2.7 Case of Load

The following cases of load must be considered in the studies of stability and respective calculations in the internal efforts.

- Case of Normal Load (CCN)
  It corresponds to all the combinations of actions that present great probability of occurrence along the durability period of the structure, during the normal operation or normal maintenance of the work, in normal hydrological conditions.

- Case of Exceptional Load (CCE)
  It correspond to any load actions of eventual occurrence of low probability of exceptional hydrological conditions, malfunctions in the drainage system, action of exceptional character, seismic effect etc.

- Case of Load of Construction (CCC)
  It corresponds to all the combination of actions that present probability of occurrence during the construction work, only, during short periods in relation to its durability and in good conditions of control. It might be due to loads of construction equipment, or, due to structures executed only partially, abnormal loads during the transportation of permanent equipment, and any other similar conditions.

Cases of load combination for stability analysis of gravity concrete dam is summarized in Table-2.3.

Table-2.3 Case of Load Combination

Case	Reservoir Level	Load mast be considered	Drain condition
CCN	N.W.L.	W, P, Pe, U	Operating
CCE	H.W.L.	W, P, Pe, I, Pd. U	Non Operating
CCC	All the load to act against dam during construction		

Note:

W	:	own weight of dam body
P	:	hydrostatic pressure forces
Pe	:	forces of the pressures due to sedimentation
I	:	seismic inertia force
Pd	:	dynamic water pressure during earthquake
U	:	uplift

2.8 Method for Stability Analysis of Concrete dam

CEMIG design criteria indicates items to be examined as shown below;

(1) Safety Coefficients to Fluctuation (C.S.F.)

Safety coefficient to the fluctuation is defined as the relation between the summitry of the gravity forces and the summitry of the Sub-pressure Forces and will be given by the expression:

$$C.S.F = \frac{\Sigma V}{\Sigma U}$$

where

C.S.F. : Safety Coefficient to the Fluctuation.
V : summitry of the gravitational forces
U : summitry of the sub-pressure forces

Allowable minimum of C.S.F. is as follows;

Table-2.4 Allowable Minimum Value of C.S.F.

Safety Coefficients	Carrying Cases		
	CCN	CCE	CCC
CSF >	1.3	1.1	1.2

(2) Safety Coefficient against the Overturning

The safety coefficient to the Overturning in any direction is defined as the relation between the stabilizing moment and the falling down moment in relation to one point or to one effective line of rotations and will be given by the expression:

$$C.S.T = \frac{\Sigma Me}{\Sigma Mt}$$

where

C.S.T. : Safety Coefficient to the Overturning
Me : summitry of stabilizer moments
Mt : summitry of falling down moments

Allowable minimum of C.S.T. is as follows;

Table-2.5 Allowable Minimum Value of C.S.T

Safety Coefficients	Carrying Cases		
	CCN	CCE	CCC
CST >	1.5	1.2	1.3

(3) Safety to the Sliding for Structures (F.S.S)

It is considered that the safety to sliding is verified if:

$$F.S.S = \frac{\dfrac{\Sigma Ni \tan(\phi i)}{CSD\phi} + \dfrac{\Sigma CiAi}{CSDC}}{\Sigma ti} \geq 1.0$$

where

CSDø : safety coefficient relatively to the friction
CSDC : safety coefficient relatively to the cohesion

Ni	:	normal strength to the surface of sliding analyzes
Øi	:	angle of friction characteristic of the sliding surface.
Ci	:	cohesion characteristic along to the sliding.
Ai	:	real area of contact of structure in the plan under analyzes.
Ti	:	resulting on the forces parallel to the sliding surface.

Values of CSDø and SDC are as follows;

Table-2.6 Safety Coefficients

Safety Coefficients	Carrying Cases		
	CCN	CCE	CCC
CSDØ	1.5 (2.0)	1.1 (1.3)	1.3 (1.5)
CSDC	3.0 (4.0)	1.5 (2.0)	2.0 (2.5)

The adoption of these figures indicates a reasonable knowledge of the resistance parameters of the involved materials.

These figures shall be increased in the cases in which such knowledge is precarious or the materials do not present constancy behavior. In this case there shall be adopted the figures between the parenthesis.

(4) Analyzes of Tensions and Deformations

The tensions and deformations analyzes shall be elaborated to all the structural elements and of foundations considering the possible cases of carrying, in a way to determine or confirming the dimensioning of the structural elements.

As this study is for basic design stage, it is enough to confirm that the vertical stress acting on the contact surface between the dam base and the foundation rock is smaller than allowable tensile stress.

The charge capacity of the foundations is related to the maximum normal tension, defined through criteria which supply the rupture conditions, and the limitations concerning the excessive repression, harmful to the behavior and perfect utilization of the structure.

The maximum normal tension on the foundations shall be obtained based on the following relation:

$$\sigma t, adm = \frac{Charge\ capacity\ of\ the foundation}{Safety Coefficient}$$

The charge capacity of the foundations material shall be determined by suitable methods, using as subsides the results of tests" in situ "and the ones from laboratory.

In relation to the safety coefficient, are recommended the figures specified as follows:

Table-2.7 Safety Coefficient for Allowable Tensile Stress

Carrying Case	Safety coefficient
CCN	3.0 (4.0)
CCC	2.0 (3.0)
CCE	1.5 (2.0)

These figures adoption points out to a reasonable knowledge of the resistance of the materials involved.

2.9 Excavation Line

The depth of the dam foundation is decided based on the imperviousness the rock strength required from the result of the stability analysis. As geological data is not enough so far, the excavation depth of the dam base should be decided according to general criteria.

(1) Dam Foundation Rock

- Concrete type dam: As a rule, C_M class rock mass is needed for the foundation of the dam with the height of 50m, though even C_L class rock mass is available where the dam height is small.
- Fill type dam : The core base needs imperviousness. C_L class rock mass is needed expecting the shielding effect by grouting. The filter base also needs C_L class rock mass. Soil and superficial sediments must be taken away from the rock base of the dam body.

(2) Excavation Slope

Average gradient of the foundation excavation slope including berm has maximum values as shown below;

Table-2.8 Maximum Excavation Slope

Inside of dam base	Outside of dam base	
	Soil	Rock
1 : 0.7	1 : 1.0	1 : 0.8

2.10 Foundation Treatment

(1) Foundation grouting

As geological data of foundation rock is not enough so far, general design values is adopted.

- Curtain Grouting

$$d = H/3 + \alpha$$

where

d	:	depth of grout hole (m)
H	:	dam height upon a grout hole (m)
α	:	constant (8-25m)

- Consolidation Grouching

Pattern of boreholes arrangement is lattice pattern with 5m distance.

(2) Drain

According to CEMIG standard, length between upstream edge and drains is more 8% of reservoir water depth.

CHAPTER 3 DESIGNS OF DAM BODY AND SPILLWAY

3.1 Alternative Design of Dam and Spillway

(1) Comparison of Dam Type

Four (4) dam types: gravity concrete dam, rock-fill dam with zone type, rock-fill dam with concrete facing type and earth-fill dam are compared. Refer to Table-3.1, Figure-3.1, Figure-3.2 and Figure-3.3. Type of Vaza Barris Dam: ***Gravity Concrete Dam Type*** is decided due to low cost and workability of construction under the following considerations:

1) Rock material and concrete aggregate are procured at the existing quarry site located near the Cajaiba Dam and 15 km far from the dam site. As necessary material volume is minimum, concrete type dam, is advantageous.

2) Due to big design flood discharge for spillway, a large scale of spillway facility is necessary. Fill type dam requires large volume of concrete for spillway facility. Spillway for concrete type dam can be installed easily on the dam body. Concrete type dam is advantageous from viewpoints of construction cost and construction workability.

3) Fill type dam can be constructed on the rather less hard rock on the ground surface. That is an advantage of fill type dam. At the dam site, the layer of this rather less hard rock, namely C_L-class rock is very thin or nothing. Concrete type dam is constructed on the hard rock foundation: C_M-class rock or more. At the dam site, the difference between fill type dam foundation and concrete type dam foundation is very small. Therefore, fill type dam has no advantage at the dam site.

4) Around the dam site, layers of soil and weathered rock are very thin. Therefore, a large area is necessary to collect core material and earth material. Considering the current land use (pastureland) near the dam site, it is impossible to collect soil material in the wide areas.

5) Water depth in flood time is very high due to low gradient of the river channel. This high water depth is disadvantage for diversion tunnel during construction period. Therefore, partial bulkhead for diversion is recommendable. Partial bulkhead method is applicable for concrete type dam because the design diversion discharge is small and concrete dam is resistant against dam-top overflow. For fill type dam, diversion during construction period is very difficult at the Vaza Barris dam site.

(2) Comparison of Spillway

Generally, for concrete dam, spillway is installed on the top of dam as one unit with dam body. For fill type dam, spillway is constructed, apart from dam body, at the most appropriate place. The location is decided considering topography, geology, workability, economic efficiency and so on. Table-3.2 shows the comparison of spillway for fill type dam. Refer to Figure-3.4.

Spillway for Vaza Barris Dam has no gate. For concrete dam, overflow spillway is installed on the top of dam. For fill type dam, Chute spillway with overflow inlet is employed. Width of spillway for fill type dam is estimated to be about 130 m, supposing overflow depth: about 6 m. From the comparison result of spillway location for fill type dam, Left-Bank plan is most recommendable.

Table-3.1 Comparison of Dam Type

Items	Gravity Concrete Dam	Rock-fill Dam		Earth Dam
		Zone Type	Concrete Facing Type	
Dam Foundation	Dam foundation at the site is possible for construction of any type of dam.			
	O	O	O	O
Material for Dam Body	Rock material including aggregate are not available near the dam site. The material for embankment and concrete has to be purchased form the existing quarry site, 15 km far from the dam site. Near the dam site, acquisition of soil material is very difficult.			
	1) Concrete Aggregate, Cement, Fly ash, Admixture	1) Core 2) Filter 3) Rock	1) Concrete 2) Transition 3) Rock	1) Earth
	O	Δ	O	X
Location of Spillway	Upper part of main dam	Left bank side, apart from dam body		
	O	Δ	Δ	Δ
Location of Low Flow Outlet	Inside of main dam	Outlet pipe cannot be installed in the dam body. Pipe and other related facilities are constructed apart from dam body.		
	O	Δ	Δ	Δ
Resistance against dam top overflow	Safety due to concrete structure.	Weak	Weak but better than zone type rock fill dam	Very weak
	O	X	Δ	X
Diversion Discharge	Q=200 m³/s (1/2 year probability)	Q=720 m³/s (1/20 year probability) For concrete facing type, design discharge can be decreased.		
	O	X	Δ	X
Diversion Method	- Partial bulkhead - Temporary hole in dam body	No practical method, due to very large scale of diversion tunnel	- Partial bulkhead - Temporary hole in dam body	No practical method, due to very large scale of diversion tunnel
	O	X	O	X
Construction Facilities	One system including concrete batch and concrete place	Total two systems: - Embankment system for dam body - Concrete batch and concrete place system for spillway		
	O	Δ	Δ	Δ
Dam Height	Base rock for fill type dam is D-class or CL-class. Base rock for concrete dam is CM-class. The rock layers of D-class and CL-class are very thin at the dam site. The dam height for concrete type and fill type is almost same, 50 m – 55 m.			
Dam Volume	275,000 m³ (1.0)	899,000 m³ (3.3)	697,000 m³ (2.5)	1,100,000 m³ (4.0)
Spillway Concrete Volume	0 m³	37,500 m³	37,500 m³	37,500 m³
Excavation Volume	373,000 m³ (1.0)	2,113,000 m³ (5.7)	1,926,000 m³ (5.1)	2,301,000 m³ (6.2)
Total Evaluation	1) Due to min. volume of dam, material cost inc. transportation is cheaper than fill type dam. 2) Good workability due to one unit inc. dam, spillway and outlet. 3) Due to big resistance against flood over top of dam, cost for diversion can be decreased.	1) Due to big diversion discharge and small slope of river channel, cost for diversion facilities is extremely expensive. 2) Acquisition of core material is difficult. 3) Cost for spillway is very larger than that of concrete type dam.	1) Due to remote quarry site, cost for rock material is very high. 2) Cost for spillway is very larger than that of concrete type dam.	1) Due to big diversion discharge and small slope of river channel, cost for diversion facilities is extremely expensive. 2) There is no earth material near dam site. 3) Cost for spillway is very larger than that of concrete type dam.
	O (Very Good)	X (Not Good)	Δ(Fair)	X (Not Good)
	Probable dam types are gravity concrete and rock-fill with concrete facing. Due to cost and workability of construction, the most promising dam type is gravity concrete dam for Vaza Barris Dam.			

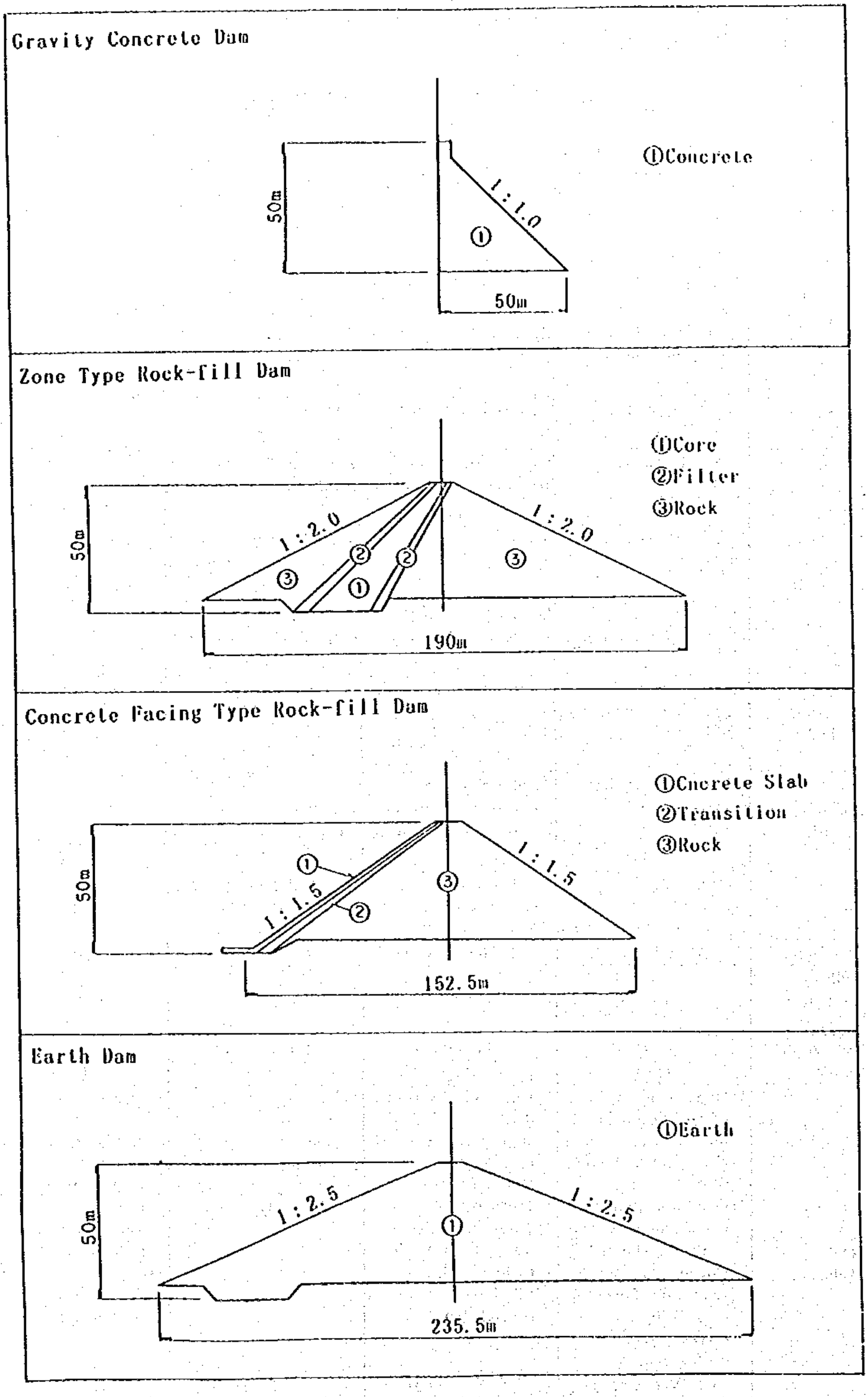
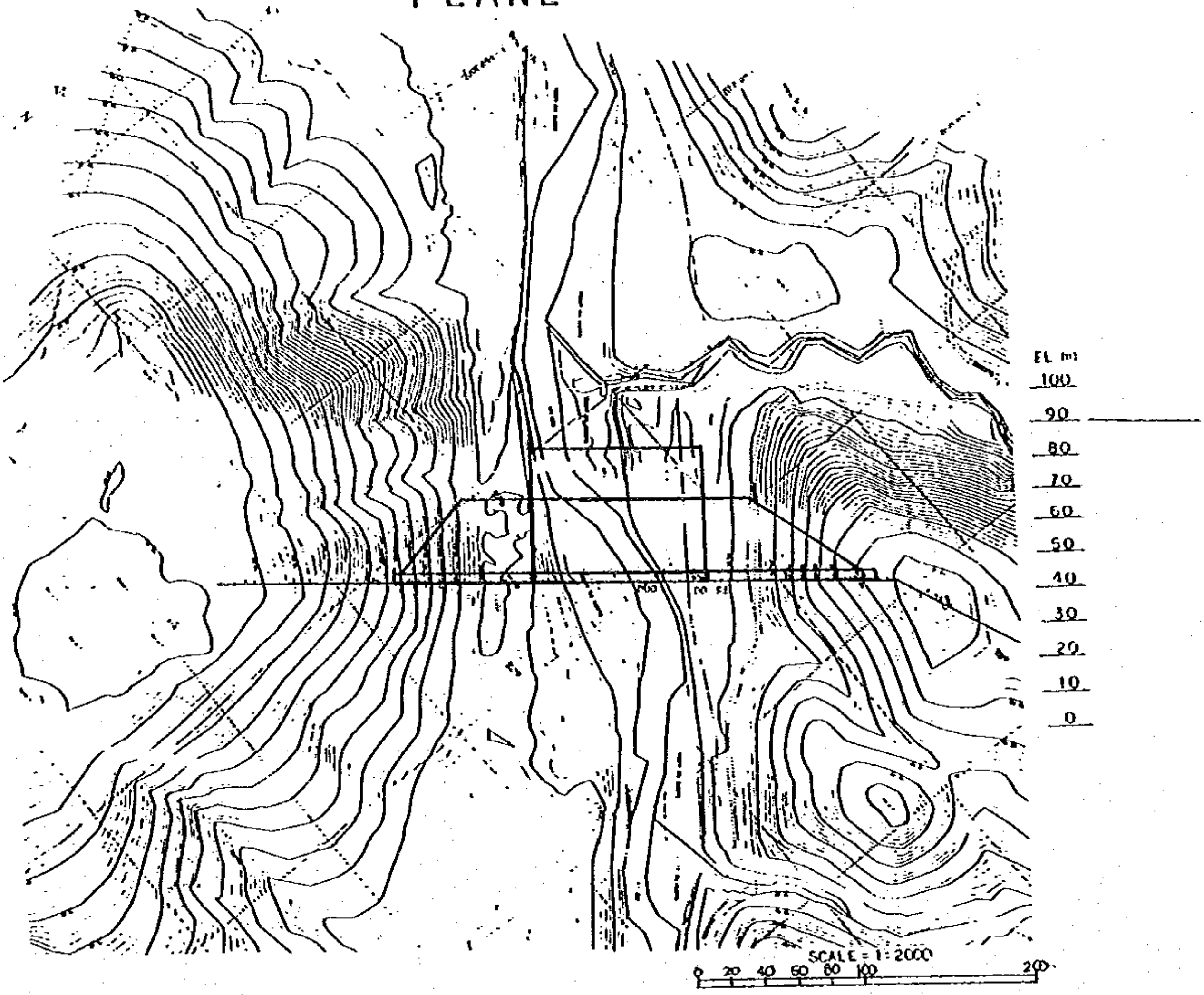
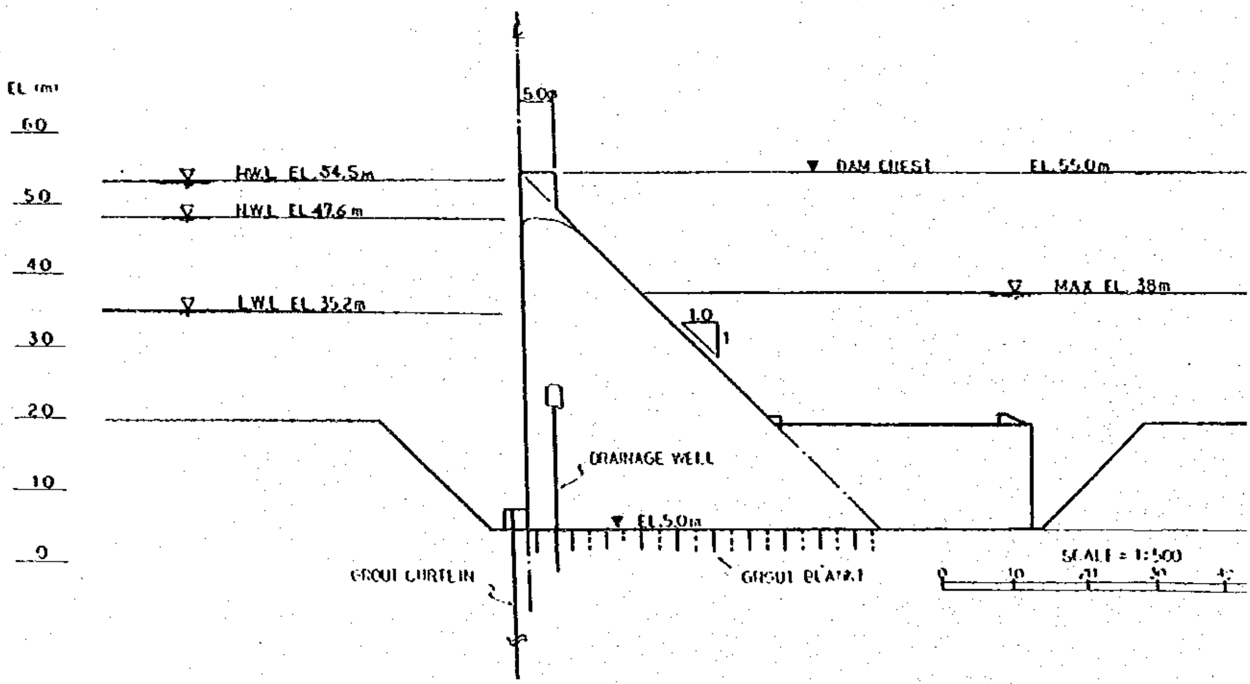


Figure-3.1 Type of Dam

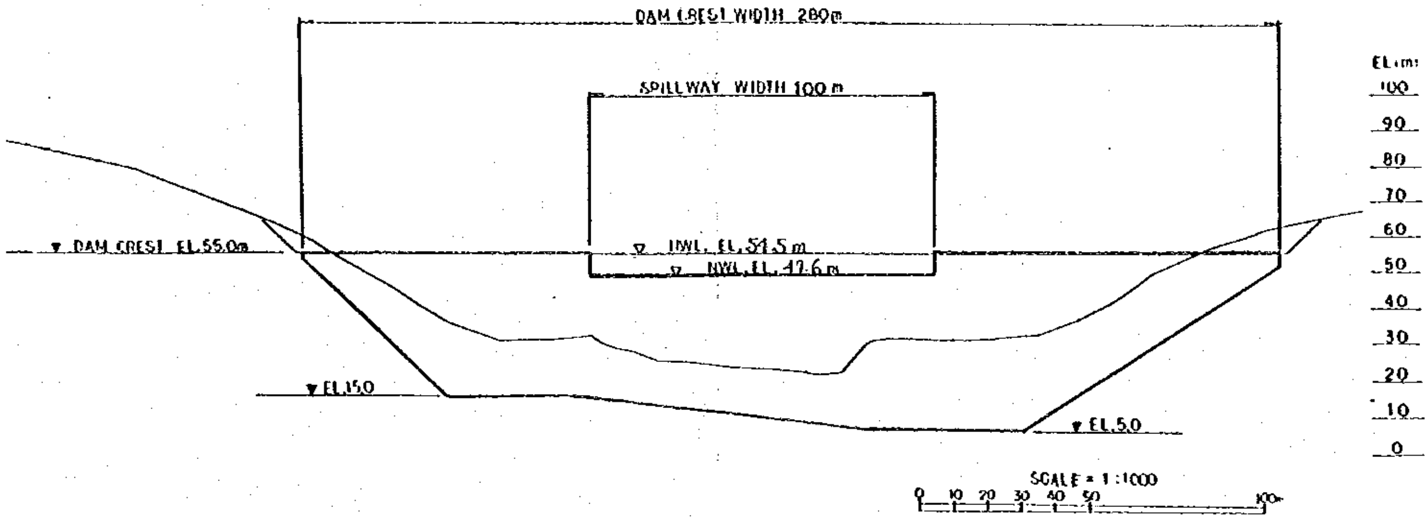
PLANE



TYPICAL CROSS SECTION



LONGITUDINAL SECTION OF DAM AXIS



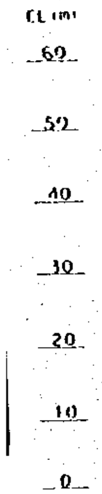


Figure-3.2 Alternative Plan (Gravity Concrete Dam)

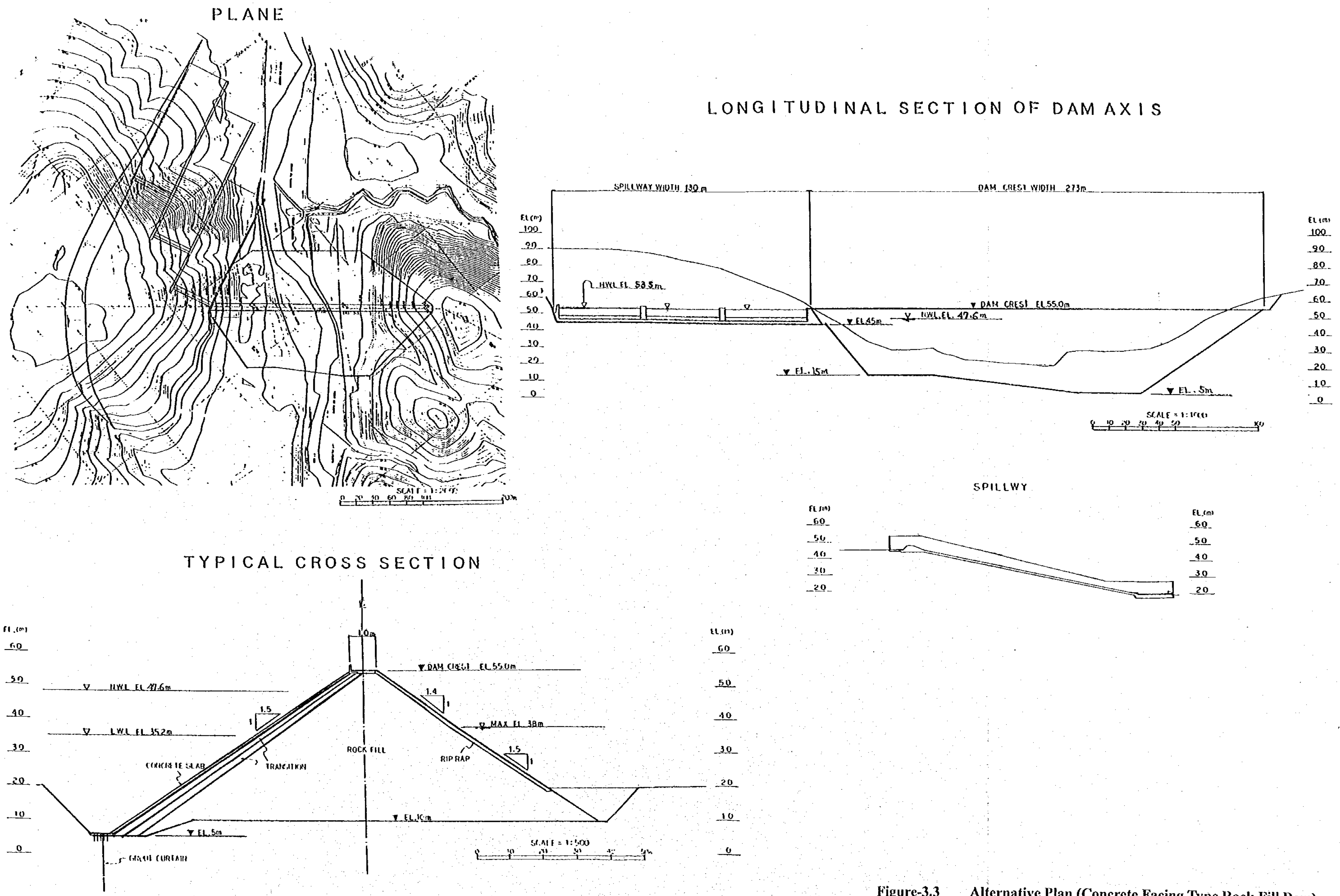
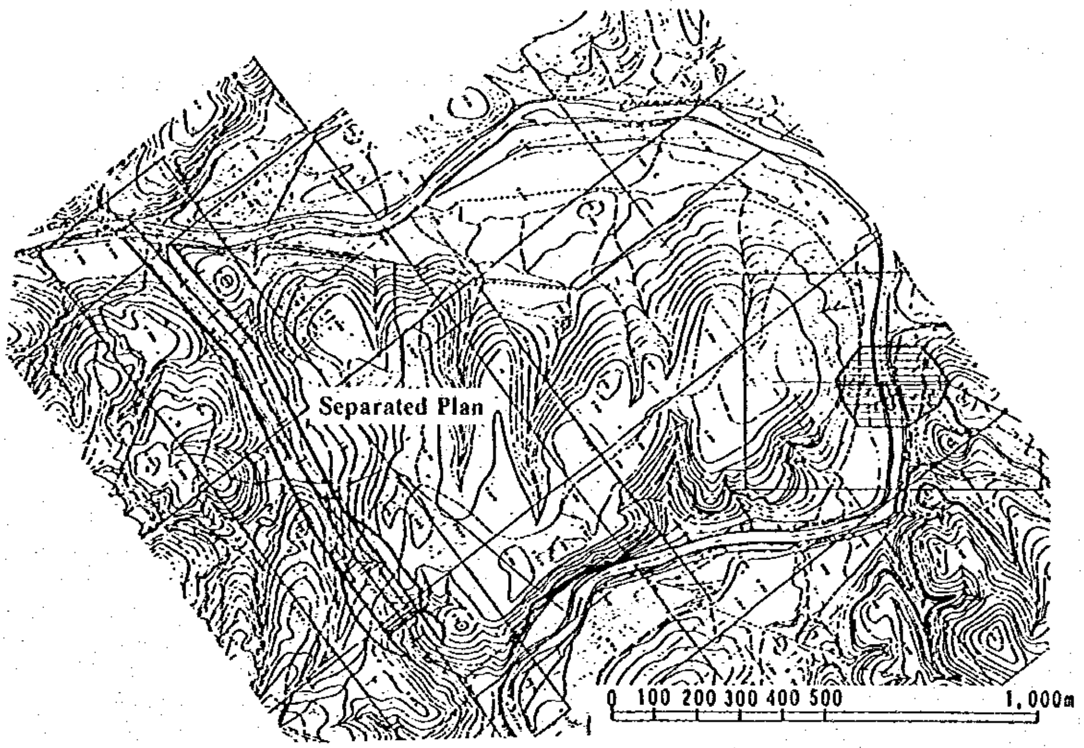


Figure-3.3 Alternative Plan (Concrete Facing Type Rock Fill Dam)

Table-3.2 Comparison of Spillway for Fill-Type Dam

Items	Left-Bank Plan	Right-Bank Plan	Separated Plan
Topography and Geology	Topographically, left bank mountain body is large enough to install spillway. There is no geological problem.	Right bank is thin ridge. Line of the ridge is inclined upstream at 30 degree from dam axis. Weathered layer is 10 - 20 m. Hill top, el. 61 m, located at the top of right abutment, is obstacle to spillway.	Location is saddle of ridge. Weathered layer is 15 m. At the downstream valley, sedimentation depth seems 15 m.
Foundation of Spillway	Inlet level is normal water level of reservoir. At the normal water level, C_L – class rock is distributed. This class rock is strong enough for foundation of spillway.		
Connection to Downstream River Channel	The center line of spillway crosses the river channel at 25 degree. No special problem.	Spilled water is directly discharged to the tributary which meets right angles to the river channel	The end of valley where spillway is installed, is flat land. No special problem.
Length of Spillway	Approx. 150 m	Approx. 100 m + 200 m training channel	Approx. 1000 m
Hydraulic Points	No special problem.	Arrangement of stilling pool is difficult due to topographic reason.	Curved chute is not appropriate to discharge supercritical flow.
Other Points	Large scale of spillway needs a large amount of excavation	Due to thin ridge and existence of lineament, there is a weak point of leakage. To prevent this leakage, there is a plan to fill the valley (upstream side of the ridge) with dumping soil material. It is not appropriate to install spillway near the dumping place.	Similar existing dams have chute with natural channel and inlet with concrete. Due to frequency of flood, large discharge and thick layer of sedimentation, natural channel – chute is not preferable.
Total Evaluation	Few problem	Large scale of spillway can not be installed.	Few merit
	O (Applicable)	X (Not applicable)	X (Not applicable)



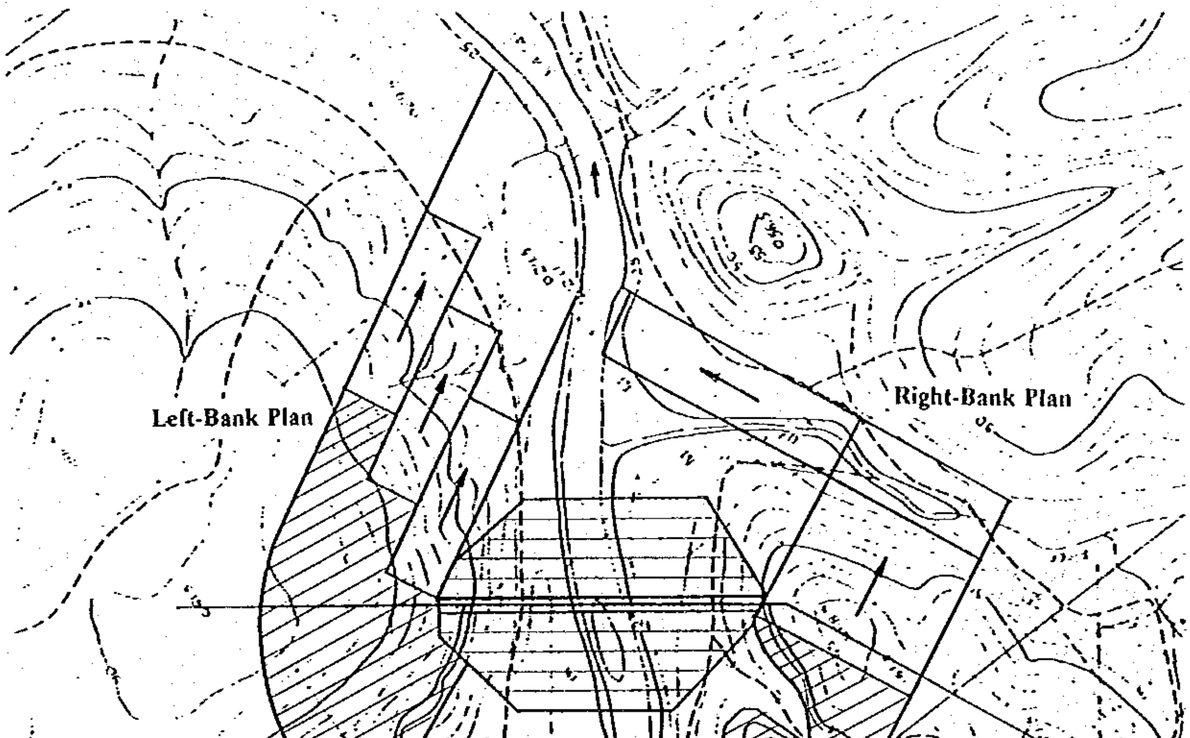


Figure-3.4 Comparison of Spillway

3.2 Design of Dam Body

(1) Design Conditions

The design conditions of the main dam is as follows:

(a) Design Water Level

1) Reservoir Water Level

- H.W.L. = EL.52.7 m
- N.W.L. = EL.47.5 m
- L.W.L. = EL.28.5 m

2) Downstream Water Level

- Normal Time: EL. 20.0 m
- Flood Time: EL. 38.2 m

(b) Elevation of Dam Top

The elevation of dam top is EL. 53.2 m, which is the larger, comparing the levels of "normal water level + concrete dam freeboard" and "flood water level + concrete dam freeboard", as shown below.

Items	Water Level	Freeboard	W/L+ Freeboard
Normal Time	N.W.L.: EL. 47.5 m	1.5 m	EL. 49.0 m
Flood Time	H.W.L.: EL. 52.7 m	0.5 m	EL. 53.2 m

(c) Dam Foundation Rock

According to the results of the geological survey covering the dam site, it is evaluated that C_M class rock at the dam site is strong enough to construct gravity concrete dam of 50 m class. Therefore, Vaza Barris Dam is installed on the C_M class rock foundation. The core boring revealed that the depth of C_M class rock is EL. 15 m at the left bank and 6 m at the right bank. It is necessary to confirm the strength of C_M class rock through the sheer test before preparation of detailed design. In this design, strength of C_M class rock in the dam site is assumed to be C = 80 ton/m^2 of cohesion and $\phi = 38^\circ$ of internal friction angle. Figure-3.5 shows relationship between rock mass class and shear strength.

(d) Dam Height

The design foundation level of C_M class rock is set at EL. 5 m. As the elevation of dam top is EL. 53.2m, the dam height is 48.2 m.

(e) Arrangement of Drain

Three (3) drains in one block (15m) were arranged at the dam foundation of 5 m downstream from the dam axis. It satisfies the drain arrangement standard, while the length between upstream edge and drains is more than 8 % of reservoir water depth (5 m / 48.2 m = 10 %).

(f) Width of Dam Top

The width of dam top is set as 5 m considering entrance of crane truck in the case that gates are installed for low flow outlet and for temporary drain hole in dam body.

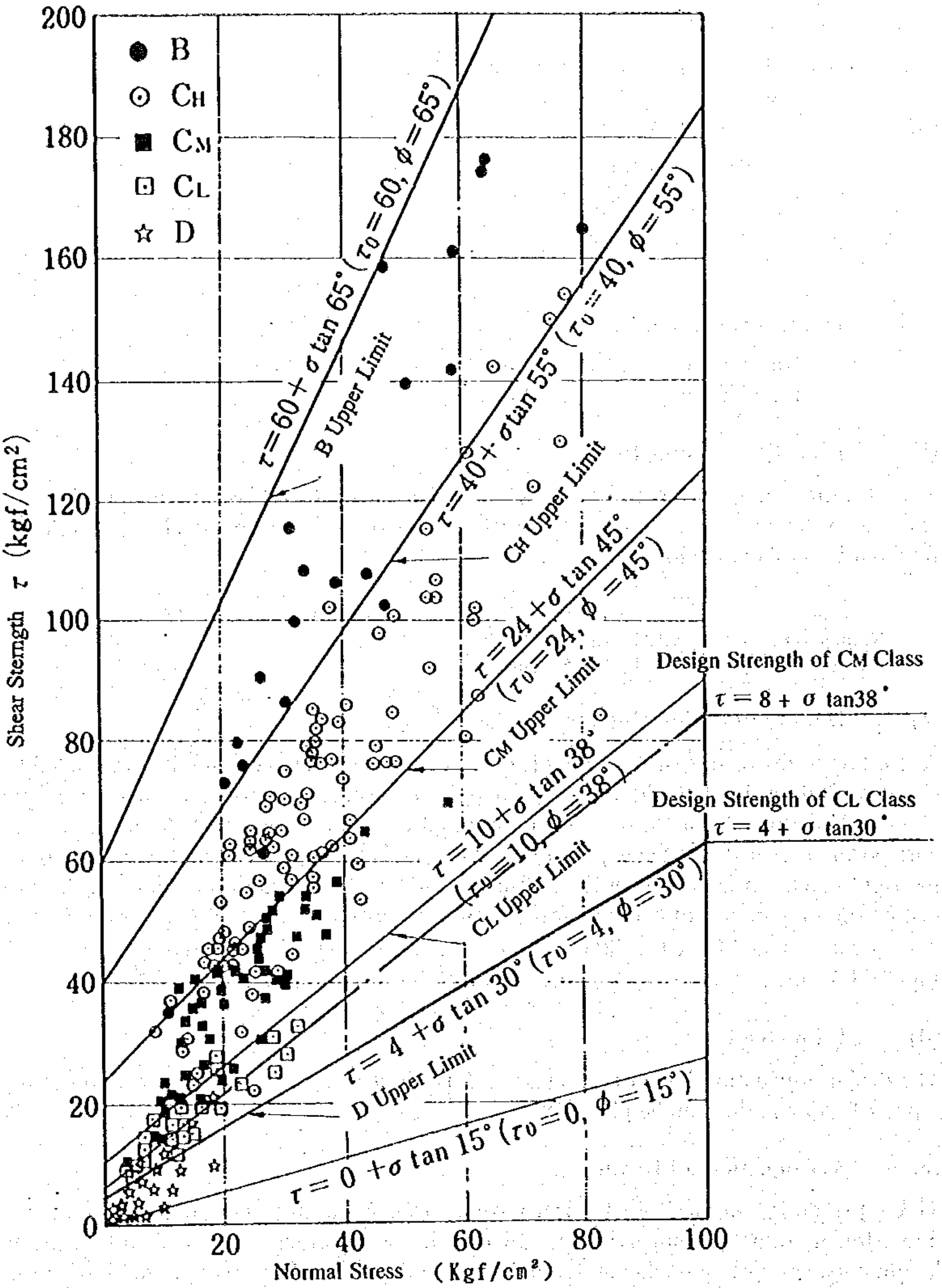


Figure-3.5 Shear Strength by Rock Class

(2) Stability Analysis

(a) Analysis Condition

Stability of the dam is analyzed for dam standard section (the maximum section of overflow section and nonoverflow section) as described in the section of "Design Criteria". The load combination of three types, namely Case of Normal Load(CCN), Case of Exceptional Load(CCE), Case of Load of Construction(CCC), are listed in the design criteria. As CCC is changeable according to construction method, this case is neglected in this Study. Experimentally it is expected that safety factor of CCC case is higher than those of CCN and CCE cases though depending on the design condition. The parameters for the calculation of CCN and CCE are shown in Table-3.3;

Table-3.3 Parameter for Stability Analysis of Dam Body

Items		Carrying Case	
		CCN	CCE
Upstream water level		EL.47.5 m (N.W.L.)	EL.52.7 m (H.W.L.)
Downstream water level		EL.20.0 m	EL.38.2 m
Sedimentation level		EL.28.6 m	EL.28.6 m
Design seismic coefficient		0	Horizontal 0.05g Vertical 0.03g
Drain	Condition	Operating	Non operating
	Position	5m from dam upstream face	
	Drainage level	EL.25 m	
Foundation design strength	C	80t/m^2	
	Ø	38°	
Unit weight	Mass concrete	2.3t/m^3	
	Water	1.0t/m^3	
	Sedimentation(sub)	0.35t/m^3	

(b) Result of stability analysis

The sectional form of the main dam, which satisfies the design standard in the dam stability analysis (see Table-3.4), was set as follows:

Upstream Slope: Vertical
Downstream Slope: 1:0.88

The allowable tensile stress of foundation rock is given by Charge Capacity / Safety Coefficient in CEMIG criteria. In order to increase the safety degree against the tensile stress within the foundation rock comprising phyllite with many joints, the sectional form of the main dam is designed so as not to cause tensile stress within the foundation rock. This study is for basic design stage and assumption above is considered enough.

Figure-3.6 and Figure-3.7 shows loads acting on the dam body by each case. Detail of the analysis is shown in case Table-3.5 to Table-3.9.

The sectional form of the dam, the drawings of Dam plan, longitudinal sections of the upstream and downstream view and standard sections, is presented in Figure-3.8, Figure-3.9 and Figure-3.10.

Table-3.4 Results of Dam Stability Analysis

Case	Section and allowable value	Safety Coefficients to Fluctuation	Safety Coefficients against the Overturning	Factor of Safety to the Sliding	Tension (t/m^2) Upstream compressive stress σvu	Tension (t/m^2) Downstream compressive stress σvd	Safety Factor by Henny
Case of Normal Load	Non-overflow section	2.53	2.76	1.61	32.23	37.63	5.1
	Overflow section	2.50	2.75	1.59	29.31	39.00	5.1
	Allowable minimum value	1.3	1.5	1.0	0	0	4
Case of Exceptional Load	Non-overflow section	1.61	1.71	2.74	0.79	48.71	5.0
	Overflow section	1.59	1.67	2.77	4.11	43.89	5.0
	Allowable minimum value	1.1	1.2	1.0	0	0	4

Nonoverflow Section

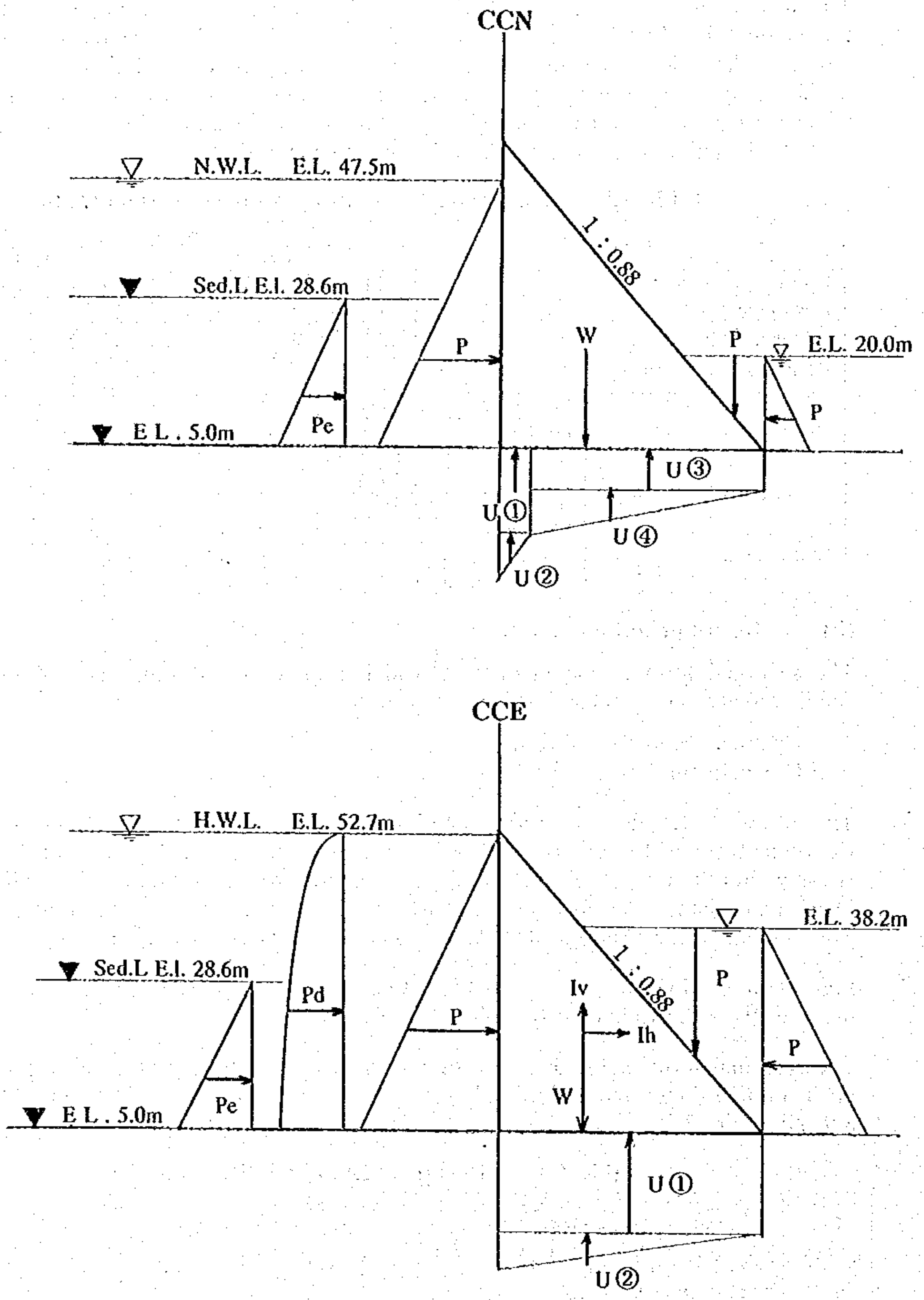
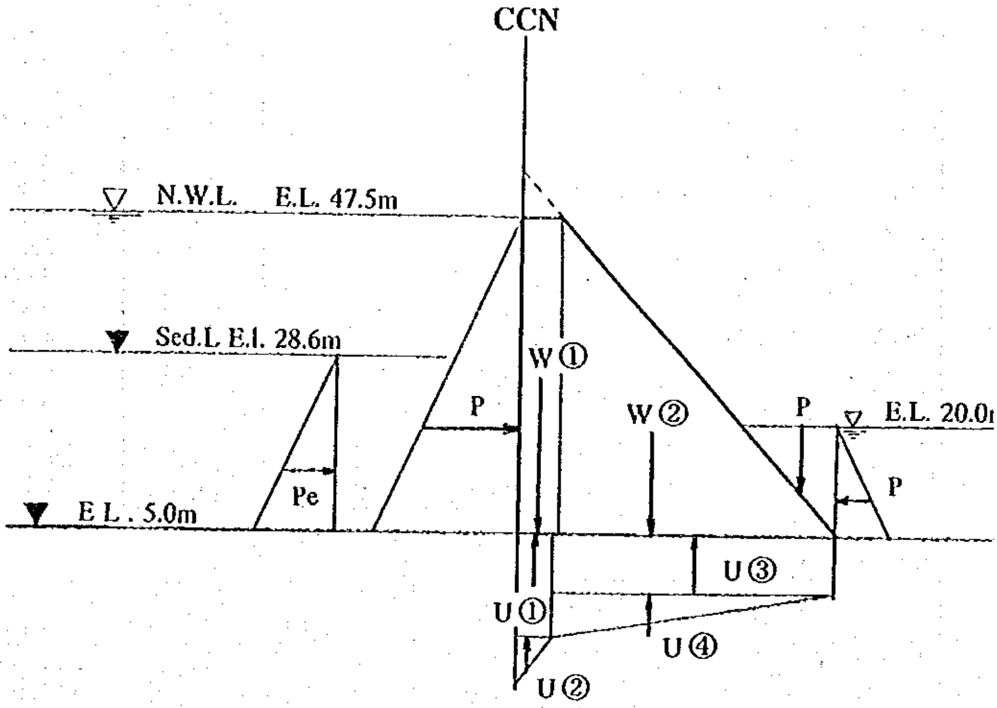


Figure-3.6 Load Acting on Non-overflow Section

Overflow Section



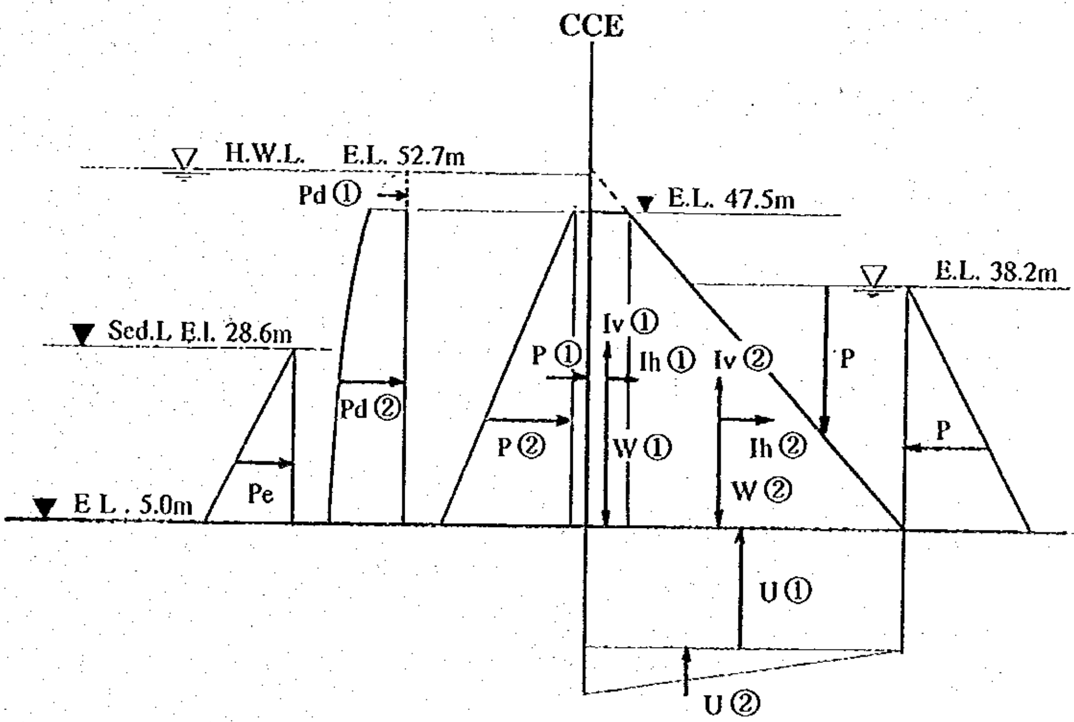


Figure-3.7 Load Acting on Overflow Section

Table-3.5 Summary of Stability Analysis of Dam Body

1. Design Load

Item		Unit	Dimension
Dam Top Level		EL.(m)	53.200
Overflow Crest Level		EL.(m)	47.500
Dam Foundation Level		EL.(m)	5.000
Downstream Slope			0.88
Design Flood Water Level		EL.(m)	52.700
Normal Water Level		EL.(m)	47.500
Sedimentation Level		EL.(m)	28.600
Downstream River Water Level	H.W.L.	EL.(m)	38.200
	N.W.L.	EL.(m)	20.000
Drain Outlet Level		EL.(m)	25.000
Distance from Upstream Face until Drain		(m)	5.000
Unit Volume Weight	Concrete	(t/m^3)	2.3
	Water	(t/m^3)	1.0
	Sedimentation	(t/m^3)	0.35
Design Seismic Coefficient	Horizontal		0.05
	Vertical		0.03
Foundation Design Strength	Cohesion	(t/m^2)	80
	Angle of Internal Friction	(deg)	38

2. Carrying Case

Item	Unit	Carrying Case	
		CCN	CCE
Upstream Water Level	EL.(m)	47.500 N.W.L.	52.700 H.W.L.
Downstream Water Level	EL.(m)	20.000	38.200
Sedimentation Level	EL.(m)	28.600	28.600
Dam Foundation Level	EL.(m)	5.000	5.000
Design Seismic Coefficient			0.05 Horizontal 0.03 Vertical
Drain Condition		Operating	Non operating

3. Results of Stability Analysis

Carrying Case	CSF	CST	FSD	σvu	σvd	n
CCN						
Nonoverflow Section	2.53	2.76	1.61	32.228	37.633	5.1
Overflow Section	2.50	2.75	1.59	29.314	38.997	5.1
Allowable Minimum Value	1.3	1.5	1.0	0.0	0.0	4.0
CCE						
Nonoverflow Section	1.61	1.71	2.74	0.792	48.710	5.0
Overflow Section	1.59	1.64	2.77	4.112	43.886	5.0
Allowable Minimum Value	1.1	1.2	1.0	0.0	0.0	4.0

Note:

CCN	:	case of normal load
CCE	:	case of exceptional load
CFS	:	safety coefficients to fluctuation
CST	:	safety coefficients against the overturning
FSS	:	factor of safety to the sliding
σvu	:	upstream compressive stress
σvd	:	downstream compressive stress
n	:	safety factor by Henny

Table-3.6 Analyzed Case ; Stability analysis for CCN of Nonoverflow Section of Main Dam

1. Carrying Case

Nonoverflow Section
CCN

2. Design Condition

Item		Dimension
Height of Dam		48.200
Downstream Slope		1 : 0.88
Upstream Head		42.500
Downstream Head		15.000
Depth of Sedimentation		23.600
Drain Condition		Operating
Uplift	Heel of Dam	42.500
	Drain section	27.425
	Toe of Dam	15.000
Design Seismic Coefficient	Horizontal	—
	Vertical	—
Dynamic Water Pressure	Cm	0.570
	C	0.570
	Pd	0.000

3. Working Load

Item		V (t)	Lv (m)	V × Lv (t×m)	H (t)	Lh (m)	H × Lh (t×m)
Own Weight		2,351.119	14.139	33,242.472			
Seismic Inertia Force							
Hydrostatic Pressure	Upstream				903.125	14.167	12,794.572
	Downstream	99.000	38.016	3,763.584	-112.500	5.000	-562.500
	Subtotal	99.000		3,763.584	790.625		12,232.072
Pressure due to Sedimentation					97.468	7.867	766.781
Dynamic Water Pressure							
Uplift	①	-137.125	2.500	-342.813			
	②	-37.688	1.667	-62.826			
	③	-561.240	23.708	-13,305.878			
	④	-232.447	17.472	-4,061.314			
	Subtotal	-968.500		-17,772.831			
Total		1,481.619		19,233.225	888.093		12,998.853

4. Stability Analysis

Summitry of the Gravitational Forces	$\Sigma V =$	2450.119 t	
Summitry of the Uplift Forces	$\Sigma U =$	968.5 t	
Safety Coefficients to Fluctuation	$C.S.F = \Sigma V / \Sigma U =$	2.53 $\geqq 1.3$	O.K
Summitry of Stabilizer Moments	$\Sigma Me =$	50567.409 t × m	
Summitry of Overturning Moments	$\Sigma Mt =$	18335.331 t × m	
Safety Coefficients against the Overturning	$C.S.T = \Sigma Me / \Sigma Mt =$	2.76 $\geqq 1.5$	O.K
Safety Coefficient relatively Ø	CSDØ =	2	
Safety Coefficient relatively C	CSDc =	4	
Normal Strength	Ni =	1481.619 t	
Angle of Friction	Øi =	38 °	
Cohesion	Ci =	80 t/m²	
Area of Contact	Ai =	42.416 m² (unit width)	
Forces Parallel to the Sliding Surface	Ti =	888.093 t	
Factor of Safety to the Sliding FSS =	(ΣNi × tanØi / CSDØ + Σc × Ai / CSDc) / Σ ti =	1.61 $\geqq 1.0$	O.K
Total of Moments	$\Sigma M = \Sigma (V \times L) + \Sigma (H \times h) =$	32232.078 t × m	
Working Point of Resultant Force	$d = \Sigma M / \Sigma V =$	21.7546333 m	
Length of Dam Base	$B =$	42.416 m	
	$e = B / 2 - d =$	-0.547 $\leqq B/6=$ 7.069	
Upstream Compressive Stress	$\sigma u = (V / B)(1 + 6e / B) =$	32.228 $\geqq 0$ t/m²	O.K
Downstream Compressive Stress	$\sigma d = (V / B)(1 - 6e / B) =$	37.633 $\geqq 0$ t/m²	O.K
Safety Factor by Henny	$n = (f \times \Sigma V + Ci \times B) / \Sigma H =$	5.1 $\geqq 4$	O.K

Table-3.7 Analyzed Case; Stability Analysis for CCE of Nonoverflow Section of main Dam

1. Carrying Case
Nonoverflow Section
CCE

2. Design Condition

Item		Dimension
Height of Dam		48.200
Downstream Slope		1 : 0.88
Upstream Head		47.700
Downstream Head		33.200
Depth of Sedimentation		23.600
Drain Condition		Non operating
Uplift	Heel of Dam	47.700
	Drain section	—
	Toe of Dam	33.200
Design Seismic Coefficient	Horizontal	0.050
	Vertical	0.030
Dynamic Water Pressure	Cm	0.570
	C	0.570
	Pd	1.359

3. Working Load

Item		V (t)	Lv (m)	V × Lv (t × m)	H (t)	Lh (m)	H × Lh (t × m)
Own Weight		2,351.119	14.139	33,242.472			
Seismic Inertia Force		-70.534	14.139	-997.280	117.556	16.067	1,888.772
Hydrostatic Pressure	Upstream				1,137.645	15.900	18,088.556
	Downstream	484.986	32.677	15,848.049	-551.120	11.067	-6,099.245
	Subtotal	484.986		15,848.049	586.525		11,989.311
Pressure due to Sedimentation					97.468	7.867	766.781
Dynamic Water Pressure					47.062	—	924.544
Uplift	①	-1,408.211	21.208	-29,865.339			
	②	-307.516	14.139	-4,347.969			
	Subtotal	-1,715.727		-34,213.308			
Total		1,049.844		13,879.933	848.611		15,569.408

4. Stability Analysis

Summitry of the Gravitational Forces	$\Sigma V =$	2,765.571 t	
Summitry of the Uplift Forces	$\Sigma U =$	1,715.727 t	
Safety Coefficients to Fluctuation	$C.S.F = \Sigma V / \Sigma U =$	1.61 $\geqq 1.1$	O.K
Summitry of Stabilizer Moments	$\Sigma Me =$	70,759.174 t × m	
Summitry of Overturning Moments	$\Sigma Mt =$	41,309.833 t × m	
Safety Coefficients against the Overturning	$C.S.T = \Sigma Me / \Sigma Mt =$	1.71 $\geqq 1.2$	O.K
Safety Coefficient relatively Ø	CSDØ =	1.3	
Safety Coefficient relatively C	CSDc =	2.0	
Normal Strength	Ni =	1,049.844 t	
Angle of Friction	Øi =	38 °	
Cohesion	Ci =	80 t/m^2	
Area of Contact	Ai =	42.416 m^2 (unit width)	
Forces Parallel to the Sliding Surface	Ti =	848.611 t	
Factor of Safety to the Sliding FSS =	$(\Sigma ni \times \tan Øi / CSDØ + \Sigma ci \times Ai / CSDc) / \Sigma ti =$	2.74 $\geqq 1.0$	O.K
Total of Moments	$\Sigma M = \Sigma (V \times L) + \Sigma (H \times h) =$	29,449.341 t × m	
Working Point of Resultant Force	$d = \Sigma M / \Sigma V =$	28.051 m	
Length of Dam Base	$B =$	42.416 m	
	$e = B / 2 - d =$	-6.843 $\leqq B/6 =$ 7.07	
Upstream Compressive Stress	$\sigma u = (V / B)(1 + 6e / B) =$	0.792 $\geqq 0$ t/m^2	O.K
Downstream Compressive Stress	$\sigma d = (V / B)(1 - 6e / B) =$	48.710 $\geqq 0$ t/m^2	O.K
Safety Factor by Henny	$N = (f \times \Sigma V + Ci \times B) / \Sigma H =$	5.0 $\geqq 4$	O.K

Table-3.8 Analyzed Case; Stability Analysis for CCN of Overflow Section of main Dam

1. Carrying Case

Overflow Section
CCN

2. Design Condition

Item		Dimension
Height of Dam		42.500
Crest Width		5.016
Downstream Slope		1 : 0.88
Upstream Head		42.500
Downstream Head		15.000
Depth of Sedimentation		23.600
Drain Condition		Operating
Uplift	Heel of Dam	42.500
	Drain section	27.425
	Toe of Dam	15.000
Design Seismic Coefficient	Horizontal	—
	Vertical	—
Dynamic Water Pressure	Cm	0.570
	C	0.570
	Pd	0.000

3. Working Load

Item		V (t)	Lv (m)	V × Lv (t × m)	H (t)	Lh (m)	H × Lh (t × m)
Own Weight	①	490.314	2.508	1,229.708			
	②	1,827.925	17.483	31,957.613			
	Subtotal	2,318.239		33,187.321			
Seismic Inertia Force							
Hydrostatic Pressure	Upstream				903.125	14.167	12,794.572
	Downstream	99.000	38.016	3,763.584	-112.500	5.000	-562.500
	Subtotal	99.000		3,763.584	790.625		12,232.072
Pressure due to Sedimentation					97.468	7.867	766.781
Dynamic Water Pressure							
Uplift	①	-137.125	2.500	-342.813			
	②	-37.688	1.667	-62.826			
	③	-561.240	23.708	-13,305.878			
	④	-232.447	17.472	-4,061.314			
	Subtotal	-968.500		-17,772.831			
Total		1,448.739		19,178.074	888.093		12,998.853

4. Stability Analysis

Summitry of the Gravitational Forces	$\Sigma V =$	2,417.239 t	
Summitry of the Uplift Forces	$\Sigma U =$	968.500 t	
Safety Coefficients to Fluctuation	$C.S.F = \Sigma V / \Sigma U =$	2.50 $\geqq$ 1.3	O.K
Summitry of Stabilizer Moments	$\Sigma Me =$	50,512.258 t × m	
Summitry of Overturning Moments	$\Sigma Mt =$	18,335.331 t × m	
Safety Coefficients against the Overturning	$C.S.T = \Sigma Me / \Sigma Mt =$	2.75 $\geqq$ 1.5	O.K
Safety Coefficient relatively Ø	$CSD\varnothing =$	2.0	
Safety Coefficient relatively C	$CSDc =$	4.0	
Normal Strength	$Ni =$	1,448.739 t	
Angle of Friction	$\varnothing i =$	38 °	
Cohesion	$Ci =$	80 t/m²	
Area of Contact	$Ai =$	42.416 m² (unit width)	
Forces Parallel to the Sliding Surface	$Ti =$	888.093 t	
Factor of Safety to the Sliding FSS =	$(\Sigma Ni \times \tan\varnothing i / CSD\varnothing + \Sigma ci \times Ai/CSDc) / \Sigma ti =$	1.59 $\geqq$ 1.0	O.K
Total of Moments	$\Sigma M = \Sigma (V \times L) + \Sigma (H \times h) =$	32,176.927 t × m	
Working Point of Resultant Force	$d = \Sigma M / \Sigma V =$	22.210 m	
Length of Dam Base	$B =$	42.416 m	
	$e = B/2 - d =$	-1.002 $\leqq B/6 =$	7.07
Upstream Compressive Stress	$\sigma u = (V / B)(1 + 6e / B) =$	29.314 $\geqq$ 0 t/m²	O.K
Downstream Compressive Stress	$\sigma d = (V / B)(1 - 6e / B) =$	38.997 $\geqq$ 0 t/m²	O.K
Safety Factor by Henny	$n = (f \times \Sigma V + Ci \times B) / \Sigma H =$	5.1 $\geqq$ 4	O.K

Table-3.9 Analyzed Case; Stability Analysis for CCE of Overflow Section of main Dam

1. Carrying Case

Overflow Section
CCE

2. Design Condition

Item		Dimension
Height of Dam		42.500
Crest Width		5.016
Downstream Slope		1 : 0.88
Upstream Head		47.700
Downstream Head		33.200
Depth of Sedimentation		23.600
Drain Condition		non operating
Uplift	Heel of Dam	47.700
	Drain section	——
	Toe of Dam	33.200
Design Seismic Coefficient	Horizontal	0.050
	Vertical	0.030
Dynamic Water Pressure	Cm	0.570
	C	0.188
	Pd	0.449

3. Working Load

Item		V(t)	Lv(m)	V×Lv (t×m)	H(t)	Lh(m)	H×Lh (t×m)
Own Weight	①	490.314	2.508	1,229.708			
	②	1,827.925	17.483	31,957.613			
	Subtotal	2,318.239		33,187.321			
Seismic Inertia Force	①	-14.709	2.508	-36.890	24.516	21.250	520.965
	②	-54.838	17.483	-958.733	91.396	1.373	125.487
	Subtotal	-69.547		-995.623	115.912		646.452
Hydrostatic Pressure	Upstream①				221.000	21.250	4,696.250
	Upstream②				903.125	14.167	12,794.572
	Downstream	484.986	32.677	15,848.049	-551.120	11.067	-6,099.245
	Subtotal	484.986		15,848.049	573.005		11,391.577
Pressure due to Sedimentation					97.468	7.867	766.781
Dynamic Water Pressure	-①				-1.695	——	-3.630
	①+②				47.062	——	924.544
	Subtotal				45.367		920.914
Uplift	①	-1,408.211	21.208	-29,865.339			
	②	-307.516	14.139	-4,347.969			
	Subtotal	-1,715.727		-34,213.308			
Total		1,017.951		13,826.439	831.752		13,725.724

4. Stability Analysis

Summitry of the Gravitational Forces	ΣV =	2,733.678 t	
Summitry of the Uplift Forces	ΣU =	1,715.727 t	
Safety Coefficients to Fluctuation	C.S.F = ΣV / ΣU =	1.59 ≧1.1	O.K
Summitry of Stabilizer Moments	ΣMe =	67,939.425 t × m	
Summitry of Overturning Moments	ΣMt =	41,311.806 t × m	
Safety Coefficients against the Overturning	C.S.T = ΣMe / ΣMt =	1.64 ≧1.2	O.K
Safety Coefficient relatively Ø	CSDØ =	1.3	
Safety Coefficient relatively C	CSDc =	2.0	
Normal Strength	Ni =	1,017.951 t	
Angle of Friction	Øi =	38 °	
Cohesion	Ci =	80 t/m²	
Area of Contact	Ai =	42.416 m² (unit width)	
Forces Parallel to the Sliding Surface	Ti =	831.752 t	
Factor of Safety to the Sliding FSS =	(ΣNi × tanØi / CSDØ + Σci × Ai/CSDc) / Σti =	2.77 ≧1.0	O.K
Total of Moments	ΣM = Σ (V × L) + Σ (H × h) =	27,552.163 t × m	
Working Point of Resultant Force	d = ΣM / ΣV =	27.066 m	
Length of Dam Base	B =	42.416 m	
	e = B / 2 - d =	-5.858 ≦B/6=	7.07
Upstream Compressive Stress	σu = (V / B) (1 + 6e / B) =	4.112 ≧0 t/m²	O.K
Downstream Compressive Stress	σd = (V / B) (1 - 6e / B) =	43.886 ≧0 t/m²	O.K
Safety Factor by Henny	n = (f × ΣV + Ci × B) / ΣH=	5.0 ≧4	O.K

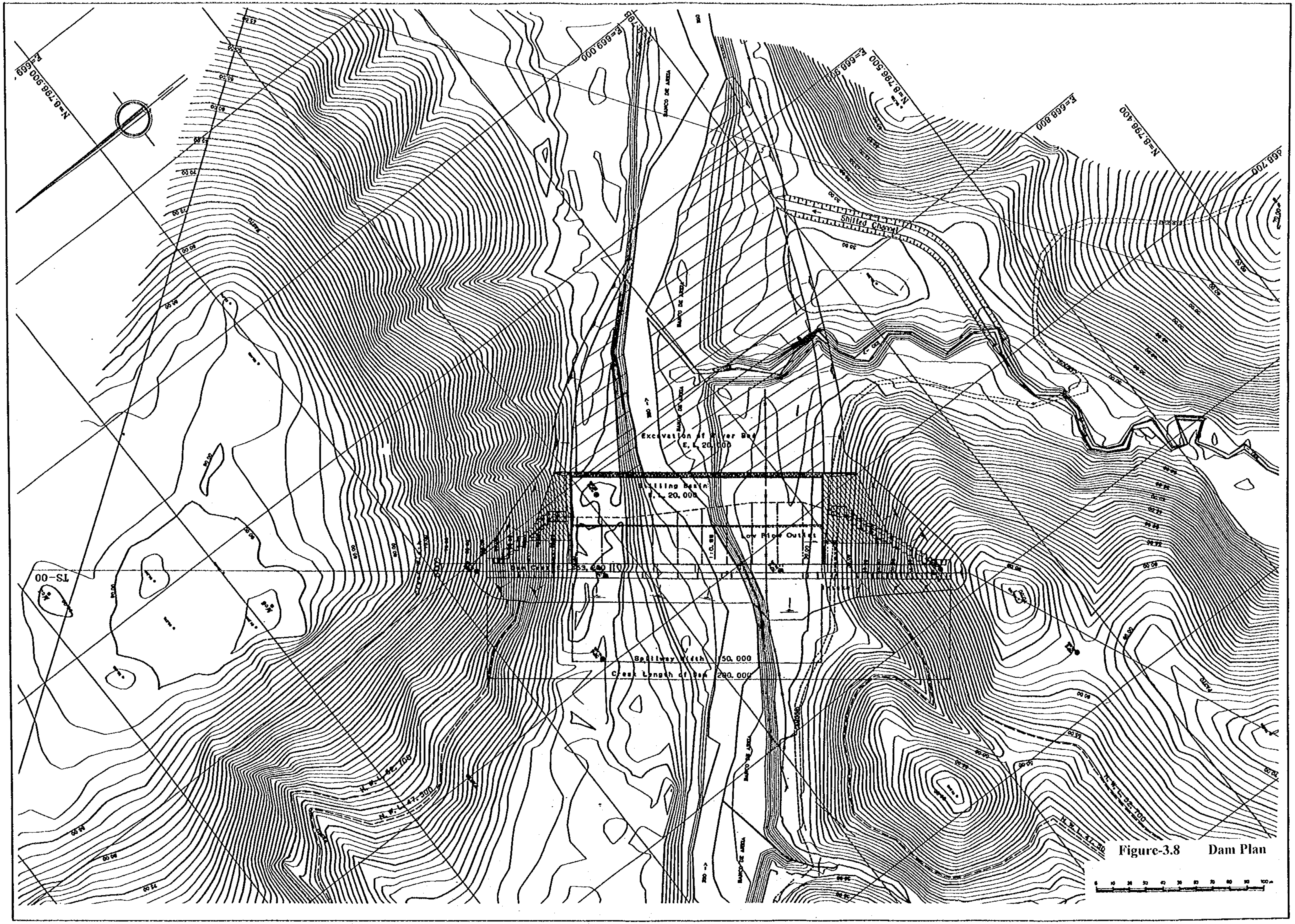
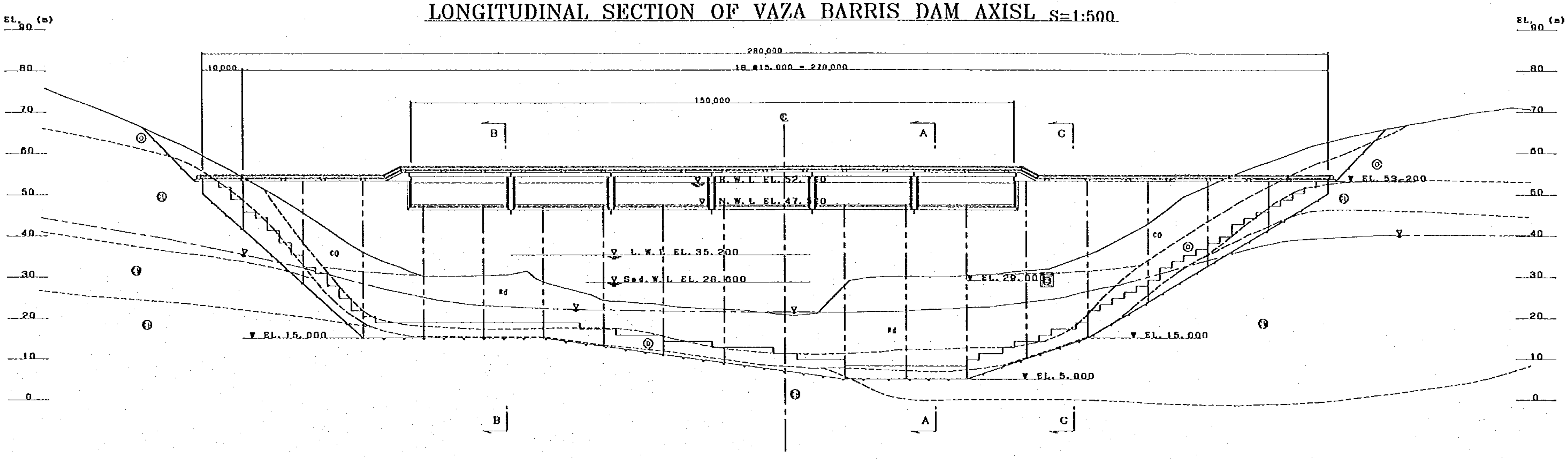
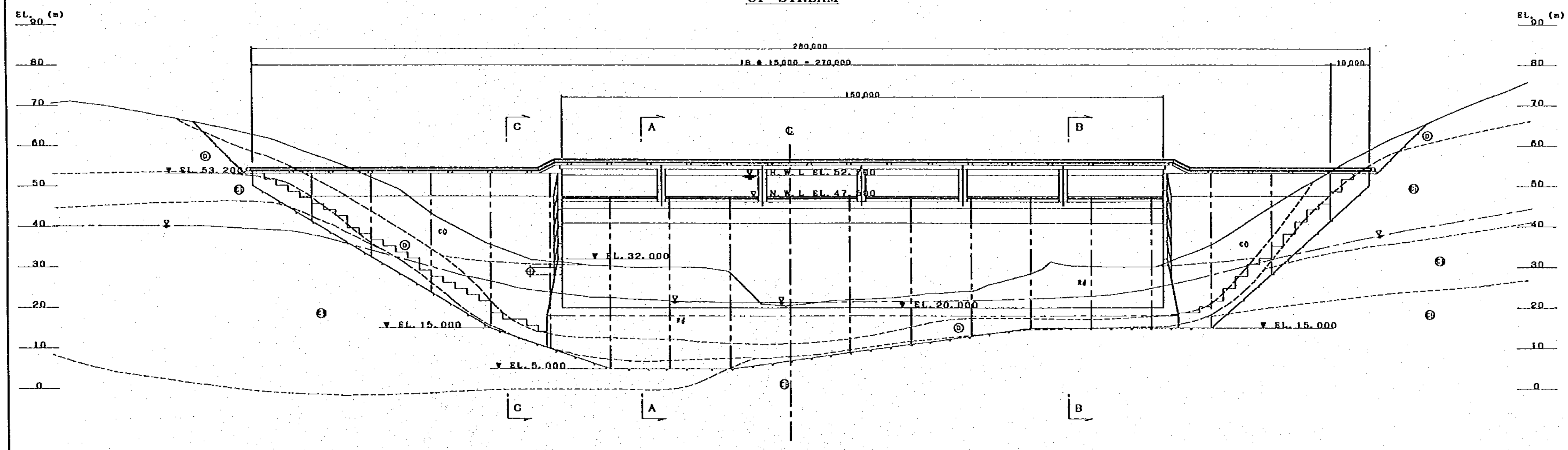


Figure-3.8 Dam Plan

LONGITUDINAL SECTION OF VAZA BARRIS DAM AXISL S=1:500



UP STREAM



DOWN STREAM

Figure-3.9 Dam Longitudinal Section of Upstream and Downstream View

CROSS SECTION OF VAZA BARRIS DAM AXISL S=1:500

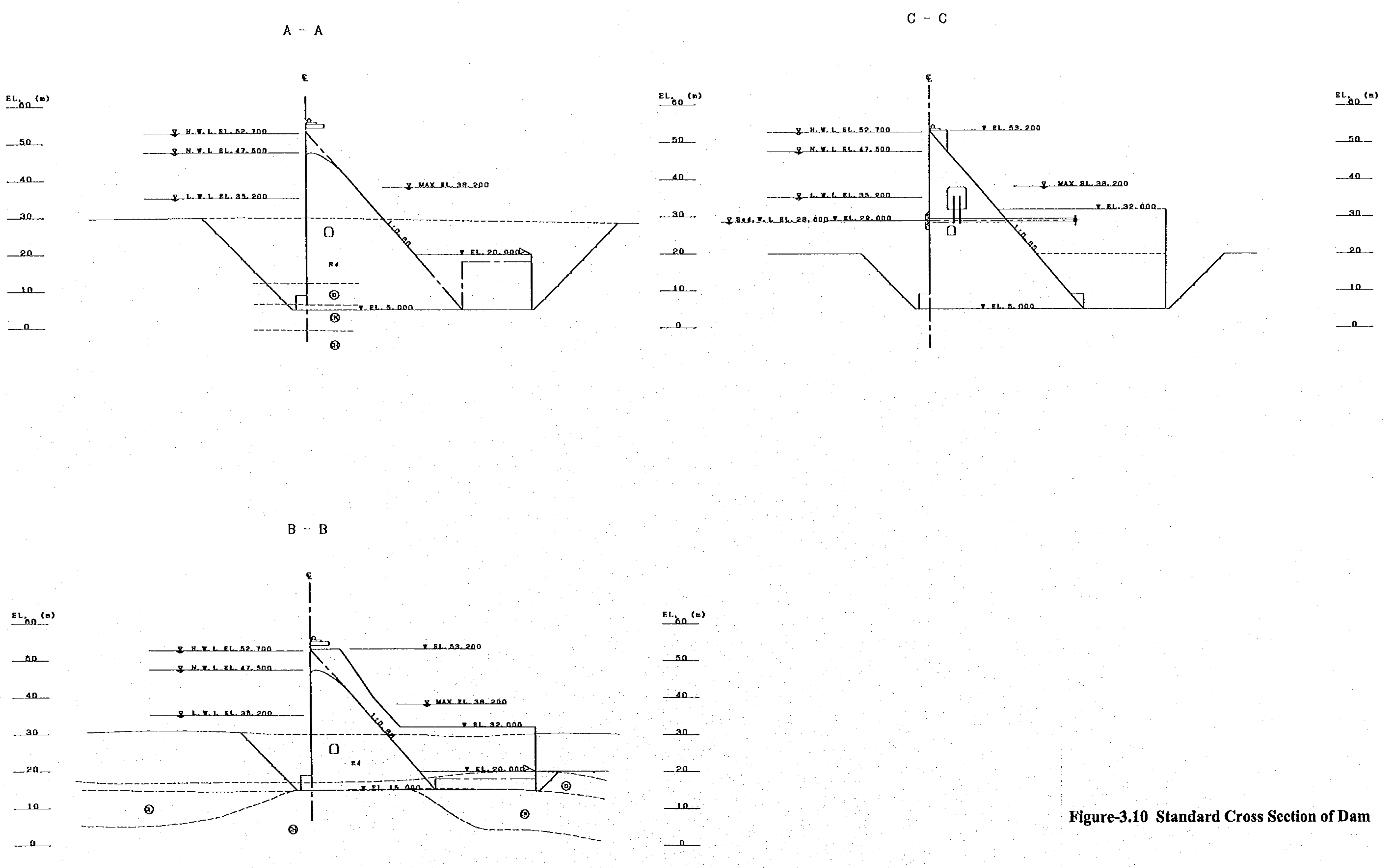


Figure-3.10 Standard Cross Section of Dam

3.3 Design of Spillway

(1) Width and Water Depth of Overflow Section

The width of overflow section (L) is set as 150 m, equivalent to the width of the current river section. As described in the above section, the depth of overflow section is given as 5.2 m at the design flood discharge Q = 3,600 m^3/s. Beside, the influence of the crest bridge pier was taken into account on the calculation of over flow section.

$$Q = CLH^{1.5}$$
$$L = L' - 2\,(Ka + nKp)\,H$$
$$C = 1.704 \times (1 + 0.648\,(H/R))^{0.5}$$

$$R = 0.9173Hd = 0.9173 \times 5.2 = 4.770 \text{ m}$$
$$C = 1.704 \times (1+0.648 \times (5.2/4.770))^{0.5} = 2.2$$
$$n = 5$$
$$L' = 150 - (2 \times 5) = 140 \text{ m}$$
$$Ka = 0.12,\ Kp = 0.01$$
$$L = 140 - 2 \times (0.12 + 5 \times 0.01) \times 5.2 = 138.2 \text{ m}$$
$$Q = 2.2 \times 138.2 \times 5.2^{1.5} = 3605 \text{ m}^3/\text{s} > 3600 \text{m}^3/\text{s} \quad \text{ok}$$

(2) Shape of Overflow Section

As the design water depth of overflow section is high as 5.2 m, the shape of overflow section is designed giving larger discharge coefficient so that the negative pressure does not occur on the overflow section. According to the standard of USCE (US Army, Corps of Engineer), the standard overflow section is employed in this design.

(3) Height of Training Wall

The heights of training wall (water depth + free board) are calculated as below: each figure is perpendicular to channel surface.

Table-3.10 Height of Training Wall

Location	Head H (m)	Velocity V (m/s)	Water depth h (m)	Freeboard Fb (m)	h + Fb (m)	Height of Training Wall (m)
EL. 40 m	12.7	14.2	1.69	1.23	2.92	3.0
EL. 30 m	22.7	19.0	1.26	1.36	2.62	2.7
EL. 20 m	32.7	22.8	1.05	1.46	2.51	2.6

Note:
- H : Reservoir Level – Location Level
- V : $0.9 \times (2 \times g \times H)^{0.5}$
- h : 3600 / 150 / V
- Fb : $0.6 + 0.037 \times V \times h^{(1/3)}$

(4) Energy Dissipater

The design conditions for the energy dissipater are as follows:

- Design discharge for energy dissipater: 1,200 m^3/s (100 year return period)
- Difference is large between current riverbed elevation: EL. 20 m and dam base rock elevation: EL. 5 m.
- Layer of deposits in riverbed is 10-20 m in thickness.
- Water depth in flood time is high. In design flood: 1,200 m^3/s, water depth is 10.8 m (EL. 30.8 m).
- Width of energy dissipater is 150 m as same width as the overflow section of the spillway.

As the riverbed deposit is thick and water depth is deep, hydraulic jump basin (with chute blocks and sills) equivalent to Basin II from USBR standard. Dimensions of the energy dissipater are designed as follows:

- Elevation of basin: EL. 20 m
- Width of basin: 150 m
- Length of basin: 26 m
- Conjugate depth of jump: 5.8 m

As the water depth of downstream river (10.8 m) is higher than the conjugate depth of jump (5.8 m), hydraulic jump occurs in the basin and flood stream is dissipated.

Design discharge; Q = 1200m³/s. Overflow depth during water flowing down is calculated as $H = (1200/2.2/140)^{(2/3)} = 2.476$m referring to the formula of $Q = CLH^{1.5}$.
Consequently, reservoir water level is EL.47.5m+2.476 = EL.50m.
Water head from the apron h = EL.50m-EL.20m = 30m.

Corresponding above, water depth at the tip of the energy dissipater; d1, velocity;V1, Froude number ; F1 are calculated as follows;

$V1 = 0.9 \times (2 \times g \times h)^{0.5} = 21.8$m/s.
$d1 = 1200 / 150 / 21.8 = 0.37$m.
$F1 = V1 / (g \times d1)^{0.5} = 21.8 / (9.8 \times 0.37)^{0.5} = 11.4$.

Water depth after hydraulic jump ; d2 is,
$d2 = 0.5 \times d1 \times ((1+8\times F1^2)^{0.5}-1) = 0.5 \times 0.37 \times ((1+8\times 11.4^2)^{0.5}-1) = 5.8$m.

Length of basin ; L is,
$L = 4.5 \times d2 = 4.5 \times 5.8 = 26$ m.

(5) Height of Basin Wall

The elevation of the basin wall is set as EL. 32 m considering water depth (30.8 m for 100 year return period) of downstream river and some freeboard. Higher elevation of the basin wall than river water depth prevents water intrusion from side to basin. Stream in basin can flow straightly as two-dimensional flow from upstream to downstream.

3.4 Design of Low Flow Outlet

Low flow outlet is composed of conduit and discharge regulating valve. To take water effectively, the low flow inlet is installed at the lower elevation where reservoir sedimentation does not disturb. According to the reservoir operation plan, the reservoir sedimentation level (L.W.L) is EL. 28.6 m. The level of the inlet is set at EL. 29 m to effectively release design low water and discharge density layer. The occurrence of the salinity water layer is not confirmed yet. As the design water head is 25.5 m in this case, high-pressure outlet facility is employed. Water from the outlet is discharged to the stilling basin. The scale of the low flow outlet, after the hydraulic study, is set as 250 mm of the gate diameter and 800 mm of the low flow pipe.

Table-3.11 shows hydraulics calculation.

3.5 Design of Foundation Treatment

The foundation rock is judged to be considerably impervious according to the result of the Lugeon tests in boreholes which shows that Lugeon Values of the foundation rock classified into more than C_M class are usually less than 2Lu. However these data are so far not enough, thus standard curtain grouting method should be designed in this study. Area to be grouted is shown in Figure-3.11.

Table-3.11 Calculation for Low Flow Discharge Facilities

Reservoir Level	EL.(m)	Normal Water level					Low water Level					Remark
		47.5	47.5	47.5	47.5	47.5	35.2	35.2	35.2	35.2	35.2	
Gate												
Diameter of Gate	Ø (mm)	350	300	250	200	150	350	300	250	200	150	
Open Rate of Gate	(%)	38	46	60	90	100	55	69	100	100	100	
Coefficient of Discharge	C	0.250	0.328	0.472	0.752	0.820	0.421	0.564	0.820	0.820	0.820	
Area	$A0(m^2)$	0.096	0.071	0.049	0.031	0.018	0.096	0.071	0.049	0.031	0.018	
Low Flow Pipe												
Roughness Coefficient	n	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.012	Steel Pipe
Diameter of Pipe	D1(mm)	800	800	800	800	800	800	800	800	800	800	
Length of Pipe	L1(m)	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	
Area	$A1(m^2)$	0.502	0.502	0.502	0.502	0.502	0.502	0.502	0.502	0.502	0.502	
Coefficient of Friction Loss	f1	0.724	0.724	0.724	0.724	0.724	0.724	0.724	0.724	0.724	0.724	
$f1/A1^2$		2.873	2.873	2.873	2.873	2.873	2.873	2.873	2.873	2.873	2.873	
$f1/A1^2 \times 0.2$		0.575	0.575	0.575	0.575	0.575	0.575	0.575	0.575	0.575	0.575	20% of Friction Loss
Summitry of Loss		3.448	3.448	3.448	3.448	3.448	3.448	3.448	3.448	3.448	3.448	
Downstream Water Level	EL.(m)	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	
Head	h(m)	18.5	18.5	18.5	18.5	18.5	6.2	6.2	6.2	6.2	6.2	
Velocity	V(m/s)	0.9	0.9	0.9	0.9	0.6	0.9	0.9	0.9	0.6	0.3	
Discharge	$Q(m^3/s)$	0.456	0.443	0.440	0.443	0.281	0.444	0.440	0.442	0.280	0.163	
Design Discharge	$Q0(m^3/s)$	0.440	0.440	0.440	0.440	0.440	0.440	0.440	0.440	0.440	0.440	
Evaluation		OK	OK	OK	OK	NG	OK	OK	OK	NG	NG	

The combination of Ø800 pipe and Ø250 gate satisfies the design discharge of .44m³/s in both normal water level and the lowest water level.

Note:

f1 : $(124.5 \times n^2 \times L1) / D^{(4/3)}$

A1 : $3.14 \times D^{0.5}$

Q : $C \times A0 \times ((2 \times g \times h) / (1 + C^2 \times A0^2 \times \Sigma(f1/A1^2)))^{0.5}$

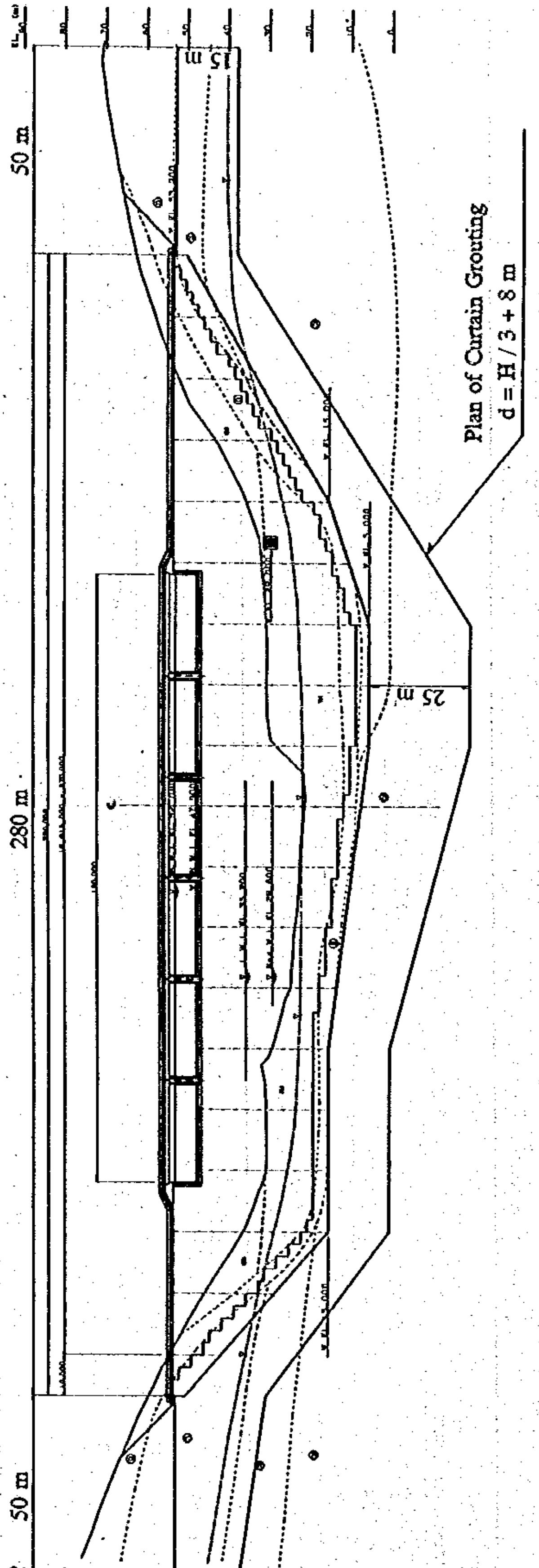


Figure-3.11 Foundation Treatment

CHAPTER 4 DESIGN OF CHECK DAM

4.1 Design Condition

(1) Type of Check Dam

Type of the check dam is set as a gravity concrete dam because of having advantages on workability of construction and resistance to flood with sediment.

(2) Location of Check Dam and Top of Dam

The location is the upstream end of reservoir where the design reservoir volume is secured at the normal water level. Also, the location is decided from viewpoint of topography and geology. The proposed location is located 29.5 km far upstream from Vaza Barris Dam. The elevation of the top of dam is EL. 63 m to secure the design sediment volume of 10,000,000 m^3.

(3) Dam Foundation and Dam Height

According to the core boring survey result at the check dam site, C_L-class rock and C_H-class rock lie on EL. 43 m and 41 m respectively. C_M-class rock is not identified at the site. The check dam is put on the C_L-class rock. Therefore, the check dam height is 20 m while the elevation of dam top is EL. 63 m. C_L-class rock is strong enough to construct this class of a concrete dam.

4.2 Design of Waterway

Considering the width of the current river, the width of waterway is set as 70 m. The water depth is 4.95 m to pass the design discharge 1,400 m^3/s (120% flood discharge of 100 year return period). The height of the water way is set as 6 m adding a freeboard to over flow depth, consequently the elevation of the top of dam is set as EL. 69 m.

Discharge capacity of Waterway is calculated using the following formula.

$$Q=(0.71h+1.77B)\times h^{1.5}$$

where

Q	:	discharge capacity
h	:	overflow depth
B	:	width of waterway

When B=70m, h=4.95m, $Q=(0.71\times 4.95+1.77\times 70)\times 4.95^{1.5}=1403 > 1400 m^3/s$ ok

4.3 Design of the Check Dam Body

(1) Design Condition

As the dam height is 20 m, the same stability analysis as that of a normal dam was carried out in the stability analysis of the check dam. The design of the check dam is different from a normal dam in the viewpoints of follows:

1) A check dam is planned to have sediment at the level of the waterway top. Since river water including sediment load flows down through a downstream face of the dam, a downstream slope should be steep to avoid abrasion by sediment. The standard downstream slope of a check dam is employed as 1:0.2 following the Japan standard.

2) Curtain grouting for water cutoff is not designed because water storage function is not needed for the check dam

Stability for the check dam body was analyzed following the design standard by CEMIG. Analysis cases are set as same as that of the main dam and the analysis section is set at over the flow section.

Table-4.1 Parameters for Stability Analysis of Check Dam

Item		Carrying case: CCN	Carrying case: CCE
Upstream water level		EL.63.0 m	EL.67.95 m
Downstream water level		EL.47.5 m	EL.50.3 m
Sedimentation level		EL.63.0 m	EL.63.0 m
Design seismic coefficient		0	Horizontal 0.05g Vertical 0.03g
Drain		None	
Uplift		Due to the lack of shielding line, Japan standard: Standard for River and Sediment Control Work is adopted to decide uplift distribution. upstream : (hd + (hu - hd)/3) γw downstream : hd × γw where: hu = upstream head, hd = downstream head, γw = unit weight of water	
Shear strength of Rock foundation (C_L class)	C	40t/m²	
	Ø	30°	
Unit weight	Mass concrete	2.3t/m³	
	Water	1.0t/m³	
	Sedimentation(sub)	0.35t/m³	

The sectional form of the check dam body, which satisfies the design standard in the dam stability analysis (see Table-4.2) as set as follows:
Loads acting on the dam body is shown in Figure-4.1 and the content of the calculation is shown in Table-4.3 to Table-4.5.

- Width of Waterway: 4 m
- Upstream Slope: 1:0.53
- Downstream Slope: 1:0.29

The dam plan and structural drawings are shown in Figure-4.2 and Figure-4.3

Table-4.2 Results of Check Dam Stability Analysis

Case	Section and Allowable Value	Safety Coefficients to Fluctuation	Safety Coefficients against the Overturning	Factor of Safety to the Sliding	Tension (t/m^2)	
					Upstream Compressive Stress σvu	Downstream Compressive Stress σvd
Case of Normal Load	Overflow section	4.89	6.47	1.88	3.189	51.982
	Allowable minimum value	1.3	1.5	1.0	-7.0	
Case of Exceptional Load	Overflow section	3.59	4.81	2.15	-13.845	66.705
	Allowable minimum value	1.1	1.2	1.0	-14.0	

According to the result of the calculation, tensile stress is caused on the dam base in CCE case, however, this stress is less than allowable value. Allowable tensile stress is set as shown below;

- Shear strength of CL class rock is assumed as $40t/m^2$.
- Generally shear strength of foundation rock is assumed that ; tension strength = compression strength/10, shear strength = compression strength/7. Hence considering above, it is assumed that tension strength = shear strength × 0.7 for foundation rock. Then;
  $\sigma t = 40 \times 0.7 = 28t/m^2$
- σt, adm = Charge Capacity / Safety Coefficient, then 28/4 = $7t/m^2$ for CCN and 28/2 = $14t/m^2$ for CCE.

4.4 Design of Sub-dam (Protection for Scoring)

Based on Japan standard, namely Standard for River and Sediment Control Works, protection works for downstream scoring are designed. The length from the main dam to the sub dam is set 33 m and the top elevation of the sub-dam is set as same elevation as the front apron elevation for the prevention of scoring.

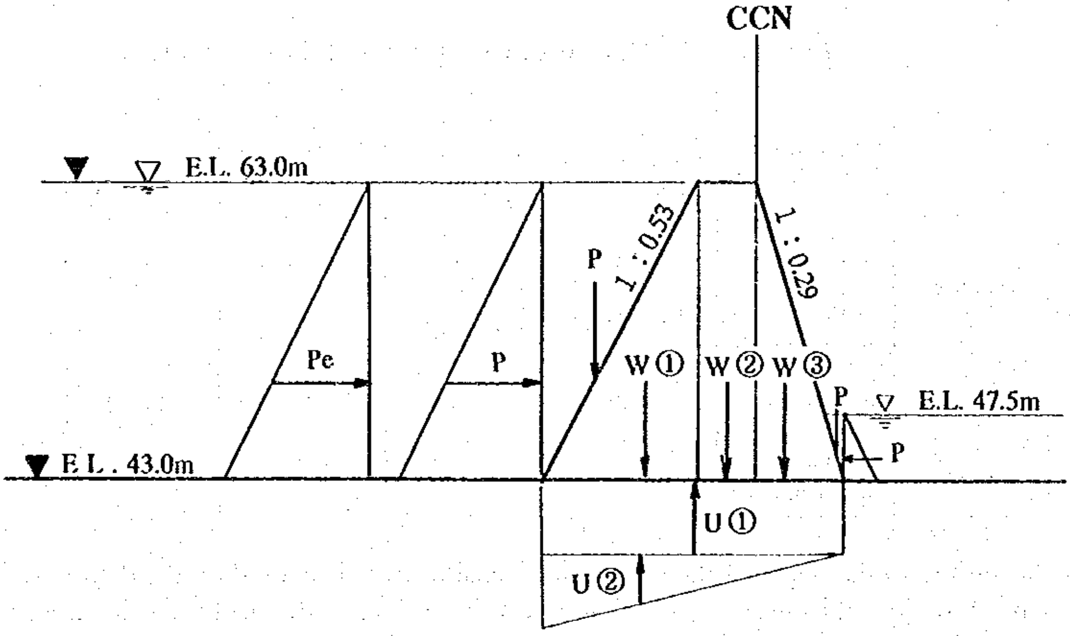
Distance between Main dam and Sub dam (L) is calculated by following formula

$$L = 1.5 (H + h)$$

where

H : height of main dam upon apron = 17m
h : overflow depth = 4.95m
L : 1.5 × (17 + 4.95) = 33m

Overflow Section



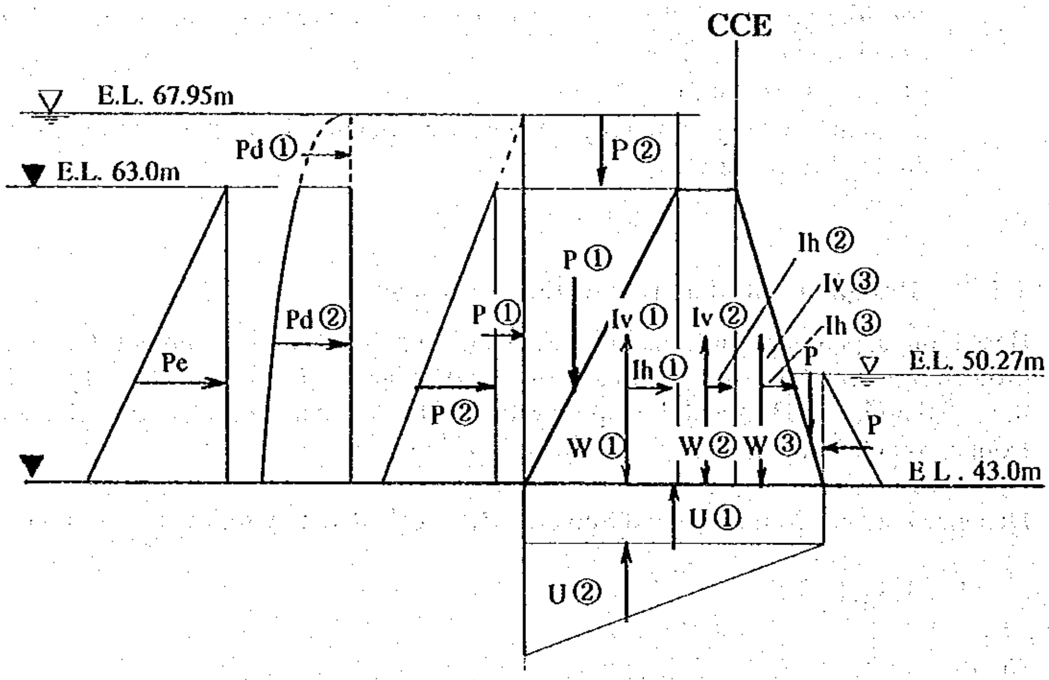


Figure-4.1 Load Acting on Check Dam

Table-4.3 Summary of Stability Analysis of Check Dam

1. Design Load

Item		Unit	Dimension
Dam Top Level		EL.(m)	63.000
Overflow Crest Level		EL.(m)	63.000
Dam Foundation Level		EL.(m)	43.000
Upstream Slope			0.29
Downstream Slope			0.53
Design Flood Water Level		EL.(m)	67.950
Normal Water Level		EL.(m)	63.000
Sedimentation Level		EL.(m)	63.000
Downstream River Water Level	H.W.L.	EL.(m)	50.270
	N.W.L.	EL.(m)	47.500
Drain Outlet Level		EL.(m)	—
Distance from Upstream Face until Drain		(m)	—
Unit Volume Weight	Concrete	(t/m^3)	2.3
	Water	(t/m^3)	1.0
	Sedimentation	(t/m^3)	0.35
Design Seismic Coefficient	Horizontal		0.05
	Vertical		0.03
Foundation Design Strength	Cohesion	(t/m^2)	40
	Angle of Internal Friction	(deg)	30

2. Carrying Case

Item	Unit	Carrying Case	
		CCN	CCE
Upstream Water Level	EL.(m)	63.000 N.W.L.	67.950 H.W.L.
Downstream Water Level	EL.(m)	50.270	47.500
Sedimentation Level	EL.(m)	63.000	63.000
Dam Foundation Level	EL.(m)	43.000	43.000
Design Seismic Coefficient			0.05 Horizontal 0.03 Vertical
Drain Condition		—	—

3. Results of Stability Analysis

Carrying Case	CSF	CST	FSD	σvu	σvd	n
CCN						
Overflow Section	4.89	6.67	1.88	3.189	51.982	4.4
Allowable Minimum Value	1.3	1.5	1	-7	-7	
CCE						
Overflow Section	3.59	4.81	2.15	-13.845	66.705	2.9
Allowable Minimum Value	1.1	1.2	1	-14	-14	

Note:

CCN	:	case of normal load
CCE	:	case of exceptional load
CFS	:	safety coefficients to fluctuation
CST	:	safety coefficients against the overturning
FSS	:	factor of safety to the sliding
σvu	:	upstream compressive stress
σvd	:	downstream compressive stress
n	:	safety factor by Henny

Table-4.4 Analyzed Case ; Stability Analysis for CCN of Overflow Section of Check Dam

1. Carrying Case

Overflow Section
CCN

2. Design Condition

Item		Dimension
Height of Dam		20.000
Crest Width		4.000
Upstream Slope		1 : 0.53
Downstream Slope		1 : 0.29
Upstream Head		9.667
Downstream Head		4.500
Depth of Sedimentation		20.000
Drain Condition		none
Uplift	Heel of Dam	9.667
	Toe of Dam	4.500
Design Seismic Coefficient	Horizontal	0.000
	Vertical	0.000

$Uu = (hd + (hu - hd) / 3) \times \gamma w$
$Ud = hd \times \gamma w$

3. Working Load

Item		V(t)	Lv (m)	V× Lv (t × m)	H(t)	Lh (m)	H × Lh (t × m)
Own Weight	①	243.800	7.067	1,722.935			
	②	184.000	12.600	2,318.400			
	③	133.400	16.533	2,205.502			
	Subtotal	561.200		6,246.837			
Seismic inertia force							
Hydrostatic Pressure	Upstream	106.000	3.533	374.498	200.000	6.667	1,333.400
	Downstream	2.936	19.965	58.617	-10.125	1.500	-15.188
	Subtotal	108.936		433.115	189.875		1,318.212
Pressure due to Sedimentation		37.100	7.067	262.186	70.000	6.667	466.690
Dynamic Water Pressure							
Uplift	①	-91.800	10.200	-936.360			
	②	-52.700	6.800	-358.360			
	Subtotal	-144.500		-1,294.720			
Total		562.736		5,647.418	259.875		1,784.902

4. Stability Analysis

Summitry of the Gravitational Forces	$\Sigma V =$	707.236 t	
Summitry of the Uplift Forces	$\Sigma U =$	144.500 t	
Safety Coefficients to Fluctuation	$C.S.F = \Sigma V / \Sigma U =$	4.89 ≧1.3	O.K
Summitry of Stabilizer Moments	$\Sigma Me =$	8,742.228 t × m	
Summitry of Overturning Moments	$\Sigma Mt =$	1,309.908 t × m	
Safety Coefficients against the Overturning	$C.S.T = \Sigma Me / \Sigma Mt =$	6.67 ≧1.5	O.K
Safety Coefficient relatively Ø	CSDØ =	1.5	
Safety Coefficient relatively C	CSDc =	3.0	
Normal Strength	Ni =	562.736 t	
Angle of Friction	Øi =	30 °	
Cohesion	Ci =	40 t/m²	
Area of Contact	Ai =	20.400 m² (unit width)	
Forces Parallel to the Sliding Surface	Ti =	259.875 t	
Factor of Safety to the Sliding FSS =	(ΣNi × tanØi / CSDØ + Σci × Ai / CSDc) / Σti =	1.88 ≧1.0	O.K
Total of Moments	$\Sigma M = \Sigma (V \times L) + \Sigma (H \times h) =$	7,432.320 t × m	
Working Point of Resultant Force	$d = \Sigma M / \Sigma V =$	13.207 m	
Length of Dam Base	$B =$	20.400 m	
	$e = B / 2 - d =$	-3.007 ≦B/6=	3.4
Upstream Compressive Stress	$\sigma u = (V / B)(1 + 6e / B) =$	3.189 ≧-7	O.K
Downstream Compressive Stress	$\sigma d = (V / B)(1 - 6e / B) =$	51.982 ≧-7	O.K
Safety Factor by Henny	$n = (f \times \Sigma V + \tau 0 \times B) / \Sigma H =$	4.4 ≧4	O.K

Table-4.5 Analyzed Case; Stability Analysis for CCE of Overflow Section of Check Dam

1. Carrying Case

Overflow Section
CCE

2. Design Condition

Item		Dimension
Height of Dam		20.000
Crest Width		4.000
Upstream Slope		1 : 0.53
Downstream Slope		1 : 0.29
Upstream Head		24.950
Downstream Head		7.270
Depth of Sedimentation		20.000
Drain Condition		none
Uplift	Heel of Dam	13.163
	Toe of Dam	7.270
Design Seismic Coefficient	Horizontal	0.050
	Vertical	0.030

$Uu = (hd+(hu-hd)/3) \times \gamma w$
$Ud = hd \times \gamma w$

3. Working Load

Item		V(t)	Lv (m)	V×Lv (t×m)	H(t)	Lh (m)	H×Lh (t×m)
Own Weight	①	243.800	7.067	1,722.935			
	②	184.000	12.600	2,318.400			
	③	133.400	16.533	2,205.502			
	Subtotal	561.200		6,246.837			
Seismic Inertia Force	①	-7.314	7.067	-51.688	12.190	6.667	81.271
	②	-5.520	12.600	-69.552	9.200	10.000	92.000
	③	-4.002	16.533	-66.165	6.670	6.667	44.469
	Subtotal	-16.836		-187.405	28.060		217.740
Hydrostatic Pressure	Upstream①	52.470	5.300	278.091	99.000	10.000	990.000
	Upstream②	106.000	3.533	374.533	200.000	6.667	1,333.400
	Downstream	7.664	19.697	150.958	-26.426	2.423	-64.030
	Subtotal	166.134		803.582	272.574		2,259.370
Pressure due to Sedimentation		37.100	7.067	262.186	70.000	6.667	466.690
Dynamic Water Pressure	-①				-1.369	—	-2.791
	①+②				14.455	—	148.530
	Subtotal				13.086		145.739
Uplift	①	-148.308	10.200	-1,512.742			
	②	-60.112	6.800	-408.762			
	Subtotal	-208.420		-1,921.504			
Total		539.178		5,203.696	383.720		3,089.539

4. Stability Analysis

Summitry of the Gravitational Forces	$\Sigma V =$	747.598 t	
Summitry of the Uplift Forces	$\Sigma U =$	208.420 t	
Safety Coefficients to Fluctuation	$C.S.F = \Sigma V / \Sigma U =$	3.59 ≧1.3	O.K
Summitry of Stabilizer Moments	$\Sigma Me =$	10,468.965 t × m	
Summitry of Overturning Moments	$\Sigma Mt =$	2,175.730 t × m	
Safety Coefficients against the Overturning	$C.S.T = \Sigma Me / \Sigma Mt =$	4.81 ≧1.5	O.K
Safety Coefficient relatively Ø	CSDØ =	1.1	
Safety Coefficient relatively C	CSDc =	1.5	
Normal Strength	Ni =	539.178 t	
Angle of Friction	Øi =	30 °	
Cohesion	Ci =	40 t/m²	
Area of Contact	Ai =	20.400 m² (unit width)	
Forces parallel to the sliding surface	Ti =	383.720 t	
Factor of Safety to the Sliding FSS =	(ΣNi × tanØi / CSDØ + Σci × Ai / CSDc) / Σti =	2.15 ≧1.0	O.K
Total of Moments	$\Sigma M = \Sigma (V \times L) + \Sigma (H \times h) =$	8,293.235 t × m	
Working Point of Resultant Force	$d = \Sigma M / \Sigma V =$	15.381 m	
Length of Dam Base	$B =$	20.400 m	
	$e = B / 2 - d =$	-5.181 |>B/6=	3.4
Upstream Compressive Stress	$\sigma u = (V / B)(1 + 6e / B) =$	-13.845 ≧-14	O.K
Downstream Compressive Stress	$\sigma d = (V / B)(1 - 6e / B) =$	66.705 ≧-14	O.K
Safety Factor by Henny	$n = (f \times \Sigma V + \tau 0 \times B) / \Sigma H =$	2.9 <4	

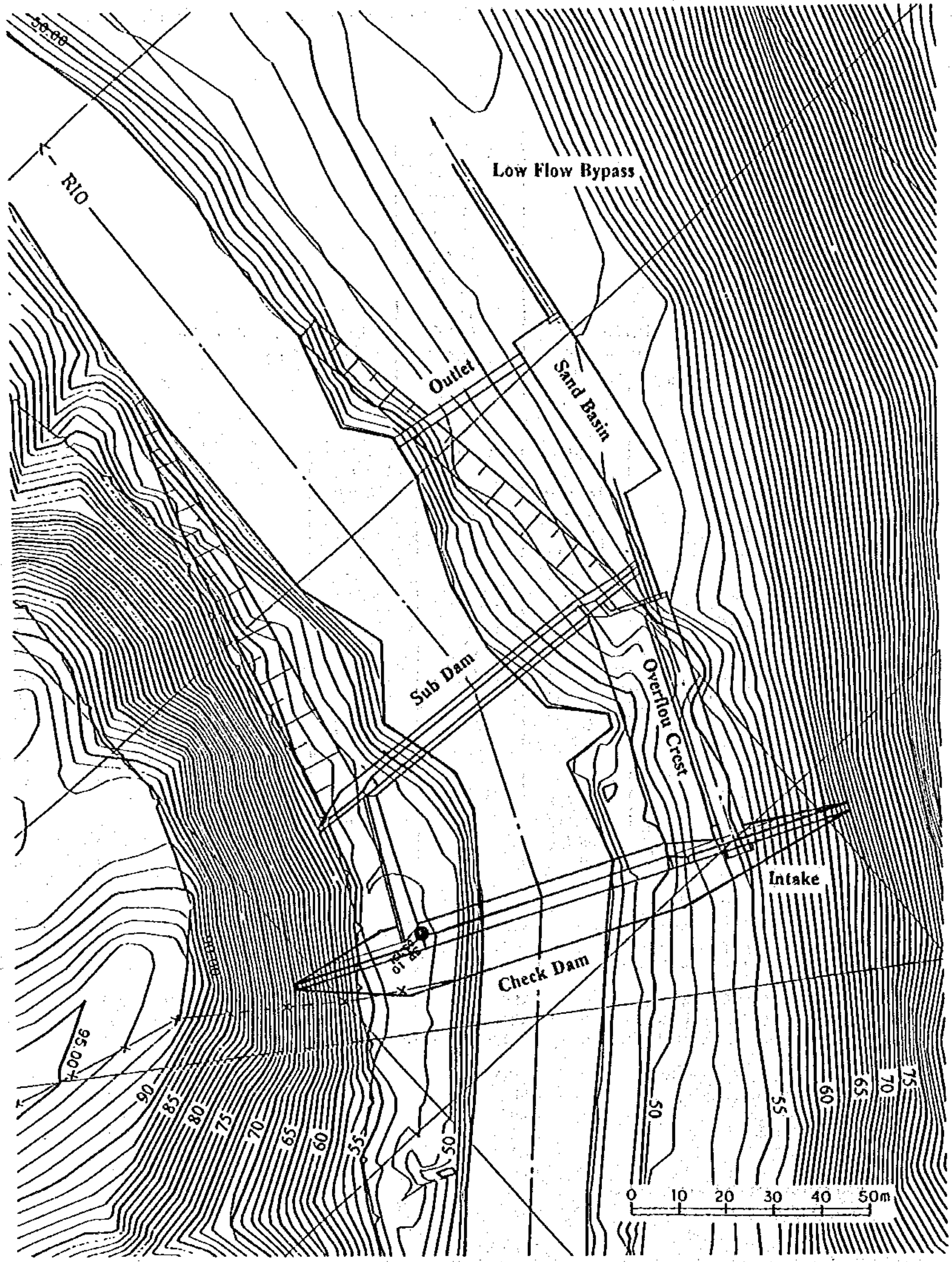


Figure-4.2 Plan of Check Dam

CHECK DAM AND INTAKE OF LOW FLOW BYPASS S=1:500

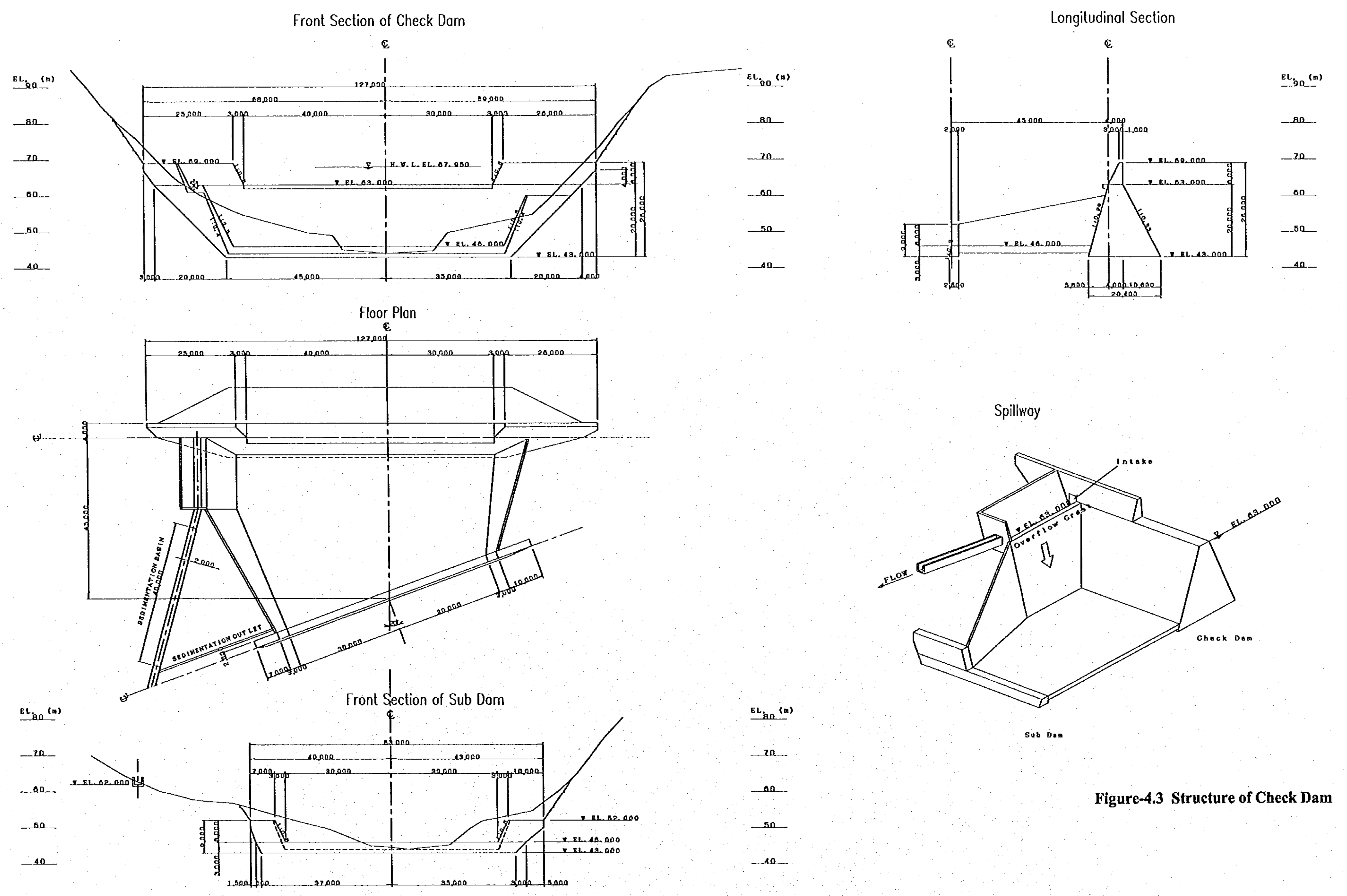


Figure-4.3 Structure of Check Dam

CHAPTER 5 PLAN AND DESIGN OF LOW FLOW BYPASS

5.1 Alternatives of Low Flow Bypass

To convey low flow water not entering reservoir and to decrease salinity of reservoir water, the low flow bypass is planned and compared.

- Open Type: Open type channel is installed along the periphery of reservoir. A channel crosses a valley or steep slope as a bridge.
- Closed Type: Closed type of pipeline or box culvert is installed in the bottom of the reservoir.

(1) Design Condition

< Water Level and Design Discharge >

- Upstream at outlet of Check Dam : EL. 63.0m
- Downstream at Vaza Barris Dam : EL. 56.0m(Open channel)
  : EL. 29.0m(Pipeline and Culvert)
- Design Discharge : 0.75m^3/s

< Coefficient of Roughness >

- Steel pipe : 0.015
- Concrete channel or culvert : 0.020
- Friction loss is taken into account in hydraulic calculation.

(2) Description of Alternatives

(a) Bypass Open Channel

Longitudinal slope of the bypass open channel is preferably constant throughout the channel route. In principle, the channel is constructed excavating ground on the middle of hill slope. At the place where the channel crosses a deep valley, a concrete bridge or earth embankment supports it in order to minimize the total length of the channel. At the place where hill slope is very steep and excessive excavation is expected, a bridge supports the channel. The bridge has a span of around 15m and is constructed by cast-in-place concrete.

Maintenance road with 3m wide is provided beside the channel. For bridge sections, the maintenance road is also constructed on hill slope apart from the bridge. The required channel dimension is W 1.5 m x H 1.5 m to convey the design discharge of 0.75 m^3/s with 80 % water depth. Channel slope is 1 : 3,750. Flow velocity is 0.7 m/s. The flow of the channel is sub-critical flow, since Froude Number of the channel flow is 0.17. Although it is rather difficult to precisely construct this kind of channel with slope required by design because the channel slope is very small, some irregularities of slope are acceptable because the flow is sub-critical flow. The construction of this type of channel affects much to environmental conditions.

(b) Bypass Steel Pipeline

Longitudinal slope of the bypass steel pipeline is not necessary to be constant because the water in pipeline flows by pressure. It is recommendable to provide downward slope as much as possible to prevent the sedimentation in the pipe. Large degree of bends in horizontal and vertical directions is avoided by means of excavating or embanking the natural ground surface along the pipeline route to secure smooth alignment. Steel pipe under water is backfilled after installation of the pipe to prevent uplift by buoyancy.

The pipe diameter to transport design discharge of 0.75 m^3/s is obtained as ø1,000mm by hydraulic calculation taking into account of the friction loss in welded steel pipes. Flow velocity inside of pipe is 1.0m/s. The cathodic protection with 80 years of service life is provided to external and internal surface of steel pipe to protect the steel pipe against corrosion. At the downstream end of pipe, the diameter of pipe is reduced to 0.45 m to regulate the discharge volume and shut gate is provided for emergency use. The affect of construction of steel pipe to environmental conditions is negligibly small since the pipe is backfilled and installed under reservoir water.

(c) Bypass Box Culvert

The same precaution is required as to the bypass steel pipeline against large degree of bends in vertical and horizontal alignment. No uplift is expected because it is heavier than uplift force by buoyancy. Hydraulic calculation shows that the required section of box culvert to transport the design discharge of $0.75m^3/s$ is W 1.05m x H 1.05m and flow velocity is 0.7 m/s. The downstream end of box culvert is connected to steel pipe with ø1,000 mm diameter and is lead to inside of dam body. The shut gate is provided as same as for bypass steel pipe. The affect of construction of box culvert to environment is also negligibly small since the box culvert is installed under water.

(3) Comparison of the Design of Low Flow Bypass

As for the alternative designs of low flow bypass, earthwork volume is calculated based on the relation between the horizontal and vertical alignment of each bypass and the natural ground surface condition along the bypass routes. The quantities of other works are also obtained based on the longitudinal and transversal sections of each bypass. Based on the construction quantities, construction cost was estimated.

The alternative designs were evaluated in viewpoints of design, workability, operation and maintenance, environment, economy and so on. Table-5.1 shows the comparative evaluation on the alternative designs of low flow bypass.

In this Feasibility Study, the box culvert bypass of the closed type was adopted as a low flow bypass, which is superior on many points although it has difficulty on maintenance.

Table-5.1 Comparison of Low Flow Bypass

Items	Open Type: Open Channel	Closed Type: Box Culvert	Closed Type: Pipeline
Flow Type	Open type	Pressure type	Pressure type
Section	Concrete rectangular channel: W1.5m × H1.5m	Concrete box culvert: W1.05m × H1.05m	Steel pipe: Ø1,000 mm
Length	30.0 km	27.7 km	27.7 km
Head	8.0 m	38.0 m	38.0 m
Flow Velocity	0.6 m/s	0.7 m/s	1.0 m/s
Dam Crossing Method	- Channel is installed on the elevation higher than the top of dam.	- A pipe crosses the dam body. - To control discharge, valve is equipped.	Same as the left
Sediment Problem	- At the intake, sedimentation basin is installed. - It is easy to clean deposit in the channel.	- At the intake, sedimentation basin is installed. - It is difficult to clean deposit in the box culvert. - Flow velocity of 0.7 m/s seems to cause no sedimentation in box culvert.	- At the intake, sedimentation basin is installed. - It is difficult to clean deposit in the pipeline. - Flow velocity of 1.0 m/s seems to cause no sedimentation in pipeline.
Maintenance	- It is easy because the facility is installed out of reservoir.	- It is difficult because the facility is submerged permanently on the bottom of reservoir.	Same as the left
Impact to Environment	- Channel installation requires a large amount of earthworks of cutting and embanking riverbanks.	- Impact to environment is very small because most of the facility is concealed under the reservoir water.	Same as the left
Impact to Reservoir Operation	- Channel divides drain system into the reservoir. It is necessary to put rainwater drain system to across underneath the channel.	- Leakage of water from reservoir into pipeline is avoided by providing careful construction.	- Leakage of water from reservoir into pipeline is avoided by providing cathodic protection to steel pipes.
Construction Workability	- Construction material is carried through the maintenance road to the construction site of channel. - The construction work of bridge with cast-in-place concrete requires much construction time and cost. - Installation of prefabricated bridge is much more difficult than cast-in-place concrete bridge.	- Access to the construction site is very easy. - Construction material is entered from the existing approach road and carried on the riverbed or river terrace. - Concrete work for box culvert is easy and simple.	- Access to the construction site is very easy. - Construction material is entered from the existing approach road and carried on the riverbed or river terrace. - Installation of pipeline in the reservoir area is easy and simple.
Construction Cost	R$ 47.1 million	R$ 32.8 million	R$ 44.5 million
Evaluation	- Maintenance for channel cleaning and repairing is easier. - Water bridge should be carefully designed and constructed based on the proper construction plan. - Impact to environment is larger. - Construction cost is higher	- Careful design is required to assure maintenance free bypass after filling of reservoir. - It is difficult to check and repair box culvert during reservoir operation. - Workability and impact to environment is much better than open channel. - Construction cost is lowest.	- Careful design is required to assure maintenance free bypass after filling of reservoir. - It is difficult to check and repair pipe during reservoir operation. - Workability and impact to environment is much better than open channel. - Construction cost is higher than box culvert.

5.2 Design of Low Flow Bypass

(1) Design of Low Flow Bypass

(a) Design Condition

- Type of Bypass : Reinforced Concrete Box Culvert
- Water Level : Upstream at outlet of Check Dam : EL. 63.0m
  : Downstream at Vaza Barris Dam : EL. 29.0m
- Flow Volume : $0.75m^3/s$
- Coefficient of roughness : Concrete surface : 0.020
  (Friction loss is taken into account in hydraulic calculation.)

(b) Required Section

Hydraulic calculation shows that the required section of box culvert to transport the design discharge of $0.75m^3/s$ is W 1.05m x H 1.05m and flow velocity is 0.7m/s. Thickness of slabs and walls is determined as 0.40m taking the external and internal water pressures into consideration. Table-5.2 shows calculation for design of box culvert and gate.

(c) Longitudinal Alignment

Longitudinal alignment of the bypass box culvert is shown in Figure-5.1. This section is prepared based on the routing plan of the bypass using the existing 1:5,000 scale topographic map along the Vaza Barris River. Large degree of bends in horizontal and vertical directions is avoided by means of excavating or embanking the natural ground surface along the bypass route to secure smooth alignment.

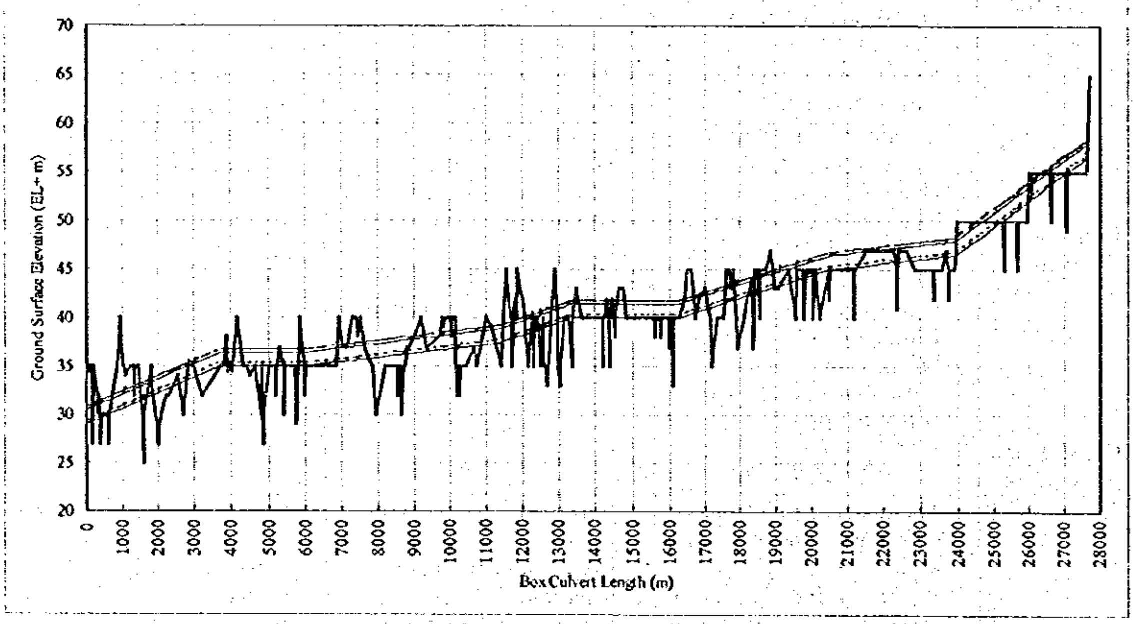


Figure-5.1 Vertical Profile of Low Flow Bypass

Table-5.2 Calculation for Box Culvert and Gate

Reservoir Level	EL.(m)	Comparison of Box Culvert				Comparison of Gate				Remark
		63.0	63.0	63.0	63.0	63.0	63.0	63.0	63.0	
Gate										
Diameter of Gate	Ø (mm)	1,100	1,050	1,000	950	450	400	350	300	
Open Rate of Gate	(%)	100	100	100	100	100	100	100	100	
Coefficient of Discharge	C	0.820	0.820	0.820	0.820	0.820	0.820	0.820	0.820	
Area	A0(m^2)	0.950	0.865	0.785	0.708	0.159	0.126	0.096	0.071	
Box Culvert										
Roughness Coefficient	n	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.020	Concrete Surface
Height and Width of Culvert	D1(mm)	1,100	1,050	1,000	950	1,050	1,050	1,050	1,050	
Length of Pipe	L1(m)	27,720.0	27,720.0	27,720.0	27,720.0	27,720.0	27,720.0	27,720.0	27,720.0	
Area	A1(m^2)	1.210	1.103	1.000	0.903	1.103	1.103	1.103	1.103	
Coefficient of Friction Loss	f1	1,215.717	1,293.511	1,380.456	1,478.170	1,293.511	1,293.511	1,293.511	1,293.511	
$f1/A1^2$		830.351	1,063.210	1,380.456	1,812.796	1,063.210	1,063.210	1,063.210	1,063.210	
$f1/A1^2 \times 0.2$		16.607	21.264	27.609	36.256	21.264	21.264	21.264	21.264	2% of Friction Loss
Summitry of Loss		846.958	1,084.474	1,408.065	1,849.052	1,084.474	1,084.474	1,084.474	1,084.474	
Downstream Water Level	EL.(m)	29.0	29.0	29.0	29.0	29.0	29.0	29.0	29.0	
Head	h (m)	34.0	34.0	34.0	34.0	34.0	34.0	34.0	34.0	
Velocity	V (m/s)	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	
Discharge	Q (m^3/s)	0.886	0.783	0.687	0.600	0.763	0.752	0.731	0.695	
Design Discharge	Q0 (m^3/s)	0.750	0.750	0.750	0.750	0.750	0.750	0.750	0.750	
Evaluation		OK	OK	NG	NG	OK	OK	NG	NG	

According to the calculation result, the minimum sections of culvert and gate for the design discharge of $0.75m^3/s$ is the combination of box culvert of 1.050mm × 1,050mm and gate of 400mm.

Note:

f1 : $(124.5 \times n^2 \times L1) / D^{(4/3)}$

A1 : $D1^2$

Q : $C \times A0 \times ((2 \times g \times h) / (1 + C^2 \times A0^2 \times \Sigma(f1/A1^2)))^{0.5}$

5.3 Design of Intake Facility

(1) Design of Sedimentation Basin

Intake facility for bypass is installed at the check dam. The intake works are composed of an inlet, a sedimentation basin, a discharge regulation spillway and a gate to stop water entering. The sedimentation basin is designed as width: 2 m x depth: 2 m x length: 40 m to settle sediment of 0.3 mm diameter. At the front of intake, screen is installed to prevent invasion of floating woods and people for safety reason.

Length of sedimentation basin; L is obtained as follow.

$L = hV / v$

where

Safety Coefficients = 1.5-2.0

h	:	depth of sedimentation basin
V	:	mean velocity
v	:	settling velocity
h	:	2.0m, V = 0.25m/s, v = 0.025m/s (sediment of 0.3 mm diameter)
L	:	$2 \times 2.0 \times 0.25 / 0.025 = 40$ m

(2) Consideration to Sedimentation

Sedimentation of soils and suspended materials in the Bypass during flood period might cause the less capability of water passage and the blockage of the bypass by sedimentation in the worst case. It is concluded that the following countermeasures are effective to prevent sedimentation in the Bypass:

1) Screen is installed at the entrance to the intake facility to prevent floating woods or particles from entering the bypass.
2) Sedimentation basin is constructed to prevent the soils from entering the bypass. Calculation shows that the grain size of soil less than or equals to 0.3 mm could be settled down in the basin with dimensions of 40 m long and 2 m deep.
3) Flow velocity in the bypass with the design discharge of 0.75 m^3/s is about 0.7m/s. The relationship between the grain size and critical flow velocity, which is the minimum velocity not to cause movement of settled soils, according to the formula presented by Justin, the critical flow velocity for 0.3 mm particles is 0.056m/s. Therefore, it is judged that the silt or clay with grain size less than or equal to 0.3 mm is washed out to the downstream of the bypass. Although the soils with larger grain sizes may enter into the bypass, it is also washed down since the critical velocity of grain size of 5 mm is 0.229 m/s.
4) The shut gate at the outlet of the bypass controls the flow volume in the bypass. Turbid water with sediment load could not inflow to the bypass when a gate is closed.
5) Aquatic plant might be grown inside of bypass when the velocity is slow. In general, the design velocity over 0.7 m/s is adopted to prevent growing aquatic plant. Although the velocity of 0.6 m/s in the box culvert is less than 0.7 m/s, it seems no problem because: 1) the flow in bypass is of high chlorine concentration, and 2) Inside of the bypass is difficult circumstance for aquatic plant to grow without sunshine.
6) Some sedimentation can be allowed by dimensioning of the box culvert larger than the hydraulically required dimensions.

5.4 Design Drawing

The design of the low flow bypass, the concrete box culvert channel is shown in Figure-5.2.