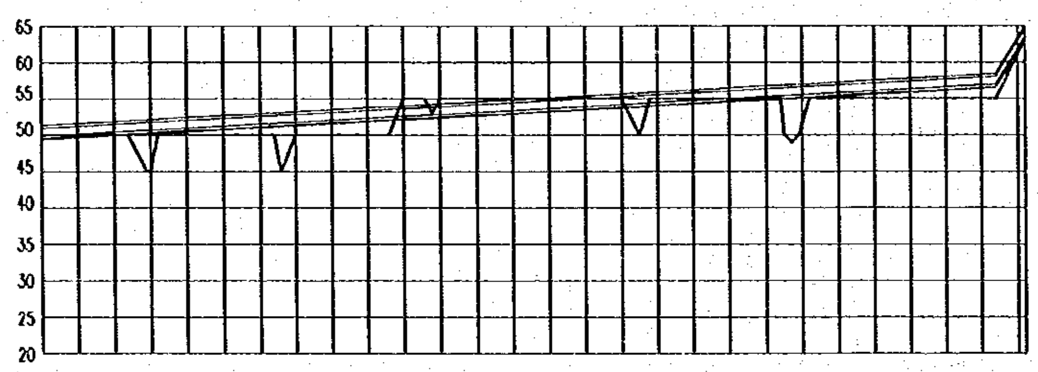



CLOSS SECTION S=1/50

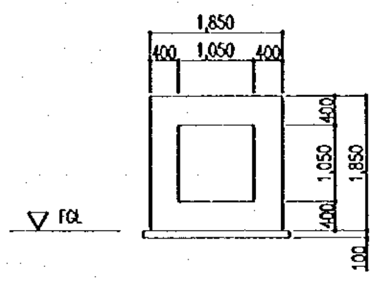
**Figure-5.2 (2/3) Profile of Bypass Box Culvert**




| STATION NO. | 1000 | 1024 | 1196 | 1250 |
|---|---|---|---|---|
| TOTAL DISTANCE (M) | | 20480 | 23920 | |
| PARTIAL DISTANCE (M) | | 4210 | 3440 | |
| GROUND ELEVATION (M) | | 45.0 | 45.0 | |
| INVERT ELEVAION (M) | | 45.4 | 46.9 | |




| STATION NO. | 1250 | 1382 | 1386 |
|---|---|---|---|
| TOTAL DISTANCE (M) | | 27640 | 27720 |
| PARTIAL DISTANCE (M) | | 3720 | 80 |
| GROUND ELEVATION (M) | | 55.0 | 63.0 |
| INVERT ELEVAION (M) | | 56.9 | 63.0 |




CLOSS SECTION S=1/50

**Figure-5.2 (3/3) Profile of Bypass Box Culvert**

# CHAPTER 6 CONSTRUCTION PLAN

## 6.1 Outline of Diversion Works

There are, in general, three methods of temporary diversion works, namely 1) Diversion flowing in half of a river section, 2) Tunnel diversion and 3) Open channel diversion. The first method, diversion flowing half of river section was adopted in this plan, taking into account of 200 $m^3/s$ of design discharge (2-year return period) and topographic feature of the dam site. In following view points, type of diversion works is designed as half cofferdam + open channel.

1) Dimension of the diversion tunnel is 8.4m in diameter and 600m in total length adopting standard horseshoe shaped section. It is difficult to apply the diversion tunnel above in view points of construction cost, construction period and safe construction work.
2) Considering the topographical characteristics that the river is winding with 50m width of the river floor while 150m width of the valley, method of half cofferdam + diversion cannel will be adopted. Dimension of the diversion tunnel is 1:1.0 in slope gradient and 15m in width of the cannel base. Considering the method of concrete placing for the dam body, the diversion channel should be constructed by excavating the river terrace deposits.

Procedure of the diversion works is shown below. Outline of the procedure is shown in Figure-6.1 and Figure-6.2. Color of the figures are; red - excavation, yellow - banking of cofferdam, green - concrete placing of dam body, light blue - river flow.

**Step-1:** Considering the hauling route for concrete aggregate and the necessary area for placing construction facilities, concrete batching plant should be placed in the left bank, and dam body construction also should be commenced from the left bank side. Consequently the diversion channel should be constructed in the right bank side as right as possible. A tributary course joining the main river at the just below the dam body from the right bank side should be shifted to the down-stream area for the effective construction works. Mucks from the excavation of the diversion channel should be used for the cofferdam construction along the upstream and the down stream of the dam body.

**Step-2:** After the river flow is transferred into the diversion cannel along the right bank side, the excavation on the right bank side and concrete placing of the dam body should be commenced.

**Step-3:** Diversion channel will be constructed inside the dam body at the elevation of 20m. The channel will be constructed to shift the river course to the left bank side.

**Step-4:** After the construction work of the diversion channel inside the dam body in the left bank side, the river flow will be shifted to the left bank side, then the dam construction will be commenced in the right bank side. The diversion channel along the right bank side will be closed at its tip and end. As the shifted river flows through the diversion channel inside the dam body, on the apron and through the channel in the downstream, an temporary training wall is needed along the energy dissipater. The abutment of the right bank side can be excavated in advance as long as it does not obstruct the diversion channel.

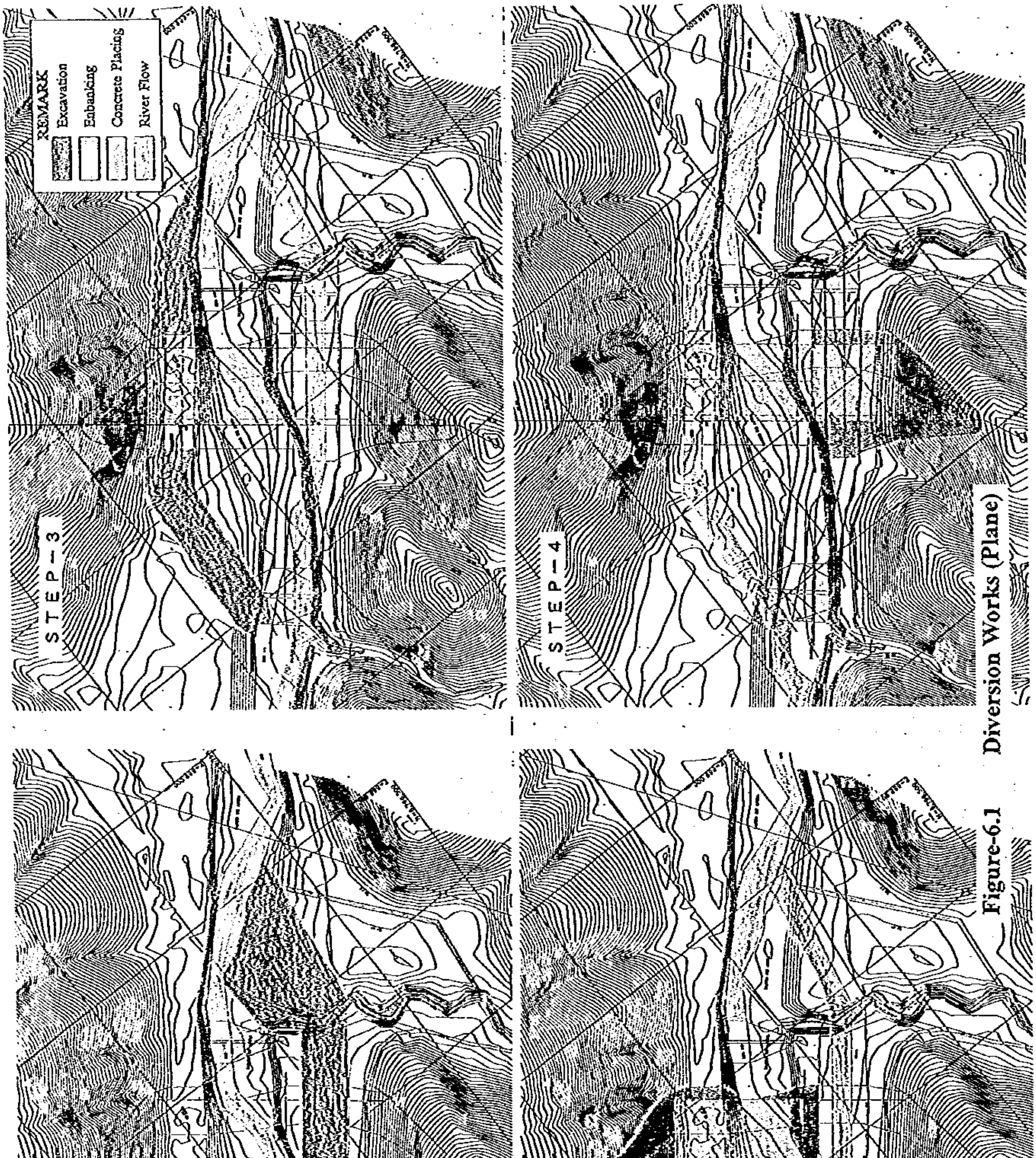



Figure-6.1 Diversion Works (Plane)

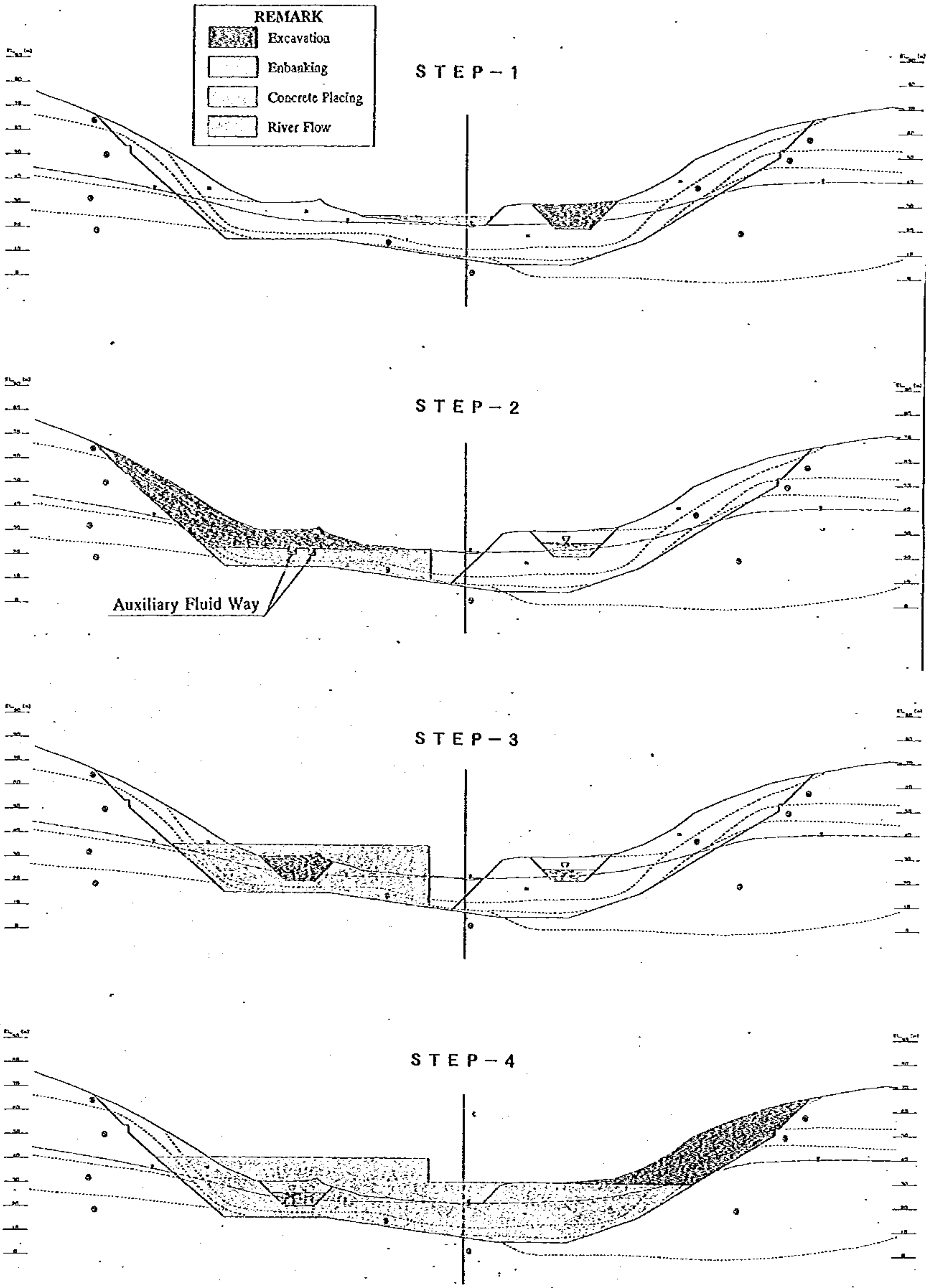



**Figure-6.2 Diversion Works (Longitudinal Section of Dam Axis, Upstream)**

## 6.2 Design of Diversion Works

### (1) Design Condition

Design discharge = $200m^3/s$ (2-year return period)
Channel bed slope = 1/1000
Channel bed elevation = EL. 20m
Coefficient of roughness = 0.022 (open channel on soil, strait, uniform section)
Slope gradient = 1:1.0

### (2) Design of Open Channel

Necessary water depth is calculated by uniform flow calculation assuming that width of the channel is 15m referring to the actual river width.

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

where

| | | |
|---|---|---|
| Q | : | design discharge ($m^3/s$) |
| A | : | flow area ($m^2$) |
| V | : | velocity (m/s) |
| n | : | coefficient of roughness |
| R | : | hydraulic mean depth (m) |
| I | : | channel bed slope |

If water depth h = 3.8m, then A = $71.44m^2$, R = 2.77m
$Q = (1/0.022) \times 71.44 \times 2.77^{(2/3)} \times 0.001^{0.5} = 202.9m^3/s > 200m^3/s$ OK

From the result of the calculation, water level is EL.20m+3.8m = EL.23.8m in case of design discharge of $200m^3/s$. However, water level becomes EL.24.2m inside the channel due to the water level of the river which is calculated as EL.24.2m. Elevation at the top of the slope of the channel is EL.25.2m which was obtained by adding 1m of freeboard as well as the case of the cofferdam.

### (3) Design of Diversion Channels in the Dam

During concrete placing of the dam body, the river flow will be shifted to the left bank side, then the river will flow through the diversion channel inside the dam body. The section of the diversion channel will be too large if it is designed as open channel, hence it should be designed as a pipeline. The discharge of the pipeline is given by formula below;

$$Q = A \times V$$
$$V = ((2gh) / (o + e + fL / 4R))^{0.5}$$
$$f = 8gn^2 / R^{(1/3)}$$

where

| | | |
|---|---|---|
| Q | : | design discharge ($m^3/s$) |
| A | : | flow area ($m^2$) |
| V | : | velocity (m/s) |
| g | : | gravity acceleration ($9.8m^2/s$) |
| h | : | head (m) |
| o | : | outlet loss |
| e | : | entrance loss |
| f | : | coefficient of friction loss |
| L | : | length of auxiliary fluid way (m) |
| n | : | coefficient of roughness (concrete = 0.015) |
| R | : | hydraulic mean depth (m) |

Assuming that water level of the downstream is EL.24.2m and the total length the diversion channel inside the dam body is 29m with EL.20m, the relationship between the dimension of the channel section and the upstream water level is as follows;

**Table-6.1 Relationship between Dimension of Channel Section and Upstream Water Level**

| Upstream water level | EL.(m) | 33.3 | 29 | 27 | 25.9 |
|---|---|---|---|---|---|
| Size of channel | W × H (m) | 3 × 3 | 3.5 × 3.5 | 4 × 4 | 4.5 × 4.5 |
| Velocity | (m/s) | 11.1 | 8.2 | 6.3 | 5 |
| Discharge | ($m^3$/s) | 100.3 | 100.3 | 101 | 100.2 |
| Number of channel | | 2 | 2 | 2 | 2 |

As the diversion channel will be constructed inside the dam body, the maximum size of the channel becomes about 5m (1/3 of the one block of 15m). However, large section of the channel will make plug works difficult, on the other hand small section can not secure the design discharge of 200$m^3$/s unless water level of upstream is made higher or the number of the channels is increased. As the design of the diverse channel gives a great influence to the plan of the construction works, it is considered enough for this study to give only the outline of the diversion channel as shown in Table-6.1.

### (4) Height of Cofferdam

Up to step-3 explained above, the channel along the right bank side will be available and the water level of the channel is EL.24.2m. If freeboard for the cofferdam is set as 1m following the CEMIG criteria, the elevation at the cofferdam crest will become EL.25.2m.

Then in step-4, after the channel is shifted to the left bank side, the water level of upstream of the dam body will rise up to higher than EL. 24.2m. Therefore, the height of the cofferdam must be raised corresponding to the water level rising.
If cofferdam is constructed using soil materials, the cofferdam must be made as high as possible because of poor resistance of the soil materials against overflow.

## 6.3 Construction Plan

Construction Plan of the dam during three years between 2004 and 2006 is shown in Table-6.2.

**Table-6.2 Construction Plan of the Dam**

| Work Item | Construction Amount | 2004 | | 2005 | | 2006 | |
|---|---|---|---|---|---|---|---|
| **Main Dam Works** | | | | | | | |
| 1) Temporary Diversion | 1 set | | | | | | |
| 2) Foundation Excavation | 380,000 $m^3$ | | | | | | |
| 3) Grouting | 4,800 m | | | | | | |
| 4) Concrete Placing | 260,000 $m^3$ | | | | | | |
| 5) Discharge Outlet facilities | 1 set | | | | | | |
| 6) Closing Works | 1 set | | | | | | |
| **Construction Facilities/Plant** | | | | | | | |
| 1) Site Road | km | | | | | | |
| 2) Construction Facilities | 1 set | | | | | | |
| Others | | | | | | | |
| 1) Check Dam | 1 set | | | | | | |
| 2) Low Flow Bypass | 1 set | | | | | | |
| 3) Test of Water Filling | 1 set | | | | | | |

# CHAPTER 7 ESTIMATION OF CONSTRUCTION QUANTITY

## 7.1 Vaza Barris Dam

### 7.1.1 Concrete

#### (1) Dan Body and Dissipater

1) Dam Body

| Section | Distance m | Area $m^2$ | Mean area $m^2$ | Volume $m^3$ | Accumulate $m^3$ |
|---|---|---|---|---|---|
| a | | 10 | | | |
| b | 40 | 654 | 332.0 | 13,280 | 13,280 |
| c | 35 | 654 | 654.0 | 22,890 | 36,170 |
| d | 90 | 1034 | 844.0 | 75,960 | 112,130 |
| e | 40 | 1034 | 1,034.0 | 41,360 | 153,490 |
| f | 75 | 20 | 527.0 | 39,525 | 193,015 |
| | | | | | |
| Total | | | | 193,015 | |

2) Footing
Footings of 3m × 3m × 3m are placed on the rock surface in the upstream and the downstream.
$V = 3m^3 \times 280m \times 2 =$ 15,120 $m^3$

3) Apron
$V = 2m \times 150m \times 32m =$ 9,600 $m^3$

4) Training wall (Dam downstream face)
$V = 5.6m^2 \times 45m \times 2 =$ 504 $m^3$

5) Training wall (Dissipater)
$V = 27m^2 \times 32m \times 2 =$ 1,728 $m^3$

6) Volume decrease of overflow section
$V = -26m^2 \times 150m =$ -3,900 $m^3$

7) Summitry of concrete volume
$V =$ 216,067 $m^3$

8) Displaced foundation concrete
$V = 1331m^2 \times 32m =$ 42,592 $m^3$

### 7.1.2 Excavation

| Section | Distance m | Area $m^2$ | Mean area $m^2$ | Volume $m^3$ | Accumulate $m^3$ |
|---|---|---|---|---|---|
| g | | 0 | | | |
| a | 12 | 154 | 77.0 | 924 | 924 |
| b | 40 | 1,600 | 877.0 | 35,080 | 36,004 |
| h | 15 | 1,125 | 1,362.5 | 20,438 | 56,442 |
| c | 20 | 1,125 | 1,125.0 | 22,500 | 78,942 |
| I | 78 | 1,309 | 1,217.0 | 94,926 | 173,868 |
| d | 12 | 2,125 | 1,717.0 | 20,604 | 194,472 |
| e | 40 | 2,125 | 2,125.0 | 85,000 | 279,472 |
| f | 75 | 324 | 1,224.5 | 91,838 | 371,310 |
| j | 12 | 0 | 162.0 | 1,944 | 373,254 |
| Total | | | | 373,254 | |

### 7.1.3 Grouting

#### (1) Consolidation Grouting

| Section | Distance<br>m | Length of dam base<br>m | Mean length<br>m | Area<br>$m^2$ | Accumulate<br>$m^2$ |
|---|---|---|---|---|---|
| a | | 5 | | | |
| b | 40 | 34 | 19.5 | 780 | 780 |
| c | 35 | 34 | 34.0 | 1,190 | 1,970 |
| d | 90 | 43 | 38.5 | 3,465 | 5,435 |
| e | 40 | 43 | 43.0 | 1,720 | 7,155 |
| f | 75 | 5 | 24.0 | 1,800 | 8,955 |
| Total | | | | 8,955 | |

Each borehole in 5m × 5m area. Borehole length is 10m depth.
L = A/(5× 5) ×10 m = 3,582 m

#### (2) Curtain Grouting

Curtain grouting area = 6825$m^2$
Interval of grout hole = 1.5m
L = 6825 / 1.5 = 4450m

### 7.1.4 Form

#### (1) Dam Body

| Section | Distance | Dam height | Form | | |
|---|---|---|---|---|---|
| | | | Joint | Upstream face | Downstream face |
| | m | m | $m^2$ | $m^2$ | $m^2$ |
| J.0 | | 3.0 | | | |
| J.1 | 10 | 11.8 | 72.3 | 74.0 | 98.6 |
| J.2 | 15 | 25.0 | 286.0 | 276.0 | 367.7 |
| J.3 | 15 | 38.2 | 653.1 | 474.0 | 631.4 |
| J.4 | 15 | 38.2 | 653.1 | 573.0 | 763.3 |
| J.5 | 15 | 38.2 | 653.1 | 573.0 | 763.3 |
| J.6 | 15 | 38.2 | 653.1 | 573.0 | 763.3 |
| J.7 | 15 | 40.2 | 722.1 | 588.0 | 783.3 |
| J.8 | 15 | 42.2 | 794.6 | 618.0 | 823.2 |
| J.9 | 15 | 44.2 | 870.6 | 648.0 | 863.2 |
| J.10 | 15 | 46.2 | 950.2 | 678.0 | 903.1 |
| J.11 | 15 | 48.2 | 1,033.2 | 708.0 | 943.1 |
| J.12 | 15 | 48.2 | 1,033.2 | 723.0 | 963.1 |
| J.13 | 15 | 48.2 | 1,033.2 | 723.0 | 963.1 |
| J.14 | 15 | 43.2 | 832.1 | 685.5 | 913.1 |
| J.15 | 15 | 28.2 | 360.9 | 535.5 | 713.3 |
| J.16 | 15 | 29.4 | 391.3 | 432.0 | 575.5 |
| J.17 | 15 | 20.6 | 197.7 | 375.0 | 499.5 |
| J.18 | 15 | 11.8 | 72.3 | 243.0 | 323.7 |
| J.19 | 15 | 3.0 | | 111.0 | 147.9 |
| Total | 280 | | 11,262.1 | 9,611.0 | 12,802.7 |

#### (2) Training Wall

A = 567$m^2$ × 4= 2268 $m^2$

## 7.2 Check Dam

### (1) Concrete

| Object | Cross section ($m^2$) | Width (m) | | | Volume ($m^3$) |
|---|---|---|---|---|---|
| | | Left bank | River bed | Right bank | |
| Main dam body | 236.0 | 20.0 | 80.0 | 20.0 | 23,600.0 |
| Main dam wing | 15.0 | 26.0 | 0.0 | 25.0 | 765.0 |
| Sub dam body | 6.9 | 3.0 | 72.0 | 1.5 | 512.3 |
| Sub dam wing | 6.0 | 10.0 | 0.0 | 7.0 | 102.0 |
| Total | | | | | 24,979.3 |

| Object | Cross section ($m^2$) | | Length (m) | Volume ($m^3$) |
|---|---|---|---|---|
| | Main dam side | Sub dam side | | |
| Left bank wing wall | 20.0 | 7.0 | 30.0 | 405.0 |
| Right bank wing wall | 27.0 | 7.0 | 60.0 | 1,020.0 |
| Total | | | | 1,425.0 |

| Object | Cross section ($m^2$) | Length (m) | Volume ($m^3$) |
|---|---|---|---|
| Spillway | 10.0 | 20.0 | 200.0 |

| Object | Area ($m^2$) | Thickness (m) | Volume ($m^3$) |
|---|---|---|---|
| Apron | 2,500.0 | 0.7 | 1,750.0 |

Summitry of concrete volume 28,354.3 $m^3$

### (2) Excavation

| Object | Area ($m^2$) | Length (m) | Volume ($m^3$) |
|---|---|---|---|
| Main dam | 958.0 | 20.0 | 19,160.0 |
| Sub dam | 536.0 | 3.0 | 1,608.0 |
| Total | | | 20,768.0 |

### (3) Form

| Object | Area ($m^2$) |
|---|---|
| Main dam | 5,029.0 |
| Sub dam | 651.0 |
| Wing wall | 1,397.0 |
| Total | 7,077.0 |

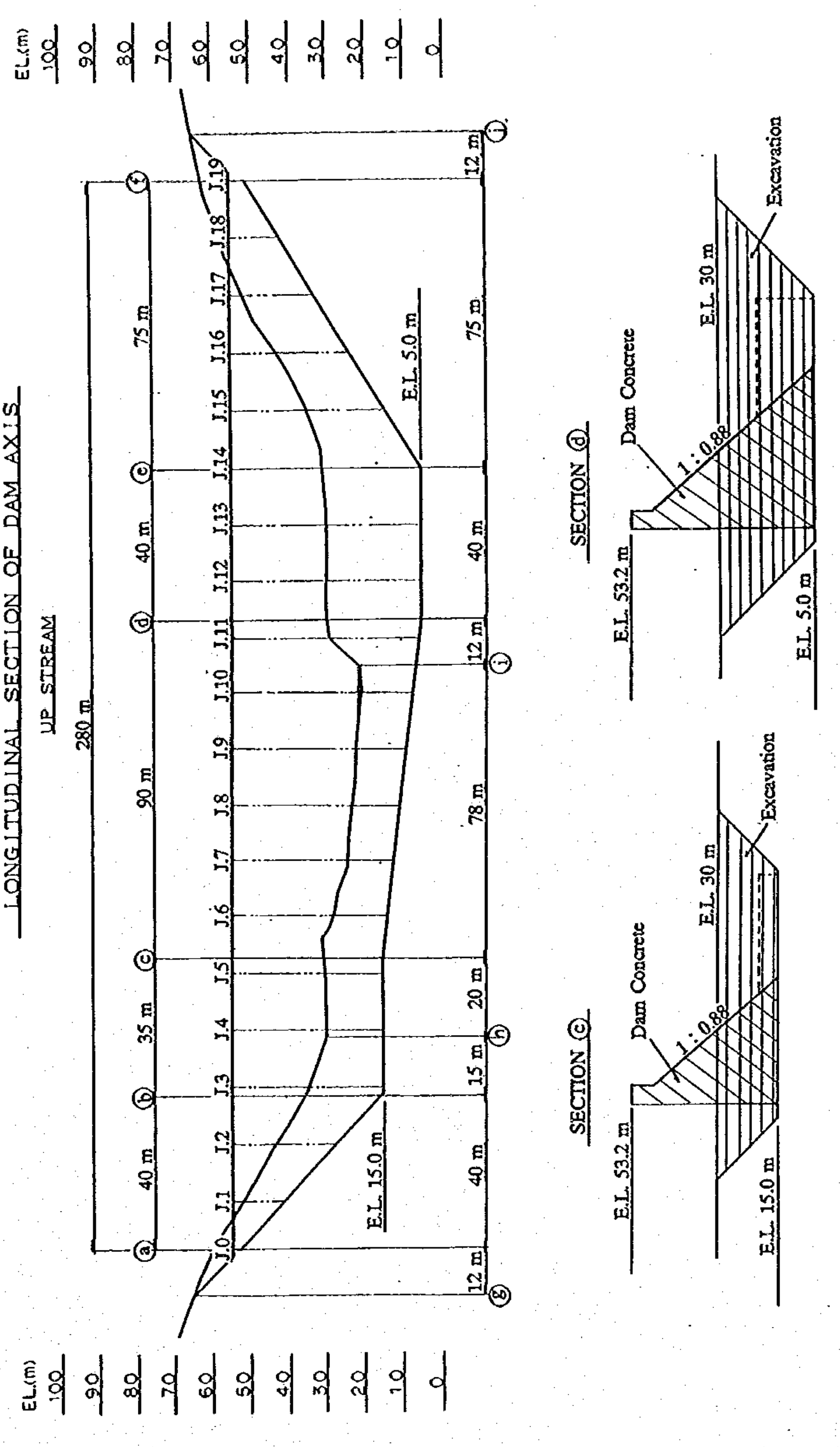



**Figure-7.1 Section for Calculation**

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

## [G] PLAN AND DESIGN OF WATER CONVEYANCE

MARCH 2000

YACHIYO ENGINEERING CO., LTD. (YEC)

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

**SUPPORTING REPORT (G)**
**PLAN AND DESIGN OF WATER CONVEYANCE**

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# CHAPTER 1 INTRODUCTION

This Supporting Report (G) explains the details of Plan and Design of Water Conveyance Facility included in the Feasibility Study on the Priority Projects in the Master Plan.

# CHAPTER 2 CRITERIA FOR PLAN AND DESIGN

## 2.1 Planning Criteria

### 2.1.1 Components Included

The components included in the Water Supply Systems are as follows:

- Vaza Barris Dam
- Water Pump Station, WPS
- Conveyance Pipeline
- Water Treatment Station, WTS
- Distribution pipeline
- Distribution network

General conceptual sketch of the system is as shown in Figure-2.1. The facilities shown by thick line in Figure-2.1 are considered as the water conveyance facility included in the Feasibility Study.

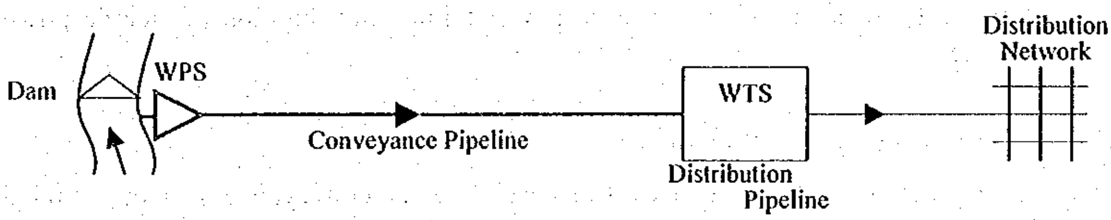



**Figure-2.1 General Conceptual Sketch of Water Supply System**

### 2.1.2 Water Pump Station, WPS

#### (1) Criteria for Selection of Location of WPS

Location of PS is selected considering the following points:

- Vicinity of Vaza Barris Dam
- Direct intake from Vaza Barris Reservoir
- Water to be conveyed by pipeline both in the right and left banks of Vaza Barris River, namely Agreste (Itabaiana) and Piauitinga (Lagarto) regions
- Existing electric power supply facility is located in the left bank near Cajaiba village
- No existing access road to construction site for WPS

### (2) Criteria for Plan and Design of Pump System

Pump system is designed taking the following items into consideration:

- Length of pipeline
- Suction head
- Total head
- Required discharge capacity of pump and adaptability to fluctuation of discharge volume
- Fluctuation of intake water level
- Siphon effect
- Easy countermeasures against water hammer
- Easy operation and maintenance
- High reliability and redundancy of pump system
- Pump head-discharge curve
- Prevention of cavitation in pump
- Specific speed of pump
- Transmission system for motor to the rotating vane and type of reduction gear
- Occupied installed area of pump

### 2.1.3 Connecting Reservoir

Connecting reservoir is required if a system with only one pump is not advantageous. Storage capacity of connecting reservoir is determined by the following items:

- Operational condition of pumps
- Frequency and condition of maintenance works of pumps
- Possibility of occurrence of electric power failure and duration of electric power outage

### 2.1.4 Water Conveyance Pipeline

Diameter of pipeline is determined by life cost of water conveyance system including pump facilities. Concept of annual cost and net present value are introduced for this evaluation.
Pipeline routing is planned based on the following criteria:

- Low life cost
- Easy installation and maintenance works
- Existing pump stations and pipelines
- Location of villages on the way to final destination
- Total length of pipeline as short as possible
- Topographic condition
- Hydraulic gradient in pipeline
- Effect of water hammer
- Thickness of pipes to be designed in accordance with loading conditions
- Auxiliary facility such as valves, drains, etc. where required

## 2.2 Design Criteria

### 2.2.1 Codes and Standards

Facilities to be required in the Feasibility Study are designed in accordance with the Codes and Standards published and authorized by federal, regional, state, municipal and/or any other public organizations or authorities in Brazil.

When applicable Codes and Standards are not specified by the related organizations mentioned above, the Codes, Standards and Regulations in Japan are used with some adjustment in accordance with the local conditions in the State of Sergipe.

The following codes and standards are used for the design of water conveyance facilities:

- Brazilian Norms (NBR), Brazilian Association of Technical Norms (ABNT)
- Japan Water Works Association (JWWA)
- Japan Society of Civil Engineers (JSCE)
- Japanese Industrial Standard (JIS)
- American Water Works Association (AWWA)
- American Standard of Testing Materials (ASTM)

### 2.2.2 Units of Measurements

Units of measurements used in design or specification of materials are in SI/MKS metric system.

### 2.2.3 Method of Design of Facilities

In principle, allowable stress design method is applied for all structural design of the facilities.

### 2.2.4 Materials to be Used

All materials to be used for construction of facilities required in the Feasibility Study are in accordance with Brazilian Technical Standards (ABNT) or equivalent Japanese Industrial Standards (JIS) or other internationally accepted standards.

### 2.2.5 Loads to be Considered

In principle, the following loads are considered in the design of structures in the Water Conveyance Systems:

- Dead Weight
- Live loads including floor loads
- Equipment loads
- Earth pressure
- Groundwater pressure
- Hydrostatic pressure
- Uplift pressure
- Wind loads
- Seismic body force : Not to be considered.
- Hydrodynamic pressure : Not to be considered.

### 2.2.6 Design Parameters and Water Supply Volume

- Projected population at the target year, P (person): Refer to Table-3.1
- Design daily water supply volume per capita :
  - Urban area q = 160 liter/capita/day
  - Large Rural area q = 120 liter/capita/day
- Coefficient of daily variation of consumption : k1 = 1.2
- Coefficient of hourly variation of consumption : k2 = 1.5
- Rate of water loss : $r_L$ = 0.42 in 1,998
  = 0.25 in 2,020
- Daily maximum water supply volume, $Q_{DM}$ (litter/day)

$$Q_{DM} = \frac{P \times q \times k1}{1 - r_L}$$

- Hourly maximum water supply volume, $Q_{HM}$ (litter/day)

$$Q_{HM} = Q_{DM} \times k2 = \frac{P \times q \times k1 \times k2}{1 - r_L}$$

### 2.2.7 Calculation of Head Loss

The head loss is the hydraulic head loss such as pipeline friction loss, shape loss and fluid friction loss.

#### (1) Friction Loss in Pipelines and Open Channels

In principle, Darcy·Weisbach formula can be applied.

$$h_f = f \cdot \frac{L}{D} \cdot \frac{v^2}{2g}$$

where $hf$ : friction loss in straight pipes (m),
$f$ : coefficient of friction loss,
$L$ : length of straight pipes around pumps (m),
$D$ : diameter of each straight pipe (m),
$v$ : velocity in pipes (m/s),
and $g$ : acceleration of gravity (m/s$^2$).

f for pipeline will be calculated by Colebrook-White formula.

$$\frac{1}{\sqrt{f}} = -2 \cdot log_{10}\left(0.27 \cdot \frac{K}{D} + \frac{2.51}{Re\sqrt{f}}\right)$$

where $f$ : coefficient of friction loss,
$D$ : diameter of each straight pipe (m),
and $K$ : equivalent roughness
New cast iron pipe K = 0.25 mm
Used cast iron pipe K = 1.00 mm.

Manning formula is applied to open channel, culvert, siphon, etc.

$$h_f = I \cdot L$$

$$I = \frac{v^2 n^2}{R^{3/4}}$$

where $hf$ : friction loss in open channel, etc. (m),
$I$ : hydraulic gradient,
$L$ : length of open channel, etc. (m),
$v$ : velocity in open channel, etc. (m/s),
$n$ : coefficient of roughness,
and $R$ : hydraulic depth (m).

### (2) Shape Loss and Fluid Friction Loss

These head loss can be expressed in the following formula:

$$h_f = f \cdot \frac{v^2}{2g}$$

where $hf$ : friction loss (m),
$f$ : coefficient of head loss,
$v$ : flow velocity (m/s),
and $g$ : acceleration of gravity (m/s$^2$).

Coefficient of head loss can be given in various technical references.

## 2.2.8 Motor Power of Pumps

Motor power of pumps can be given by calculation using the following formula:

$$L = \frac{\rho \cdot g \cdot Q \cdot H \cdot (1+\alpha)}{\eta_P \cdot \eta_G}$$

where $L$ : Motor power (kW),
$\rho$ : density of liquid (t/m$^3$),
$g$ : acceleration of gravity (m/s$^2$),
$Q$ : discharge capacity of pump (m$^3$/s),
$H$ : total head (m),
$\alpha$ : redundancy rate,
$\eta P$ : pump efficiency,
and $\eta G$ : transmission efficiency.

# CHAPTER 3 WATER CONVEYANCE PLAN

## 3.1 Water Demand

### 3.1.1 Population

Water supply population in the Project areas is in accordance with the Population Projection discussed in the Master Plan Study. Population increase is based on the Strategic scenario proposed by JICA study team.

**Table-3.1 Projected Population of Benefited Area: 1997 – 2020**

Unit: 1,000 persons

| Benefited Area | Census | Projected Population | | | | |
|---|---|---|---|---|---|---|
| | 1996 | 1997 | 2000 | 2010 | 2020 | % (*) |
| Agreste | | | | | | |
| Areia Branca | 14.0 | 14.9 | 22.3 | 47.1 | 77.2 | 7.4 |
| C. do Brito | 14.9 | 15.3 | 17.7 | 25.9 | 36.2 | 3.8 |
| Itabaiana | 72.2 | 73.8 | 84.8 | 123.7 | 173.2 | 3.7 |
| Macambira | 5.4 | 5.5 | 5.9 | 7.3 | 9.1 | 2.2 |
| S. Domingos | 8.3 | 8.5 | 9.1 | 11.2 | 14.1 | 2.2 |
| Sub-Total | 114.8 | 118.0 | 139.8 | 215.2 | 309.8 | 4.2 |
| Piauitinga | | | | | | |
| Poco Verde | 17.6 | 17.6 | 17.6 | 18.5 | 21.0 | 0.7 |
| Simao Dias | 33.7 | 34.0 | 35.1 | 39.6 | 46.4 | 1.3 |
| Lagarto | 75.3 | 76.0 | 82.8 | 107.3 | 139.0 | 2.6 |
| R. do Dantas | 17.8 | 18.0 | 18.5 | 20.6 | 23.3 | 1.1 |
| Sub-Total | 144.4 | 145.6 | 154.0 | 186.0 | 229.7 | 2.0 |
| Grand-Total | 259.2 | 263.6 | 293.8 | 401.2 | 539.5 | 3.1 |

Note: * Population Growth Rate between 1996 and 2020

### 3.1.2 Total Required Water Supply Volume

Total water supply volume required in the Project areas is in accordance with the Population Projection shown in Table-3.1.

**Table-3.2 Municipal/Industrial Water Supply in Piauitinga and Agreste Areas**

| Area Covered By | Agreste Integrated Pipeline System (System 2) | | Piauitinga Integrated Pipeline System (System 1) | | Total Supply | | Total Source* |
|---|---|---|---|---|---|---|---|
| | ($m^3$/day) | ($m^3$/s) | ($m^3$/day) | ($m^3$/s) | ($m^3$/day) | ($m^3$/s) | ($m^3$/s) |
| Required Water Supply Volume in 2020 | 74,286 | 0.860 | 79,664 | 0.922 | 153,950 | 1.782 | 2.138 |
| Present Capacity | 12,810 | 0.148 | 12,130 | 0.141 | 24,940 | 0.289 | 0.346 |
| PROAGUA Project | 22,200 | 0.257 | 30,200 | 0.349 | 52,400 | 0.606 | 0.728 |
| Vaza Barris Dam Project | 39,276 | 0.455 | 37,334 | 0.432 | 76,610 | 0.887 | 1.064 |

Note : Total Source = k1 x Total Supply

## 3.2 Water Conveyance Plan

### 3.2.1 Pipeline Routing

#### (1) Agreste Pipeline

Pipeline is installed, as much as possible, along the existing pathways or tracks and the existing pipelines passing from Ribeira through Cajaiba and Carrilho Villages leading to Itabaiana. Pipeline route is also planned so as to reduce the number of summits (high points) and concave sections (low points) in pipeline. The route and vertical profile of the Agreste pipeline is shown in Figure-3.1 and Figure-3.2, respectively.

#### (2) Piauitinga Pipeline

Pipeline is installed, as much as possible, along the existing pathways or tracks which run near the construction site of Pump Station and lead toward Jenipapo. From Jenipapo to Lagarto, the pipeline is laid along the existing roads. Jenipapo, Brasilia, Acuvelho and Urubutinga villages are located along the pipeline route. Pipeline route is planned so as to reduce the number of summits (high points) and concave sections (low points) in the pipeline. The route and vertical profile of the Piauitinga pipeline is shown in Figure-3.1 and Figure-3.3, respectively.

### 3.2.2 Pump Station, PS

#### (1) Selection of Location of PS

Water Intake pump station is installed facing the Vaza Barris Reservoir near to Vaza Barris Dam in order to pump up water directly from the Reservoir.
Although the Agreste and Piauitinga pipelines run in right and left banks of Vaza Barris River, respectively, the Water Intake Pump Station for both pipelines, WIPS, is located jointly on the right bank of the river due to the following reasons:

- Water Intake Pump Station for Vaza Barris Irrigation Project, WIPSI, is constructed on the right bank to facilitate the water supply to the Irrigation Project Area located on the right bank of Vaza Barris River.
- The pump capacity for WIPSI is much larger than those required for WIPS.
- In the case that the pump station WIPS is constructed separately on the right and left banks of the Vaza Barris reservoir, the cost for construction, operation and maintenance of the pump stations, including the construction of substation for electric power supply for the pumps are much higher than three pump stations constructed at the same place on the right bank.

#### (2) Type of Pumps

Water intake pump is a vertical flow type to cope with the high suction head of 30m and high fluctuation of seasonal change of 19m in water intake level in the reservoir. Number of the water intake pumps is three for normal operation plus one for stand-by considering the assurance of high reliability and redundancy of pump system and high applicability for fluctuation of water supply volume. Uni-type pump system composed of vertical pumps to cover the whole conveyance system is not recommendable when the high total head of the system, which is caused by geographical difference in intake and discharge water levels and high friction head due to long distance pipeline, will be expected. In order to mitigate the water hammer effect in the system and to make the pump system simple, two types in function of pumps are introduced in each System, namely,

- Water intake pumps composed of vertical pumps
- Water lift pumps composed of horizontal pumps

The above two systems are connected with the connecting reservoir as specified in the proceeding section.

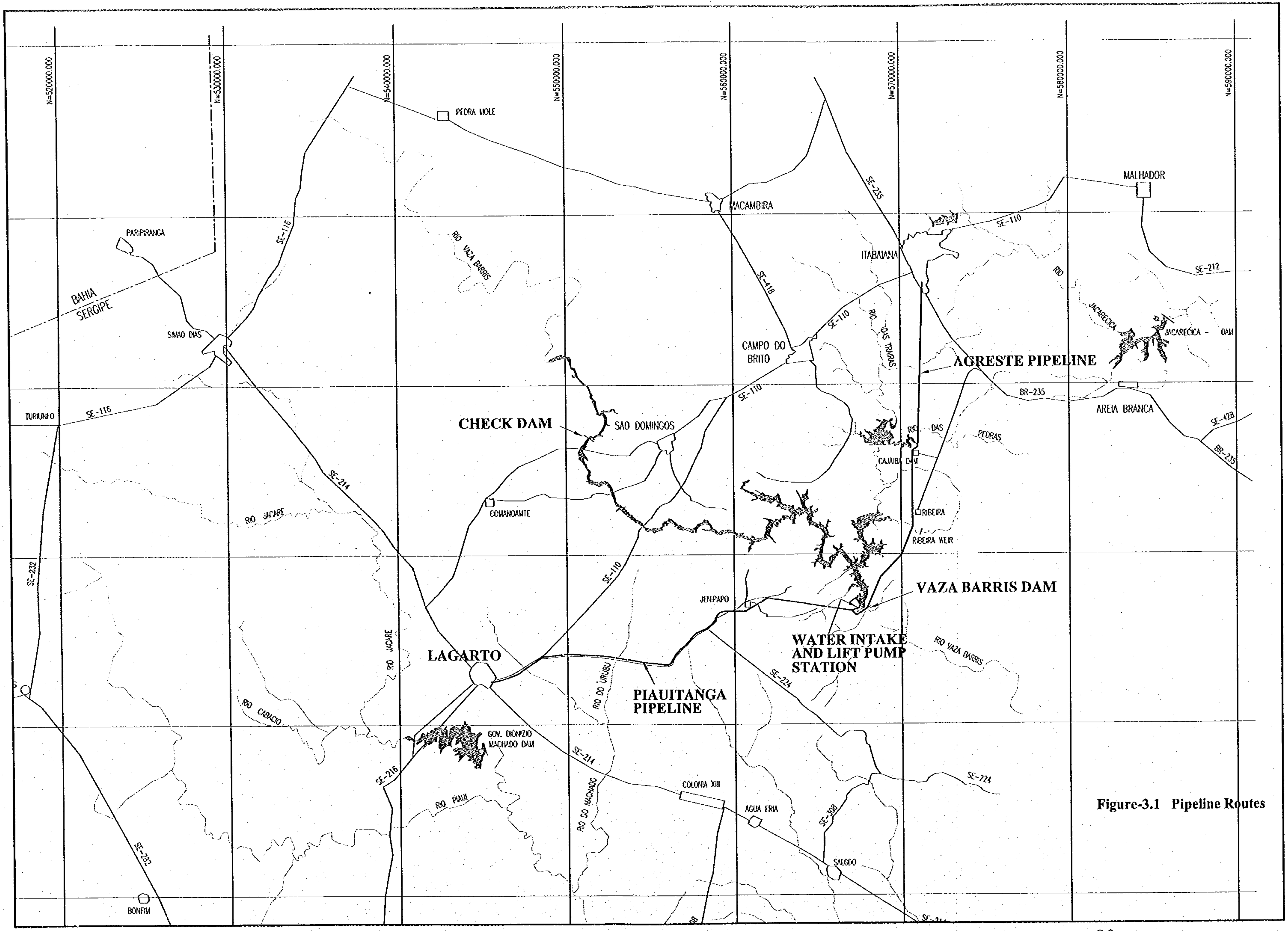



Figure-3.1 Pipeline Routes

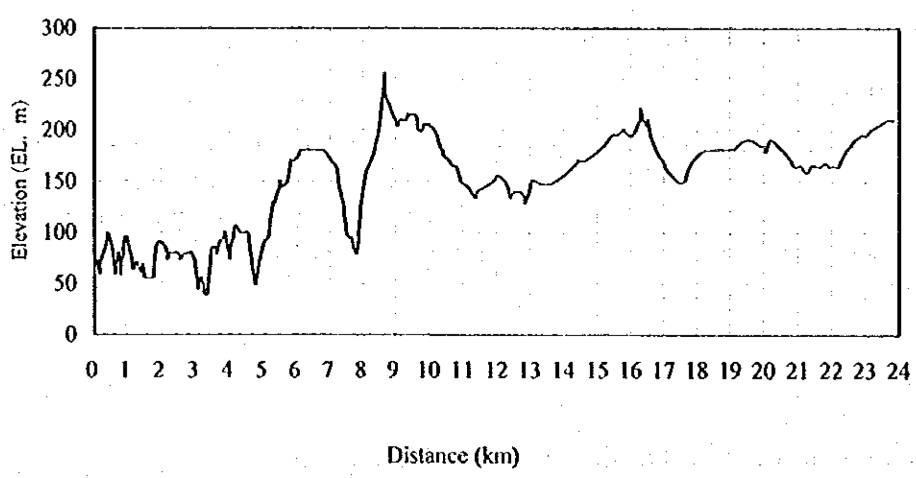



**Figure-3.2 Vertical profile of Agreste Pipeline**

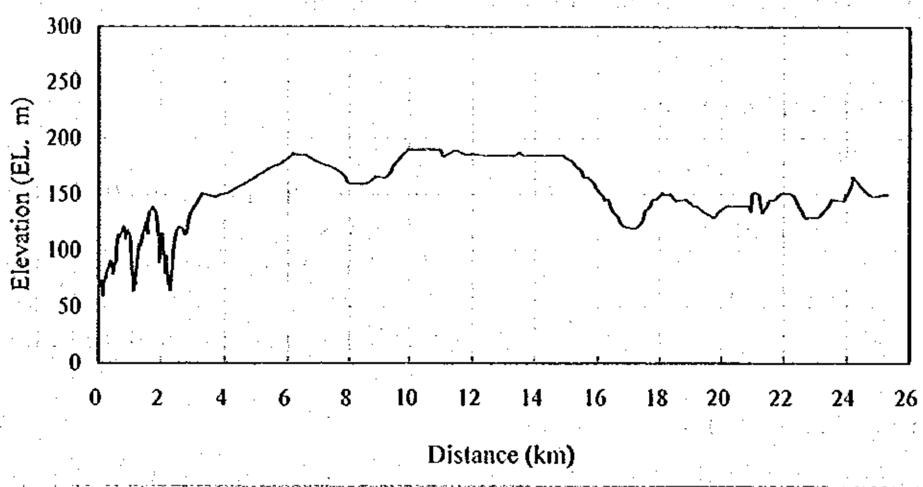



**Figure-3.3 Vertical profile of Piauitinga Pipeline**

### 3.2.3 Connecting Reservoir

Volume of connecting reservoir is capable of storage of water to be discharged at least in one hour and half of operation of pumps. This duration of time is determined based on the possible operational condition of pumps and the possibility of occurrence of electric power outage. The electric power outage is expected to occur on average once in every 25 days, and its averaged duration of 1.5 hour at Cajaiba S/S. For details of electric power supply condition, refer to Section 4.6.

### 3.2.4 Other Required Facilities

The following countermeasures will be provided where required to mitigate the technical problems caused by the long distance and the frequent vertical ups and downs in the pipelines and to make easy maintenance work of pipelines:

- Air release valves
- Level invert tee
- Stop valves
- Surge tanks

# CHAPTER 4 DESIGN OF WATER CONVEYANCE FACILITY

## 4.1 Water Intake Pump Station

### 4.1.1 General

Piauitinga and Agreste Water conveyance systems are used for domestic and industrial water supply. The construction works are implemented in two phases in each route. Water intake pumps are a vertical type composed of 3 pumps for normal operation and 1 pump for stand-by for each phase. These pumps are used commonly for water supply for both regions.

### 4.1.2 Design Conditions

**(1) Water Intake Level in the Vaza Barris Reservoir**

- High Water Level : EL. 52.7 m
- Normal Water Level : EL. 47.5 m
- Low Water Level : EL. 35.2 m
- Sedimentation Surface Level : EL. 28.6 m

**(2) Water Intake Volume**

- Phase 1 stage : 0.533m$^3$/s (0.260 for Piauitinga and 0.273 for Agreste)
- Phase 2 stage : 0.533m$^3$/s (0.260 for Piauitinga and 0.273 for Agreste)
- Total : 1.066m$^3$/s (0.520 for Piauitinga and 0.546 for Agreste)

### 4.1.3 Location of WIPS

WIPS is constructed in the Vaza Barris Reservoir at the right bank of the river. The location of WIPS is selected as shown in Figure-4.1 taking the following technical aspects into consideration:

- To avoid the interference with dam construction work
- To secure the lowest water intake level of EL. 28.6m
- To secure smooth water intake from the reservoir
- To intake as much as possible the water from Trairas River Basin

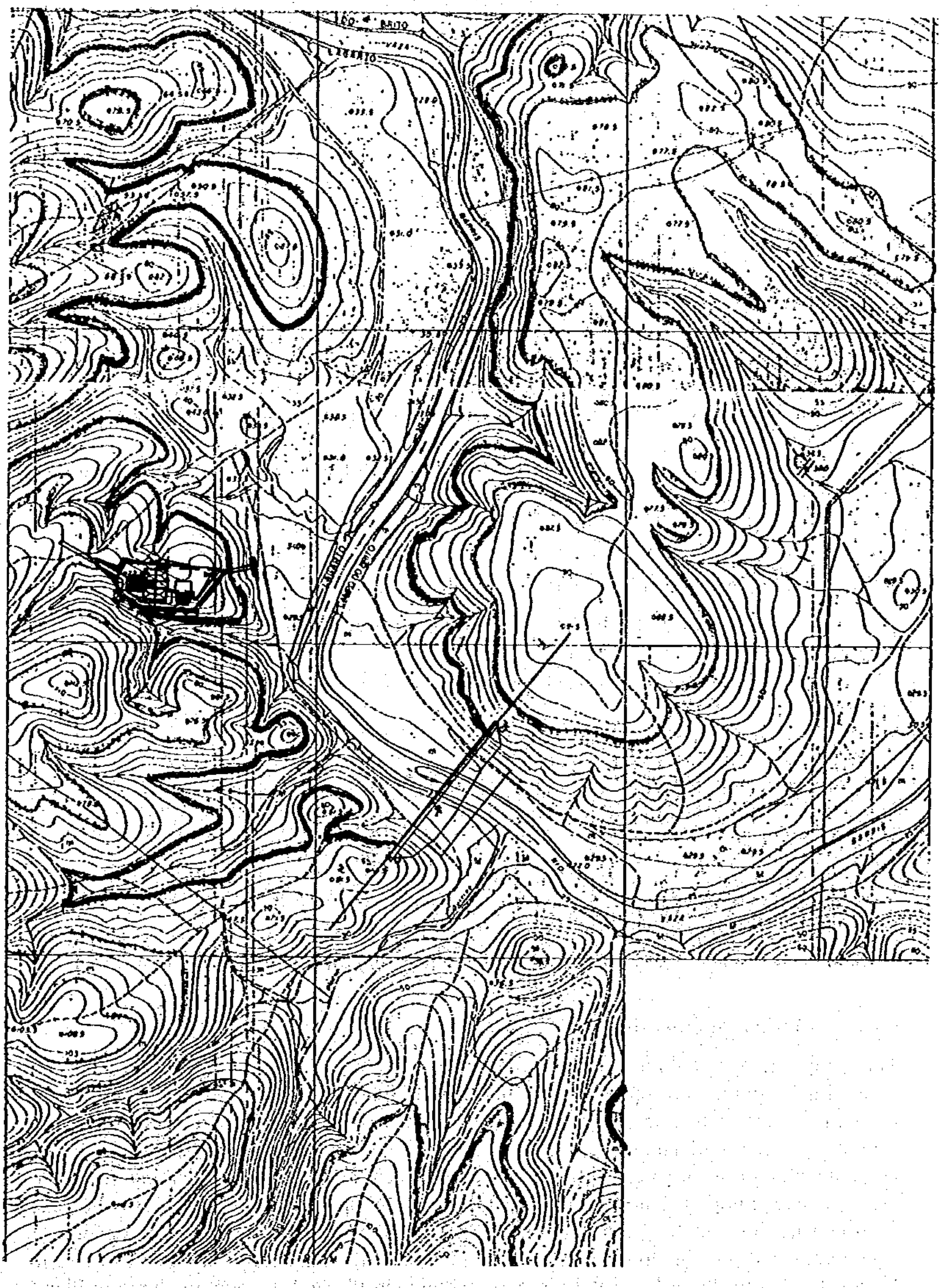


**Figure-4.1 Location of Water Intake Pump Station**

### 4.1.4 Design of Water Intake Pump House

Water Intake Pump House is designed as shown in Figure-4.2 considering the following technical points:

- The house is designed as reinforced concrete structure.
- Intake tower is required to assure direct intake from the reservoir by vertical pumps.
  Bridge to the House is required for transportation of equipment and materials as well as support for pipelines to the Connecting Reservoirs.
- Floor level is almost 5 meters above H.W.L. so as not to be affected by the waves in the reservoir
  Floor level is taken as EL. 57.5m.
- Surface water intake is assured during normal operation
  Stop logs are used for surface water intake. Screens are also installed.
- Installation and maintenance of pumps, fittings, accessories, stop logs, screens, etc. are performed by over-head travelling crane.
- Intake tower and spaces for pumps in Phase 2 are constructed in Phase 1 stage.
  Pumps in Phase 2 are installed in dry work by closing water intake tower by stop logs.
- Intermediate stage or floor is required to support the shafts of vertical pumps in position.
  Intermediate floors at around 10m in height are provided.

### 4.1.5 Technical Description of Water Intake Pumps

**Table-4.1 Technical Description**

| Pump Name | WIP 1 & 2 | |
|---|---|---|
| Use | Domestic and Industrial | |
| Treatment | Raw | |
| Phase | 1 | 2 |
| Quantity | 3 + 1 | 3 + 1 |
| Total head | 41 m | 41 m |
| *Actual head* | *40 m* | *40 m* |
| *Head loss* | *1 m* | *1 m* |
| Max. Suction head | 30 m | 30 m |
| Discharge | 0.533 (0.178) | 0.533 (0.178) |
| Pipeline Dia. | 700 mm | 700 mm |
| Pipeline Length | 80 m | 80 m |

Note 1 : Discharge of pumps to be a total discharge with 3 pumps in operation.
Discharge in ( ) to be per one pump
Unit of discharge : $m^3/s$

Note 2 : The pipeline in Phase 2 is constructed beside the pipeline constructed in Phase 1.

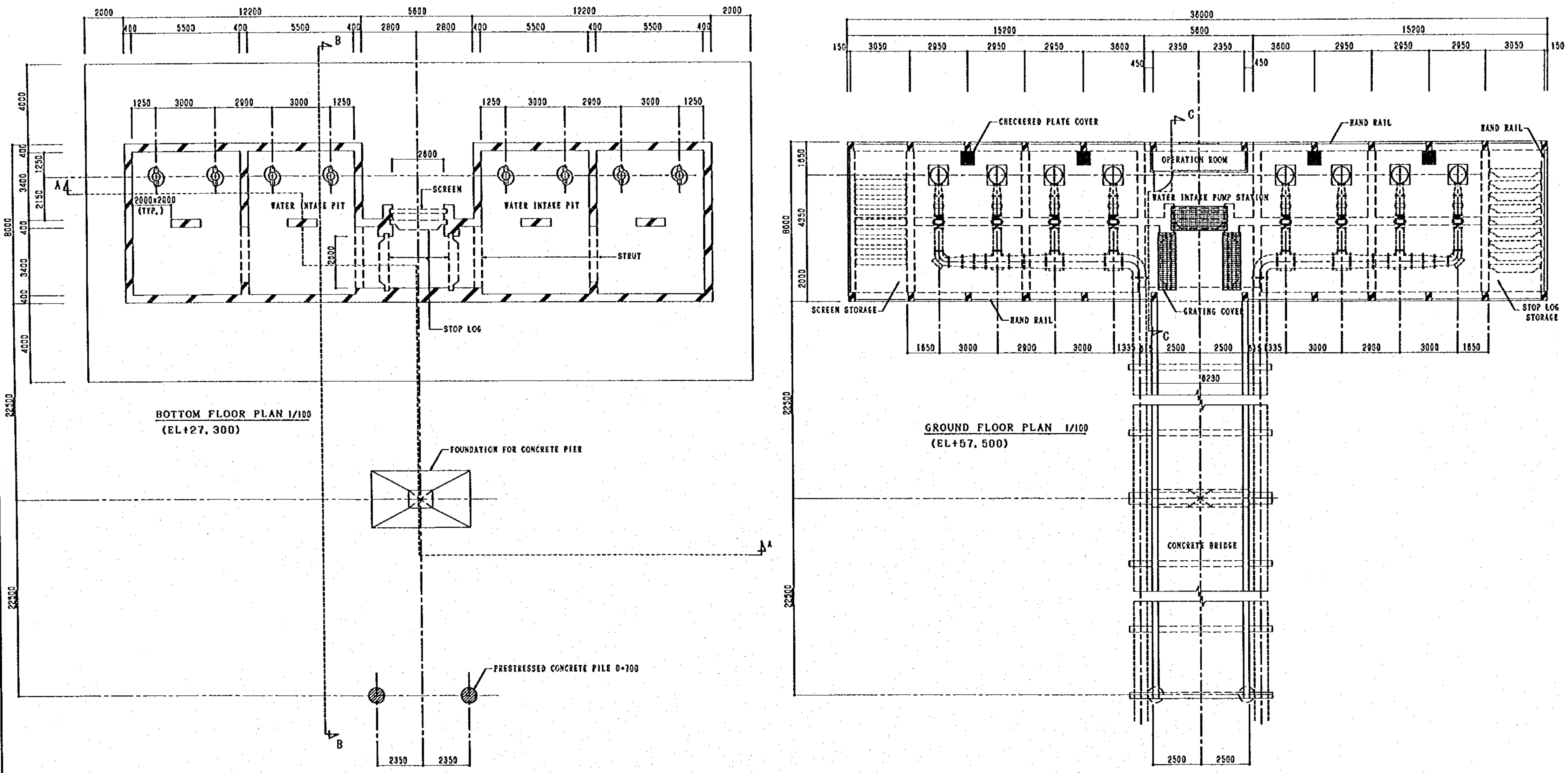
Vertical profile and system curve of the system are shown in Figure-4.3 and Figure-4.4.

### 4.1.6 Method of Operation

The pumps are operated for 24 hours a day.

Water intake pumps are operated automatically in accordance with the discharge water level in the connecting reservoir, which is constructed down stream of water lift pump station. Numbers of pumps in operation at water intake pump station are also selected automatically in accordance with the discharge water level in the connecting reservoir. Water intake pumps are also controlled to stop their operation automatically when the suction water level in the dam reservoir falls down to the Sedimentation Surface Level.

The pumps and pipeline to be constructed in Phase 2 are completely the same as those to be constructed in Phase 1. Total discharge volume in Phase 2 is double volume in Phase 1. The electrical stoppage is expected to occur in every 25 days and its duration is 1.5 hours on average according to the registered record in 1998 and 1999 as shown in 4.6.




| EXTERIOR FINISH SCHEDULE | | INTERIOR FINISH SCHEDULE | |
|---|---|---|---|
| ROOF | FIBERCEMENT ROOFING CANALETE 49 TYPE OR EQUIVALENT | FLOOR | CONCRETE STEEL TROWEL WITH SILICON PAINT |
| WALL | PLASTER WITH LATEX PAINT | WALL | PLASTER WITH LATEX PAINT |
| | | COLUMN | SILICON PAINT |
| | | BEAM | DITTO |
| | | | |

**Figure-4.2(1/3) Water Intake Pump House**

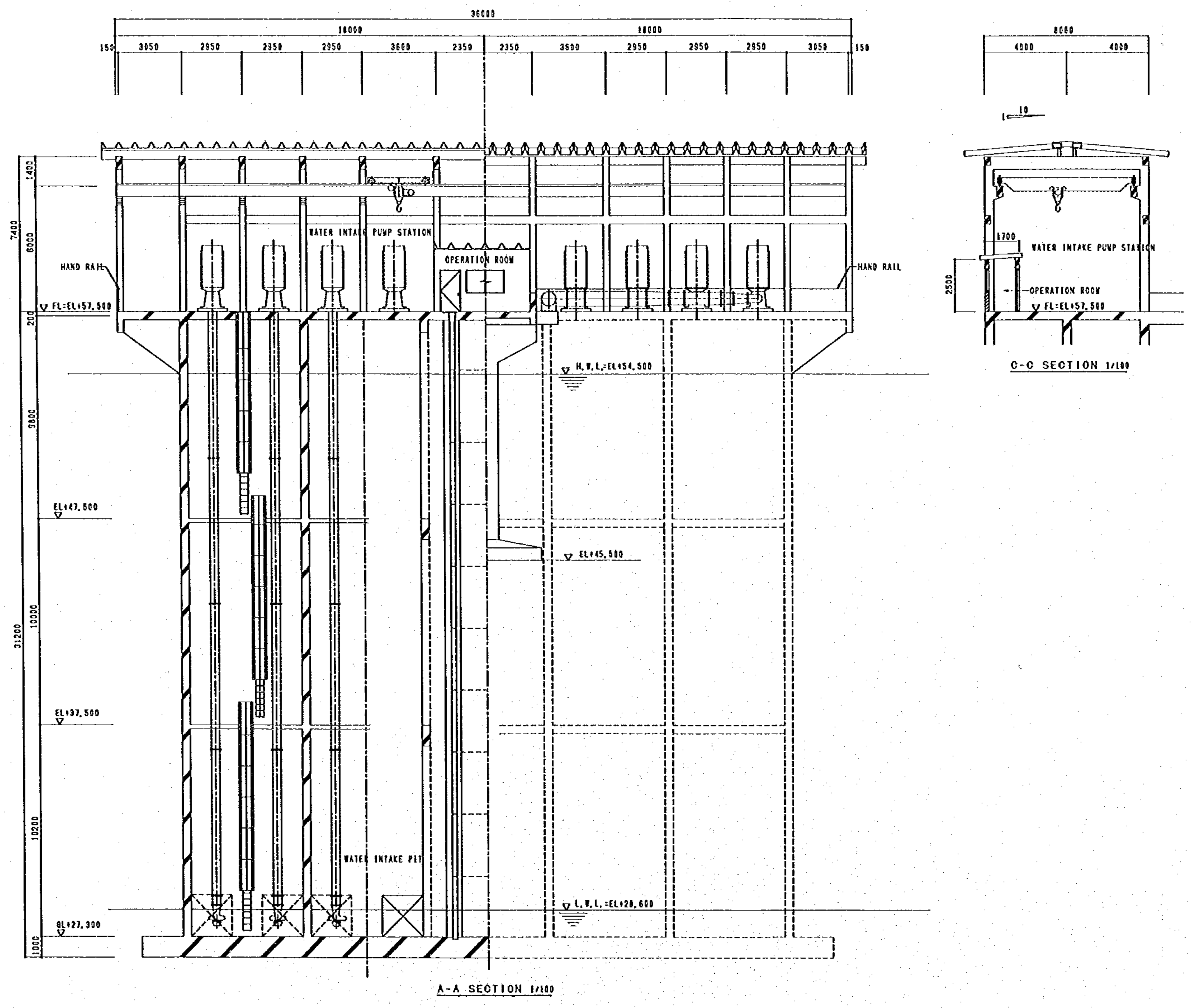
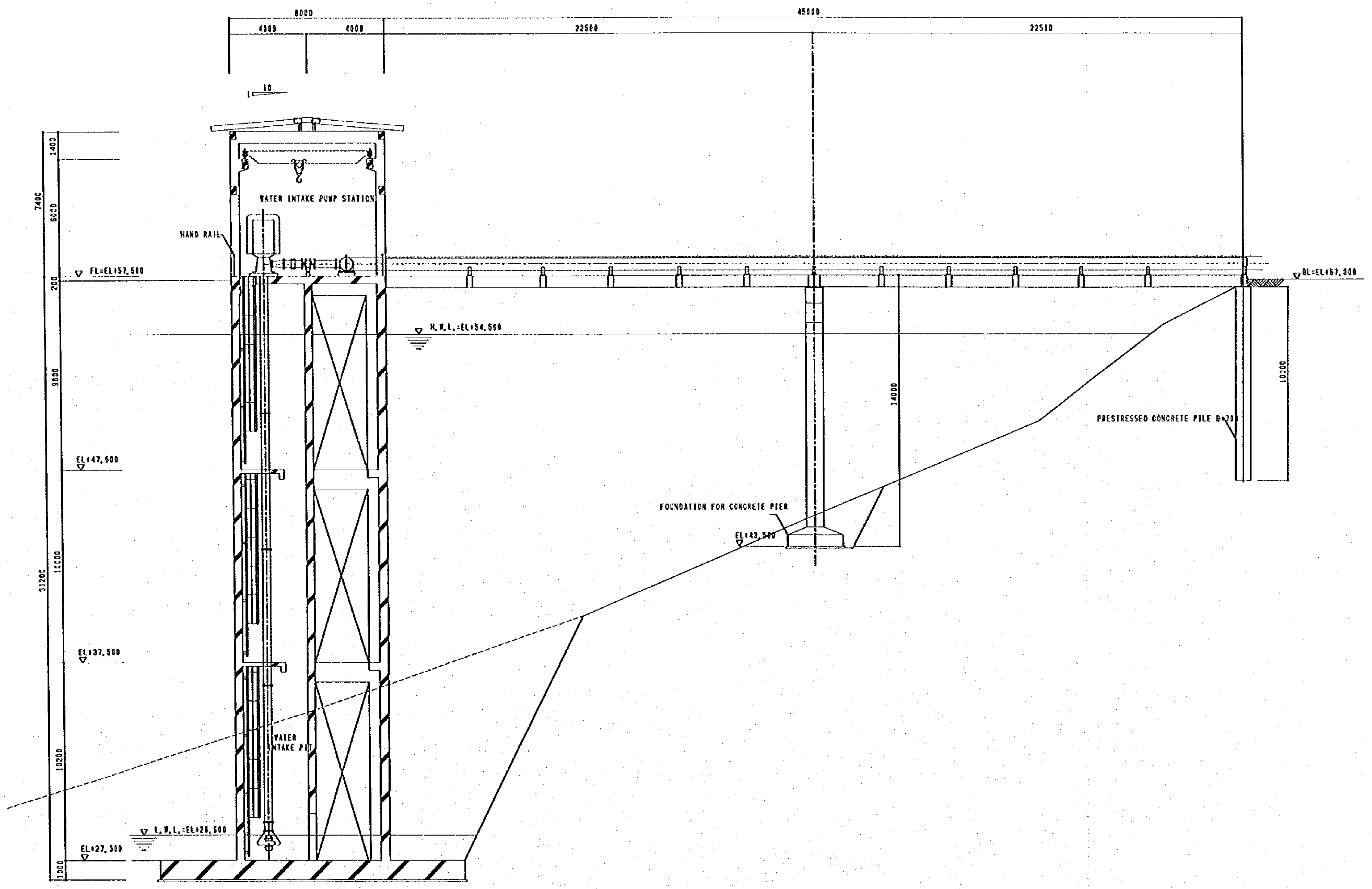



Figure-4.2(2/3) Water Intake Pump House




B-B SECTION 1/100

**Figure-4.2(3/3) Water Intake Pump House**

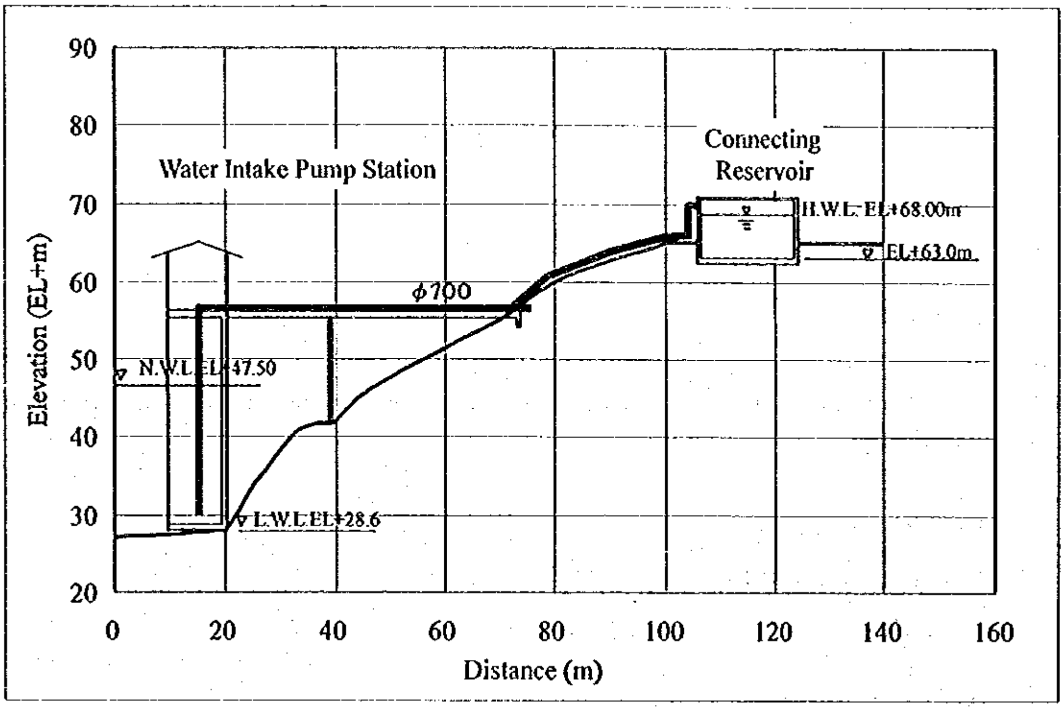



**Figure-4.3 Vertical profile of Pipeline**

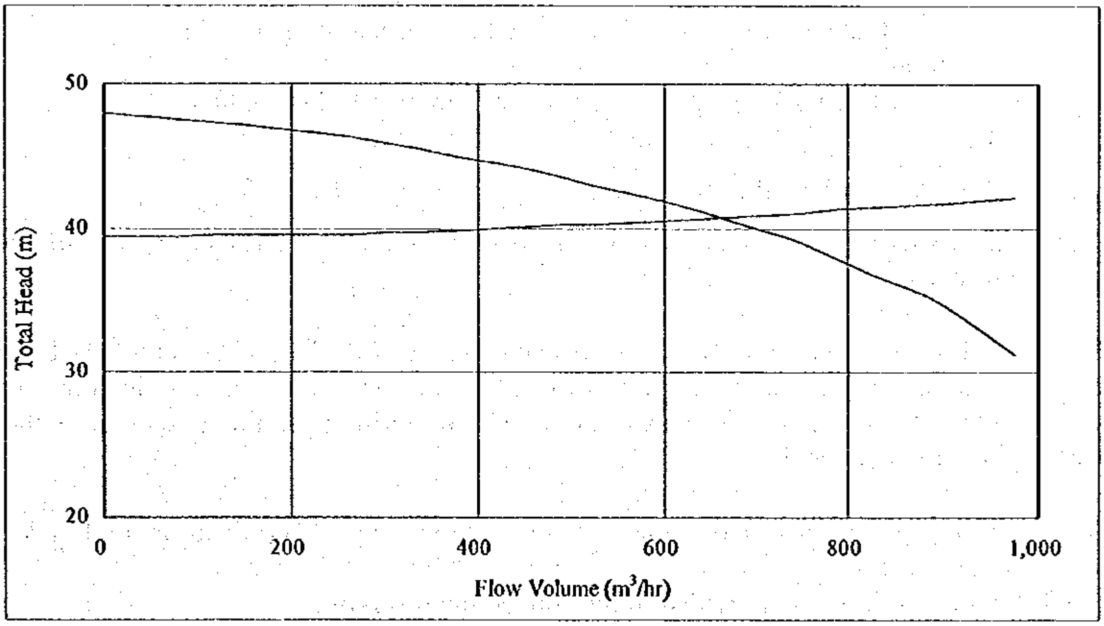



**Figure-4.4 System Curve for Water Intake Pump System**

### 4.1.7 Pump Unit Specification

#### (1) Pump

- Units : 3 + 1R per one construction phase
- Type : vertical mixed flow
- Discharge : 0.178 $m^3/s$ per one pump
- Total head : 41 m

### (2) Electric Motor

- Rotation : 1160 rpm
- Voltage : 440V
- Power : 110kW (150 PS)

## 4.2 Connecting Reservoir

### 4.2.1 General

Connecting reservoir are required for smooth conveyance of water from WIPS to the Water Lift Pump Stations and for provision of buffer reservoir against troubles in operation of Water Intake Pumps.

### 4.2.2 Design Conditions

### (1) Finished Ground Level

- F.G.L. : EL. 65.0m

### (2) Required Storage Capacity

- Phase 1
  max(0.260,0.273)$m^3$/s x 60min/s x 60 min/hr x 1.5hr ≒ 1,500$m^3$
- Phase 2
  max(0.260,0.273)$m^3$/s x 60min/s x 60 min/hr x 1.5hr = 1,500$m^3$
- Total
  max(0.519,0.546)$m^3$/s x 60min/s x 60 min/hr x 1.5hr = 3,000$m^3$

### 4.2.3 Design of Connecting Reservoir

Connecting Reservoir is designed as shown in Figure-4.5. The reservoir is reinforced concrete reservoir.

Water Level in the Connecting Reservoir is decided as H.W.L. EL. 68.0m and L.W.L. EL. 63.0m based on the finished ground level of EL. 65.0m. The reservoir is divided into two basins, so that the easy maintenance work of one basin during operation is secured without interruption of water supply.

Therefore, the dimensions of one basin of the reservoir are determined as 17.4m(W) x 17.3m(L) x 5.0m(H).
The following facilities are provided in the connecting reservoir.

- Inlet and outlet pipes
- Overflow pit and pipe for emergency overflow
- Drain pit for discharge at maintenance work
- Gate valve with float for water level control in the reservoir
- ventilation pipes

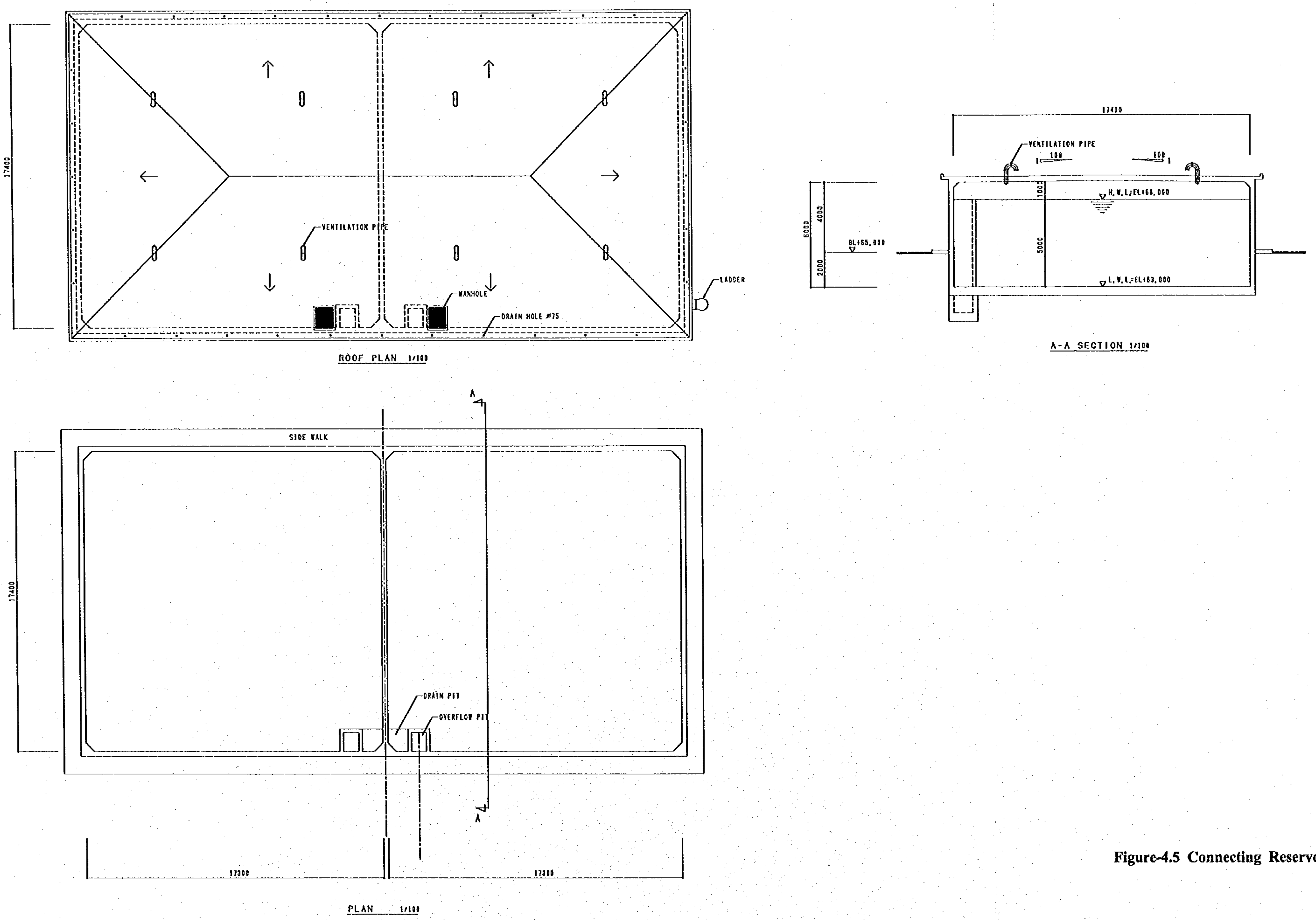



Figure-4.5 Connecting Reservoir

### 4.2.4 Sequence of Construction and Method of Operation

Connecting Reservoir with the storage capacity of 3,000m³ is required for each Water Conveyance System, namely CR1 for Agreste and CR2 for Piauitinga, after completion of Phase 2 construction work. In Phase 1 construction stage, the Connecting Reservoir CR1 is constructed and is used one basin for Agreste and the other for Piauitinga. In Phase 2, CR2 is constructed for Piauitinga use and CR1 is converted for Agreste use only. The conversion work is managed by combination of valve operation.

## 4.3 Water Lift Pump Station

### 4.3.1 General

Water lift pumps are composed of horizontal type pumps suitable for high total head with long distance conveyance. Pumps for Agreste and Piauitinga regions are operated separately with 2 pumps for normal operation and 1 pump for stand-by in each route in each phase. Pumps installed in Phase 1 are used continuously in Phase 2 with higher total head than in Phase 1.

### 4.3.2 Preliminary Study

#### (1) Cases for the Study

The preliminary study on the selection of combination of different diameters of pipelines and the associated pumps was elaborated considering the technical requirement for the systems described in 2.1. The result of Piauitinga pipeline is shown in this report. The Agreste pipeline has almost the same result because of the similar vertical profile.

The following three cases of combination of pipes were studied:

CASE-1 : All construction works of the system are finished in Phase 1. Diameter of pipe is 700 mm, which gives the optimum life cost for the system with specified water supply volume, in the whole length of the pipeline. The conveyance of water from the connecting reservoir to Brasilia is by pressure and the rest of pipeline is by gravity. Although the construction works are divided into two phases due to the financial limitation of the Government of Sergipe, this case is also studied for reference.

CASE-2 : Construction works are divided into two phases. Exactly the same system is constructed in each phase, resulting in the parallel operation of two water conveyance systems after the completion of Phase 2. Diameter of pipe is 500 mm and 600 mm in part to decrease the friction head loss in the pipeline, so that the water level in the connecting reservoir at Brasilia is acceptable. The conveyance of water from Brasilia to Lagarto is by gravity as same as in CASE-1.

CASE-3 : Combination of CASE-1 and CASE-2. The pipeline from the connecting reservoir to Brasilia has the diameter of 700 mm and is constructed in Phase 1. This pipeline is used in Phase 2 to convey water supply volume of 2 times greater than in Phase 1. The rest of the pipeline is constructed in each phase with the same diameter, resulting in the parallel operation of two water conveyance systems after the completion of Phase-2 works. Diameters of pipe are 500 mm and 600 mm to reduce the total friction head loss in the pipeline.

Cases for the preliminary study are summarized in Table-4.2.

**Table-4.2 Summary of Cases for Study**

| Case | Phase | Diameter of Pipelines: Dam – Brasilia | Brasilia – SE-104 | SE-104 – Lagarto |
|---|---|---|---|---|
| CASE-1 | 1 | 1-$\phi$700 | 1-$\phi$700 | 1-$\phi$700 |
| CASE-2 | 1 | 1-$\phi$500 | 1-$\phi$500 | 1-$\phi$600 |
| | 2 | 1-$\phi$500 | 1-$\phi$500 | 1-$\phi$600 |
| CASE-3 | 1 | 1-$\phi$700 | 1-$\phi$500 | 1-$\phi$600 |
| | 2 | - | 1-$\phi$500 | 1-$\phi$600 |

## (2) Result of the Preliminary Study

Investment, Annual Expense and Net Present Value (NPV) are calculated for each case and are summarized as shown in Table-4.3. Details of calculation of annual expense and net present value for each case are shown in Table-4.4 and Table-4.5 for Piauitinga and Agreste, respectively.

**Table-4.3 Result of the Preliminary Study**

Unit : millions R$

| Case | Investment (R$ million) | | | | Annual Expense | | | Net Present Value |
|---|---|---|---|---|---|---|---|---|
| | Pipeline | Pump Civil | Pump Equipment | Total | Interest | O & M | Total | |
| CASE-1 | 14.86 | 0.92 | 3.69 | 19.47 | 1.747 | 1.390 | 3.137 | 33.748 |
| CASE-2 | 17.16 | 1.02 | 4.07 | 22.25 | 1.997 | 1.636 | 3.633 | 33.417 |
| CASE-3 | 16.57 | 0.90 | 3.80 | 21.27 | 1.912 | 1.548 | 3.460 | 32.317 |

Although CASE-1 gives the lowest construction cost and annual expense, NPV is highest among all cases. This means that CASE-1 is not competitive in the view point of investment. In addition, CASE-1 is not feasible due to financial reason mentioned above. CASE-3 is the most competitive for the investment in spite of its construction cost, annual expense and NPV are higher than in CASE-1.

**Table-4.4 Investment, Annual Expense and Net Present Value for Piauitinga Pipeline**

| Case | Pipeline Route | Flow Volume | Water Elevation | Highest Elevation | Geographical difference in Elevation | Length of Pipeline | Pipe Dia. | Head Loss | Total Head | Pump Power | Investment (R$ millions) | | | | Annual Expense (R$ millions) | | | NPV (R$: millions) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | m³/s | m | m | m | km | m | m | m | kW | Pipe line | Pump Civil | Pump Equip | Total | AEc | AEo | Total | |
| CASE-1 | One phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Brasilia | 0.519 | 35.2 | 195.1 | 159.9 | 11.1 | 0.70 | 28.6 | 188.5 | 399.5 | 6.47 | 0.92 | 3.69 | 11.08 | 0.977 | 1.390 | 2.367 | |
| | Brasilia - Lagarto | 0.519 | 195.1 | 158.0 | 0.0 | 14.5 | 0.70 | 37.1 | 37.1 | | 8.39 | 0.00 | 0.00 | 8.39 | 0.770 | 0.000 | 0.770 | |
| | Total | | | | | | | | | | 14.86 | 0.92 | 3.69 | 19.47 | 1.747 | 1.390 | 3.137 | 33.748 |
| CASE-2 | 1st Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Brasilia | 0.260 | 35.2 | 202.7 | 167.5 | 11.1 | 0.50 | 40.9 | 208.4 | 221.3 | 3.53 | 1.02 | 2.04 | 6.59 | 0.561 | 0.818 | 1.379 | |
| | Brasilia - IMP | 0.260 | 202.7 | 158.6 | 0.0 | 10.9 | 0.50 | 39.9 | 39.9 | | 3.46 | 0.00 | 0.00 | 3.46 | 0.317 | 0.000 | 0.317 | |
| | IMP - Lagarto | 0.260 | 158.6 | 155.0 | 0.0 | 3.6 | 0.60 | 5.4 | 5.4 | | 1.59 | 0.00 | 0.00 | 1.59 | 0.146 | 0.000 | 0.146 | |
| | Subtotal | | | | | | | | | | 8.58 | 1.02 | 2.04 | 11.64 | 1.023 | 0.818 | 1.841 | |
| | 2nd Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Brasilia | 0.260 | 35.2 | 202.7 | 167.5 | 11.1 | 0.50 | 40.9 | 208.4 | 221.3 | 3.53 | 0.00 | 2.04 | 5.57 | 0.511 | 0.818 | 1.329 | |
| | Brasilia - IMP | 0.260 | 202.7 | 158.6 | 0.0 | 10.9 | 0.50 | 39.9 | 39.9 | | 3.46 | 0.00 | 0.00 | 3.46 | 0.317 | 0.000 | 0.317 | |
| | IMP - Lagarto | 0.260 | 158.6 | 155.0 | 0.0 | 3.6 | 0.60 | 5.4 | 5.4 | | 1.59 | 0.00 | 0.00 | 1.59 | 0.146 | 0.000 | 0.146 | |
| | Subtotal | | | | | | | | | | 8.58 | 0.00 | 2.04 | 10.62 | 0.973 | 0.818 | 1.791 | |
| | Total | | | | | | | | | | 17.16 | 1.02 | 4.07 | 22.25 | 1.997 | 1.636 | 3.633 | 33.418 |
| CASE-3 | 1st Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Brasilia | 0.260 | 35.2 | 200.2 | 165.0 | 11.1 | 0.70 | 8.0 | 173.0 | 183.7 | 6.47 | 0.90 | 1.69 | 9.06 | 0.793 | 0.737 | 1.530 | |
| | Vaza Barris - Brasilia | 0.228 | 35.2 | 200.2 | 165.0 | 11.1 | 0.70 | 28.6 | 193.6 | 180.2 | 0.00 | 0.00 | 0.00 | 0.00 | 0.000 | 0.710 | 0.710 | |
| | Brasilia - IMP | 0.260 | 200.2 | 160.4 | 0.0 | 10.9 | 0.50 | 39.9 | 39.9 | | 3.46 | 0.00 | 0.00 | 3.46 | 0.317 | 0.000 | 0.317 | |
| | IMP - Lagarto | 0.260 | 160.4 | 155.0 | 0.0 | 3.6 | 0.60 | 5.4 | 5.4 | | 1.59 | 0.00 | 0.00 | 1.59 | 0.146 | 0.000 | 0.146 | |
| | Subtotal | | | | | | | | | | 11.52 | 0.90 | 1.69 | 14.11 | 1.256 | 0.710 | 1.966 | |
| | 2nd Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Brasilia | 0.291 | 35.2 | 200.2 | 165.0 | 11.1 | 0.70 | 28.6 | 193.6 | 230.0 | 0.00 | 0.00 | 2.11 | 2.11 | 0.194 | 0.837 | 1.031 | |
| | Brasilia - IMP | 0.260 | 200.2 | 160.4 | 0.0 | 10.9 | 0.50 | 39.9 | 39.9 | | 3.46 | 0.00 | 0.00 | 3.46 | 0.317 | 0.000 | 0.317 | |
| | IMP - Lagarto | 0.260 | 160.4 | 155.0 | 0.0 | 3.6 | 0.60 | 5.4 | 5.4 | | 1.59 | 0.00 | 0.00 | 1.59 | 0.146 | 0.000 | 0.146 | |
| | Subtotal | | | | | | | | | | 5.05 | 0.00 | 2.11 | 7.16 | 0.657 | 0.837 | 1.494 | |
| | Total | | | | | | | | | | 16.57 | 0.90 | 3.80 | 21.27 | 1.912 | 1.548 | 3.460 | 32.317 |

## Table-4.5 Investment, Annual Expense and Net Present Value for Agreste Pipeline

| CASE- | Pipeline Route | Flow Volume | Water Elevation | Highest Elevation | Geographical difference in Elevation | Length of Pipeline | Pipe Dia. | Head Loss | Total Head | Pump Power | Investment (R$:millions) | | | | Annual Expense (R$:millions) | | | NPV (R$: millions) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | m³/s | m | m | m | km | m | m | m | kW | Pipe line | Pump Civil | Pump Equip | Total | AEc | AEo | Total | |
| CASE-1 | One phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Ribeira | 0.546 | 35.2 | 257.8 | 222.6 | 8.8 | 0.70 | 24.7 | 247.3 | 551.4 | 5.09 | 1.27 | 5.09 | 11.44 | 0.996 | 1.734 | 2.730 | |
| | Ribeira - Itabaiana | 0.546 | 257.8 | 215.0 | 0.0 | 15.2 | 0.70 | 42.8 | 42.8 | | 8.81 | 0.00 | 0.00 | 8.81 | 0.808 | 0.000 | 0.808 | |
| | Total | | | | | | | | | | 13.90 | 1.27 | 5.09 | 20.26 | 1.805 | 1.734 | 3.539 | 38.275 |
| CASE-2 | 1st Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Ribeira | 0.273 | 35.2 | 257.0 | 221.8 | 8.8 | 0.50 | 35.3 | 257.1 | 286.6 | 2.78 | 1.32 | 2.65 | 6.76 | 0.564 | 0.958 | 1.523 | |
| | Robeira - IMP | 0.273 | 257.0 | 228.3 | 0.0 | 7.1 | 0.50 | 28.7 | 28.7 | | 2.27 | 0.00 | 0.00 | 2.27 | 0.207 | 0.000 | 0.207 | |
| | IMP - Itabaiana | 0.273 | 228.3 | 215.0 | 0.0 | 8.1 | 0.60 | 13.3 | 13.3 | | 3.55 | 0.00 | 0.00 | 3.55 | 0.326 | 0.000 | 0.326 | |
| | Subtotal | | | | | | | | | | 8.60 | 1.32 | 2.65 | 12.58 | 1.098 | 0.958 | 2.056 | |
| | 2nd Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Ribeira | 0.273 | 35.2 | 257.0 | 221.8 | 8.8 | 0.50 | 35.3 | 257.1 | 286.6 | 2.78 | 0.00 | 2.65 | 5.43 | 0.497 | 0.958 | 1.455 | |
| | Robeira - IMP | 0.273 | 257.0 | 228.3 | 0.0 | 7.1 | 0.50 | 28.7 | 28.7 | | 2.27 | 0.00 | 0.00 | 2.27 | 0.207 | 0.000 | 0.207 | |
| | IMP - Itabaiana | 0.273 | 228.3 | 215.0 | 0.0 | 8.1 | 0.60 | 13.3 | 13.3 | | 3.55 | 0.00 | 0.00 | 3.55 | 0.326 | 0.000 | 0.326 | |
| | Subtotal | | | | | | | | | | 8.60 | 0.00 | 2.65 | 11.25 | 1.031 | 0.958 | 1.989 | |
| | Total | | | | | | | | | | 17.20 | 1.32 | 5.30 | 23.83 | 2.129 | 1.916 | 4.045 | 37.284 |
| CASE-3 | 1st Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Ribeira | 0.273 | 35.2 | 257.0 | 221.8 | 8.8 | 0.70 | 6.9 | 228.7 | 254.9 | 5.09 | 1.23 | 2.34 | 8.66 | 0.743 | 0.889 | 1.632 | |
| | Vaza Barris - Ribeira | 0.234 | 35.2 | 257.0 | 221.8 | 8.8 | 0.70 | 24.7 | 246.5 | 235.5 | 0.00 | 0.00 | 0.00 | 0.00 | 0.000 | 0.829 | 0.829 | |
| | Robeira - IMP | 0.273 | 257.0 | 228.3 | 0.0 | 7.1 | 0.50 | 28.7 | 28.7 | | 2.27 | 0.00 | 0.00 | 2.27 | 0.207 | 0.000 | 0.207 | |
| | IMP - Itabaiana | 0.273 | 257.0 | 215.0 | 0.0 | 8.1 | 0.60 | 13.3 | 13.3 | | 3.55 | 0.00 | 0.00 | 3.55 | 0.326 | 0.000 | 0.326 | |
| | Subtotal | | | | | | | | | | 10.91 | 1.23 | 2.34 | 14.48 | 1.277 | 0.829 | 2.106 | |
| | 2nd Phase | | | | | | | | | | | | | | | | | |
| | Vaza Barris - Ribeira | 0.312 | 35.2 | 257.0 | 221.8 | 8.8 | 0.70 | 24.7 | 246.5 | 314.0 | 0.00 | 0.00 | 2.90 | 2.90 | 0.265 | 1.054 | 1.319 | |
| | Robeira - IMP | 0.273 | 257.0 | 228.3 | 0.0 | 7.1 | 0.50 | 28.7 | 28.7 | | 2.27 | 0.00 | 0.00 | 2.27 | 0.207 | 0.000 | 0.207 | |
| | IMP - Itabaiana | 0.273 | 257.0 | 215.0 | 0.0 | 8.1 | 0.60 | 13.3 | 13.3 | | 3.55 | 0.00 | 0.00 | 3.55 | 0.326 | 0.000 | 0.326 | |
| | Subtotal | | | | | | | | | | 5.82 | 0.00 | 2.90 | 8.72 | 0.799 | 1.054 | 1.853 | |
| | Total | | | | | | | | | | 16.72 | 1.23 | 5.24 | 23.19 | 2.076 | 1.884 | 3.959 | 36.741 |

### 4.3.3 Water Lift Pump Station for Piauitinga, WLPS2

#### (1) Design Conditions

1) Water Intake Level : EL. 68.0m

2) Water Discharge Volume

- Phase 1 : 0.260 m³/s
- Phase 2 : 0.260 m³/s
- Total : 0.520 m³/s

#### (2) Location of WLPS2

WLPS2 is constructed next to the connecting reservoir CR2 as shown in Figure-4.6.

#### (3) Design of Water Lift Pump House

Water Lift Pump House is designed as shown in Figure-4.7 considering the following technical points:

- The house is designed as reinforced concrete frame with masonry wall.
- Reinforced suction pit is provided for smooth intake of water from the connecting reservoir.

  Internal dimension of suction pit is 1.2m(W) x 17.8m(L) x 6.1m(H). H.W.L. is EL. 68.0m as same as in the connecting reservoir and bottom level is EL. 62.5m to secure water flow by gravity from the connecting reservoir.
- Floor level of pump pit is EL. 62.5 m as same as in the suction pit.
- Installation and maintenance of pumps, fittings, accessories, etc. are performed by manual monorail chain block of 2 ton capacity.

## (4) Description of the Pump System

**Table-4.6 Description of Pump System**

| Pump Name | Lift 1 | | |
|---|---|---|---|
| Use | Domestic and industrial | | |
| Treatment | Raw | | |
| Phase | 1* | 1* | 2 |
| Quantity | 2 + 1 | 2 + 1 | 2 + 1 |
| Total head | 140 m | 161 m | 161 m |
| *Actual head* | *132 m* | *132 m* | *132 m* |
| *Head loss* | *8 m* | *29 m* | *29 m* |
| Suction head | 2 m | 2 m | 2 m |
| Discharge | 0.260 | 0.519 | 0.519 |
| | (0.130) | (0.130) | (0.130) |
| Pipeline Dia. | 700 mm | 700 mm | 700 mm |
| Pipeline Length | 11.1 km | 11.1 km | 11.1 km |

Note 1 : Discharge of pumps to be total discharge with 2 pumps in operation. Discharge in ( ) to be per one pump. Unit of discharge in $m^3/s$

Note 2 : Pumps installed in Phase 1 stage to be operated with higher total head in Phase 2 stage.

Vertical profiles and system curve of the system is shown in Figure-4.8, Figure-4.9 and Figure-4.10.

## (5) Method of Operation

In principle, the pumps are operated for 24 hours a day. Water lift pumps are stopped their operation automatically when the water level in the connecting reservoir is lower than three meters below normal water level in the connecting reservoir. The pipeline constructed in Phase 1 is used in Phase 2 with the flow volume 2 times greater than that in Phase 1, which results in the increase in total head in the system. The pumps installed in Phase 1 are operated continuously in Phase 2 with total head higher than that in Phase 1. The pumps installed in Phase 2 are operated parallel with the pumps installed in Phase 1. The condition of electrical stoppage is the same as for the water intake pumps.

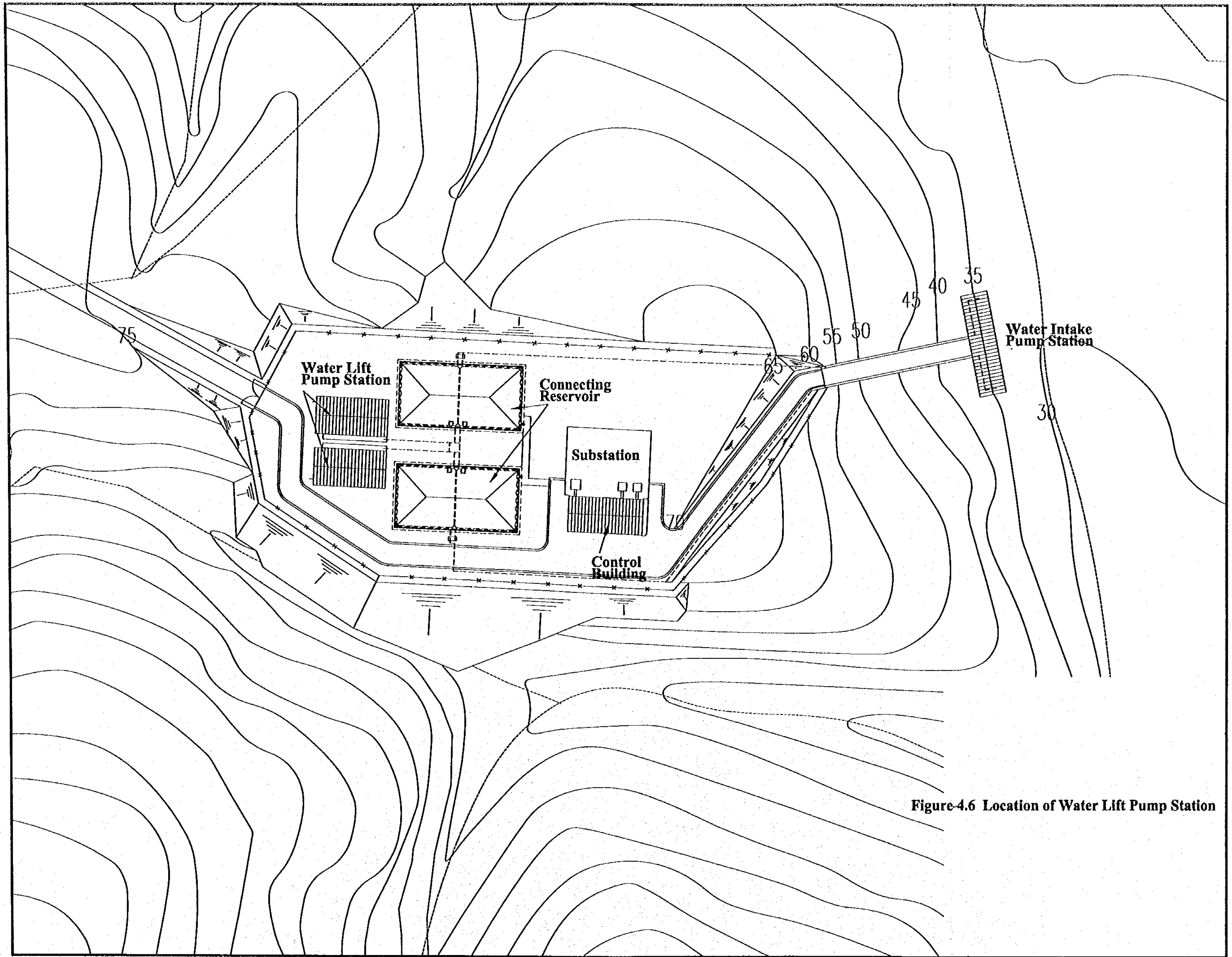
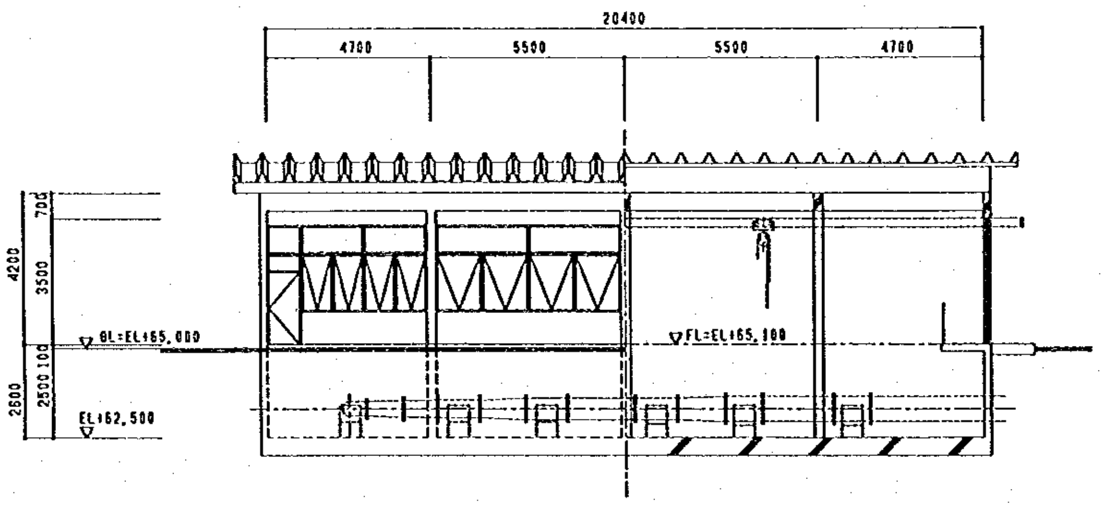
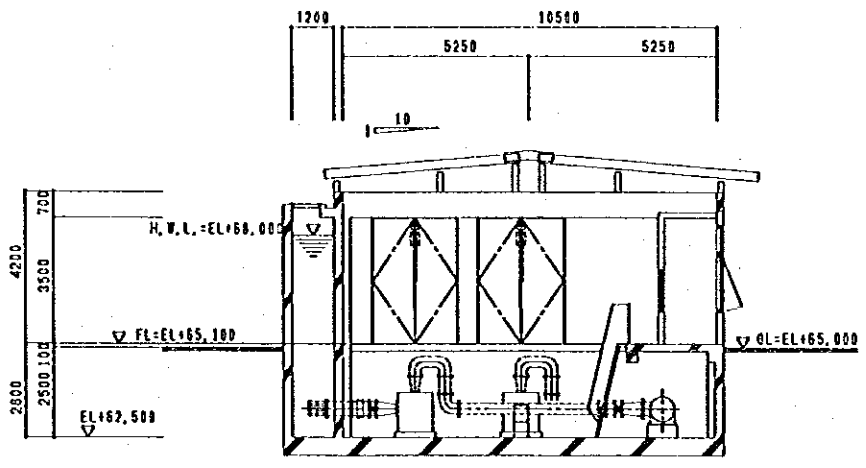


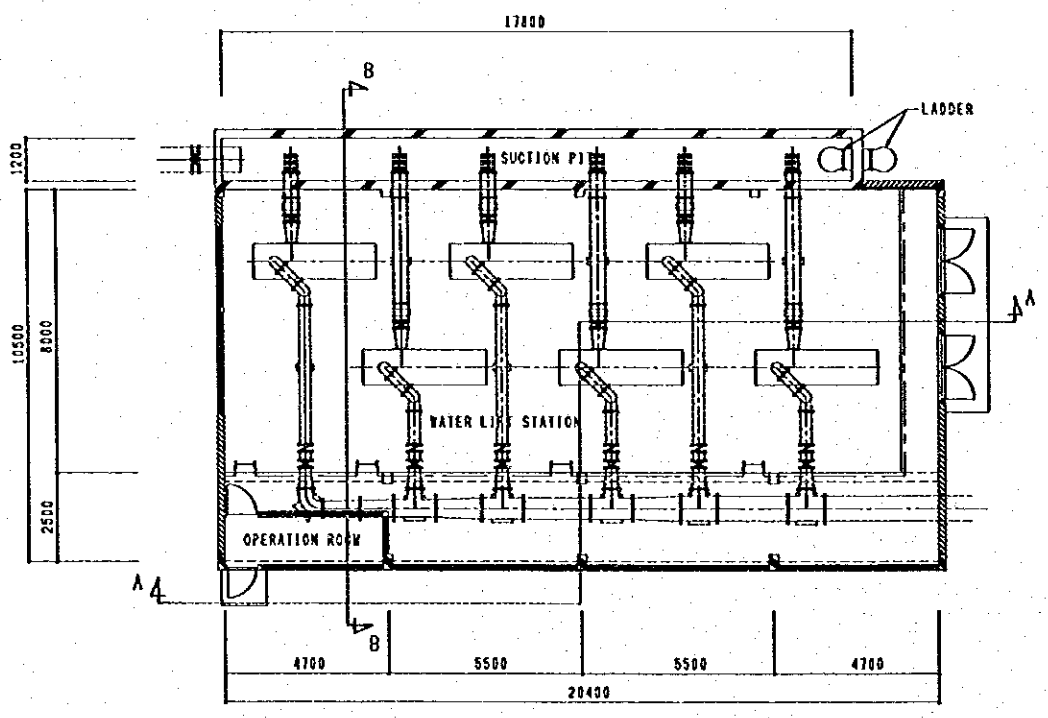

Figure 4.6 Location of Water Lift Pump Station




A-A SECTION 1/100




B-B SECTION 1/100




GROUND FLOOR PLAN 1/100

| EXTERIOR FINISH SCHEDULE | | INTERIOR FINISH SCHEDULE | |
|---|---|---|---|
| ROOF | FIBERCEMENT ROOFING CANALETE 49 TYPE OR EQUIVALENT | FLOOR | CONCRETE STEEL TROWEL WITH SILICON PAINT |
| WALL | PLASTER WITH LATEX PAINT | WALL | PLASTER WITH LATEX PAINT |
| | | COLUMN | SILICON PAINT |
| | | BEAM | DITTO |
| | | SUCTION PIT | WATER PROOFING WITH ACRYLIC BASE CRYSTALIZED CEMENT |

**Figure-4.7 Water Lift Pump House**

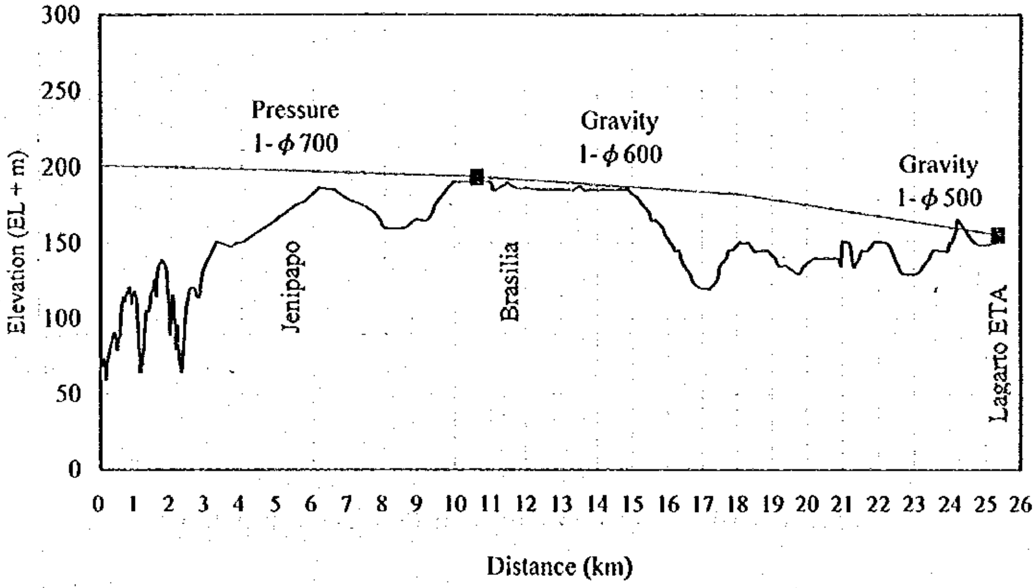



**Figure-4.8 Vertical Profile of Piauitinga Pipeline in Phase 1**

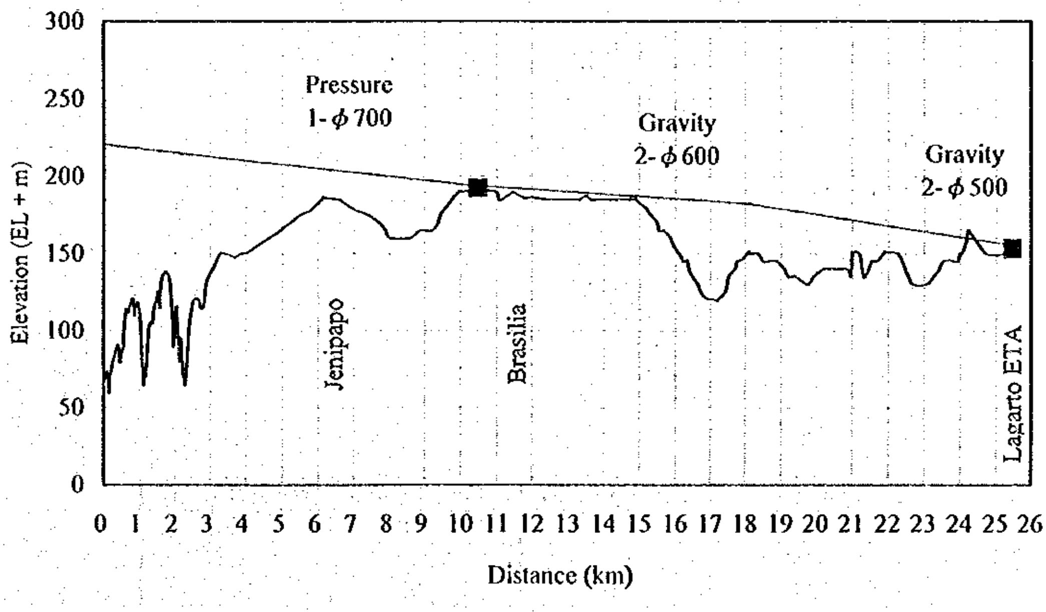



**Figure-4.9 Vertical Profile of Piauitinga Pipeline in Phase 2**

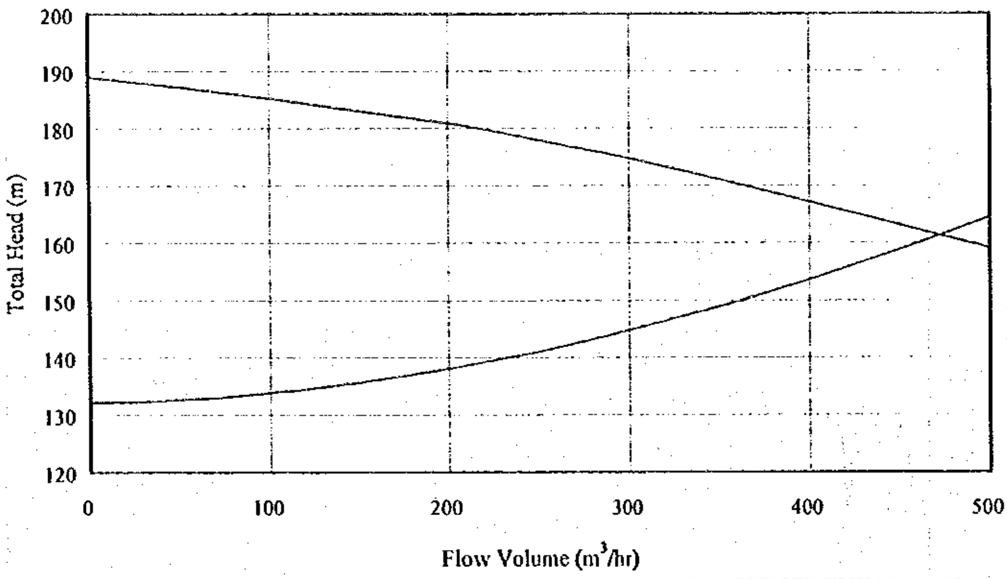



**Figure-4.10 System Curve of Piauitinga Pipeline**

## (6) Water Hammer

In general, water hammer is expected to occur when designing a high head pump, large capacity pump, pump conveying water to a long distance, etc. Water lift pumps have the characteristics most likely to occur water hammering. Water hammer analysis was executed to grasp the occurrence of water hammering in the pump system.

### (a) Water Hammer Analysis without Countermeasures

1) Input data

– Pipeline data
  length : 11,100 m
  diameter : 700 mm
  material : ductile cast iron
  flow rate : 31.2 $m^3$/min

– Pump Data
  quantity : 4
  total head : 161 m
  flow rate : 7.8 $m^3$/min
  motor : 294 kW
  $GD^2$ : 30kg·$m^2$
  speed : 1,780 rpm

– Profile of pipeline

| Length (m) | Level (EL. m) | Length (m) | Level (EL. m) |
|---|---|---|---|
| 0 | 68 | 4,000 | 150 |
| 800 | 122 | 6,200 | 185 |
| 1,100 | 70 | 8,000 | 165 |
| 1,750 | 135 | 9,300 | 170 |
| 2,300 | 140 | 10,000 | 185 |
| 2,600 | 123 | 11,000 | 185 |
| 3,300 | 150 | 11,100 | 200 |

2) Result of analysis

The result of analysis is shown in Figure-4.11. It is obvious that negative pressure is generated in most of the pipeline and the countermeasures against water hammer are required.

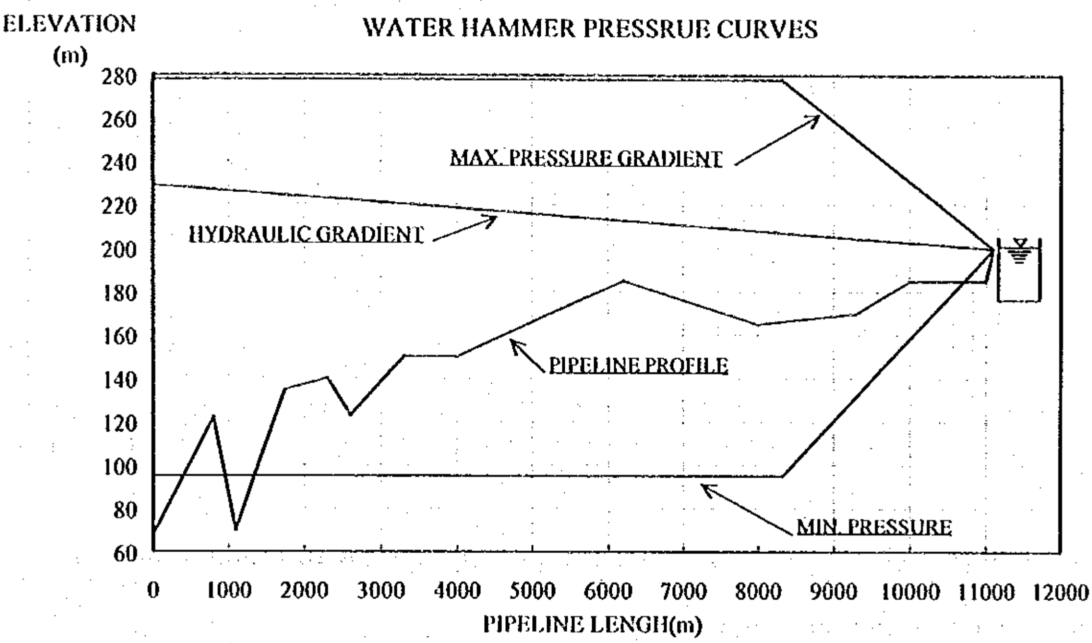



**Figure-4.11 Result of Water Hammer Analysis without Countermeasures**

**(b) Water Hammer Analysis with Countermeasures**

1) Countermeasures against water hammer

Through various trial-and-error of water hammer analysis, it was found that the following countermeasures are effective.

- provision of flywheel to the pumps
- one way surge tanks in two locations

2) Input data

- Pipeline data
  no change in input data
- Pump Data
  no change in input data except for flywheel $GD^2$ : $100 kg \cdot m^2$
- One Way Surge Tank

| No. | Head | Section Area ($m^2$) |
|---|---|---|
| 1 | 92 | 10 |
| 2 | 127 | *10* |

- Profile of pipeline
  no change in input data

3) Result of analysis

The result of analysis is shown in Figure-4.12. Although still there exists some negative pressure zones, it is judged that the combination of flywheel and one way surge tanks is effective.

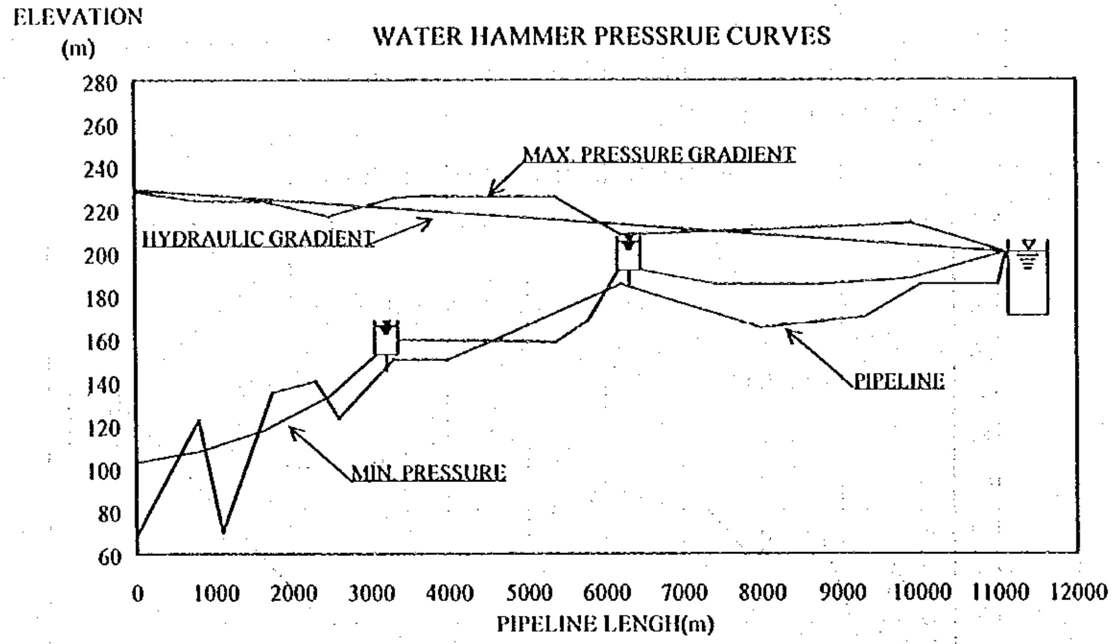



**Figure-4.12 Result of Water Hammer Analysis with Countermeasures**

**(c) Countermeasures to be adopted**

- flywheel with GD2=100kg·m²
- one way surge tank

| No. | Location(m) | W.L.(EL. m) | Cross section Area ($m^2$) | Height(m) |
|---|---|---|---|---|
| 1 | 3,300 | > 160.0 | 10 | > 3.0 |
| 2 | 6,200 | > 195.0 | 10 | > 5.0 |

**(7) Pump Specification**

**(a) Pump**

- Units : 2 + 1R
- Type : horizontal
- Discharge : 0.130 $m^3/s$ per one pump
- Total head : 140 m (Phase 1) and 161 m (Phase 2)
- Flywheel : 100 kg·$m^2$

**(b) Motor**

- Rotation : 1,775 rpm
- Voltage : 440 V
- Power : 294 kW (400 PS)

### 4.3.4 Water Lift Pump Station for Agreste, WLPS1

**(1) Design Conditions**

**(a) Water Intake Level** : EL. 68.0m

**(b) Water Discharge Volume**

- Phase 1 : 0.273 $m^3/s$

- Phase 2 : 0.273 $m^3/s$
- Total : 0.546 $m^3/s$

### (2) Location of WLPS1

WLPS1 is constructed next to the connecting reservoir CR1 as shown in Figure-4.6.

### (3) Design of Water Lift Pump House

Water Lift Pump House is designed as same as WLPS2 shown in Figure-4.7.

### (4) Description of the Pump System

**Table-4.7 Description of Pump System**

| Pump Name | Lift 2 | | |
|---|---|---|---|
| Use | Domestic | | |
| Treatment | Raw | | |
| Phase | 1* | 1* | 2 |
| Quantity | 2 + 1 | 2 +1 | 2 + 1 |
| Total head | 196 m | 214 m | 214 m |
| *Actual head* | *189 m* | *189 m* | *189 m* |
| *Head loss* | *7 m* | *25 m* | *25 m* |
| Suction head | 2 m | 2 m | 2 m |
| Discharge | 0.273 | 0.546 | |
| | (0.137) | (0.137) | (0.137) |
| Pipeline Dia. | 700 mm | 700 mm | |
| Pipeline Length | 8.8 km | 8.8 km | |

Note 1 : Discharge of pumps to be total discharge with 2 pumps in operation. Discharge in ( ) to be per one pump. Unit of discharge in $m^3/s$

Note 2 : Pumps installed in Phase 1 stage to be operated with higher total head in Phase 2 stage.

Vertical profiles and system curve of the system are shown in Figure-4.13, Figure-4.14 and Figure-4.15.

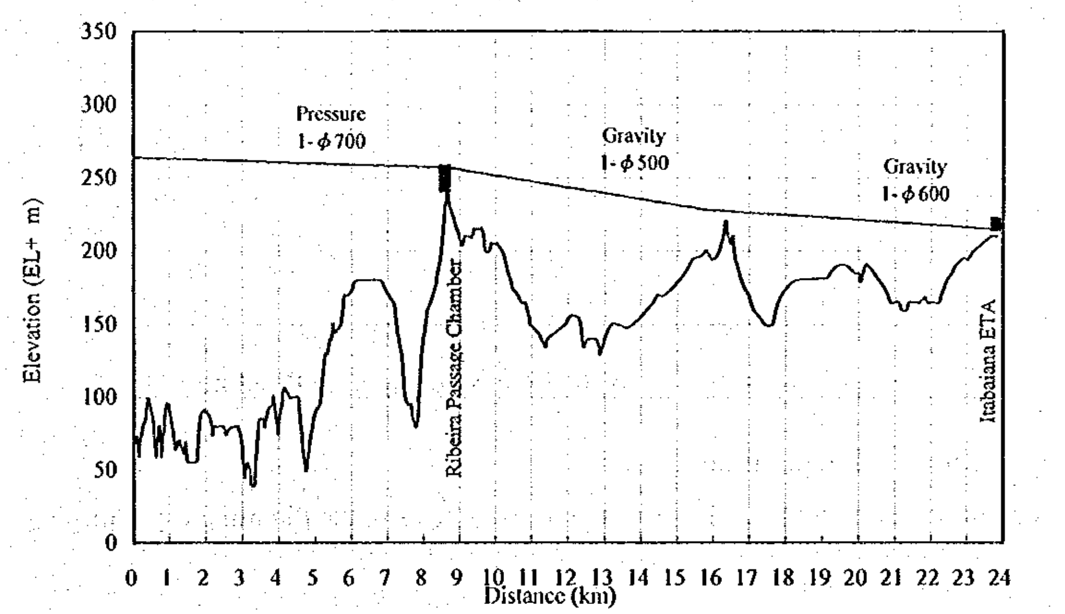



**Figure-4.13 Vertical Profile of Agreste Pipeline in Phase 1**

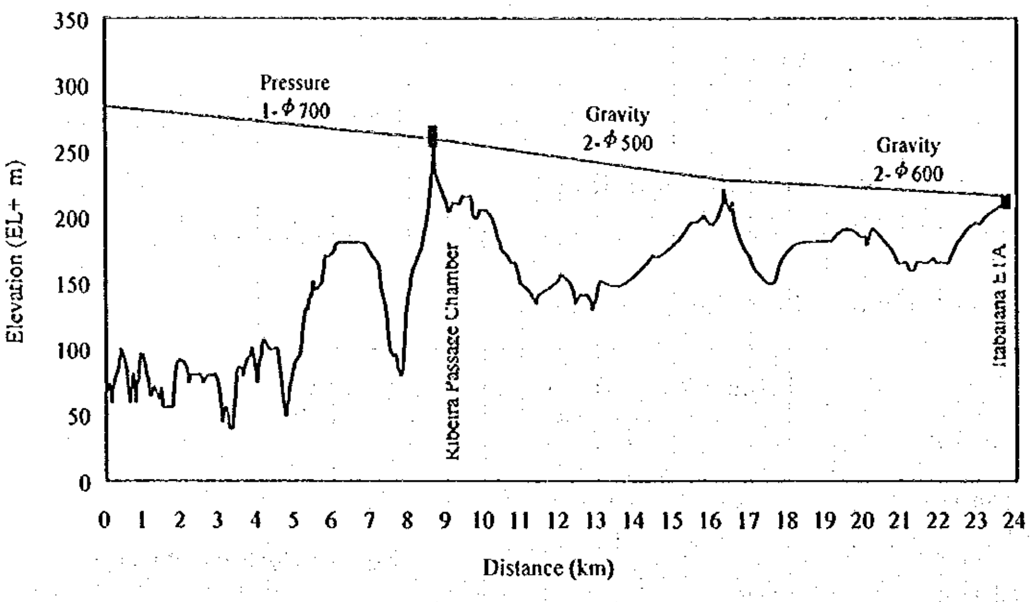



**Figure-4.14 Vertical Profile of Agreste Pipeline in Phase 2**

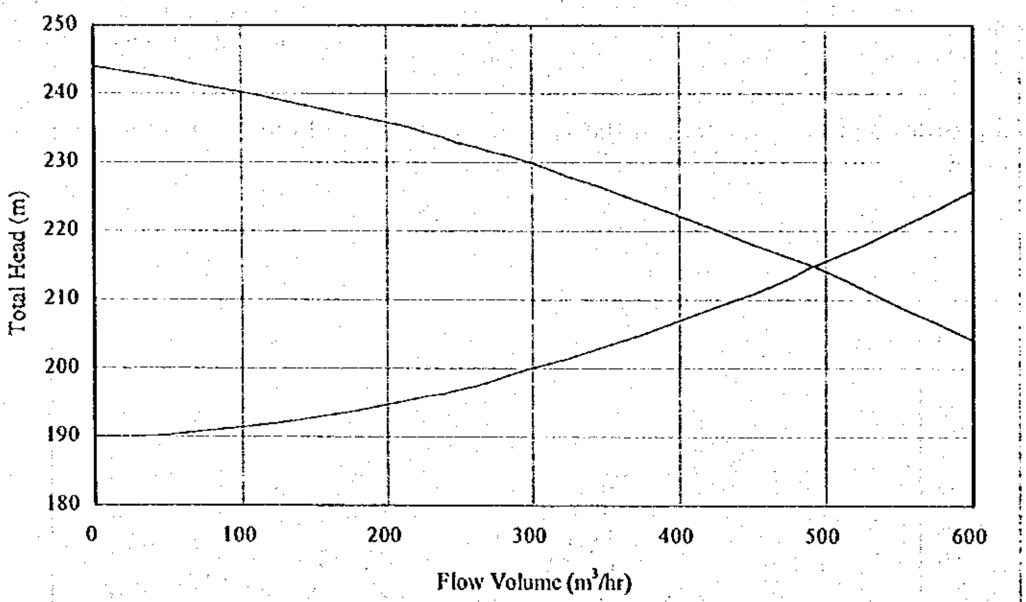



**Figure-4.15 System Curve of Agreste Pipeline**

## (5) Method of Operation

Method of operation in the Agreste system is as same as in the Piauitinga system.

## (6) Water Hammer

Water hammer analysis was executed to grasp the occurrence of water hammering in the pump system.

**(a) Water Hammer Analysis without Countermeasures**

1) Input data

- Pipeline data
  - length : 8,800 m
  - diameter : 700 mm
  - material : ductile cast iron
  - flow rate : 32.8 $m^3$/min
- Pump Data
  - quantity : 4
  - total head : 214 m
  - flow rate : 8.2 $m^3$/min
  - motor : 294 kW
  - GD2 : 50kg·$m^2$
  - speed : 3,500 rpm
- Profile of pipeline

| Length (m) | Level (EL. m) | Length (m) | Level (EL. m) |
|---|---|---|---|
| 0 | 68 | 4,100 | 105 |
| 450 | 100 | 4,500 | 100 |
| 800 | 60 | 4,800 | 50 |
| 1,000 | 100 | 6,000 | 180 |
| 1,700 | 60 | 6,900 | 180 |
| 2,000 | 90 | 7,800 | 80 |
| 3,000 | 75 | 8,700 | 235 |
| 3,300 | 40 | 8,800 | 257 |

2) Result of analysis

The result of analysis is shown in Figure-4.16. It is obvious that negative pressure will be occurred some of the pipeline and the countermeasure against water hammer will be required.

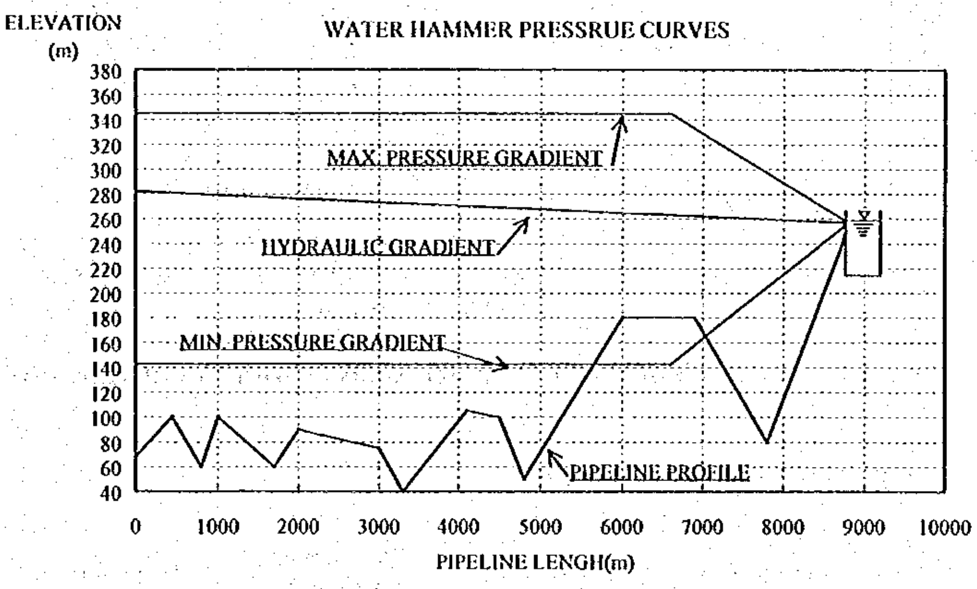



**Figure-4.16 Result of Water Hammer Analysis without Countermeasures**

## (b) Water Hammer Analysis with Countermeasures

1) Countermeasures against water hammer

Through various trial-and-error of water hammer analysis, it was found that the following countermeasures are effective.

- one way surge tank in one location

2) Input data

- Pipeline data

  no change in input data

- Pump Data

  no change in input data

- One Way Surge Tank

| No. | Head | Section Area ($m^2$) |
|---|---|---|
| 1 | 120 | 20 |

- Profile of pipeline

  no change in input data

3) Result of analysis

The result of analysis is shown in Figure-4.17. It was shown that one way surge tank is effective.

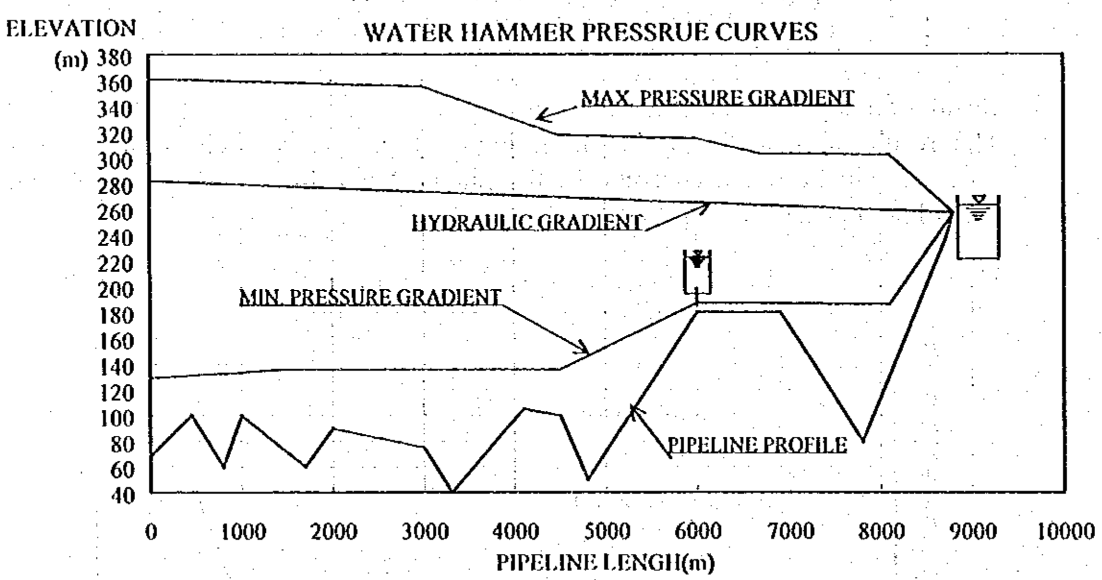



**Figure-4.17 Result of Water Hammer Analysis with Countermeasures**

## (c) Countermeasures to be Adopted

- one way surge tank

| No. | Location(m) | W.L.(EL. m) | Cross section Area ($m^2$) | Height(m) |
|---|---|---|---|---|
| 1 | 6,000 | > 188.0 | 20 | > 2.0 |

**(7) Pump Specification**

**(a) Pump**

- Units : 2 + 1R
- Type : horizontal
- Discharge : 0.137 $m^3/s$
- Total head : 196 m (Phase 1) and 214 m (Phase 2)

**(b) Motor**

- - Rotation : 3,500 rpm
- - Voltage : 440 V
- - Power : 294 kW (400 PS)

## 4.4 Piauitinga Water Conveyance Pipeline

### 4.4.1 Characteristics of Pipeline

Characteristics of Pipeline route determined based on the water conveyance plan in CHAPTER 3 are as follows:

**(1) Design Conditions**

Service Pressure in the Pipeline

- Permissible Service Pressure : 1.58Mpa
- Maximum Service Pressure : 1.58Mpa

**(2) Materials**

Unless otherwise specified, all pipe materials are ductile cast iron.
Class of Push-on Joint Type pipe is K7 in accordance with the Service pressure in the pipeline.

**(3) Length**

$\phi$700 : 10,416 m
$\phi$600 : 7,783 m per Phase (total length in two phases 15,566 m)
$\phi$500 : 7,206 m per Phase (total length in two phases 14,412 m)

**(4) Ground Elevation**

Vertical profile of the pipeline is shown in Figure-4.18.
Lowest elevation : EL. 60m
Highest elevation : EL. 190m

**(5) Conditions of Ground Surface in Pipeline Route**

Type of ground surface and its length is as follows:

Pasture : 4,324 m
Bare land : 20,911 m
Pavement : 80 m
Over-pass : 90 m

**(6) Details of Over-pass**

**(a) Urubu River Crossing**

- 40m of river crossing
- Pipes to be supported by the steel support anchored at each 6m to the side of existing reinforced concrete bridge with span length of approximately 35m

**(b) Urubutinga River Crossing**

- 6m of river crossing
- Pipes to span over the river without any support from the existing reinforced concrete box culvert with span length of approximately 6m

**(c) Machado River Crossing**

- 40m of river crossing
- Pipes to be supported by the steel support anchored at each 6m to the side of existing reinforced concrete bridge with span length of approximately 35m

**(7) Road Crossings**

Major road crossings are as follows:

**(a) SE-214**

Crossing with SE-214 coming from Itaporanga d'Ajuda at the intersection of road from Jenipapo. Crossing width is 20m.

**(b) Brasilia**

Crossing with branch road from SE-214 in Brasilia. Crossing width is 30m.

**(c) SE-104**

Crossing with SE-104 at intersection of road from Brasilia. Crossing width is 10m.

**(d) Lagarto Water Treatment Station**

Crossing with internal road just before Lagarto Water Treatment Station. Crossing width is 20m.