JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

MINISTRY OF SETTLEMENT AND REGIONAL DEVELOPMENT
THE REPUBLIC OF INDONESIA

THE DETAILED DESIGN OF FLOOD CONTROL, URBAN DRAINAGE AND WATER RESOURCES DEVELOPMENT IN SEMARANG IN THE REPUBLIC OF INDONESIA

FINAL REPORT

COMPONENT B: JATIBARANG MULTIPURPOSE DAM CONSTRUCTION

VOLUME II DESIGN CRITERIA

AUGUST 2000

CTI ENGINEERING INTERNATIONAL CO., LTD.
IN ASSOCIATION WITH
PACIFIC CONSULTANTS INTERNATIONAL
AND
PASCO INTERNATIONAL INC.

CONSTITUTION OF THE REPORT

1. SUMMARY
2. COMPONENT A : WEST FLOODWAY/GARANG RIVER IMPROVEMENT

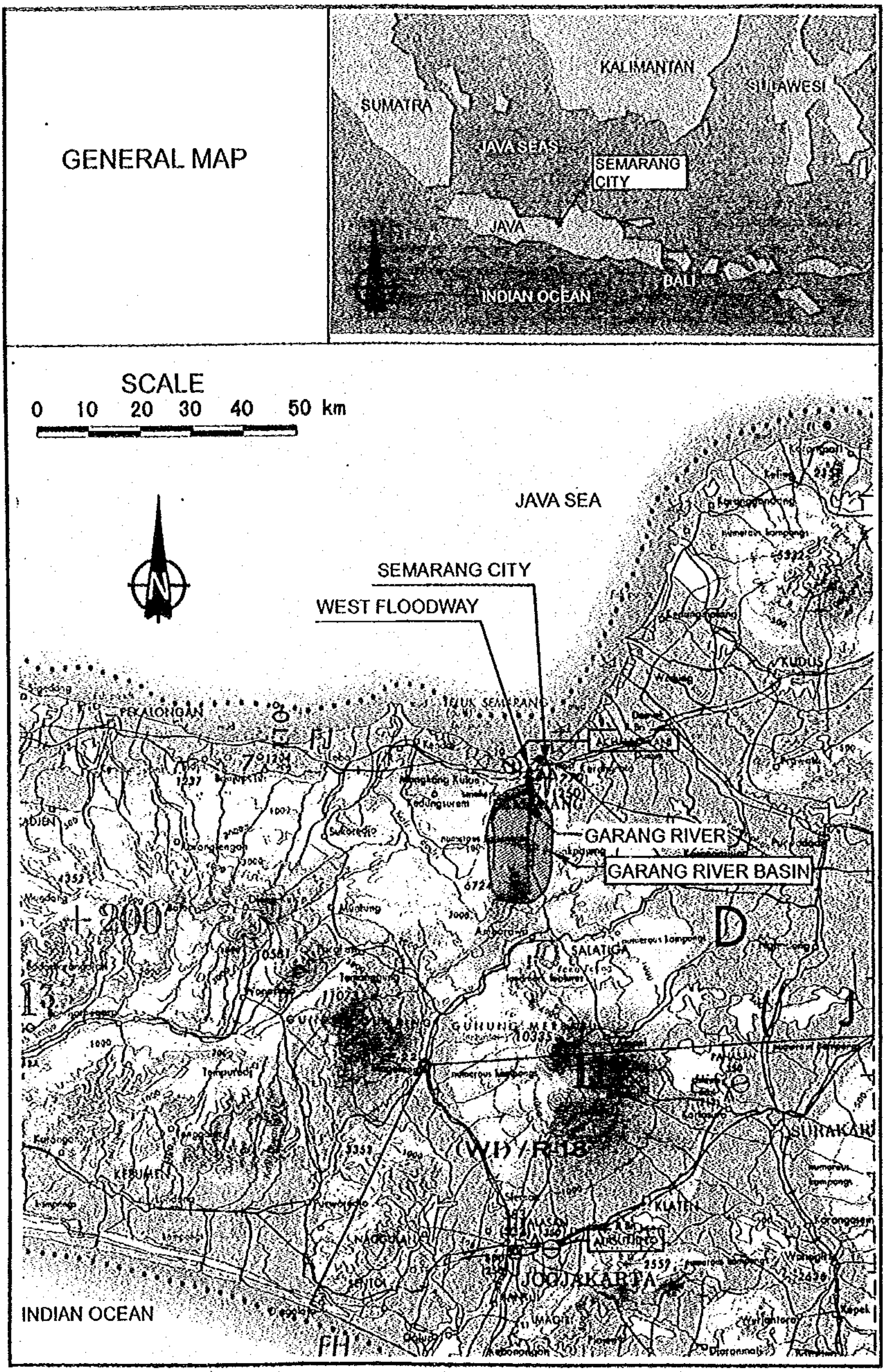
VOLUME I	MAIN REPORT
VOLUME II	DESIGN CRITERIA
VOLUME III	DESIGN NOTES
VOLUME IV	WORK QUANTITY CALCULATION
VOLUME V	CONSTRUCTION PLANNING
VOLUME VI	COST ESTIMATE
VOLUME VII	DATA BOOK

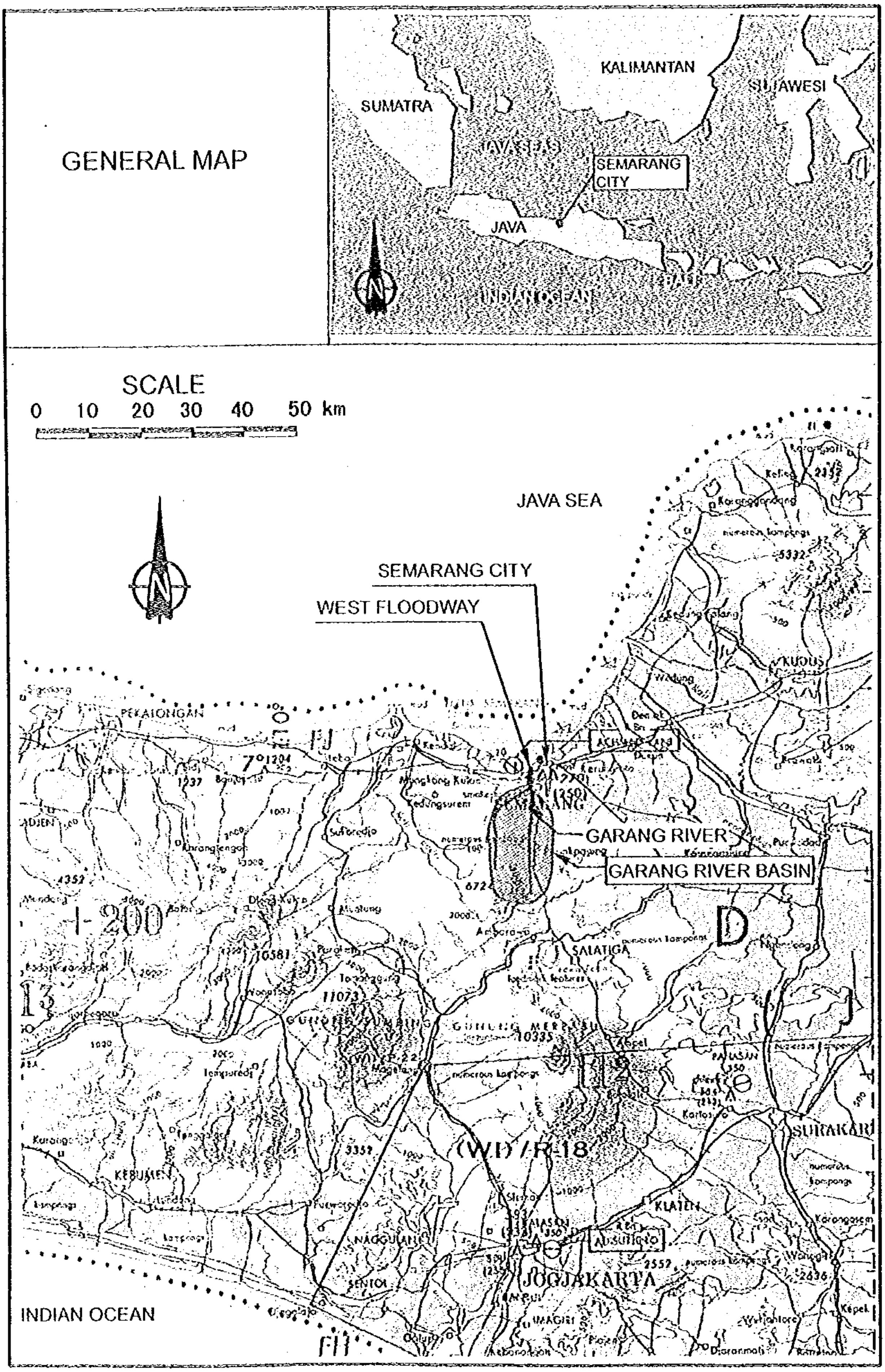
3. COMPONENT B : JATIBARANG MULTIPURPOSE DAM CONSTRUCTION

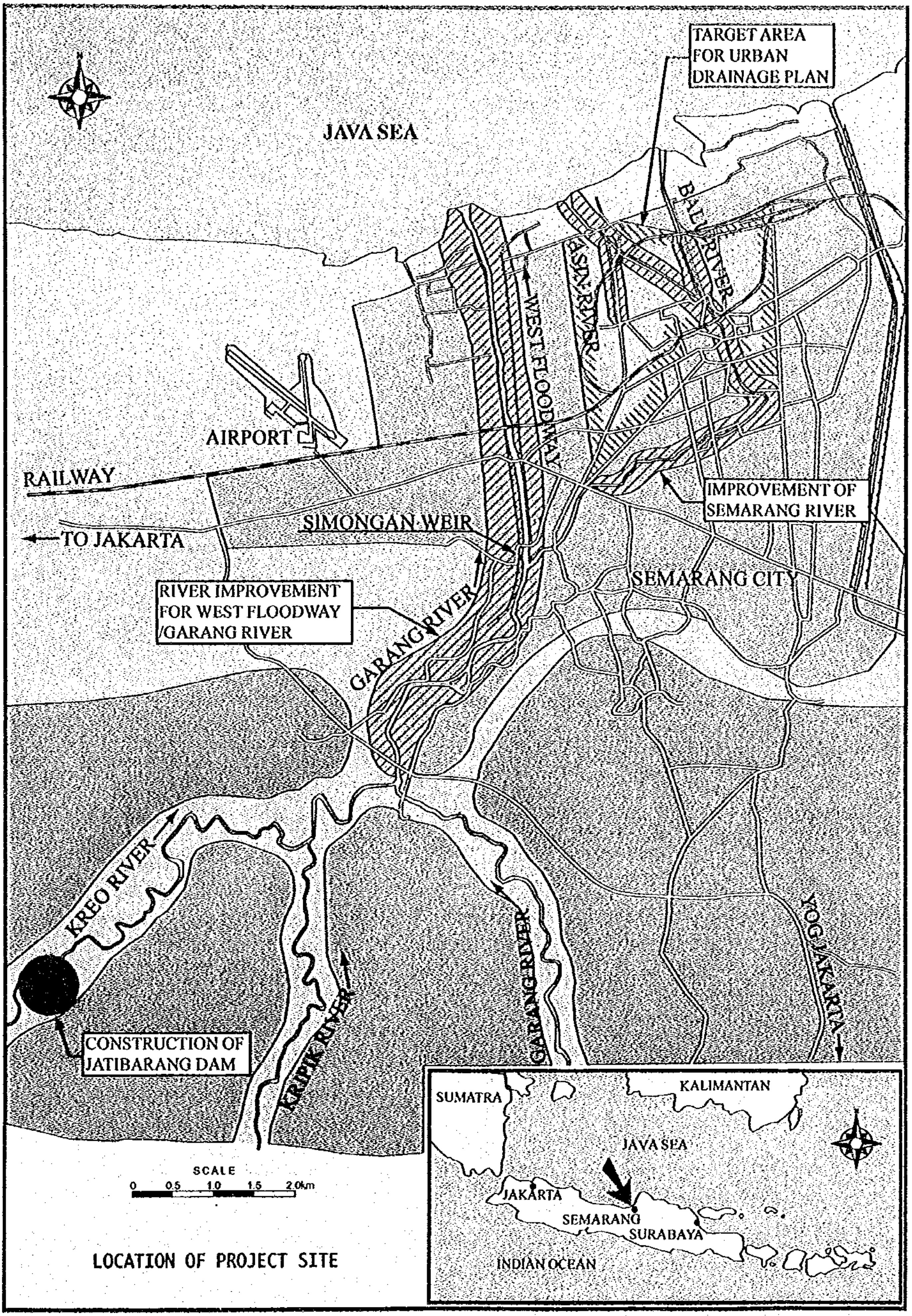
VOLUME I	MAIN REPORT
VOLUME II	DESIGN CRITERIA
VOLUME III	DESIGN NOTES
VOLUME IV	WORK QUANTITY CALCULATION
VOLUME V	CONSTRUCTION PLANNING
VOLUME VI	COST ESTIMATE
VOLUME VII	DATA BOOK
VOLUME VIII	ANNEX

4. COMPONENT C : URBAN DRAINAGE SYSTEM IMPROVEMENT

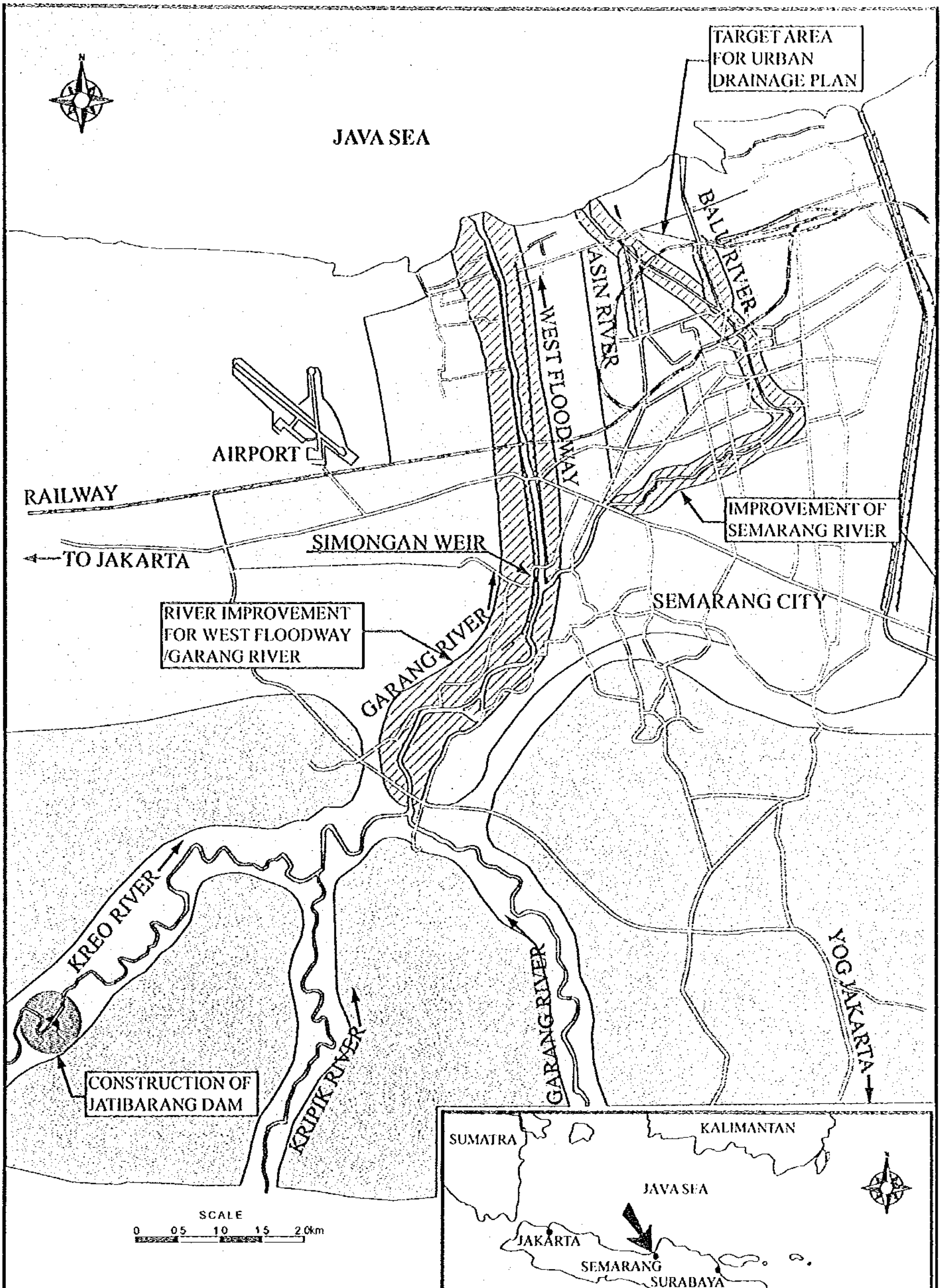
VOLUME I	MAIN REPORT
VOLUME II	DESIGN NOTES
VOLUME III	WORK QUANTITY CALCULATION
VOLUME IV	CONSTRUCTION PLANNING
VOLUME V	COST ESTIMATE
VOLUME VI	DATA BOOK


GENERAL MAP
KALIMANTAN
SULAWESI
SUMATRA
JAVA SEAS
SEMARANG CITY
JAVA
BALI
INDIAN OCEAN
SCALE
0 10 20 30 40 50 km
JAVA SEA
SEMARANG CITY
WEST FLOODWAY
GARANG RIVER
GARANG RIVER BASIN
PEKALONGAN
KUDUS
SALATIGA
SURAKARTA
KLATEN
JOGJAKARTA
INDIAN OCEAN


GENERAL MAP
KALIMANTAN
SULAWESI
SUMATRA
JAVA SEA
SEMARANG CITY
JAVA
INDIAN OCEAN
SCALE
0 10 20 30 40 50 km
JAVA SEA
SEMARANG CITY
WEST FLOODWAY
GARANG RIVER
GARANG RIVER BASIN
PEKALONGAN
KUDUS
SALATIGA
KEBUMEN
KLATEN
JOGJAKARTA
INDIAN OCEAN



LOCATION OF PROJECT SITE



LOCATION OF PROJECT SITE

VOLUME II DESIGN CRITERIA

TABLE OF CONTENTS

VOLUME II DESIGN CRITERIA

TABLE OF CONTENTS

PART-II DRAWING STANDARD

Page

LIST OF TABLE

LIST OF FIGURE

PART I DESIGN CRITERIA

CHAPTER 1 GENERAL

1.1 Introduction

Jatibarang Multipurpose Dam planned on Kreo River is located in the southwest of Semarang City at about 13 km upstream from the confluence of Garang River. It will primarily function flood control, public water supply of Semarang City and hydropower generation.

Detailed discussion on selection of dam type has been given in the Main Report (VOLUME I). Based on the technical appraisal as well as construction cost, the center core rockfill type was found the most suitable for Jatibarang Multipurpose Dam. It was discussed between JICA Study Team and Indonesian Government and accepted by Indonesian Government in the Meeting held on 23 February 1999 in Jakarta.

The Design Criteria (VOLUME II) was prepared, serving for the succeeding detailed design of structures subject to Jatibarang Multipurpose Dam. The criteria contain the codes/design standards, formulas, properties of structural materials, safety factors to be adopted for stability analysis, hydraulic design and structural details.

In designing the objective structure, the Indonesian guidelines and local conditions in Semarang area are considered as much as possible. The minor engineering assumption and judgment of design which are not mentioned in the criteria, will be considered and decided by each design engineer during the detailed design work.

1.2 Objective Structures

The design criteria will be applied to the detailed design of the following structures:

(1) The main dam consisting of the foundation excavation, internal gallery, and main dam embankment with impervious zone, semi–pervious zone, and inner and outer pervious zone.

(2) The spillway structure which is provided with side channel spillway with the bathtub type overflow weir, control portion, chute, stilling basin and spillway bridge.

(3) The diversion arrangement consisting of cofferdam, diversion tunnel and inlet structure with overflow weir and guide wall.

(4) Outlet facilities that is provided with inclined intake structure with bulkhead and low water outlet gates, outlet tunnel, steel conduit and control gate.

(5) Hydropower station consisting of generating room, after bay and tailrace channel, tail race stoplog, penstock (portion for generating only).

The features resulting from the definitive plan are summarized hereinafter.

Reservoir

Catchment Area	:	53.0 km^2
Reservoir Surface Area	:	1.10 km^2
Maximum Water Surface	:	EL. 155.300 m
Surcharge Water Surface	:	EL. 151.800 m
Normal Water Surface	:	EL. 148.900 m
Low Water Surface	:	EL. 136.000 m
Gross Storage Capacity	:	20,400,000 m^3
Effective Storage Capacity	:	13,600,000 m^3
Flood Control Capacity	:	3,100,000 m^3
Water Use Capacity	:	10,500,000 m^3
Sediment Capacity	:	6,800,000 m^3

Dam

Height above Foundation	:	77.0 m
Crest Elevation	:	EL. 157.000 m
Foundation Elevation	:	EL. 80.000 m
Crest Length	:	200.0 m
Crest Width	:	10.0 m

Spillway

Design Flood (Inflow)		
Probable Maximum Flood	:	1,600 m^3/s (inflow into reservoir)
100-year Probability	:	290 m^3/s (inflow into reservoir)
Design Discharge for Energy Dissipater	:	340 m^3/s (100-year probable flood)
Design Discharge for Sidewall Height	:	1,310 m3/s (PMF outflow from reservoir)
Overflow Crest (Service Spillway)		
Crest Elevation	:	EL. 148.900 m
Crest Width	:	15.0 m

Overflow Crest (Emergency Spillway)		
Crest Elevation	:	EL. 151.800 m
Total Crest Width	:	60.0 m
Total Length of Spillway	:	307 m
Stilling Basin	:	24.0 m wide x 60.0 m long
Spillway Bridge (PC Girder Type)	:	5.0 m wide x 23.94 m long

Outlet Facilities

Maximum Design Discharge	:	6.0 m^3/s
Minimum Design Discharge	:	0.26 m^3/s
Intake Structure	:	Inclined Type
Bulkhead Gate	:	Clear Span 2.0 m x Clear Height 1.4 m
Low Water Outlet Gate	:	Clear Span 2.0 m x Clear Height 1.4 m
Steel Outlet Pipe	:	393 m long x 1.4 m dia.
Control Gate	:	650 and 250 mm dia.

Diversion Facilities

Design Discharge	:	280 m^3/s (25-year probable flood)
Tunnel Section	:	Horseshoe with the diameter of 5.6 m
Longitudinal Gradient	:	1/30
Tunnel Length	:	441 m
Tunnel Inlet Elevation	:	EL. 98.500 m
Crest of Temporary Cofferdam	:	EL. 113.000 m

Hydropower Generation

Maximum Plant Discharge	:	3.0 m^3/s
Maximum Gross head	:	65.5 m
Installed Capacity	:	1,560 kW
Number of Generator at Future Stage	:	No extension
Annual Energy	:	6,020 MWh
Power Station	:	450 m^2

1.3 Codes and Standards

The design and computation are based on internationally accepted codes, standards as well as conformity with Indonesian codes, standards and practice.

The following codes and standards are principally used in establishing design conditions of each structure.

Indonesian Standards

- Standar Industri Indonesia (SII), 1986 (Indonesian Industrial Standards)
- Peraturan Beton Bertulang Indonesia PBBI 71-Ni-2, 1971 (Indonesian Guideline for Reinforced Concrete)
- Pembebanan Rencana Rumah dan Gedung, SNI-1727-1989F (Building Design Load Code)
- Pedoman Perencanaan Pembebanan Jalan dan Jembatan, 1989 (Guideline for Highway and Bridge Design Loads, SNI-1725-1989)
- Peraturan Perencanaan Bangunan Baja Indonesia, 1984 (Indonesian Steel Structures Code)
- Peraturan Bangunan Nasional dan Pelengkap (Regulation of National Building and Finishes 1978)
- Pedoman Perencanaan Ketahanan Gempa Untuk Rumah dan Gedung (Guideline for Earthquake Resistant Building Design, SNI-1726-1989)
- Pedoman perencanaan Tahan Gempa Untuk Jembatan Jalan Raya, 1986 (Guideline for Earthquake Resistant Highway Bridge Design)
- Flood Control Manual, Ministry of Public Works, Government of Indonesia
- Standar Perencanaan Irigasi, Departemen Pekerjaan Umum. 1986 (Design Standards of Irrigation, Ministry of Public Works, Government of Indonesia)
- Peraturan Perencanaan Teknik Jembatan May 1992 BINAMARGA (BMS) (Bridge Design Code)
- Design Manual, December 1992 BINAMARGA
- Peta Zona Gempa Dan Cara Penggunaannya Sebagai Usulan Dalam Perencanaan Bangunan Pengairan Tahan Gempa, Puslitbang Pengairan (IHE-Bandung, 1994)

In addition, the following standards/specifications are used to supplement the design codes/standards mentioned above.

Japanese Standards

- Technical Standards for River and Sabo Works, River Association of Japan
- Specifications for Design and Construction of Road Bridge, Japan Road Association
- Design Specifications for River Gate, River Association of Japan
- Specifications Highway Bridges, Part-I Common Specifications, Part-II Concrete Bridges, Part-IV Substructures, Highway Association of Japan
- Japanese Industrial Standard (JIS), Japanese Standards Association
- Design Criteria for Dams, Japanese National Committee on Large Dams
- Manual for River Works in Japan, Design of Dams, River Bureau, Ministry of Construction
- Technical Standards for Gates and Penstock, Hydraulic Gate and Penstock Association
- Technical Standards for Gate Facilities of Dams and Weirs, Japan Association of Dam and Weir Equipment Engineering
- Mountain Tunnel Version of Standard Specification for Tunneling, Japan Society of Civil Engineers

Other References

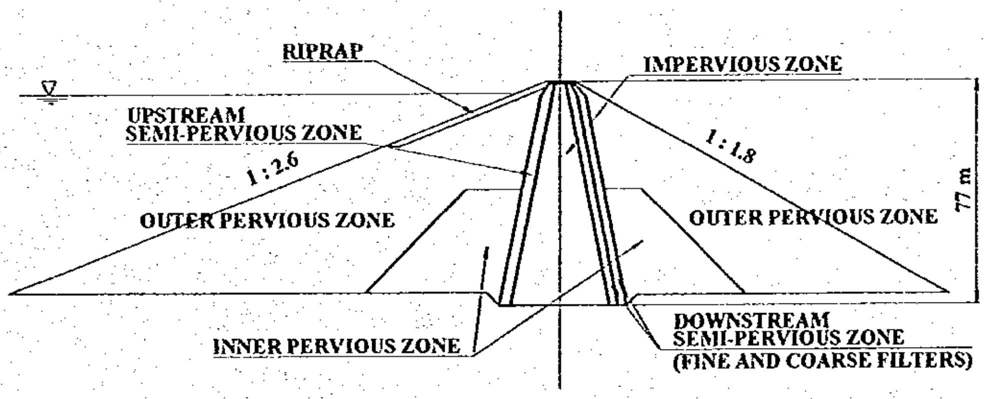
- Pacific Consultants International in Association with SINOTECH Eng., Consultants. INC and Others, "Design Criteria for Keuliling Irrigation Dam", Consulting Service for Detailed Design, Krueng Aceh Irrigation Project, Directorate of West Region Implementation, DGWRD, June 1997
- Puslitbang Pengairan (IHE-Bandung), "Kriteria Desain, Bendungan Kendang", Penyelidikan dan Perencanaan Teknis, PPWS, Bengawan Solo, January 1987
- Suyono Sosrodarsono DR. & Kensaku Takeda, "Bendungan Type Urugan", PT. Pradaya Paramita, Jakarta, 1989
- United States Department of the Interior Bureau of Reclamation (USBR), "Design of Small Dams"
- M. MacDonald International Ltd., Cambridge, UK in association with Nippon Koei Co. Ltd. and others, "Annex A Design Criteria for Civil Works, Mujur Dam and Appurtenance Design Report", Volume I, DGWRD, MPW, May 1997

CHAPTER 2 DAM EMBANKMENT MATERIAL

2.1 Constitution of Zones

The center core rockfill dam can be divided into three zones, impervious zone, semi-pervious zone and pervious zone. The permeability of each zone increases toward the outer slopes.

Impervious zone filled with earth material provides watertightness. Inner and outer pervious zones filled with rock of all sizes support the less stable impervious material and provide the stability of the dam body. Semi-pervious zone of sand-gravel or fine rock is embanked between impervious zone and pervious zone, to be served as a transition and filter. The downstream semi-pervious zone consists of two zones, fine and coarse filters. Riprap zone is provided to prevent the upstream slope from being eroded.



Typical Constitution of Zones in Zoned Rockfill Dam

2.2 Design Values

The design values of materials to be used in stability analysis are adopted from the limited test results on smaller samples. Cohesion and internal friction angle in terms of effective stresses are directly determined from the test results. Wet density and saturated density obtained from the laboratory test can be converted into the design values considering a content ratio of a gravel coarser than the maximum size (19.0 mm) of samples.

2.2.1 Impervious Material

Earth material for impervious zone will be selected to satisfy the requirements of low permeability, sufficient shearing strength and low compressibility after compaction. It will be easily compacted and free from deleterious substances such as organic matter. The design values will be determined from a series of laboratory test using the mixed materials.

The gradation limits and the design values of the mixed impervious material are shown as below:

Gradation Limits of Impervious Material

Diameter of Particle (mm)	Passing by Weight (%)
150.0	100
53.0	72 – 100
19.0	50 – 80
4.75	35 – 65
0.075	15 – 25

Properties of Impervious Material

No.	Item	Unit	Design Value
1	Average Specific Gravity (Gs)	tf/m³	2.72
2	Average Natural Moisture Content (W)	%	12.6
3	Dry Density (γ_d)	tf/m³	1.87
4	Wet Density (γ_t)	tf/m³	2.11
5	Saturated Density (γ_{sat})	tf/m³	2.19
6	Effective Internal Friction Angle (ϕ')	°	25.0
7	Effective Cohesion (C')	tf/m²	1.0

2.2.2 Semi-pervious Material

Semi-pervious zones are provided at both sides of impervious zone. The upstream and downstream semi-pervious zones are designed to prevent the impervious zone from washing out the fine particles and to provide adequate drainage of impervious zone.

The downstream semi-pervious zone consists of two zones, fine and coarse filters. Material for semi-impervious zone shall consist of a well-graded and non plastics mixture of sand-gravel or fine rock in which the individual particles are hard and durable, free from clay, silt and organic material.

The following filter criteria developed by many years of experience are used to design the downstream semi-pervious zones that will prevent the movement of the protected impervious material into the semi-pervious material. This criterion is mainly studied by USSCS (United States Soil Conservation Service) based on the grain-size relationship between the protected soil and the filter. In the following table, the lower case "d" is used to represent the grain size for the protected material and the upper case "D" the grain size for the filter material.

Filter Criteria (Downstream Semi-pervious Zone)

Protected Material *1			Semi-pervious (Filter) Material			
Category		Percent Finer than 0.075 mm	D_5 Size	D_{15} Size	D_{60} Size	D_{100} Size (Dmax.)
1	Fine silts and Clays	More than 85 % finer	≥ 0.075 mm	$\leq 9 \cdot d_{85}$	≤ 4.75 mm or $\leq 20 \cdot D_{10}$ *4	≤ 50 mm
2	Sands, silts, clays, and silty and clayey sands	40 to 85 % finer	≥ 0.075 mm	≤ 0.7 mm	≤ 4.75 mm or $\leq 20 \cdot D_{10}$	≤ 50 mm
3	Silty and clayey Sands and Gravels	15 to 40 % finer	≥ 0.075 mm	$\leq (40-A)/(40-15) \cdot (4 \cdot d_{85} - 0.7\text{mm}) + 0.7\text{mm}$ *2	≤ 4.75 mm or $\leq 20 \cdot D_{10}$	≤ 50 to 150 mm
4	Sands and gravels	Less than 15 % finer	≥ 0.075 mm	$\leq 4 \cdot d_{85}$ *3	$\leq 20 \cdot D_{10}$	≤ 50 to 150 mm

Notes *1 : Category designation for the protected material containing particles finer than 0.075 mm is determined from a gradation curve of the base protected material that has been adjusted to 100 % passing the 4.75 mm sieve.

*2 : $15 \leq A < 40$, A = percent passing the 0.075 mm sieve after any regrading. When $4 \cdot d_{85}$ is less than 0.7 mm, use 0.7 mm.

*3 : d_{85} can be based on the total protected material before regrading.

*4 : It means that the uniformity coefficient D_{60}/D_{10} should not exceed 20 (D_{60} on coarse limit of filter, D_{10} on fine limit of filter).

For the semi-pervious material, cohesion is assumed to be zero. The gradation limits and the design values of the mixed semi-pervious materials are shown as below:

Gradation Limits of Downstream Semi-pervious Material (Fine Filter)

Diameter of Particle (mm)	Passing by Weight (%)	Description
50.0	100	$D_{100} \leq 50$ mm
4.75	60 – 85	$D_{60} \leq 4.75$ mm
0.7	15 – 40	$D_{15} \leq 0.7$ mm
0.25	0 – 10	$D_{60} \leq 20 \cdot D_{10}$
0.075	not more than 5	$D_5 \geq 0.075$ mm

Gradation Limits of Downstream Semi-pervious Material (Coarse Filter)

Diameter of Particle (mm)	Passing by Weight (%)	Description
150.0	100	$D_{100} \leq 150$ mm
53.0	60 – 100	-
19.0	15 – 50	$D_{15} \leq 4 \cdot d_{85}$
2.65	0 – 10	$D_{60} \leq 20 \cdot D_{10}$
0.075	not more than 5	$D_5 \geq 0.075$ mm

Gradation Limits of Upstream Semi-pervious Material

Diameter of Particle (mm)	Passing by Weight (%)
150.0	100
53.0	60 – 100
19.0	30 – 60
4.75	20 – 40
0.075	not more than 5

Properties of Semi-pervious Material

No.	Item	Unit	Design Value		
			Upstream	Down-stream (fine)	Down-stream (coarse)
1	Specific Gravity (Gs)	tf/m^3	2.56	2.58	2.54
2	Natural Water Content (W)	%	1.6	2.0	1.0
3	Dry Density (γ_d)	tf/m^3	2.08	1.86	1.92
4	Wet Density (γ_t)	tf/m^3	2.11	1.90	1.94
5	Saturated Density (γ_{sat})	tf/m^3	2.27	2.14	2.16
6	Effective Internal Friction Angle (ϕ')	°	35.0	35.0	45 ($0<\sigma' \leq 2.6$) 42 ($2.6<\sigma' \leq 6.3$) 37 ($6.3 < \sigma'$)
7	Effective Cohesion (C')	tf/m^2	0.0	0.0	0.0

2.2.3 Pervious Material

Pervious zone is subdivided into two zones, inner pervious zone and outer pervious zone. Both sides of inner zone are arranged at the outer side of semi-pervious zone of both upstream and downstream. Rock material for pervious zone especially filled in outer rock zone, shall consist of a well graded mixture of hard and durable particles, and shall be slightly weathered to fresh rock to secure the stability of dam body.

The rock materials for the pervious zone have high friction angle at low stress levels. The upper envelope of the circles on a Mohr diagram is typically concave downward with a slope that is steepest in the lower range of normal stress that decreases gradually with increasing stress.

For the rock materials for outer pervious zone, three (3) fixed values of friction angle depending on stress levels are adopted. Internal friction angle of them in inner pervious zone is reduced by about 5 % because the soft rocks from the required excavations, which have less desirable properties and are more erratic, are allowed to be mixed in this zone.

For the pervious materials, cohesion is assumed to be zero. The design values for stability calculation are given as below:

Gradation Limits of Pervious Material

Diameter of Particle (mm)	Passing by Weight (%)	
	Inner Zone	Outer Zone
750.0	100	
100.0	not more than 70	not more than 60
	not less than 15	
4.75	not more than 20	not more than 10
0.075	not more than 5	

Properties of Pervious Material

No.	Item	Unit	Design Value	
			Inner Zone	Outer Zone
1	Specific Gravity (Gs)	tf/m³	2.54	2.54
2	Natural Water Content (W)	%	1.0	1.0
3	Void Ratio		0.325	0.325
4	Dry Density (γ_d)	tf/m³	1.92	1.92
5	Wet Density (γ_t)	tf/m³	1.94	1.94
6	Saturated Density (γ_{sat})	tf/m³	2.16	2.16
7	Effective Internal Friction Angle (ϕ')	°	43 ($0<\sigma' \leq 2.6$)	45 ($0<\sigma' \leq 2.6$)
			40 ($2.6<\sigma' \leq 6.3$)	42 ($2.6<\sigma' \leq 6.3$)
			35 ($6.3 < \sigma'$)	37 ($6.3 < \sigma'$)
8	Effective Cohesion (C')	tf/m²	0.0	0.0

Notes : σ' = Effective normal stress acting on the failure surface (kgf/cm²)

2.2.4 Riprap

1.0 m thick of riprap zone with selected large size rocks is provided to prevent the upstream slope from being eroded. It is placed above the Low Water Surface.

The material for the riprap zone shall have the following properties:

- The riprap zone shall have average rock size not less than 50 cm and the maximum rock size about 100 cm.

- The rock material is required to have a minimum bulk dry density of 2.5 tf/m³ and maximum water absorption of 4 %.

The other design values for stability calculation are same as the pervious material.

CHAPTER 3 DAM AND FOUNDATION

3.1 Seismic Coefficient

The design Seismic Coefficient for the dam body stability analysis should be estimated referring to the treatise "Seismic Zone Map and Guidance for Design of Water Resources Structure against Earthquake (by Najoan and others)," which was brought up by Directorate of Technical Guidance, Ministry of Public Works.

The Seismic Coefficient for Jatibarang Dam is calculated by using the following equation:

$$K = z \times Ac \times v/g$$

Where,

K	:	seismic coefficient
z	:	coefficient of zone (z = 0.8 for Semarang City)
Ac	:	basic earthquake acceleration (cm/sec²) (Ac = 215.81 for 200-year return period, large dam)
v	:	correction factor of ground (v = 0.9 for rock, v = 1.0 for diluvium)
g	:	gravity acceleration (g = 980 cm/sec²)
Ad	:	earthquake design acceleration (cm/sec²)
	=	$Z \times Ac \times v$

Classification of Water Resources Structures

No.	Dam Scale	Reservoir Capacity ($1,000,000 m^3$)	Height (m)	Potential Damage in the Case of Failure	Category
1	Large	> 60	> 20	Sacrifice of human lives and serious property damage	1
	Medium	1 - 60	10 - 30	Not many sacrifice of human lives	2
	Small	< 1	< 15	No sacrifice of human lives	3

Standard Earthquake Load

		No Damage Condition			Small Damage and No Failure Condition	
Category	Life Time (years)	Design Earthquake Acceleration (cm/sec^2)	Return Period (years)	Method of Analysis	Return Period (years)	Method of Analysis
1	50 - 100	Ad.min=0.1g Ad.max=0.4g	100 -200	Seismic Coefficient	10,000 (MCE)	Dynamic Analysis
2	20 - 50	Ad.min=0.1g Ad.max=0.4g	50 - 100	Seismic Coefficient	1,000 - 5,000 (MCE)	Dynamic Analysis
3	20	Ad.min=0.1g Ad.max=0.4g	20 - 50	Seismic Coefficient		

Note : MCE stands for Maximum Credible Earthquake

Coefficient of Zone

Zone	Coefficient Z	Remarks
A	1.90 - 2.00	
B	1.60 - 1.90	
C	1.20 - 1.60	
D	0.80 - 1.20	
E	0.40 - 0.80	Semarang
F	0.20 - 0.40	

Correction Factor

Rock Type	Factor V
Rock Foundation	0.9
Diluvium	1.0
Alluvium	1.1
Soft Alluvium	1.2

Earthquake Base Acceleration

Unit : (cm/sec^2)

Return Period T (years)	Earthquake Base Acceleration Ac
10	98.42
20	119.62
50	151.72
100	181.21
200	215.81
500	271.35
1,000	322.35
5,000	482.80
10,000	564.54

The Earthquake design acceleration (Ad) should be compared to Ad_{min} and Ad_{max} from the above tables for classification of the water resources structures and standard earthquake load.

Seismic coefficient K was estimated as follows:

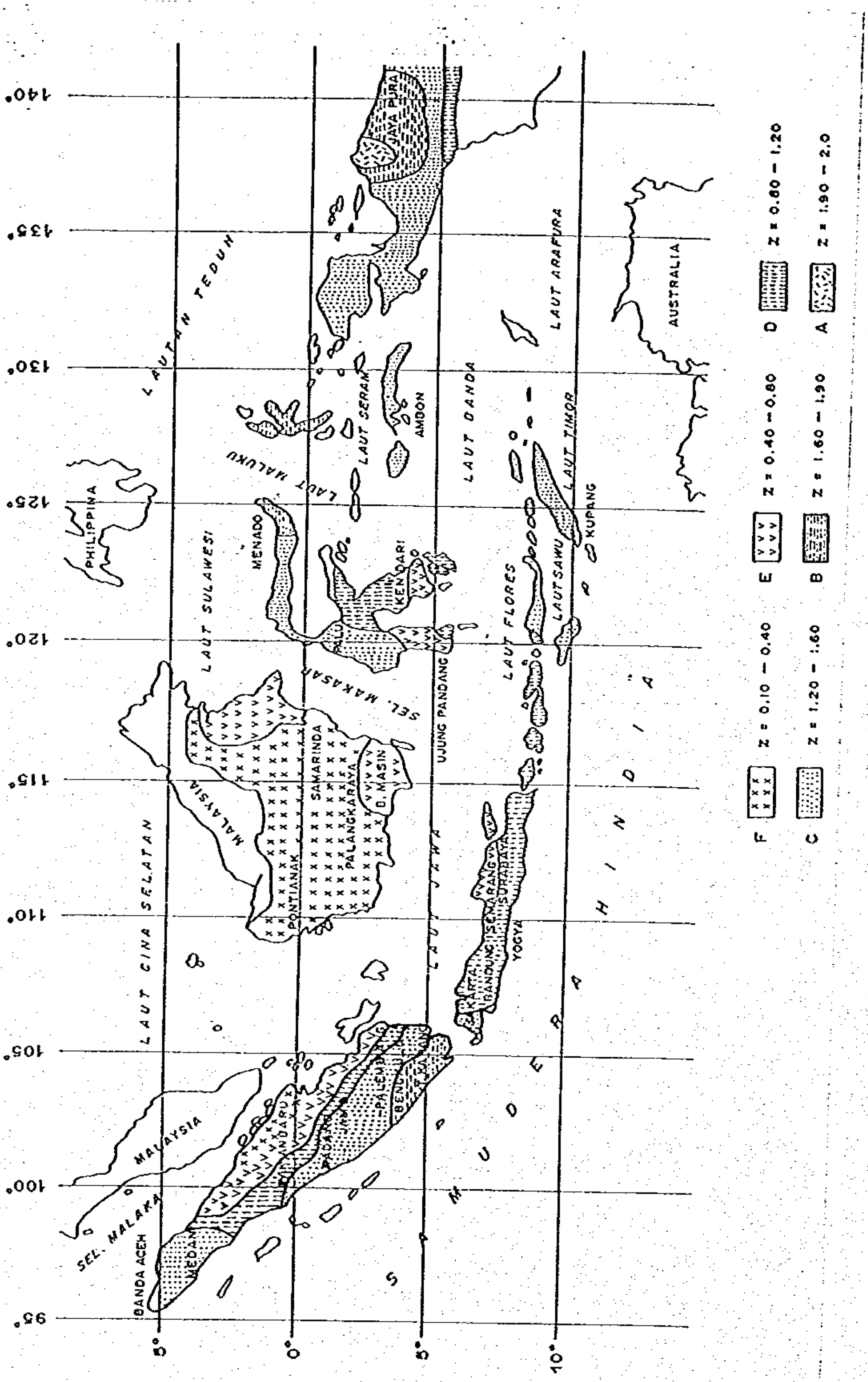
Items	Symbol	Unit	Rock Foundation (Concrete Structure)	Diluvium (Rock Fill Dam)
Coefficient of zone	Z		0.80	0.80
Earthquake base acceleration	Ac	cm/sec^2	215.81	215.81
Correction factor	V		0.90	1.00
Gravity acceleration	g	cm/sec^2	980	980
Seismic coefficient (Z*Ac*V/g)	K		0.16	0.18

The lateral earthquake force is calculated by using the following formula :

$$F = K \cdot W$$

Where,

F	:	lateral earthquake force (tf)
K	:	seismic coefficient
W	:	self weight (tf)



Geographic Position and Factors Z

3.2 Freeboard and Crest Elevation

The dam crest level of non-overflow portion of a dam should be equal or higher than the height which consists of the freeboard and the Maximum Water Level. The highest value should be determined among the Normal Water Surface, the Surcharge Water Surface and the Maximum Water Surface, plus freeboard according to the following criteria:

Case (a) The Normal Water Surface (EL. 148.9 m), freeboard is $Hf = h_e + h_w$

Case (b) The Surcharge Water Surface (EL. 151.8 m), freeboard is $Hf = h_e/2 + h_w$

Case (c) The Maximum Water Surface (EL. 155.3 m), freeboard is $Hf = h_w$

For the ungated Spillway, the minimum freeboard is 0.75 m

Where,

Hf	:	freeboard
h_w	:	the height of wave due to wind
h_e	:	the height of wave due to earthquake

The height of wave due to earthquake should be determined by the following formula:

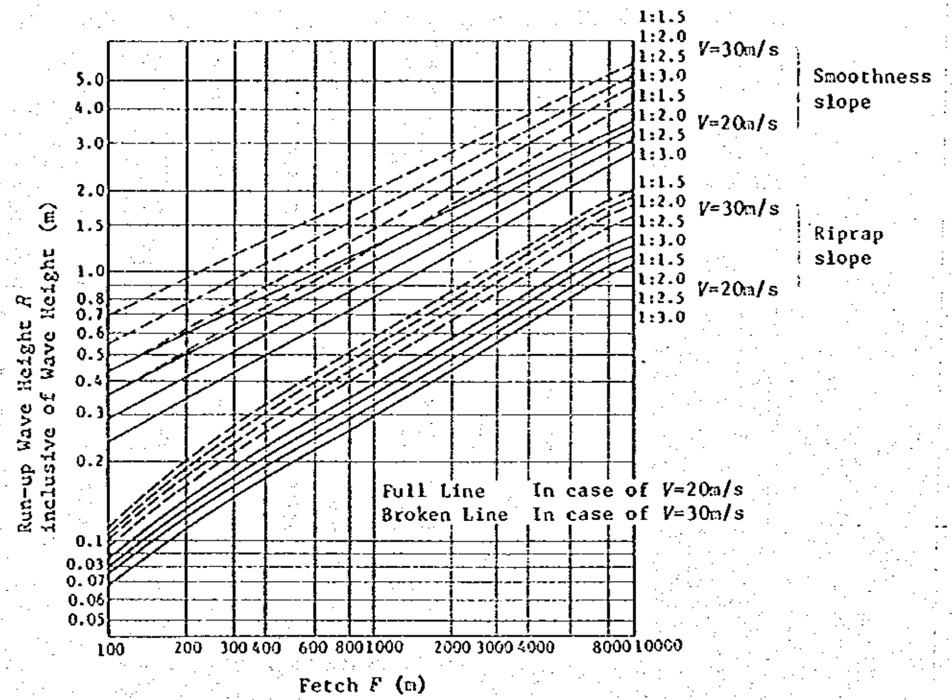
$$H_e = \frac{K\tau}{2\pi} * \sqrt{g\,Ho}$$

Where,

H_e	:	height of seismic water wave (m)
k	:	seismic coefficient
τ	:	period of earthquake wave, assuming to be 1.0 sec.
Ho	:	reservoir water depth (m)
g	:	gravity acceleration (m/sec²)

The wave height (h_w) due to wind will be calculated by using the SMB Method. When the upstream face is inclined as a fill type dam, the run-up height of the wave along the dam is calculated using Saville's Method. The following figure shows diagram for the run-up wave height obtained by a combined use of the SMB Method and Saville's Method. Calculation of water wave in reservoir is shown in the table below.

Item	Symbol	Value	Remarks
Wind Velocity	V	20 m/s	
Maximum Fetch	F	2,100 m	
Upstream Slope		1 : 2.6	Riprap Slope
Height of Wave by Wind	hw	0.5 m	refer to diagram as shown below
Earthquake coefficient	k	0.18	
Seismic Frequency	t	1 sec	
Reservoir Depth	Ho	68.9	Ho = NWL 148.9–Base 80.0 m
Height of Wave by Earthquake	he	0.8 m	$He = k*t*(9.8*Ho)^{0.5}/2/3.14=0.745m$



Run-Up Wave Height (inclusive of wave height) Obtained by A Combined Use of The SMB Method and Saville's Method

3.3 Safety of Dam Body against Sliding

3.3.1 Loading Condition to be considered

The varieties and combination of loads to be considered in embankment stability against sliding failure shall be determined in accordance with the reservoir water surface and seismic condition. In principle, self weight, hydrostatic pressure, pore pressure and seismic body force shall be considered.

Load to be considered for each condition of the dam are tabulated below:

Case	Condition of Dam	Combination of Loads	Required Safety Factor
1	Reservoir stage is at Normal Water Surface and seepage is steady.	Self weight Hydrostatic pressure Pore pressure 100 % of seismic body force	1.20
2	Reservoir stage is at Normal Water Surface and seepage is steady.	Self weight Hydrostatic pressure Pore pressure 0 % of seismic body force	1.50
3	Reservoir stage is at Maximum Water Surface	Self weight Hydrostatic pressure Pore pressure 0 % of seismic body force	1.20
4	Reservoir is being rapidly drawn down from Normal Water Surface to Low Water Surface and there is residual pore pressure	Self weight Hydrostatic pressure Pore pressure 100 % of seismic body force	1.10
5	Reservoir is being rapidly drawn down from Normal Water Surface to Low Water Surface and there is residual pore pressure.	Self weight Hydrostatic pressure Pore pressure 0 % of seismic body force	1.25
6	At the end of construction, there is residual pore pressure.	Self weight Pore pressure 50 % of seismic body force	1.20

3.3.2 Loads

(1) Self Weight

Self weight for analyzing the safety of the dam at the end of construction is calculated based on the wet density of materials. Thus, those at the Maximum Water Surface and Low Water Surface of reservoir are estimated on the wet density and saturated density used for the portion above and below the seepage water line respectively.

Self weight will be calculated by following equation:

$$G = W \cdot V$$

Where,

G : self weight (tf)
W : wet or saturated density (tf/m^3)
V : volume of dam body (m^3)

(2) Hydrostatic Pressure

Hydrostatic pressure acts perpendicularly on the surface of the embankment and its value will be determined using the following equation:

$$P = W_o \cdot h$$

Where,

P	:	hydrostatic pressure (tf/m^2)
W_o	:	unit weight of water (1.0 tf/m^3)
h	:	depth of water (m)

(3) Pore Pressure

Pore pressure is assumed to act perpendicularly on sliding faces. In relation to the condition of the dam, three cases are considered in the calculation of pore pressure. At the end of construction, pore pressure is considered, which will be estimated by using 50 % of the self-weight above the point of the sliding face. At the Normal Water Surface, pore pressure that develops by seepage is considered. At rapid drawdown residual pore pressure is considered.

(4) Seismic Body Force

For the seismic body force, the value of weight of the embankment multiplied by a seismic coefficient is applied and treated to act horizontally. The force can be calculated as follows:

$$G_k = G \cdot k$$

Where,

G_k	:	seismic body force (tf)
G	:	self weight (tf)
k	:	seismic coefficient

3.4 Slope Stability Analysis

(1) Slip Circle Method

The circular slip surface by using the methods of slices will be applied for checking the safety factor against sliding failure in the six conditions mentioned above (see 3.3). The stability analysis in carried out using the effective stress method. The following formula will be used:

$$SF = \frac{\Sigma\{C' * L + (N - U - Ne) * \tan \phi'\}}{\Sigma(T + Te)}$$

Where,

SF : safety factor

N : normal force acting on slip circle (tf/m)

T : tangential force acting on slip circle (tf/m)

U : pore pressure acting on slip circle (tf/m)

Ne : normal force of earthquake load acting on slip circle (tf/m)

Te : tangential force of earthquake load acting on slip circle (tf/m)

ϕ' : effective angle of Internal friction (°)

C' : effective cohesion of materials on slip circle (tf/m^2)

L : arc length of slip circle (m)

(2) Surface Sliding Method

The factor of safety against plane surface sliding of cohesionless material is given by the following equation. This equation is derived under the assumption that the slope of cohesionless material extends uniformly and semi-infinitely. This equation can be applied for the case of the reservoir being full and the upstream slope being fully saturated. In case of the downstream slope, γ_{sat} and γ_{sub} are equal.

$$SF = \frac{\{1 - k * (\gamma_{sat} / \gamma_{sub}) * \tan \theta\}}{\{\tan \theta + k * (\gamma_{sat} / \gamma_{sub}) * \tan \phi\}}$$

Where,

SF : safety factor

θ : slope gradient (°)

k : seismic coefficient

ϕ : effective internal friction angle (°)

γ_{sat} : saturated density (tf/m^3)

γ_{sub} : submerged density (tf/m^3)

(3) Safety Factor against Sliding

Safety Factor of stability analysis must satisfy following conditions, in accordance with the Indonesian criteria.

Reservoir water Surface	Earthquake	Safety Factor
Normal Water Surface	100 %	1.20
Normal Water Surface	0 %	1.50
Maximum Water Surface	0 %	1.20
Rapid Drawdown to Low Water Surface	100 %	1.10
Rapid Drawdown to Low Water Surface	0 %	1.25
End of Construction	50 %	1.20

3.5 Foundation Rock

The foundation rock consists of Damar Formation from the later period of Tertiary to Quaternary. River bed deposit and talus deposit are distributed as secondary sediment.

Damar Formation formulates alternation, which consist of volcanic breccia units, tuff, sandstone and conglomerate. It is divided into two (2) strata of Pyroclastic Rock Units and three (3) strata of Sedimentary Rock Units and they show almost level geological structure.

The Pyroclastic Rocks mainly consist of volcanic breccia, and mafic tuff and andesite lava, which exist partially. Sedimentary rock forms complicated alternation, which consist of tuff, conglomeration and sandstone.

(1) Estimated Shear Strength Elastic Modulus and Deformation Modulus

The foundation rock distribution at the damsite composed of soft rock. It is classified into four (4) classes based on their hardness and cementation, namely D class as the worst class, CL Class, CM-L class and CM-H class as the best class.

The estimated shear strength and modulus of each rock unit were decided by the results of the rock tests, in-situ shearing tests and loading tests for each geological unit and rock class. The estimated values are summarized bellow.

Geological Unit	Rock Class	Elastic Modulus	Deformation Modulus	Estimated Shear Strength
Upper Sedimentary Rock	CL class	3,000 – 5,000 kgf/cm^2	1,500 – 3,000 kgf/cm^2	τ_o=30 tf/m^2 (f=0.7)
	CM-L class	9,000 – 12,000 kgf/cm^2	5,000 – 7,000 kgf/cm^2	τ_o=45 tf/m^2 (f=0.8)
Upper Pyroclastic Rock	CM-H class	–	–	τ_o=50 tf/m^2 (f=0.8)
Middle Sedimentary Rock	CM-L class CM-H class	–	–	τ_o=45 tf/m^2 (f=0.8)
Lower Pyroclastic Rock	CM-H class	–	–	τ_o=50 tf/m^2 (f=0.8)

Note f : coefficient of internal friction

(2) Permeability

The permeability of the foundation rock at the damsite is described below based on the lugeon test result.

(a) Riverbed

The permeability of the lower pyroclastic rock unit that exists between EL.50 m and EL.80 m is small. Impervious rock foundation with less than 5 lugeon is confirmed below EL.60 m.

The permeability of the lower sedimentary rock unit that exists deeper thanEL.50 m shows more than 20 lugeon and confined groundwater with maximum pressure of 2.0 kgf/m^2 is confirmed.

(b) Left and Right Abutment

The permeability of the upper pyroclastic rock unit and middle sedimentary rock unit which are distributed between EL.80 and EL.120 m shows less than 10 lugeon. Seventeen percent of the lugeon test results in them shows average critical pressure of 6 kgf/cm^2.

At the left and right abutments, less pervious rock zone with less than 5 lugeon rises up to as high as the normal water level of the reservoir. However, the groundwater level has not yet observed at the level higher than the normal water level at the left abutment.

3.6 Settlement Analysis

Most settlement is completed during the construction period. In recent year, particularly, sufficient compaction is performed by heavy duty compaction machine, and the settlement occurring after dam completion is very small or less than 1 % in most cases. To estimate total settlement the following of elasticity formula should be applied:

$$S_{tot} = 0.001 \cdot H^{3/2}$$

$$S_{tot} = \frac{\gamma}{2E} \cdot H^2 \cdot T$$

$$E = \frac{(P_o - P_x)(1 + e_o)}{e_o - e_x} = \frac{1}{-m_v}$$

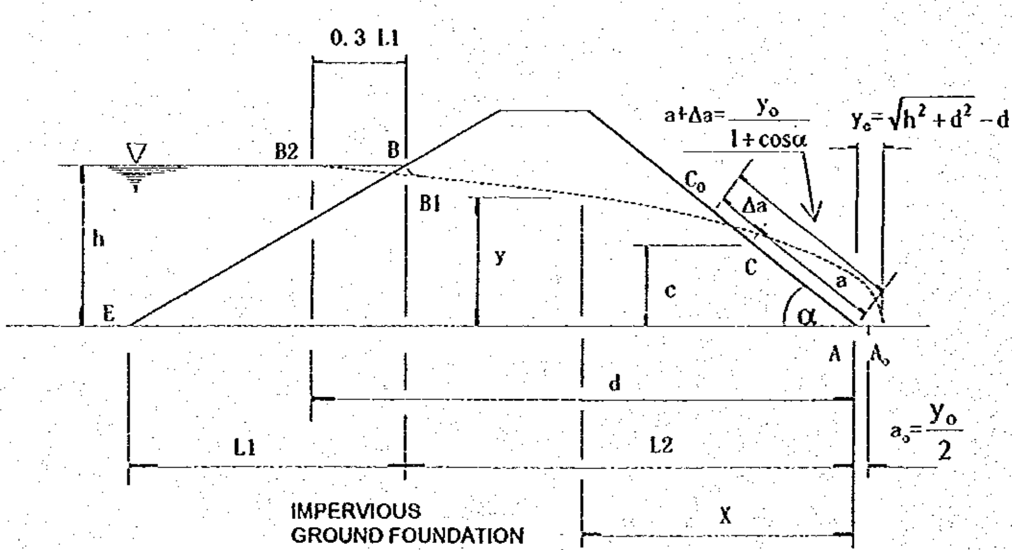
Where,

H_e	:	height of seismic water wave (m)
S_{tot}	:	total Settlement (m)
H	:	dam height (m)
γ	:	specific gravity of fill material
E	:	elasticity from consolidation test (tf/m²)
T	:	coefficient of settlement
P_o	:	initial loading pressure (tf/m²)
P_x	:	pressure after thickness of fill at x meter (tf/m²)
e_o	:	pore value at P_o
e_x	:	pore value at P_x
m_v	:	coefficient of volume compressibility

Extra embankment is provided on the dam crest to maintain the freeboard compensating for possible future settlement on the dam body and the foundation.

3.7 Seepage Analysis

3.7.1 Flow Net Method



Free Surface of Seepage Flow

According to A. Casagrande, the free surface of seepage flow (hereafter: top flow line) in the dam body (to be adequate distance from both upstream and downstream slopes) coincides with standard parabola developed by J.S. Kozeny for a dam with downstream slope at 30° degree to the horizontal.

As shown in the figure, this parabola starts at point B2, slightly upstream of point B, while C_o is obtained from the intersection of parabola with the downstream slope locates slightly higher than C, the actual breakout point of seepage on the downstream slope. The fundamental parabola concerned with the top flow line is as follows:

$$x = \frac{y^2 - y_o^{\ 2}}{2y_o}$$

$$y = \sqrt{2y_o x + y_o^{\ 2}}$$

$$y_o = \sqrt{h^2 + d^2} - d$$

Where,

h	:	vertical distance between A and B (m)
d	:	horizontal distance between B_2 and A (m)
L_1	:	horizontal distance between B and E (m)
L_2	:	horizontal distance between B and A (m)
A	:	toe of downstream slope on pervious portion
A_o	:	origin of coordinate which is $y_o/2$ downstream from A
B	:	intersection of water level and upstream slope
B_1	:	intersection of the parabola with vertical line through B
B_2	:	the point located at 0.3 L_1 upstream from B

However, to determine the top flow line, some corrections to the parabola obtained in the above manner must be made. One, the entrance point to dam body, is at right angle to upstream slope that is simultaneously an equipotential line. Other, the breakout point so as the parabola, does not appear outside of the slope.

The top flow line (B-C-A) is obtained by corrections to the fundamental parabola (B_2-C_0-A_0) for which the entrance point is as described above and C is lowered to C_0 with slope of Δa. The Δa exhibits a different value according to the angle of slope on the discharge face (at breakout point) and can be found by the following equation.

$$a + \Delta a = \frac{y_o}{1 - \cos\alpha}$$

Where,

a	:	slope distance between point A and C (m)
Δa	:	slope distance between point C_0 and C (m)
α	:	slope angle on discharge face (degree)

In the case of α < 30 degree, a can be obtained by the following equation.

$$a = \sqrt{h^2 + d^2} - \sqrt{d^2 - h^2 \cot^2 \alpha}$$

3.7.2 Finite Element Method

The velocity with which water seeps under pressure gradient through the void spaces of a fine and porous soil is directly related to the first power of the hydraulic gradient. Darcy's law that is written as below can express this theory.

$$V = k \cdot i = k \frac{dh}{dl}$$

Where,

V	:	discharge velocity of seeping water (m/sec)
k	:	Coefficient of permeability (m/sec)
i	:	hydraulic gradient
h	:	the pressure head (m)
l	:	length of the seepage path (m)
A	:	cross-sectional area through which the water is seeping (m^2)
Q	:	quantity of seepage per unit of time (m^3/sec)

A seepage analysis of dam embankment and foundation is conducted based on Darcy's law applying the finite element method (FEM) in the two dimensional porous model. The base equation is as follows:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right) + Q = 0$$

Where,

h	:	total head (= $p/\gamma_w + y$) (m)
t	:	time (sec)
k_x, k_y	:	permeability coefficient at x, y direction (m/sec)
p	:	hydrostatic pressure at the point (tf/m^2)
γ_w	:	unit weight of fluid (tf/m^3)
y	:	elevation (m)
Q	:	discharges (m^3/sec)

In case of $k_x = k_y$

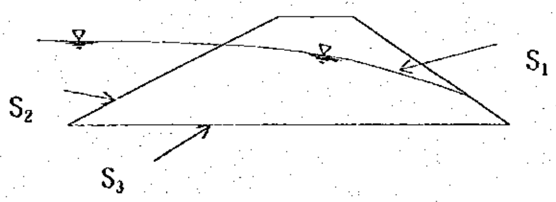
$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+Q=0$$

In case of the steady-stade seepage, the above equation becomes Laplace equation.

$$Q=0$$

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=0$$

To solve above equation, the following boundary conditions are considered:



(1) Each free water surface, S_1;

$h = Y(x, y, t)$

(2) on the dam body under the reservoir water, S_2;

$h = h(t)$

(3) on the foundation of the dam body, S_3;

$$k_x \frac{\partial h}{\partial x} l_x + k_y \frac{\partial h}{\partial y} l_y = q$$

Where,

q	:	inflow – outflow quantity from unit surface (m^3/sec)
l_x, l_y	:	direction cosine of alignment at boundary surface

Finite element equation for each element can be obtained as follows:

$$[K]\{h\}+\{F\}=0$$

Where,

$[K]$	:	seepage matrix
$\{h\}$	:	unknown potential head at FEM each contact point

Then velocity and quantity of flow to each element is obtained from a value of head at each node through the solution of the equation.

3.7.3 Permeability of Foundation and Dam Body

The permeability of the foundation and dam body were estimation as follows:

Location	Lugeon Value	Coefficient of Permeability (cm / sec)
Foundation	$Lu \leq 2$	2.7×10^{-5}
	$2 < Lu \leq 5$	6.7×10^{-5}
	$5 < Lu \leq 10$	1.3×10^{-4}
	$10 < Lu \leq 20$	2.7×10^{-4}
	$20 < Lu$	6.7×10^{-4}
Grouted Foundation	Curtain Grouting	6.7×10^{-5} (Lu = 5)
	Blanket Grouting	1.3×10^{-4} (Lu = 10)
Dam Body	Impervious Zone	1.0×10^{-5}

3.7.4 Piping Analysis

The dam and its foundation are not designed to fully prevent leakage. It permits leakage within an allowed range. Therefore, the pervious pressure and the hydraulic gradient must be checked so as not to cause seepage failure such as piping. Seepage failure may occur at a non-uniform section of the dam and its foundation, and at the contact area between a crack and the foundation rock.

A theoretical treatment regarding seepage failure is generally different. However, the following method can be used as a reference.

(1) Exit Gradient

Under steady-state condition, the water pressure acting on soil grain is only the pore water pressure. However, for flowing water the soil grain is exposed to the percolation water pressure. When the hydraulic gradient increases over a certain limit, blowup or boiling can occur with the possibility of potential for development of piping failure. The safety factor (SF) of the exit gradient against blowup or boiling can be expressed by the following equation:

$$SF = \frac{1}{\text{actual gradient}}$$

The required safety factor should be at least 3 and preferably 5.

(2) Critical Velocity

When the velocity of seepage flow reaches a certain value (critical velocity) movement of soil particles occurs within the dam body and foundation. The allowable velocity of seepage flow in dam body or foundation shall be determined by referring to the critical velocity obtained by the following theoretic equation advocated by Justin:

$$V = \sqrt{\frac{W_1 \cdot g}{F \cdot \gamma_w}}$$

Where,

V	:	velocity of water (cm/sec)
W_1	:	effective weight of particles in water = $1/6\ (G_s - 1) \cdot \pi\ d^3$ (gf)
G_s	:	specific gravity of soil particle
d	:	diameter of soil particle (cm)
g	:	gravity acceleration (980 cm/sec^2)
F	:	Area of particle exposed to flow = $1/4\ \pi\ d^2$ (cm^2)
γ_w	:	weight of water = (gf/cm^3)

Therefore, $C = \sqrt{\frac{2}{3}(G_s - 1) \cdot d \cdot g}$

3.7.5 Volume of Seepage Water

The rate of seepage through the dam body and foundation is obtained from the following equation:

$$q = k \cdot y_o$$

Where,

q	:	rate of seepage per unit length in axial direction
k	:	permeability coefficient (cm/sec)
y_o	:	$\sqrt{h^2 + d^2} - d$ (refer to (3.7.3))

CHAPTER 4 HYDRAULIC DESIGN

4.1 Spillway

Side channel spillway with bathtub type overflow weir is selected considering landform in the neighborhood of damsite. This type mainly consists of five (5) portions, namely, overflow weir, side channel, control portion, chute and stilling basin.

The design formulas for each portion are described hereunder.

4.1.1 Freeboard and Clearance

(1) Freeboard of Spillway

Emergency spillway should be designed to accommodate the Probable Maximum Flood (PMF) for fill type dam (by request latter from Directorate of Technical Guidance on 19 Feb. 1998).

The minimum freeboard for spillway without gate should not be less than 0.75 m, and for a spillway with gate 1.25 m.

(2) Clearance

Bridges will be designed above an emergency spillway at the dam crest. Clearance of bridge girders above the maximum flow surface should not be less than 0.75 m for a fill type dam (by the request letter from Directorate of Technical Guidance of DGWRD on 19 Feb. 1998).

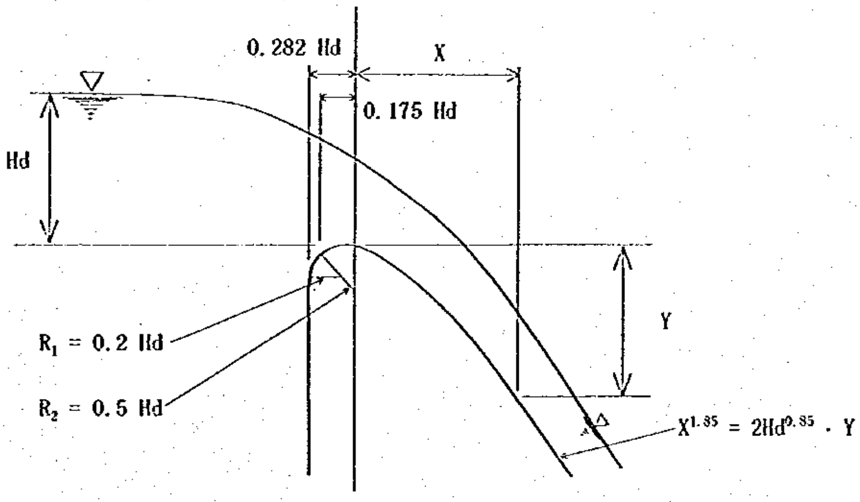
4.1.2 Overflow Weir

<u>Shape of Overflow Crest</u>

Shape of overflow crest shall be based on the underside surface of free nappe over knife edged weir for the purpose of obtaining coefficient of overflow discharge as large as possible without creating negative pressure at the overflow crest.

The standard type of overflow crest is a shape, which simulate to that of underside surface of free nappe over vertical knife edge weir, assuming design head (Hd) as overflow head during design flood.

For standard type, Harrold's shape is used mostly as shown below:



Harrold's Standard Type of Overflow Crest

Overflow Discharge

Basically the overflow discharge is given by the following formulas:

$$Q = C \cdot B \cdot H^{3/2}$$

$$C = 1.60\frac{1+2a(H/Hd)}{1+a(H/Hd)}$$

$$a = \frac{Cd-1.6}{3.2-Cd}$$

$$Cd = 2.20 + 0.416\,(Hd/W)^{0.99}$$

Where,

Q	:	overflow discharge (m^3/sec)
C	:	coefficient of overflow discharge
B	:	overflow width (m)
H	:	overflow head at crest (m)
a	:	constant value
Hd	:	design head (m)
Cd	:	coefficient of overflow discharge when H = Hd
W	:	height of overflow weir (m)

4.1.3 Side Channel

Set up on Side Channel Section

(1) Side channel slope at weir side is assumed to be 1 : 0.7, and channel wall of the opposite bank (as a rule : in-situ rock) is assumed to be vertical wall. But, when the opposite bank is not assumed to be vertical wall, some slope to the opposite bank is assumed. However, in this case, some hydraulic model test should be carried out for determination of hydraulic dimension.

(2) Bottom slope of side channel, i_1, is assumed to be $\leq 1/13$

(3) Ratio of bottom width at the downstream end of the side channel *B*, to the water depth, *d*, (d/B) is assumed to be about 0.5.

(4) Froude number at the end of side channel is assumed to be Fr < 0.5, generally Fr = 0.44.

$$\mathrm{Fr} = \frac{V}{\sqrt{gD}}$$

Where,

$$D = \left(\frac{A}{T}\right)$$: hydraulic depth (m)

A : cross section area (m^2)

T : width of water surface (m)

g : gravity acceleration (m/sec^2)

(5) Water depth (over the weir crest) at the upper end of the side channel is assumed to be lower than 1/2.5 of overflow depth.

(6) Floodway with gentle slope is succession to the side channel should be designed under the formula mentioned in (4) above and must be gentle in enough.

(7) Overflow weir at the downstream end of the floodway with gentle slope is in succession to the steep channel.

Overflow side wall at the junction between side channel and floodway with gentle slope is considered to be well in gentle or rapid shortening.

The Calculation Formula is as follows:

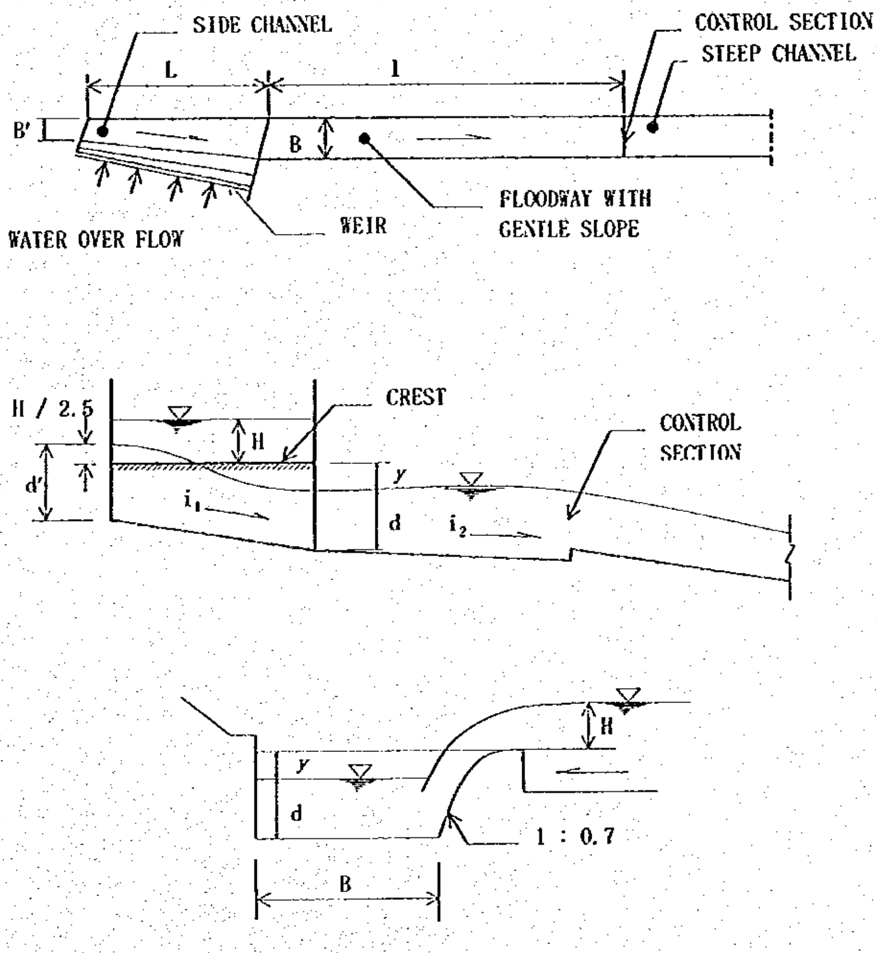
$$\frac{d^3B^2\left(1+\frac{m}{2}\frac{d}{B}\right)^3}{\left(1+m\frac{d}{B}\right)}=\frac{Q^2}{gFr^2}$$

If value on, m = 0.7, d/B = 0.5, are selected, the following formula can be introduced.

$$d=0.463\left(\frac{Q}{Fr}\right)^{2/5}\ (\text{unit}: m)$$

Also, Fr = 0.44 to 0.50, formula is as follows.

$$d=(0.643\sim 0.61)\,Q^{2/5}\ (\text{unit}: m)$$



Explanation Figure on Side Channel

On formula mentioned above, the condition introduced is assumed to be d/B = 0.5, and then following formula is formed.

$$B = 2d$$

And also, section area A is indicated by the following formula.

$$A = \left(B + \frac{0.7d}{2} \right) d$$

Where, bottom width Bx, at distance, x, from the downstream end of the side channel is obtained by formula.

$$Bx = B\left\{1 - (1 - \alpha)\frac{x}{L}\right\}$$

Where,

- α : B' / B, generally, B'/B is assumed to be 0.5.
- B : bottom width at the downstream end of the side channel (m)
- B' : bottom width at the upstream end of the side channel (m)
- x : distance from beginning at the downstream end to upper. (m)
- L : the full length of the side channel (m)

Bottom height of the side channel can be obtained by the following formula.

$$Zx = i_1 \cdot x$$

Where,

- Zx : bottom height of x point (in standard as bottom height at the downstream end of the side channel).
- i_1 : bottom slope of side channel; $i_1 \leq 1/13$

Full length, l, of gentle slope channel in succession to the side channel is as follows.

$$l \geq 4d$$

Where,

d is water depth at the downstream end of the side channel.

The bottom slope of gentle slope channel is given by the following formula in the case of rectangular form channel.

$$i_2 = g * n^2 Fr^2 \frac{\left(1+2\frac{d}{B}\right)^{4/3}}{d^{1/3}}$$

Where,

- B : bottom width at the downstream end of the side channel (m)
- i_2 : gradient of gentle slope channel (floodway)
- g : gravity acceleration (9.8 m/sec^2)
- n : Manning's n roughness coefficient of canal
- Fr : Froude number

Calculation on Flow Profile

The water surface line in the side channel is calculated based on Momentum Equation as following.

$$\Delta h = \frac{Q_1(v_1+v_2)}{g(Q_1+Q_2)}\left(\Delta v + \frac{q * v_2 * \Delta x}{Q_1}\right)$$

Where,

- Δh : a rise in water level in the Δx section (m)
- Q_1 : discharge of downstream section (m^3/sec)
- Q_2 : discharge of upstream section (m^3/sec)
- v_1 : average velocity at the downstream section (m/sec)
- v_2 : average velocity at the upstream section (m/sec)
- q : inflow per unit width (overflow discharge) (m^2/sec)
- Δv : $v_1 - v_2$ (m/sec)
- g : gravity acceleration (9.8 m/sec^2)

Calculation on flow profile is carried out by the trial method, from downstream to upstream section.

4.1.4 Control Portion

The channel flow in the floodway with gentle slope should be maintained at sub critical stage for good hydraulic performance. This can be achieved by establishing a control section downstream from the floodway. The critical depth for flow at the Control section can be determined as follows:

$$h_c = \sqrt[3]{\frac{Q^2}{gB^2}}$$

Where,

h_c : the critical depth (m)
Q : design discharge (m^3/sec)
g : gravity acceleration (m/sec^2)
B : bottom width at the downstream end of the side channel (m)

4.1.5 Chute Structure

Basic Formula of Flow Velocity

Type of flow in the Chute Structure depend on the froude number as follow :

$$Fr = \frac{v}{\sqrt{gd}}$$

Where,

Fr : Froude number
v : flow velocity (m/sec)
g : gravity acceleration (9.80 m/sec^2)
d : depth of flow (m)

The type of flow is as follows:

If	$Fr < 1$	means sub critical flow type
	$Fr = 1$	means critical flow type
	$Fr > 1$	means flow fall type

The flow velocity will be determined by Manning formula:

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

Where,

R : hydraulic radius = A/P (m)
A : flow section area (m^2)
P : wetted perimeter (m)
n : Manning's roughness coefficient
S : energy gradient

Velocity and Flow Depth in Chute

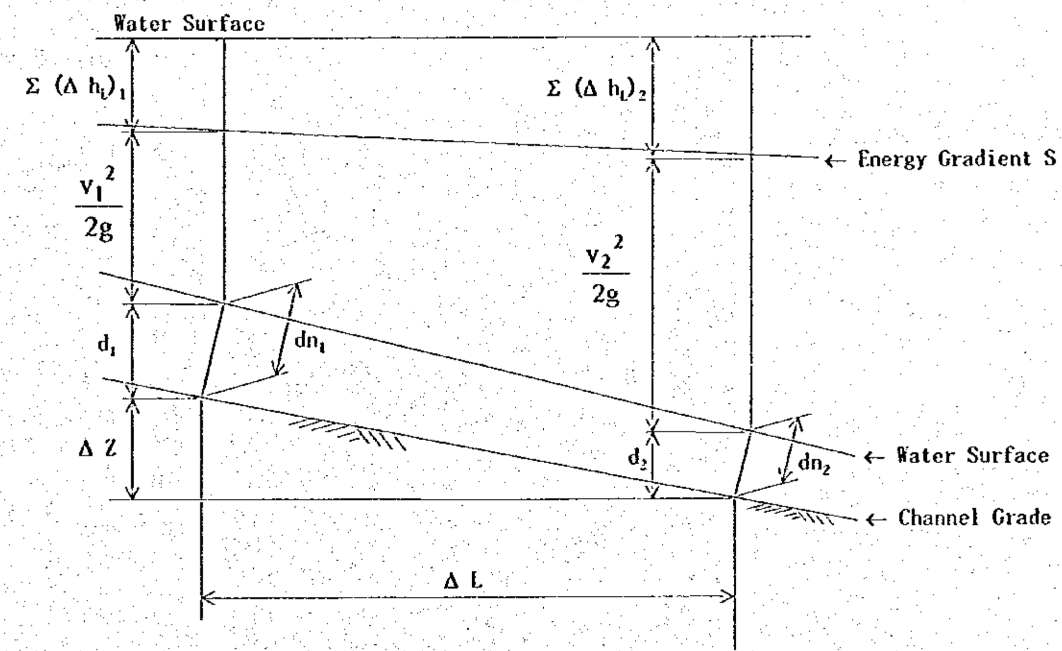
The velocity and depth of free surface flow conform to the principle of the conservation of energy as expressed by the Bernoulli's theorem. It states "The absolute energy of flow at any cross section is equal to the absolute energy at a downstream section plus intervening loses of energy". As applied to the figure as shown below, this relationship can be expressed as follows:

$$\Delta Z + d_1 + \frac{v_1^2}{2g} = d_2 + \frac{v_2^2}{2g} + \Delta h_L$$

$$\Delta h_L = S \cdot \Delta L = \frac{S_1 + S_2}{2} \cdot \Delta L$$

Where,

ΔZ : difference of elevation (m)
d_1, d_2 : vertical depth of flow (m)
v_1, v_2 : velocity (m/sec)
g : gravity acceleration (9.8 m/sec^2)
Δh_L : energy losses (m)
S : average energy gradient in the section
S_1, S_2 : energy gradient at the section
ΔL : distance between sections (m)



Flow Depth in Chute

The coefficient of roughness n, will depend on the nature of the channel surface. For conservative design the frictional loss should be maximized when evaluating the depth of flow and minimized when evaluating the energy content of the flow. For determining the depth of flow in a concrete line Channel, a value of n about 0.018 should be assumed, in order to account for air swell, wave action, etc. For determining specific energies of flow needed for designing the dissipating device, a value of n of about 0.008 should be assumed.

Freeboard of Chute

In a chute conducting flow at supercritical stage, the surface roughness, wave action and air bulking are related to the velocity and energy content of the flow. The energy per meter of width can express in terms of velocity and depth of flow.

An empirical expression based on this relationship which gives a reasonable indication of desirable freeboard values is as follows:

$$F = 0.61 + 0.037 \cdot v \cdot d^{1/3}$$

Where,

F : freeboard (m)
v : velocity (m/sec)
d : depth of flow (m)

Profile of Chute

To avoid the water to spring away from the floor and reduce the surface contact pressure, the floor shape for convex curvature should be made slightly flatter than the trajectory of a free-discharging jet. It is issuing under a head equal to the specific energy of flow as it enters the curve. The curvature should approximate a shape defined by the following equation:

$$y = x\tan\theta + \frac{x^2}{K[4*(d+hv)\cos^2\theta]}$$

Where,

θ : slope angle of floor upstream from the curve (degree)
d : depth of flow (m)
hv : energy of velocity (m/sec)

To assure positive pressure along the entire contact surface of the curve K should be equal to or greater than 1.5.

4.1.6 Stilling Basin

The design discharge for stilling basin should be the largest figures of he following discharges.

- Outflow capacity at the Surcharge Water Surface (150 m^3/s)
- 100-year probable flood discharge (340 m^3/s), which is estimated by the hydrological model of the dam site itself having the catchment area of 53 km^2.

In this case, the design discharge of stilling basin becomes 340 m^3/s.

Regarding the energy dissipater type, a hydraulic jump basin with endsill will be employed.

The height of side wall shall be designed so as not to overflow it when the maximum outflow discharge (Q_d = 1,290 m^3/s) after regulating the design discharge for emergency spillway (PMF) flowed into the stilling basin.

The design formula for stilling basin is given as follows:

$$v_1 = 0.95\sqrt{2g(H_S - H_b)}$$

$$h_1 = \frac{Q}{A} = \frac{Q}{B \cdot v_1}$$

$$Fr = \frac{v_1}{\sqrt{gh_1}}$$

$$h_2 = \frac{h_1}{2}\left(\sqrt{1+8Fr^2} - 1\right)$$

$$L \geq 4.5\ h_2$$

$$\frac{D}{h_1} = \frac{\left(1+2Fr^2\right)\sqrt{1+8Fr^2} - 1 - 5Fr^2}{1+4Fr^2 - \sqrt{1+8Fr^2}} - \left(\frac{\sqrt{g}}{2}Fr\right)^{\frac{2}{3}}$$

$$H = \left(\frac{Q_d}{2B}\right)^{\frac{2}{3}}$$

Where,

v_1 : velocity at entrance (m/s)

g : gravity acceleration (9.8 m/s^2)

H_s : water surface in gentle slope channel (EL. 152.083 m)

H_b : apron elevation (EL. m)

h_1 : water depth at entrance (m)

Q : design discharge (340 m^3/s)

B : width of apron (m)

Fr : Froude number

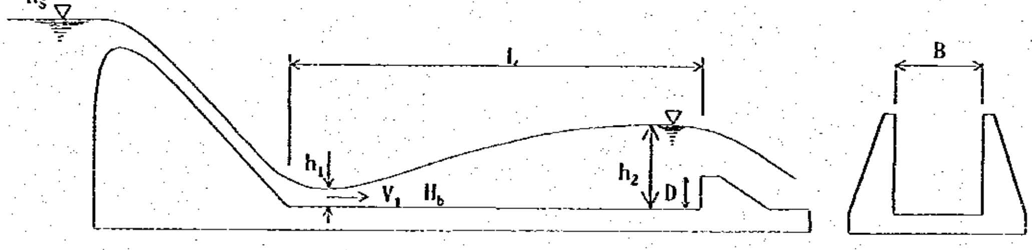
h_2 : conjugate depth (m)

L : length of apron (m)

D : height of endsill (m)

H : overflow depth (m)

Q_d : design discharge for side wall height (1,290 m^3/s)



Hydraulic Jump Basin with Endsill

4.2 Diversion Structures

Diversion structures consist of cofferdam, inlet structure and diversion tunnel. These components will be designed to accommodate the 25-year probable flood.

4.2.1 Cofferdam

The crest elevation of cofferdam is determined by taking freeboard of 0.5 m against 25-year probable flood with the peak discharge of 280 m^3/sec. The dam body of cofferdam is to be filled with excavated materials.

The stability of the cofferdam was also examined by the same method as that for main dam.

4.2.2 Inlet Structure

The inlet structure consists of the overflow weir and the inlet guide wall. The water will be dammed up at the entrance of the tunnel so that design discharge of 280 m^3/s can be introduced into the tunnel smoothly by supercritical flow. The inlet guide wall may act as barrier and prevent the tunnel from flowing sand and cobbles.

Dimension of the inlet structure will be designed in accordance with the following formula:

$$H = (1 + fe)\frac{V_2^{\,2}}{2g} + h_2$$

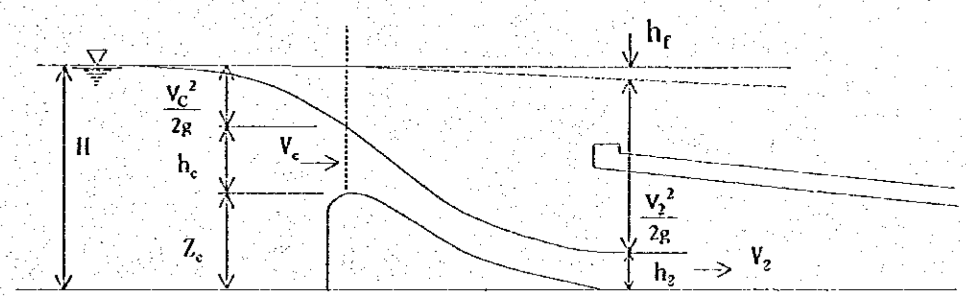
$$Z_C = (1 + fe)\frac{V_2^{\,2}}{2g} + h_2 - \frac{Vc^2}{2g} - hc$$

$$V_C = \frac{Q_C}{B \cdot h_C}$$

$$h_C = \sqrt[3]{\frac{Q_C^{\,2}}{gB^2}}$$

Where,

- H : total water depth (m)
- fe : entrance energy loss coefficient (0.2)
- V_2 : flow velocity at entrance of tunnel (m/s)
- h_2 : flow depth at entrance of tunnel (m)
- g : gravity acceleration (9.8 m/s^2)
- V_C : critical flow velocity at top of overflow crest (m/s)
- h_C : critical water depth at overflow crest (m)
- Q_C : design discharge (m^3/s)
- B : bottom width of overflow crest (m)
- Z_C : height of overflow weir (m)



Flow on Overflow Weir of Inlet Structure

4.2.3 Diversion Tunnel

The tunnel slope, dimension, roughness and the inlet and outlet geometry define the flow capacity of diversion tunnel.

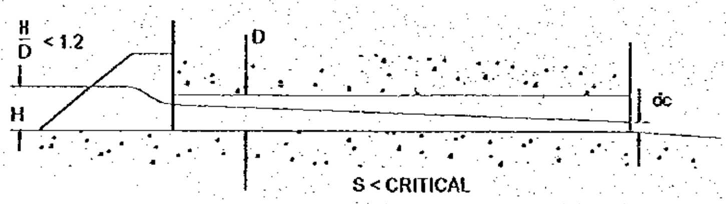
The tunnel cross section will be designed for the following criteria:

- Shape of the cross section	:	standard horseshoe with 2 r
- Design discharge	:	25-year probable flood of 280 m^3/sec
- Type of flow	:	open channel (less than 80 % of tunnel height)
- Longitudinal gradient	:	i = 1/30
- Roughness coefficient	:	n = 0.015 (concrete lining)

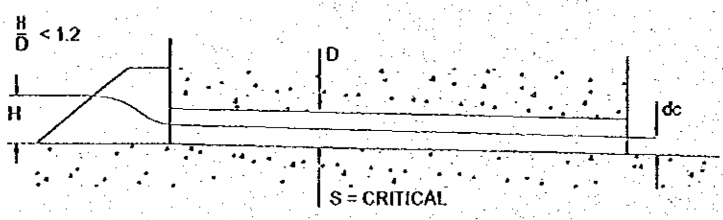
For either the mild or steep slope, the control may be either at the inlet or the outlet depending on the relation between entrance geometry and head.

When H/D is smaller than 1.2, the various conditions which may govern a particular flow are illustrated as shown below.

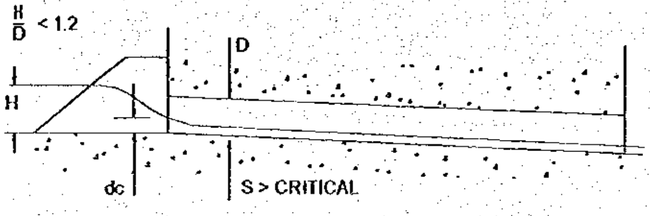
1) Mild Slope. Subcritical flow, control at critical depth at outlet



2) Critical slope. Subcritical flow control at critical depth at outlet



3) Steep slope. Supercritical flow, control at critical depth at inlet



Typical Flow Condition of Diversion Tunnel

When the tunnel is on a mild slope and the outlet discharges freely the flow will be controlled by critical depth at the outlet. This condition is shown as 1) in the figure.

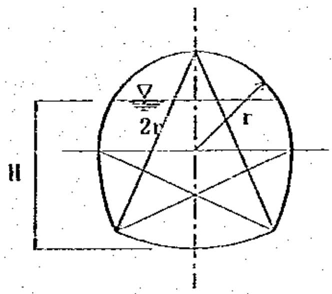
When the tunnel is on a steep slope, the flow will be controlled by critical depth at the inlet, as indicated by conditions 3) in the figure.

Discharge amount was calculated in accordance with the hydraulic formula as shown below:

$$Q = \frac{1}{n} A \cdot R^{2/3} \cdot i^{1/2}$$

Where,

R : hydraulic radius (m) = A/P
A : flow sectional area (m^2)
P : wetted perimeter (m)
n : Manning's roughness coefficient (0.015)
i : longitudinal gradient



H/r	A/r²	R/r	(R/r)^2/3
2.0	3.317	0.507	0.635
1.9	3.258	0.578	0.693
1.8	3.153	0.600	0.711
1.7	3.021	0.611	0.720
1.6	2.870	0.613	0.721
1.5	2.703	0.608	0.717
1.4	2.524	0.598	0.709
1.3	2.337	0.584	0.699
1.2	2.143	0.564	0.681
1.1	1.946	0.541	0.663
1.0	1.746	0.514	0.641
0.9	1.546	0.484	0.616
0.8	1.348	0.450	0.587
0.7	1.150	0.412	0.553
0.6	0.957	0.370	0.515
0.5	0.767	0.322	0.469
0.4	0.583	0.268	0.415
0.3	0.404	0.205	0.347
0.2	0.233	0.133	0.260

4.2.4 Friction Loss

The Energy Losses from frictional resistance of the tunnel will be determined by the following formula:

$$H_f = S \cdot \Delta L$$

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

Where,

g	:	gravity acceleration (9.8 m/s²)
S	:	energy gradient
n	:	Manning's n roughness coefficient
R	:	hydraulic radius (m)
v	:	flow velocity (m/sec)
ΔL	:	Length of section over which losses are being computed (m)

or,

$$H_f = \frac{124.5\,n^2}{D^{1/3}} \cdot \frac{\Delta L}{D} \cdot \frac{V^2}{2g} \quad \text{(for circular section)}$$

$$H_f = \frac{2 \cdot g \cdot n^2}{R^{1/3}} \cdot \frac{\Delta L}{R} \cdot \frac{V^2}{2g} \quad \text{(for non circular section)}$$

Where,

D = Diameter of the tunnel (m)

n = Manning's roughness coefficient

The maximum and minimum values of n that will be used to determine the tunnel size and the energy of flow are as follows:

Material	Max. Value	Min. Value
Concrete pipe or cast in place tunnel	0.014	0.01
Steel pipe with welded joints	0.012	0.01

4.3 Outlet Facilities

4.3.1 Diameter of Outlet Pipe

In accordance with the reservoir purpose, outlet facilities are designed to release water for domestic use and power generation needs.

The intake facilities consist of the inclined inlet structure, the outlet pipe to be connected with the power station and control gate. A branched pipe is connected with the outlet pipe just before the Power Station so as to enable to release the water directly without passing through the power station. The design discharge is given as shown below:

- Minimum Discharge : 0.26 m^3/sec
- Maximum Discharge : 6.0 m^3/sec

The diameter of the outlet pipe is determined at $D = \left(\frac{4 \cdot Q}{\pi \cdot V}\right)^{0.5} = 1.4\ \mathrm{m}$, using the condition of the discharge Q = 6.0 m^3/sec and the velocity V = 4.0 m/sec.

Computation procedure to determine the flow condition in the pipe will be carried out by using continuity equation and Bernoulli's energy equation.

4.3.2 Discharge Coefficient of Control Gate

The basic equation of the discharge through the control gate is expressed as follows:

$$Q = C \cdot A \cdot \sqrt{2 \cdot g \cdot (H - \Sigma h_i)}$$

Where,

Q	:	discharge (m^3/sec)
A	:	area of clear opening of control gate (m^2)
C	:	discharge coefficient
H	:	depth from reservoir water surface to control gate centerline (m)
Σh_i	:	total losses of head (m)
g	:	gravity acceleration (9.8 m/sec^2)

For Jatibarang dam, the jet flow gate type will be used. Discharge coefficient (C) of control gate (jet flow gate) is shown as follows:

Gate open percent (%)	Discharge Coefficient	
	In Air	In Water
10	0.04	0.04
20	0.10	0.10
30	0.18	0.17
40	0.27	0.26
50	0.37	0.35
60	0.47	0.47
70	0.57	0.57
80	0.67	0.67
90	0.77	0.75
100	0.85	0.81

4.3.3 Losses of Head

<u>Entrance Losses</u>

The loss of head at the entrance of a conduit will be determined by the following approach.

$$H_e = K_e \cdot \frac{V^2}{2g}$$

Where,

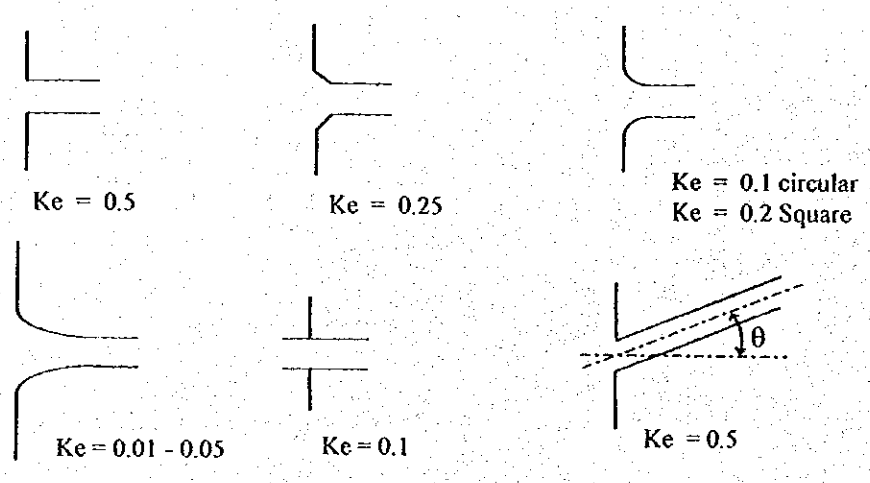
He : entrance loss (m)

K_e : entrance loss coefficient

V : flow velocity (m/s)

g : gravity acceleration (9.8 m/s²)

The values of K_e are as follows:

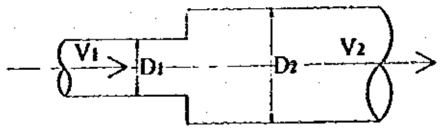


Entrance Loss Coefficient

Transition Losses

(1) Sudden Expansion Loss

$Hse = (V_1 - V_2)^2/(2g) = Kse \cdot V_1^2/(2g)$



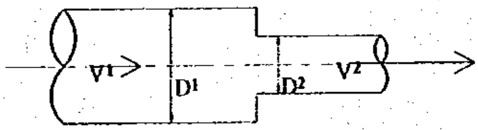
Sudden Expansion Loss

Where,

- Hse : sudden expansion loss (m)
- V_1 : velocity before expansion (m/s)
- V_2 : velocity after expansion (m/s)
- Kse : sudden expansion loss coefficient = $(1 - (A_1/A_2))^2$
  - A_1 : pipe area before expansion (m^2)
  - A_2 : pipe area after expansion (m^2)

(2) Sudden Contraction Loss

$Hsc = (1/Cc - 1)^2 \cdot V_2^2/(2g) = Ksc \cdot V_2^2/(2g)$



Sudden Contraction Loss

Where,

- Hsc : sudden contraction loss (m)
- Cc : coefficient of contraction
- V_2 : velocity after contraction (m/s)
- Ksc : sudden contraction loss coefficient = $(1/Cc - 1)^2$

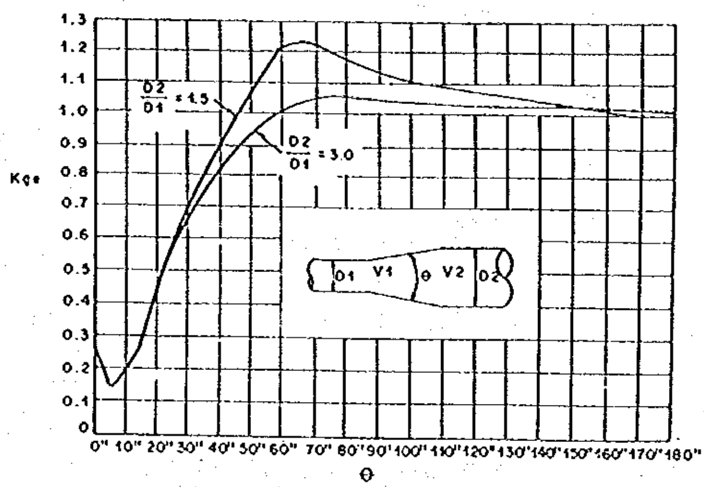
A_1/A_2	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Cc	0.61	0.62	0.63	0.65	0.67	0.70	0.73	0.77	0.84	1.00
Ksc	0.41	0.38	0.34	0.29	0.24	0.18	0.14	0.09	0.04	0.00

(3) Gradual Expansion Loss

$$H_{ge} = K_{ge} \cdot \frac{(V_1 - V_2)^2}{2g}$$

Where,

- H_{ge} : gradual expansion loss (m)
- K_{ge} : gradual expansion loss coefficient
- V_1 : velocity before expansion (m/s)
- V_2 : velocity after expansion (m/s)



Gradual Expansion Loss

(4) Gradual Contraction Loss

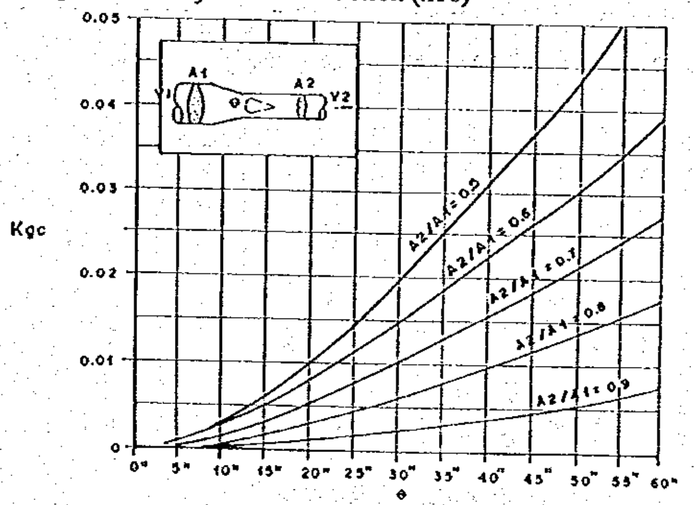
$$H_{gc} = K_{gc} \cdot \frac{V_2^2}{2g}$$

Where,

H_{gc} : gradual contraction loss (m)

K_{gc} : gradual contraction loss coefficient

V_2 : velocity after contraction (m/s)



Gradual Contraction Loss

Bending head losses

$Hb = Kb_1 \cdot Kb_2 \cdot V^2/(2g)$

Where,

Hb : bending head loss (m)

Kb_1 : loss coefficient determined by the ratio bending radius ρ to the pipe diameter D

ρ/D : in case that a center angle of bending is 90°

Kb_2 : ratio of the loss for a center angle θ to the loss for a center angle of 90°

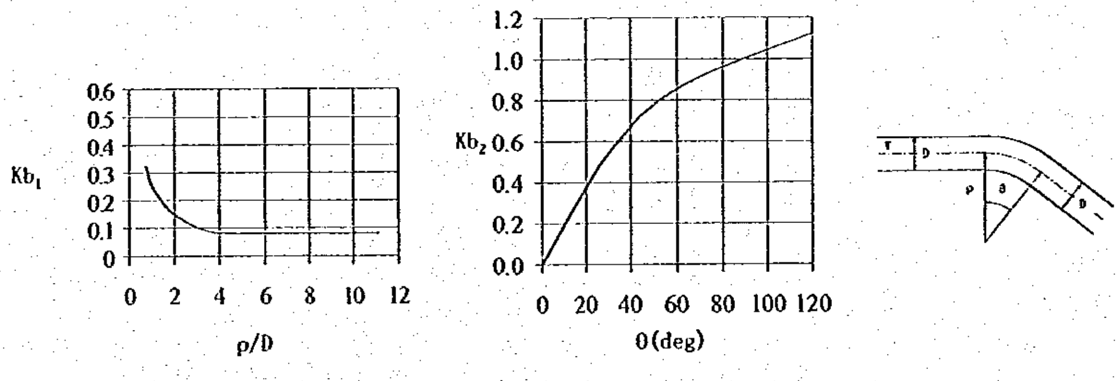
V : flow velocity (m/s)

The following empirical formula is frequently used for Kb_1 and Kb_2:

Kb_1 : $0.131 + 0.1632 \cdot (D/\rho)^{7/2}$

Kb_2 : $(\theta/90°)^{1/2}$

Hb given by the above formula does not include the friction head loss.



(a) Value of Kb_1 (90°)

(b) Value of Kb_2

Bending Head Loss

<u>Branching and Junction Losses</u>

(1) Branching loss

$$H_\alpha - H_\beta = Kd_\beta \cdot V_\alpha^2 / (2g)$$

$$H_\alpha - H_\gamma = Kd_\gamma \cdot V_\alpha^2 / (2g)$$

$$Kd_\beta = 0.95\,(1 - q_\beta)^2 + q_\beta^2\,(1.3\cot(\theta/2) - 0.3 + (0.4 - 0.1\cdot\phi)/\phi^2)(1 - 0.9\sqrt{(\rho/\phi)})$$

$$+0.4\,q_\beta\,(1 - q_\beta)(1 + 1/\phi)\cot(\theta/2)$$

$$Kd_\gamma = 0.58\,q_\beta^2 - 0.26\,q_\beta + 0.03$$

Where,

H_α : working pressure head before branching (m)
(sum of location head and pressure head)

H_γ : working pressure head in branched main pipe (m)

H_β : working pressure head in branched pipe (m)

V_α : flow velocity before branching (m/s)

Kd_β, Kd_γ : head loss coefficient by branching

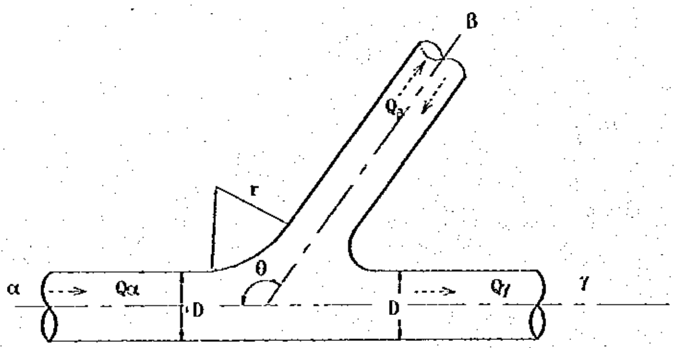
θ : intersecting angle made by original pipe and branched pipe (degree)

ϕ : ratio of sectional area of branched pipe to original pipe

ρ : ratio of tangential radius at branching to diameter of original pipe $= r/D$

r : radius at the portion illustrated below (m)

q_β : ratio of branched discharge Q_β to original discharge Q_α ($= Q_\beta / Q_\alpha$)



(2) Junction loss

$$H_\alpha - H_\gamma = Kc_\alpha \cdot V_\gamma^2 / (2g)$$
$$H_\beta - H_\gamma = Kc_\beta \cdot V_\gamma^2 / (2g)$$
$$Kc_\alpha = -q_\beta^2 (2.59 + (1.62 - \sqrt{\rho})((\cos\theta)/\phi - 1) - 0.62\,\phi) - q_\beta (1.94 - \phi) + 0.03$$
$$Kc_\beta = -q_\beta^2 ((1.2 - \sqrt{\rho})((\cos\theta)/\phi - 1) + 0.8 (1 - 1/\phi^2) - (1 - \phi)(\cos\theta)/\phi)$$
$$-(1 + q_\beta)(0.92 + q_\beta (2.92 - \phi))$$

Where,

H_α : working pressure head in main pipe before junction (m)

H_γ : working pressure head in main pipe after junction (m)

H_β : working pressure head in subsidiary pipe before junction (m)

V_γ : flow velocity in main pipe after junction (m/s)

Kc_α, Kc_β : head loss coefficient of junction

θ : intersection angle made by original pipe and branched pipe (deg)

ϕ : ratio of sectional area of branched pipe to original pipe

ρ : ratio of tangential radius at branching to diameter of original pipe $= r/D$

q_β : ratio of subsidiary pipe discharge Q_β to junction discharge Q_α, given by minus value ($= -Q_\beta / Q_\alpha$)

Gate or Valve loss

$$Hv = Kv \cdot V^2 / (2g)$$

Where,

- Hv : Gate or Valve loss (m)
- Kv : Gate or Valve loss coefficient
- V : Flow velocity (m/s)

Gate or Valve	Kv
High pressure slide gate	$(0.03)^{*1}$ 0.06
Ring follower gate	0.0
Sluice valve	0.06

*1 In case of same structure as main control gate

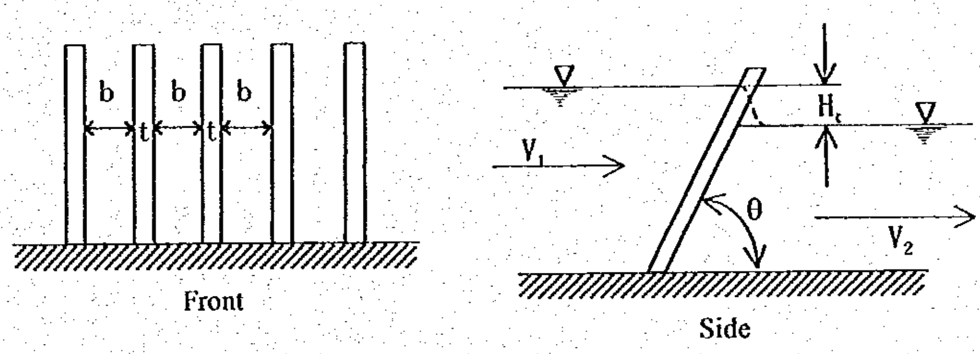
Head Loss through Trash Racks

$$K_t = \beta \cdot \sin\theta \cdot \left(\frac{t}{b}\right)^{4/3}$$

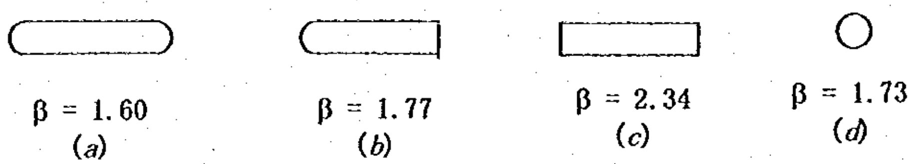
$$H_t = \alpha \cdot K_t \cdot \frac{V_1^2}{2g}$$

Where,

- Ht : head loss through Trash Rack (m)
- K_t : trash racks loss coefficient
- V_1 : mean velocity before trash rack (m/s)
- β : bar shape coefficient
- θ : trash rack inclination (degree)
- t : width of bar (m)
- b : clear span between bars (m)
- α : safety factor for clogging due to trash (=3.0)



Trash Racks



Bar Shape Coefficient

Friction head loss

$$H_f = Kf \cdot L/R \cdot V^2/(2g)$$

$$K_f = 2g \cdot n^2/R^{1/3}$$

Where,

H_f : friction head loss (m)
n : Kutter's coefficient of roughness
L : length of pipe (m)
V : flow velocity (m/s)
R : hydraulic depth of pipe = A/S (m)
A : flow area (m^2)
S : wetted perimeter (m)

For circular pipes, taking D (m) as the internal diameter of the pipe:

$$Hf/L = 124.5n^2/D^{4/3}$$

As for the value of n, 0.01 to 0.014 is applicable to normal steel pipes. In case of calculation for welded steel pipe area, n = 0.012.

The Other head losses

Slight head loss is generated around expansion joints, manholes, etc., and so it is desirable to add some allowances to the sum of head loss.

4.4 Hydropower Generation

General civil design criteria are mostly applied to design of power station. Excavation work of powerhouse foundation is related to that of stilling basin of dam body. Powerhouse structure is designed in accordance with general civil design criteria. Design of outlet gate of powerhouse is carried out in the similar manner as that of intake gate of dam body. Design criteria for steel pipe of water supply is applied also for penstock of powerhouse. In this clause, therefore, calculation of water hammer and stability condition of hydraulic turbine is described as characteristic design criteria for hydropower generation.

(1) Calculation of water hammer

Allievi's formula is employed for calculation of water hammer in steel penstock for power generation.

$$\xi_{x\theta} - \xi_{x1\theta1} = 2\rho(q_{x\theta} - q_{x1\theta1})$$

Where,

ξ = H / H_o
H : water hammer
H_o : static head at the end point of pipe line before closing of valve
x : distance from the location of valve
θ : dimensionless time = t / T = t / (2L/a)
L : length of pipe line
a : propagation speed of water hammer
suffix 1,2 -- t = T, t = 2T –
ρ = $a Q_o / 2gf H_o$
Q_o : discharge in pipe line before closing of valve
g : acceleration of gravity
F : cross area of pipe line
$q_{x\theta}$ = $v_{x\theta} / V_o$
v_o = Q_o / f

(2) Stability condition of hydraulic turbine

$$T_M > 2\, T_w^2$$

Where,

H : water hammer
T_M : $(\pi N_o / 60)^2 (GD^2/P_o)$
T_W : $LQ_o / gf H_o$
N_0 : rated rotation of hydraulic turbine (rpm)
GD^2: fly-wheel effect (tonf-m2)
P_0 : power output (KW)