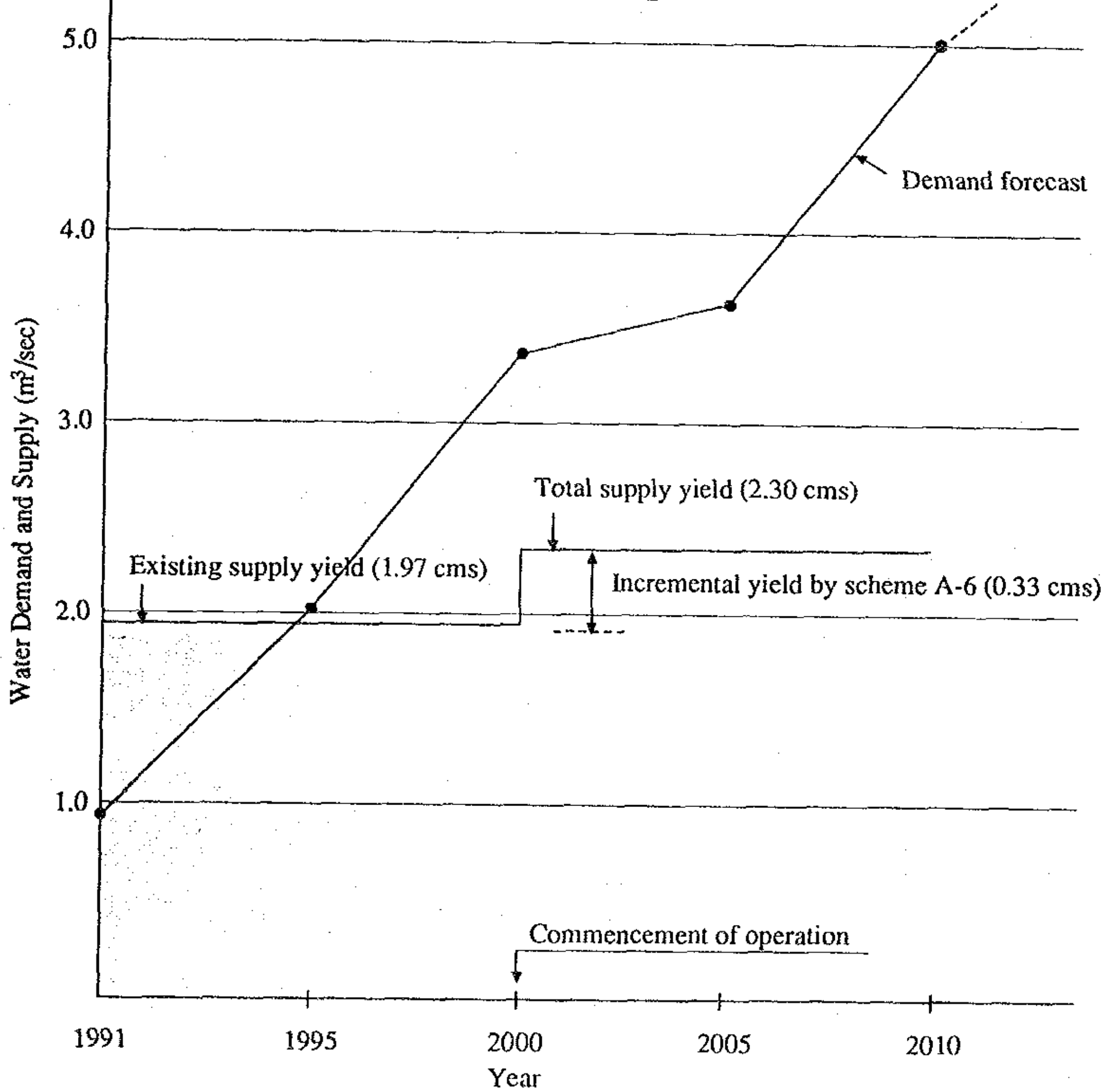
Fig. F-43

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	2.30	3.30	- 1.00
2005	2.30	3.67	- 1.37
2010	2.30	4.94	- 2.64



Water Demand and Supply Program by Scheme A-6

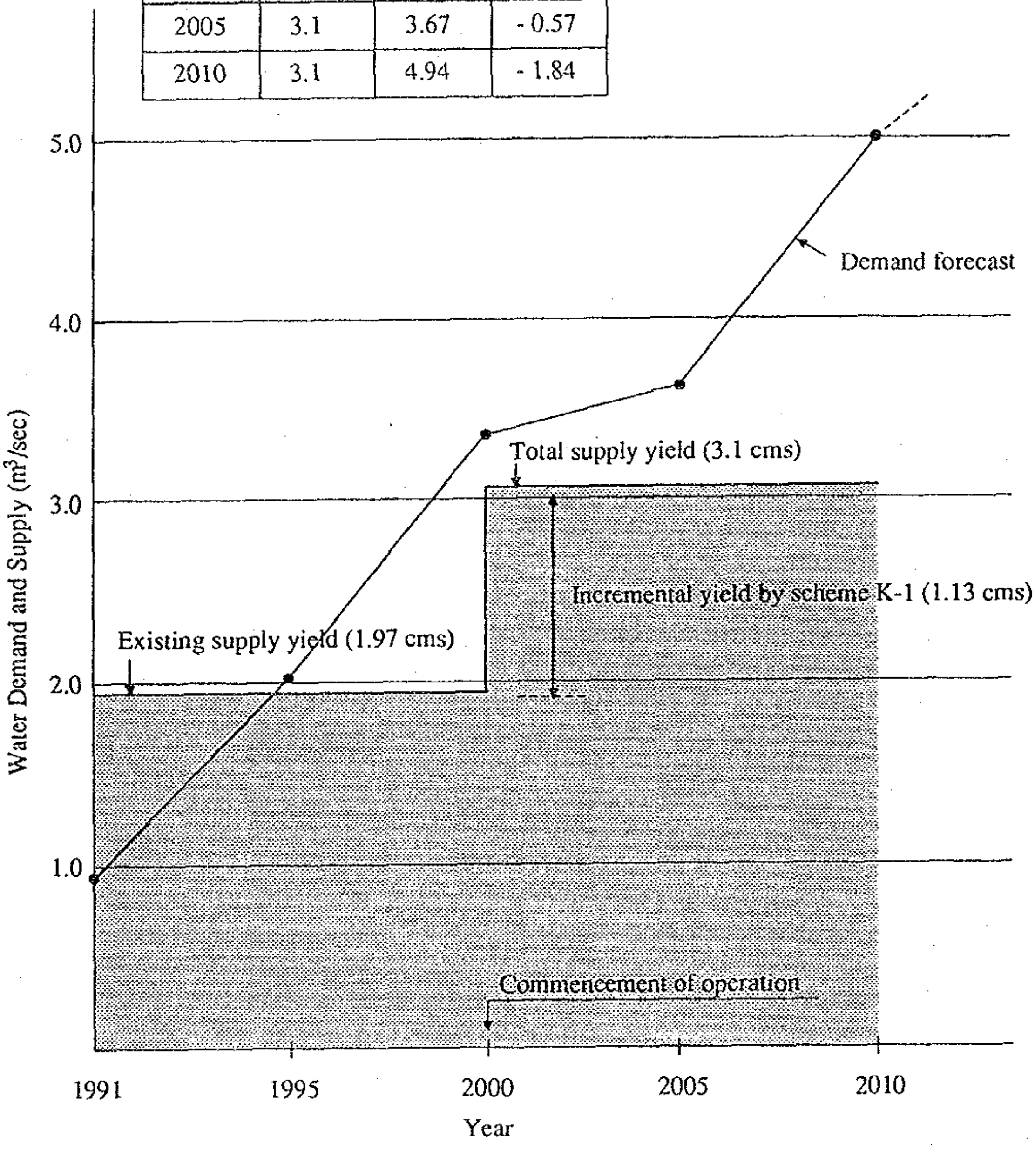
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-44

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.1	3.30	- 0.20
2005	3.1	3.67	- 0.57
2010	3.1	4.94	- 1.84

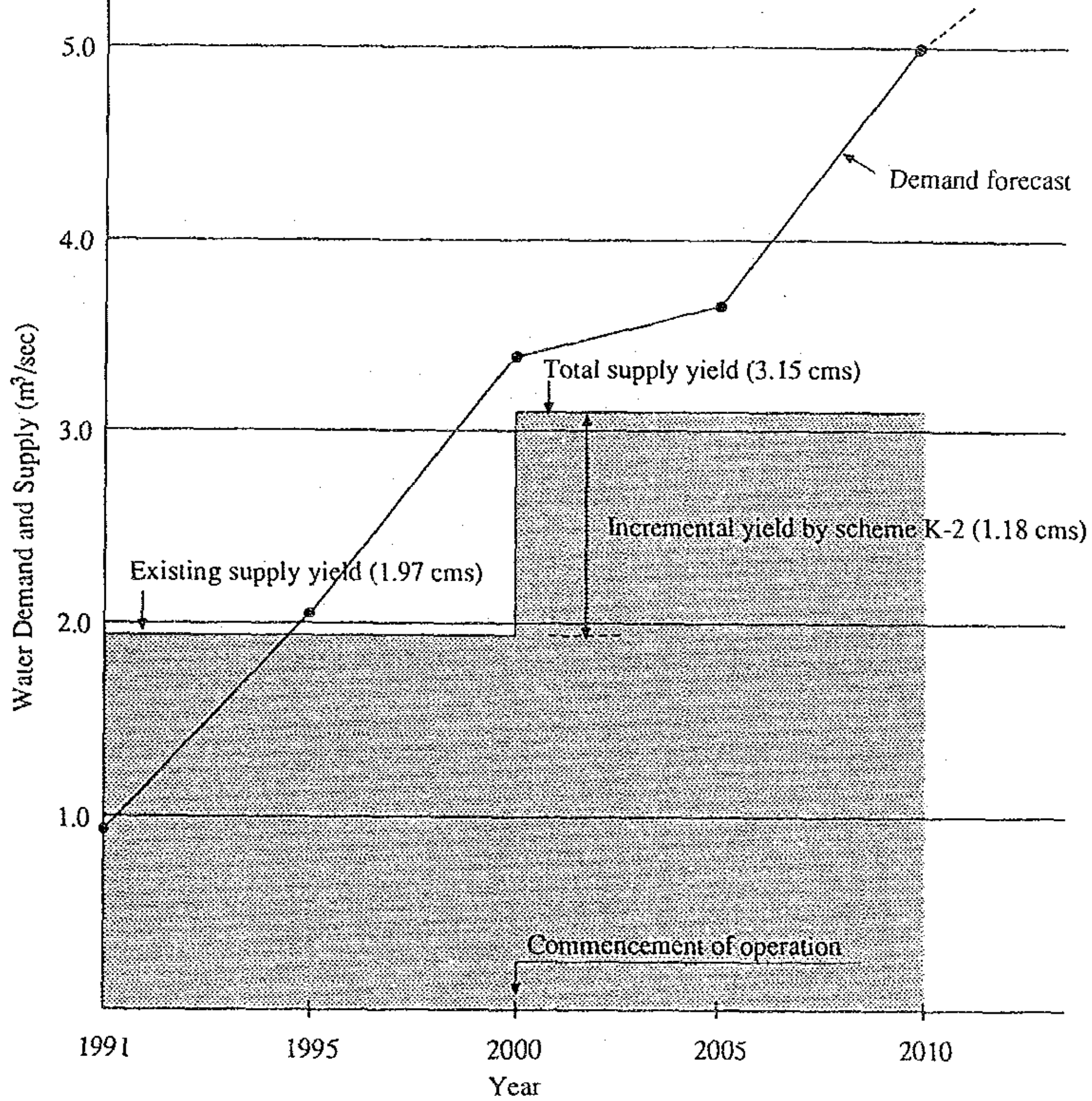


Water Demand and Supply Program by Scheme K-1

Fig. F-45

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.15	3.30	- 0.15
2005	3.15	3.67	- 0.52
2010	3.15	4.94	- 1.79



Water Demand and Supply Program by Scheme K-2

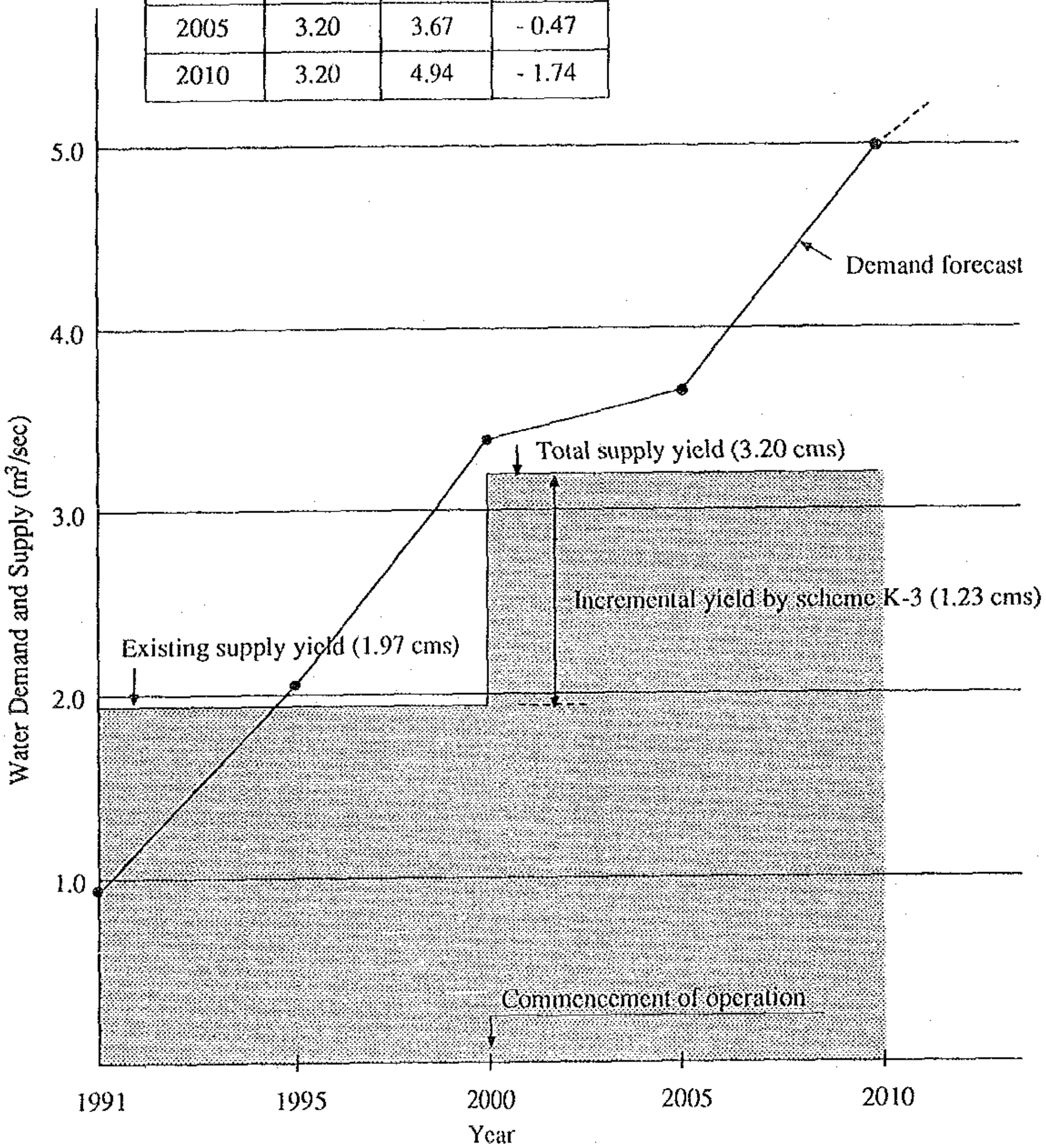
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-46

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.20	3.30	- 0.10
2005	3.20	3.67	- 0.47
2010	3.20	4.94	- 1.74



Water Demand and Supply Program by Scheme K-3

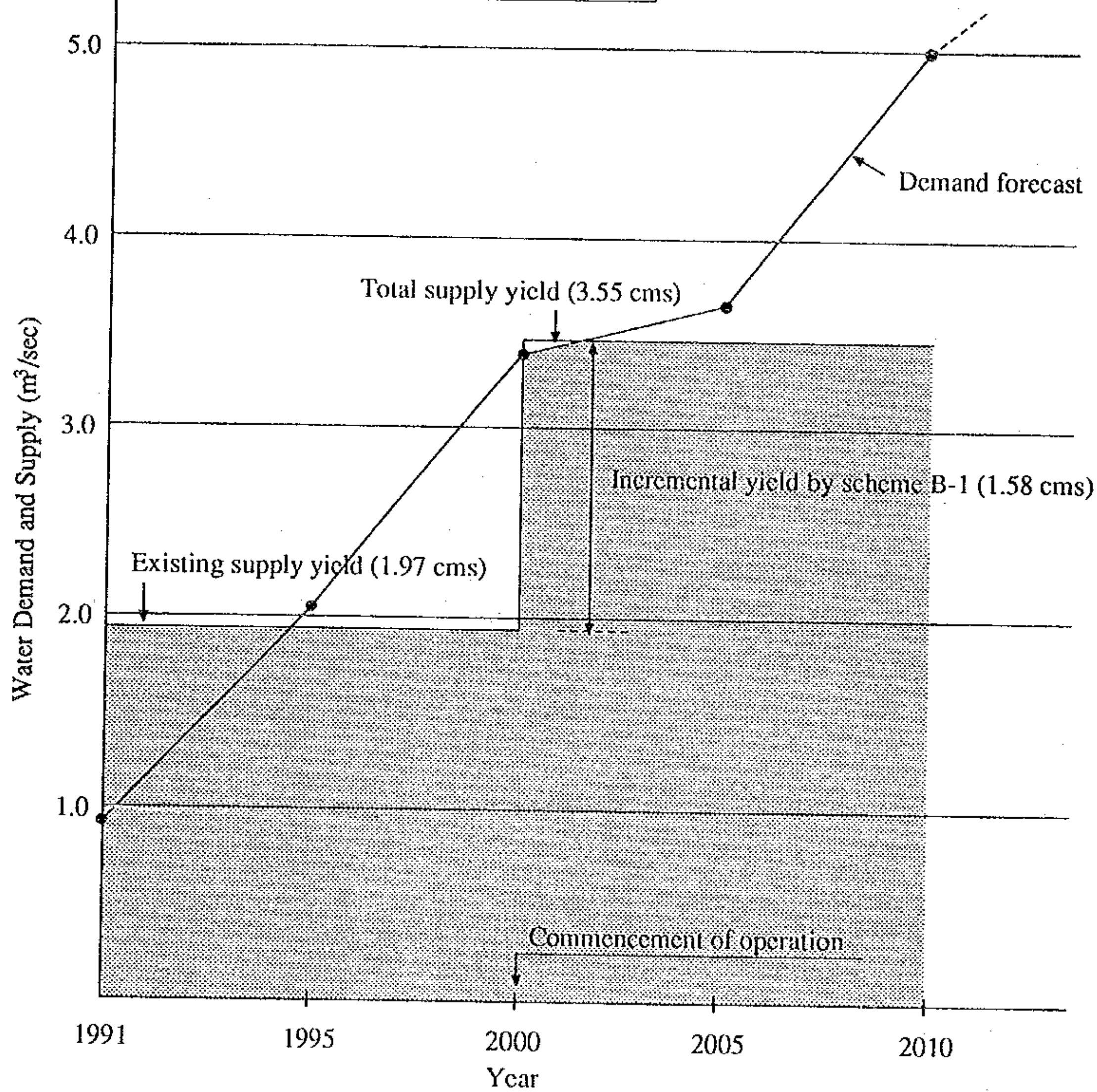
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-47

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.55	3.67	- 0.12
2010	3.55	4.94	- 1.39



Water Demand and Supply Program by Scheme B-1

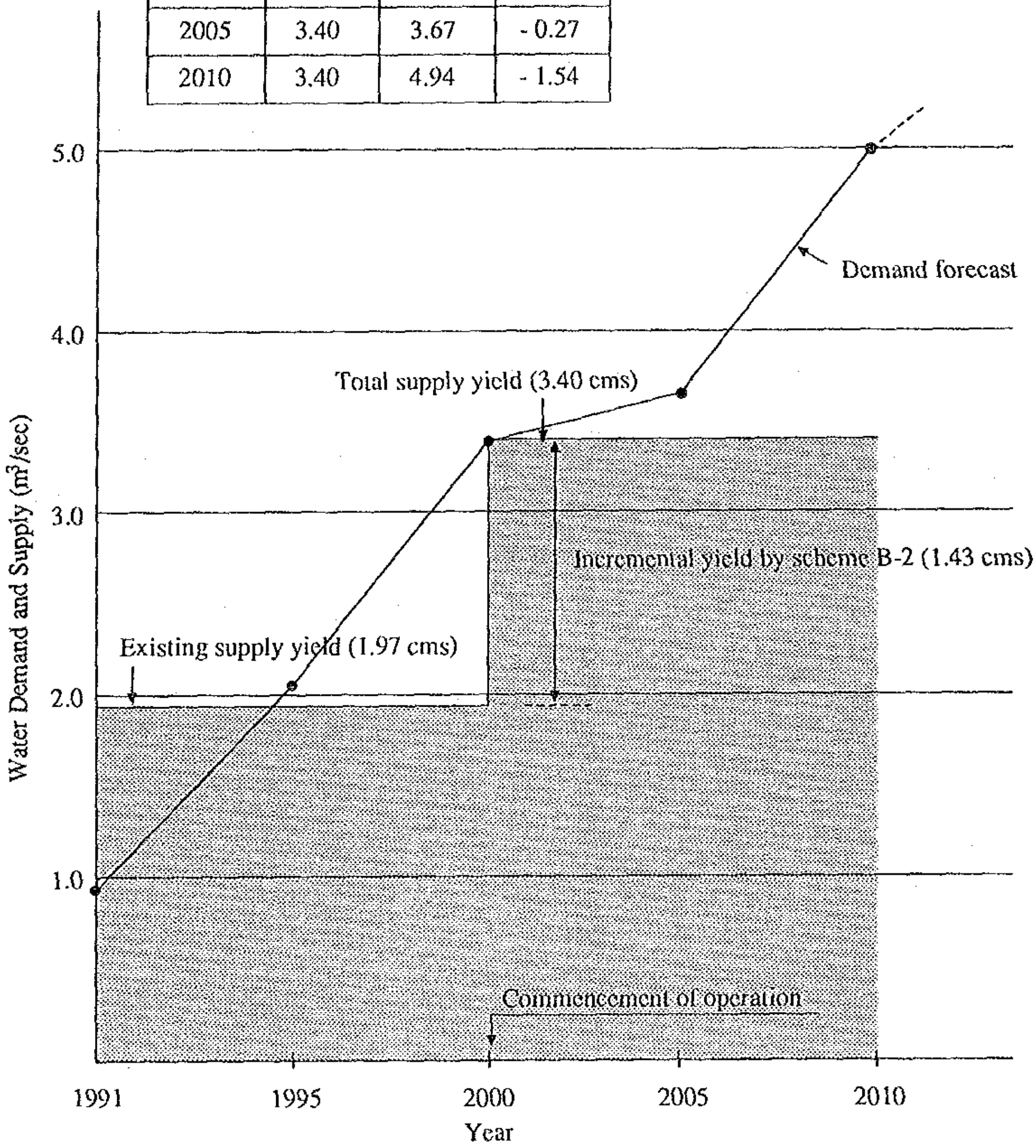
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-48

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.40	3.67	- 0.27
2010	3.40	4.94	- 1.54



Water Demand and Supply Program by Scheme B-2

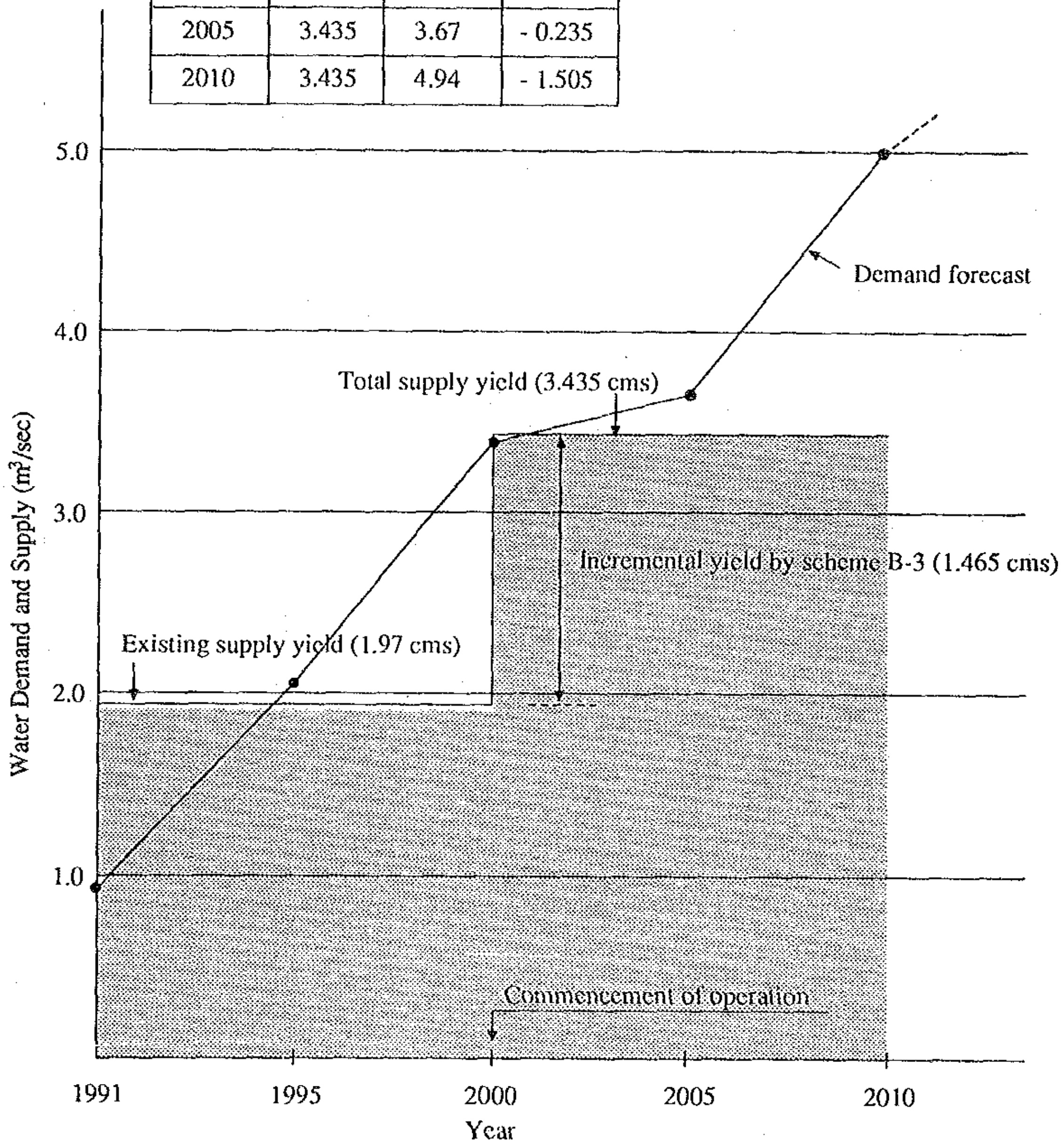
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-49

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.435	3.67	- 0.235
2010	3.435	4.94	- 1.505



Water Demand and Supply Program by Scheme B-3

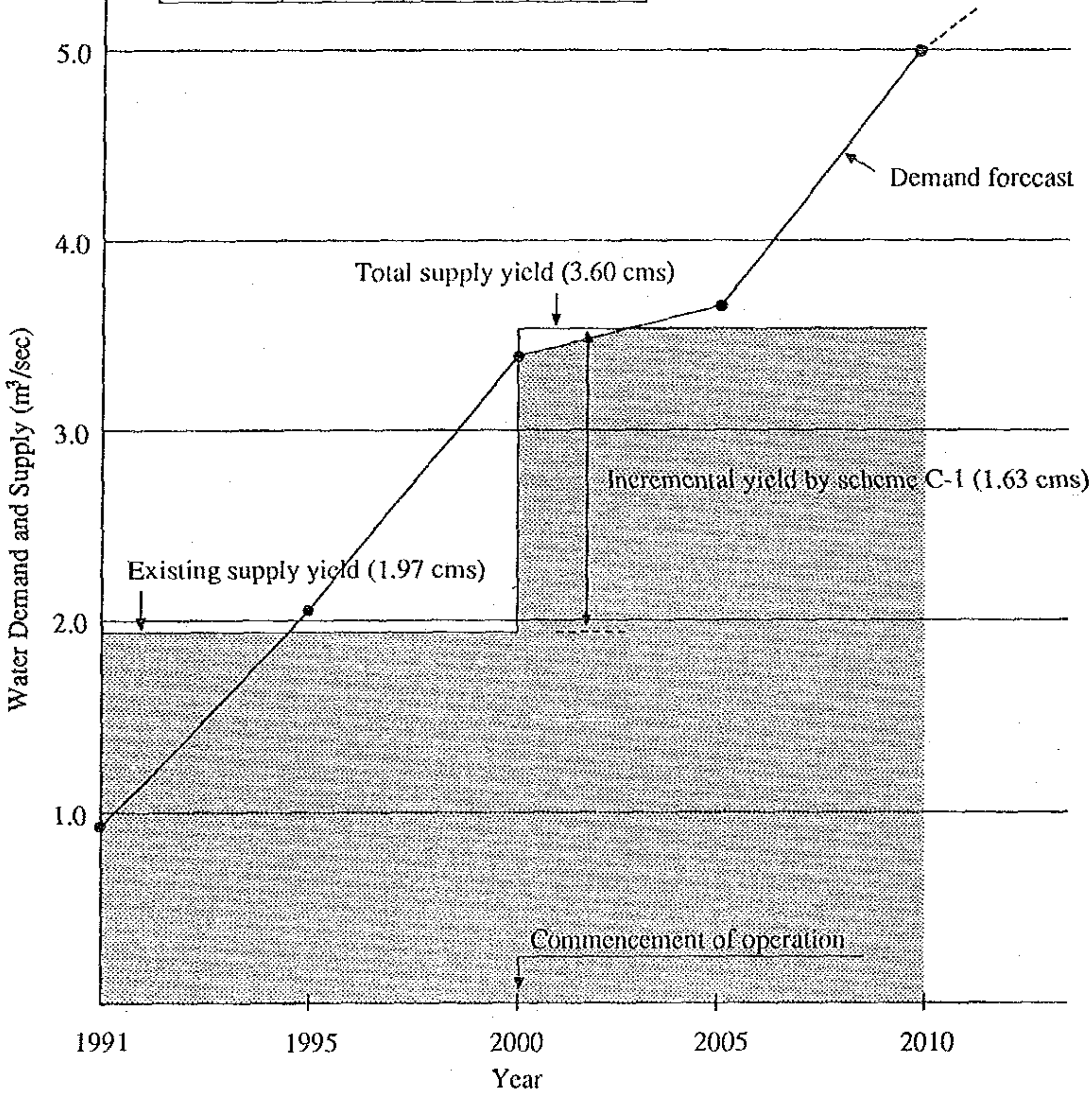
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-50

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.60	3.67	- 0.07
2010	3.60	4.94	- 1.34



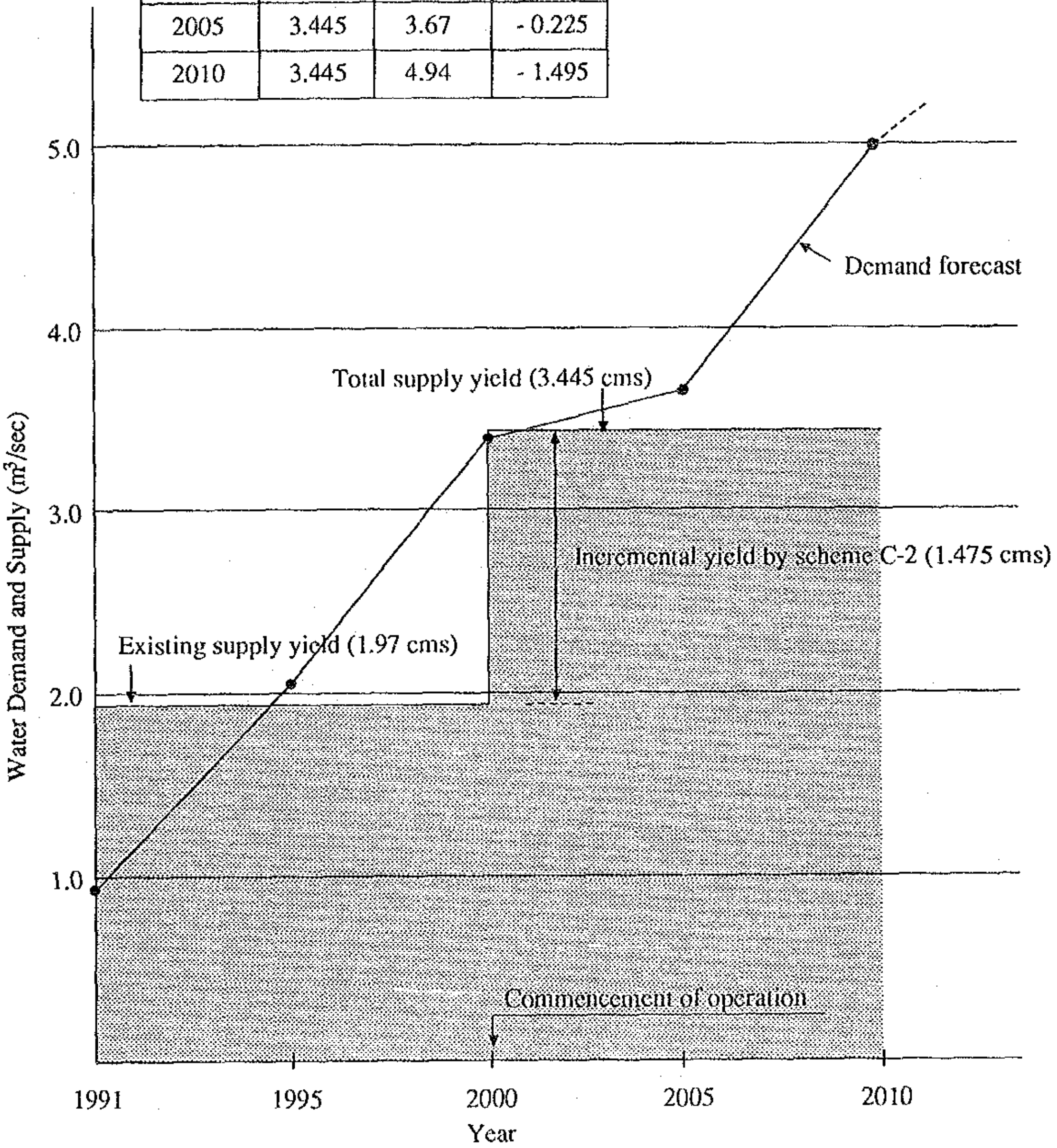
Water Demand and Supply Program by Scheme C-1

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.445	3.67	- 0.225
2010	3.445	4.94	- 1.495



Water Demand and Supply Program by Scheme C-2

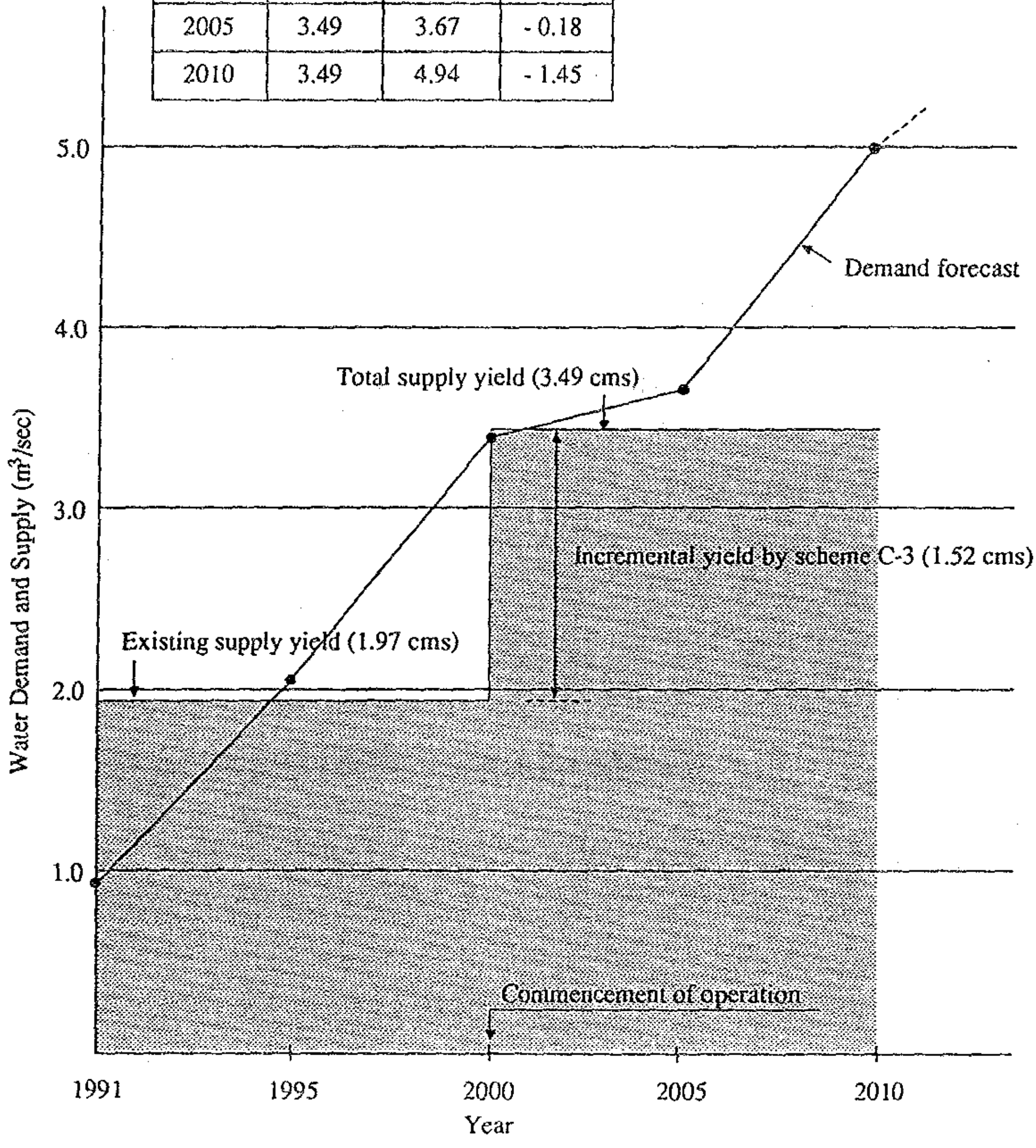
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-52

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.49	3.67	- 0.18
2010	3.49	4.94	- 1.45



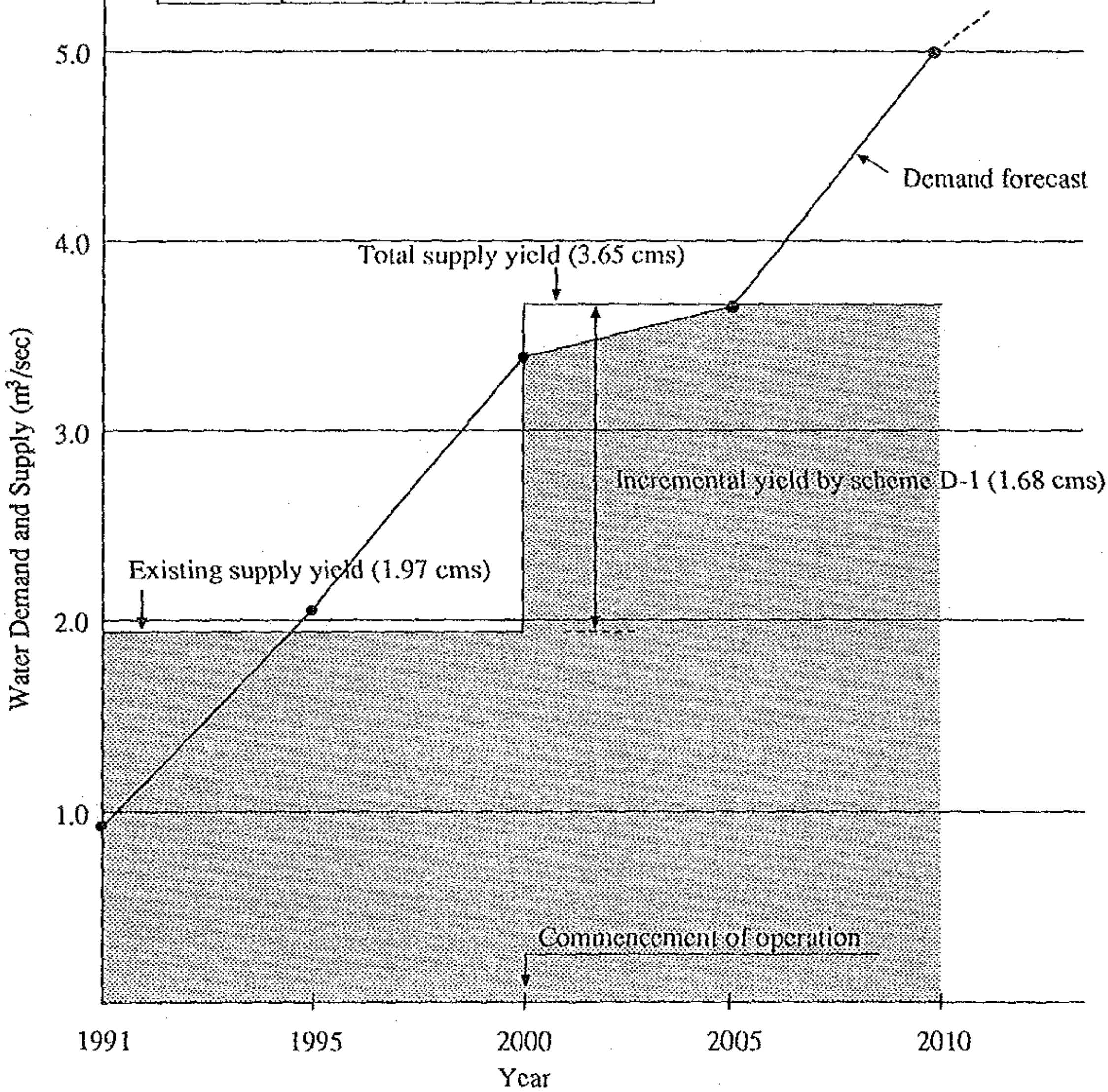
Water Demand and Supply Program by Scheme C-3

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.65	3.67	-0.02
2010	3.65	4.94	- 1.29



Water Demand and Supply Program by Scheme D-1

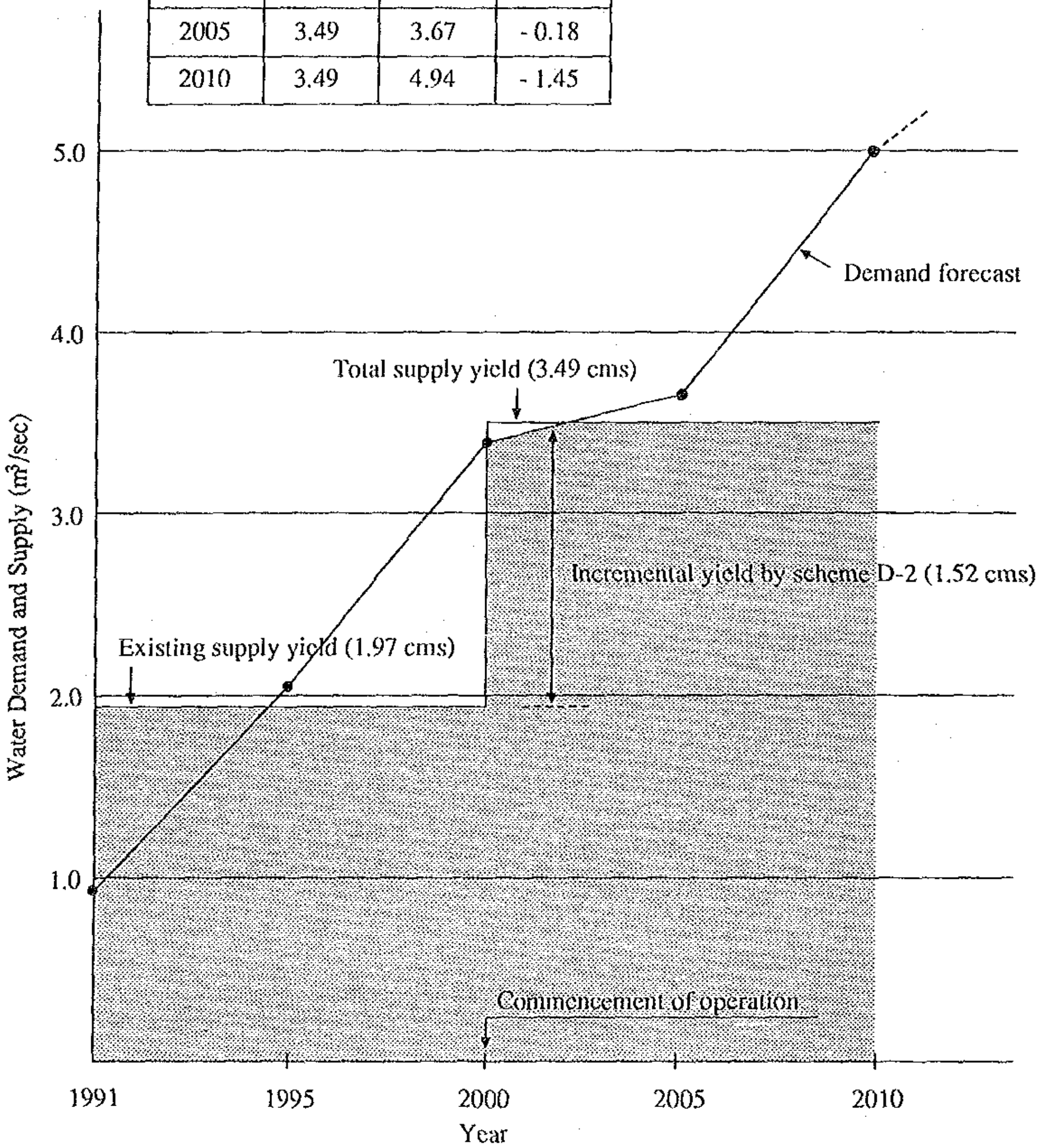
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-54

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.49	3.67	- 0.18
2010	3.49	4.94	- 1.45



Water Demand and Supply Program by Scheme D-2

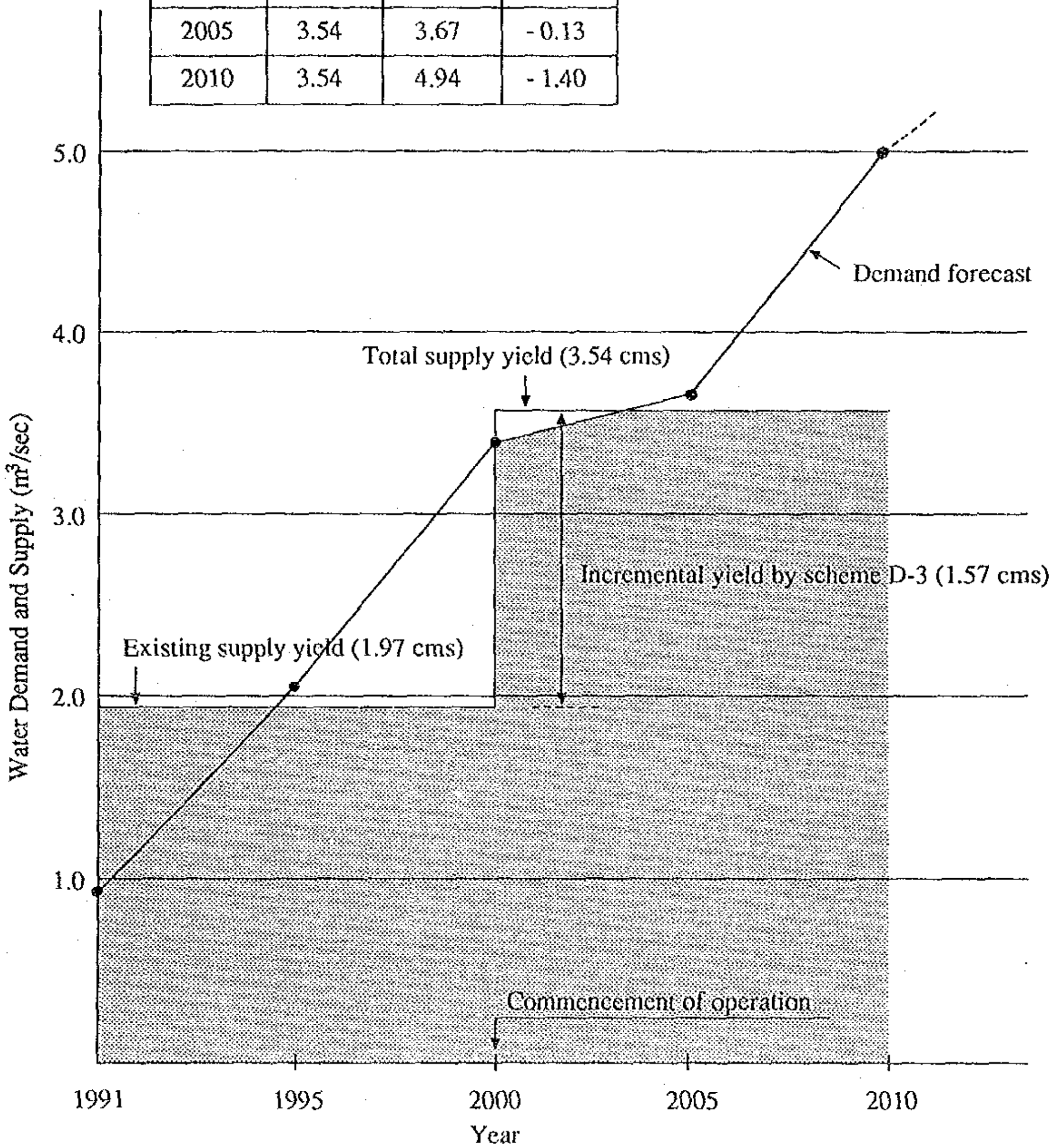
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-55

Unit: cms)

Year	Supply	Demand	Balance
1991	1.97	0.86	+1.11
1995	1.97	2.06	- 0.09
2000	3.30	3.30	0
2005	3.54	3.67	- 0.13
2010	3.54	4.94	- 1.40

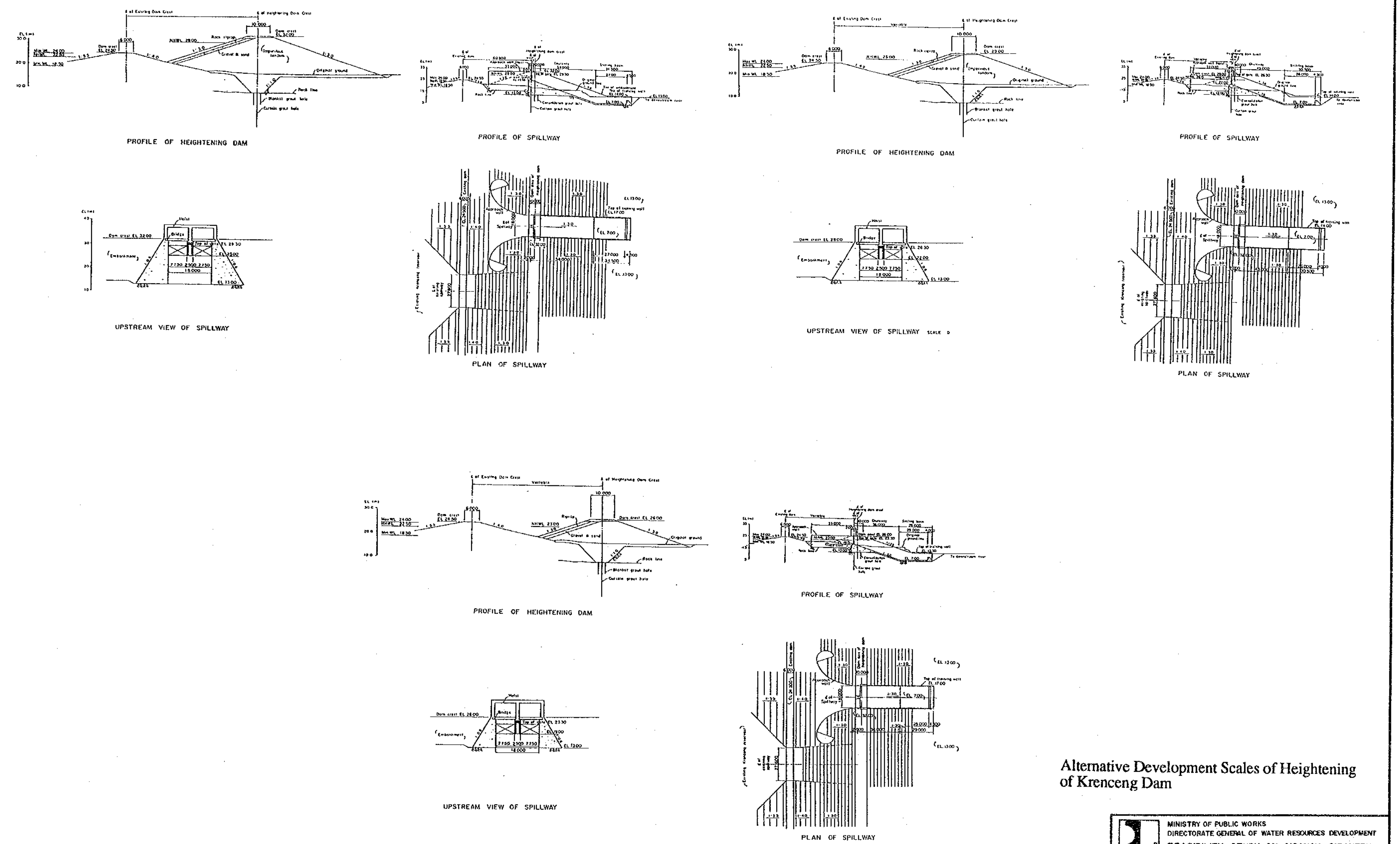


Water Demand and Supply Program by Scheme D-3

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-56

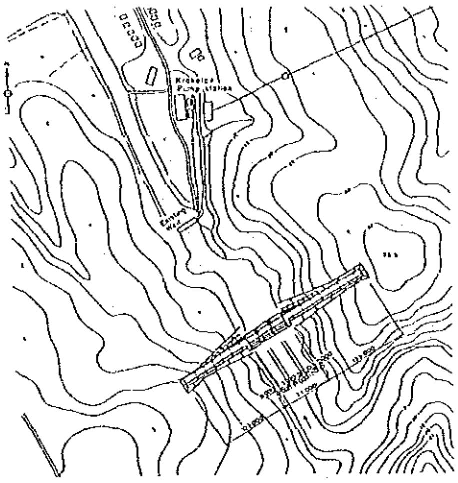


Alternative Development Scales of Heightening of Krenceng Dam

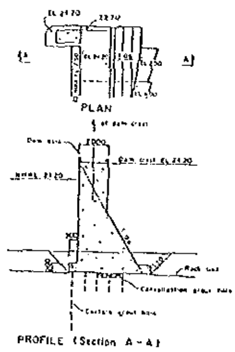
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

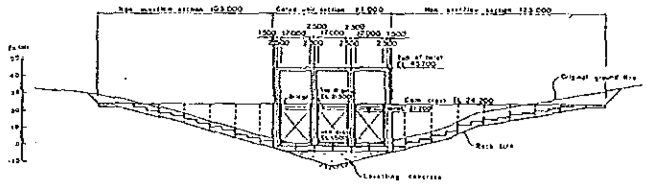
Fig. F-57



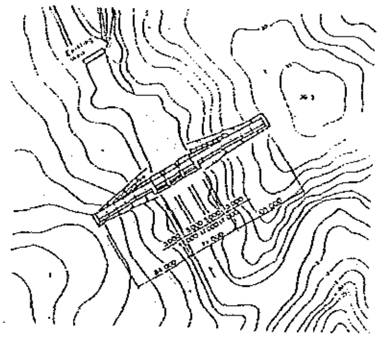
PLAN



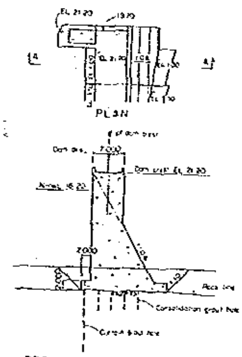
PROFILE & PLAN OF NON OVERFLOW SECTION



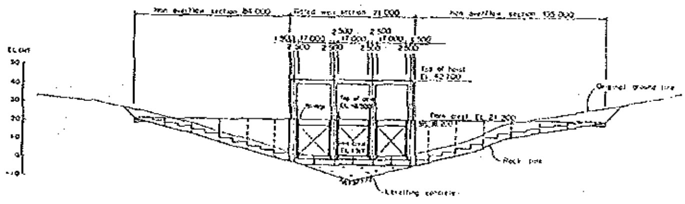
UPSTREAM ELEVATION



PLAN



PROFILE & PLAN OF NON OVERFLOW SECTION

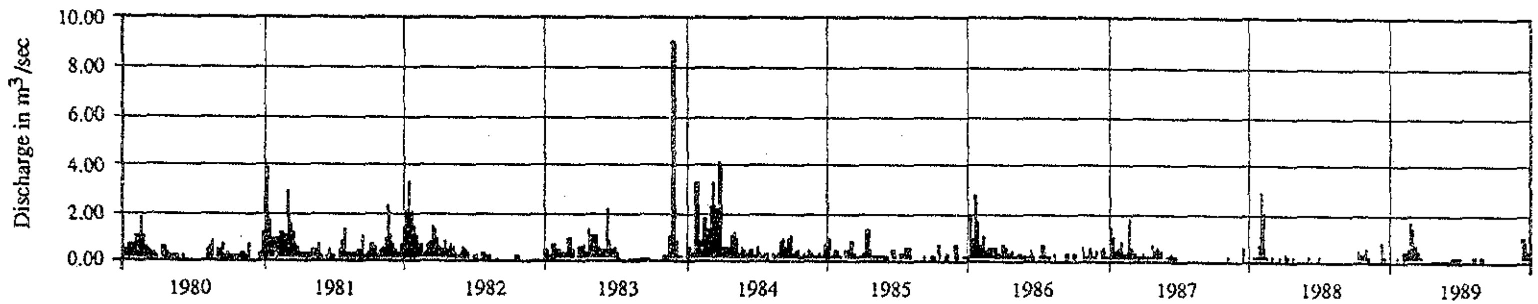


UPSTREAM ELEVATION

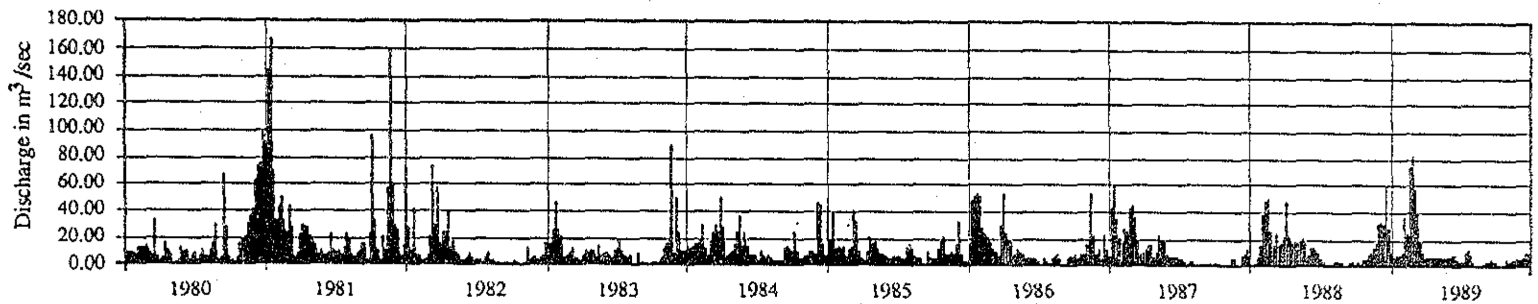
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

Alternative Dam Types for Cidanau Gated Weir

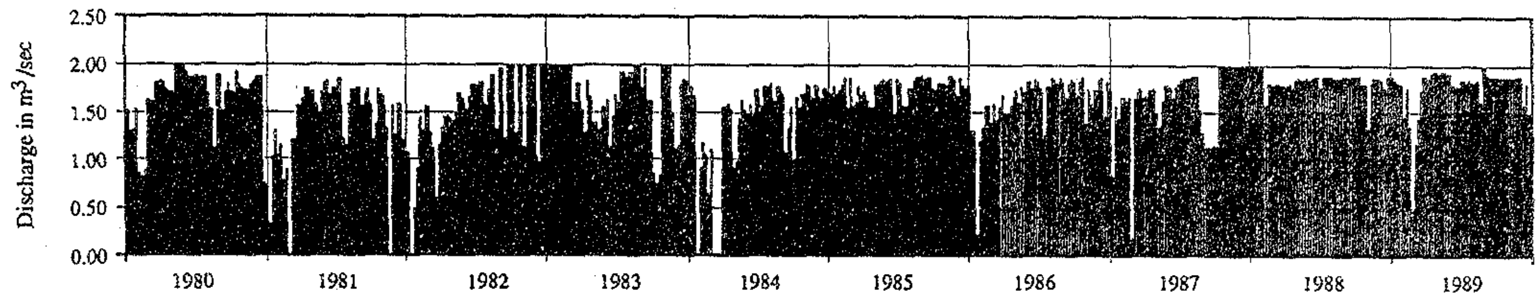
JAPAN INTERNATIONAL COOPERATION AGENCY



Five (5)-day Mean Inflow at Krenceng Dam Site



Five (5)-day Mean Inflow Discharge at Krakatau Intake

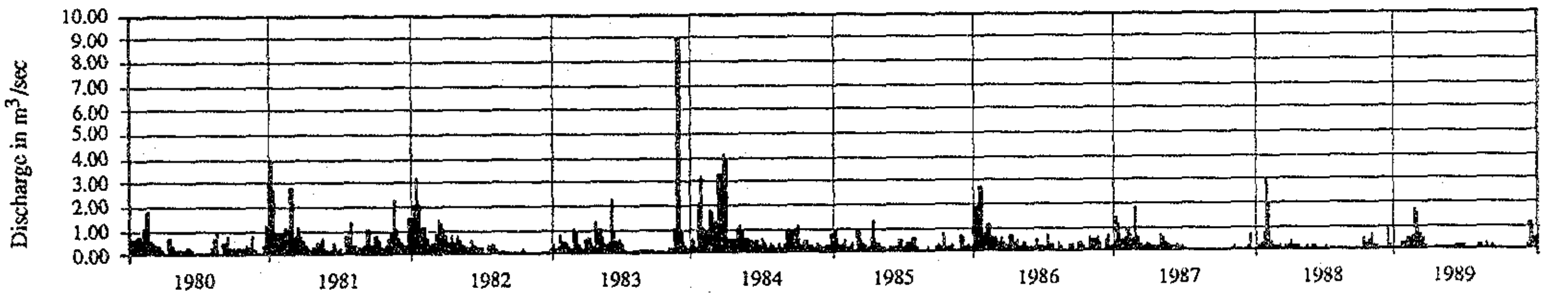


Five (5)-day Mean Water Conveyance from Krakatau Intake to Existing Krenceng Reservoir through 27.2 km long Pipeline

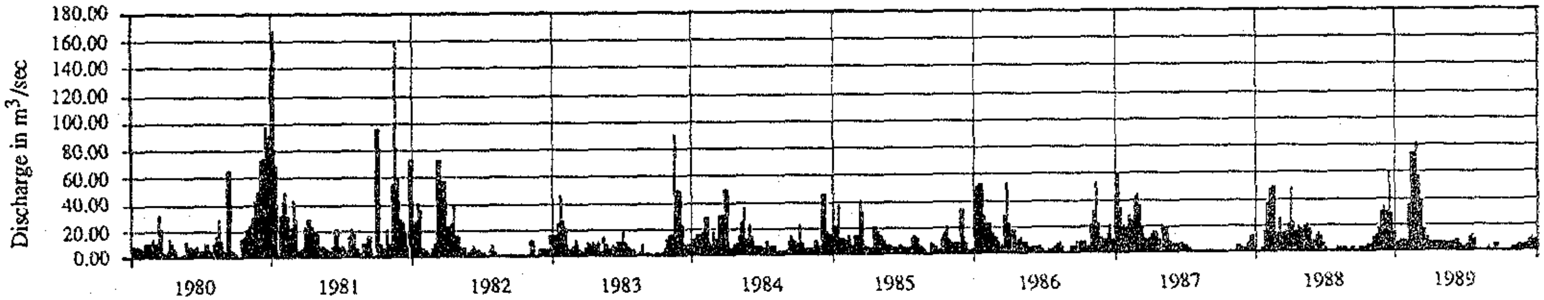
INFLOW DISCHARGE FOR EXISTING KRENCENG DAM

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU - CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

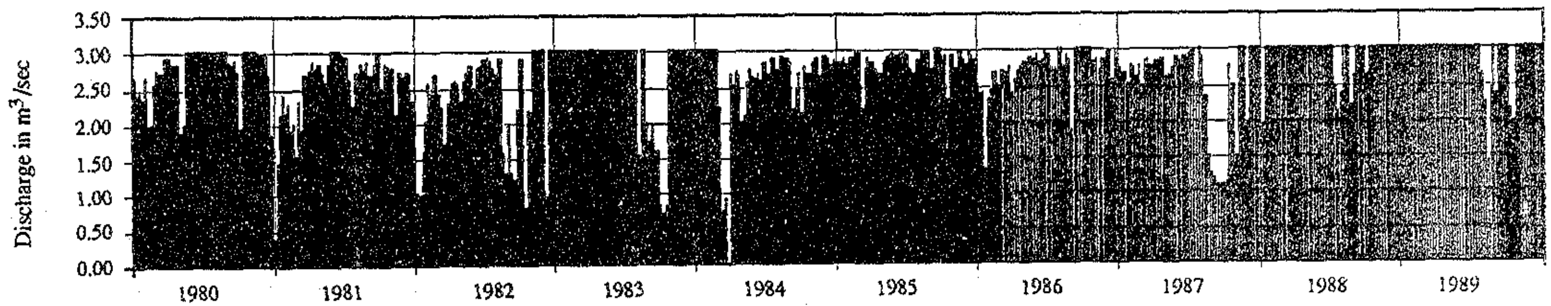
JAPAN INTERNATIONAL COOPERATION AGENCY



Five (5)-day Mean Inflow Discharge at Krenceng Dam Site



Five (5)-day Mean Inflow Discharge at Krakatau Intake



Five (5)-day Mean Water Conveyance from Krakatau Intake to Heightening of Krenceng Reservoir through 27.2 km long Pipeline

INFLOW DISCHARGE FOR HEIGHTENING OF KRENCENG DAM
(without Beroeng Diversion Tunnel)

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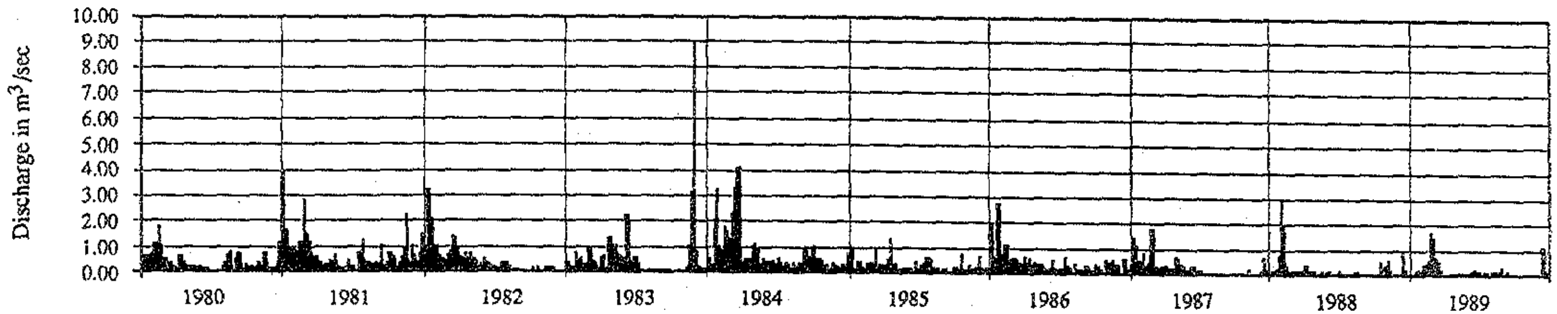
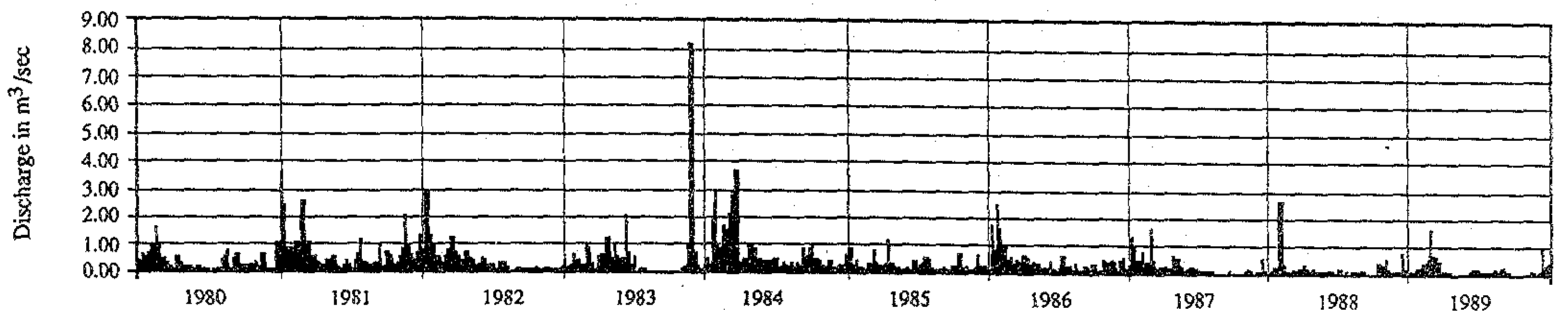
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

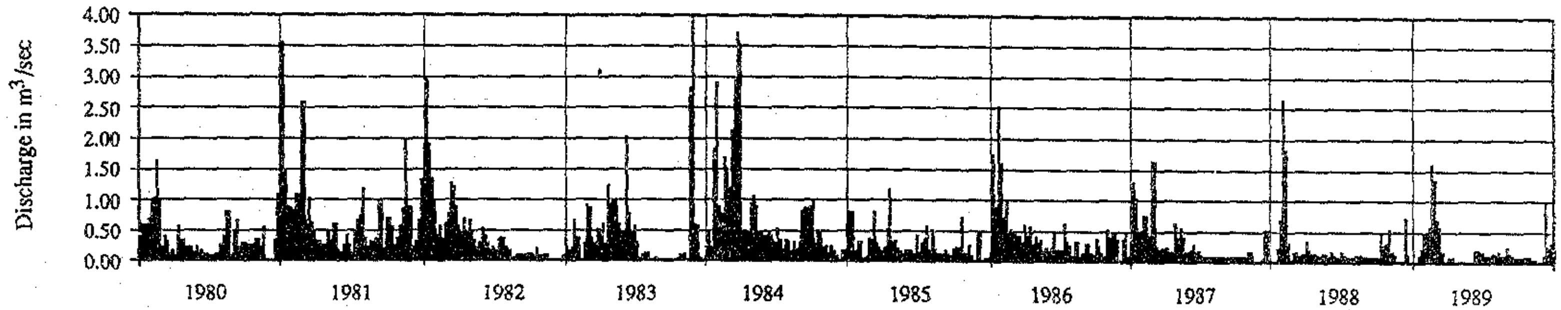
Fig. F-60 (1)



Five (5)-day Mean Inflow Discharge at Krenceng Dam Site



Five (5)-day Mean Inflow Discharge at Beroeng Diversion Weir Site



Five (5)-day Mean Water Conveyance from Beroeng River to Krenceng River through Beroeng Diversion Tunnel

INFLOW DISCHARGE FOR HEIGHTENING OF KRENCENG DAM WITH BEROENG DIVERSION TUNNEL

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DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
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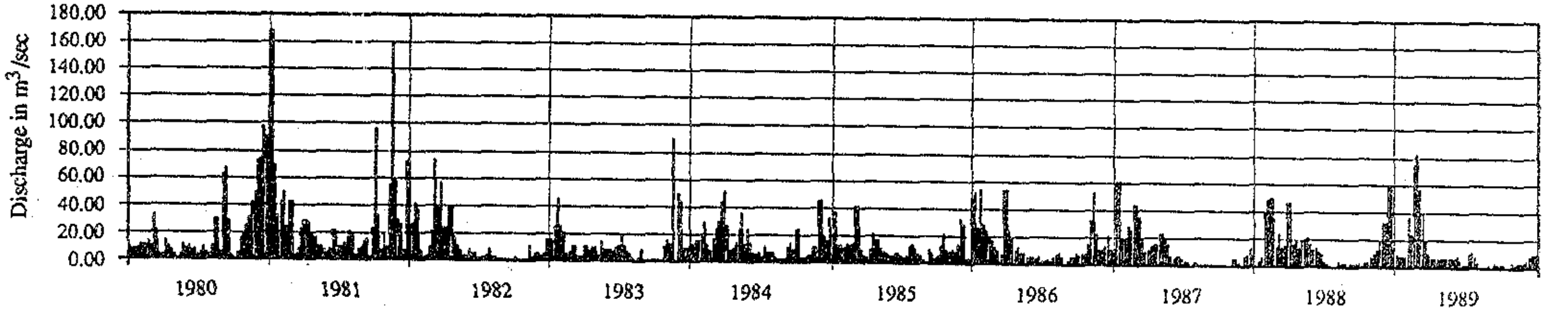
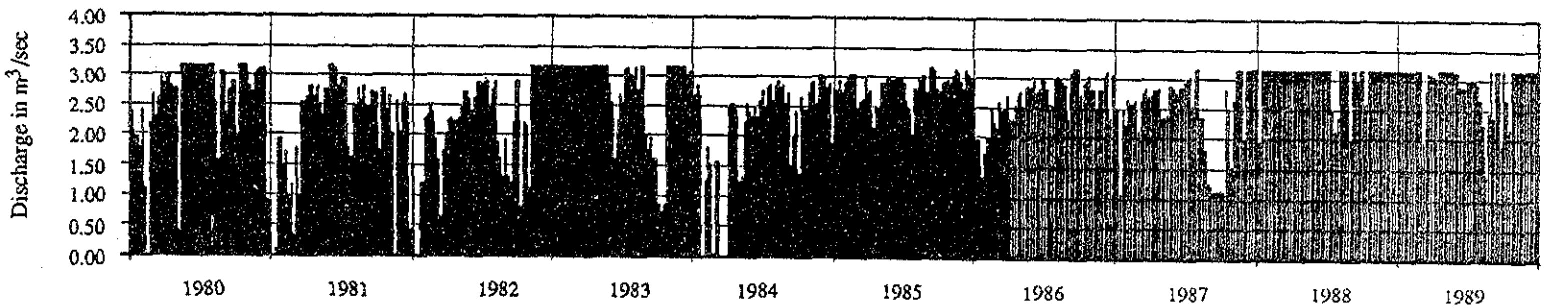


Five (5)-day Mean Inflow Discharge at Krakatau Intake



Five (5)-day Mean Water Conveyance from Krakatau Intake to Heightening of Krenceng Reservoir through 27.2 km long Pipeline

INFLOW DISCHARGE FOR HEIGHTENING OF KRENCENG DAM WITH BEROENG DIVERSION TUNNEL

MINISTRY OF PUBLIC WORKS
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FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

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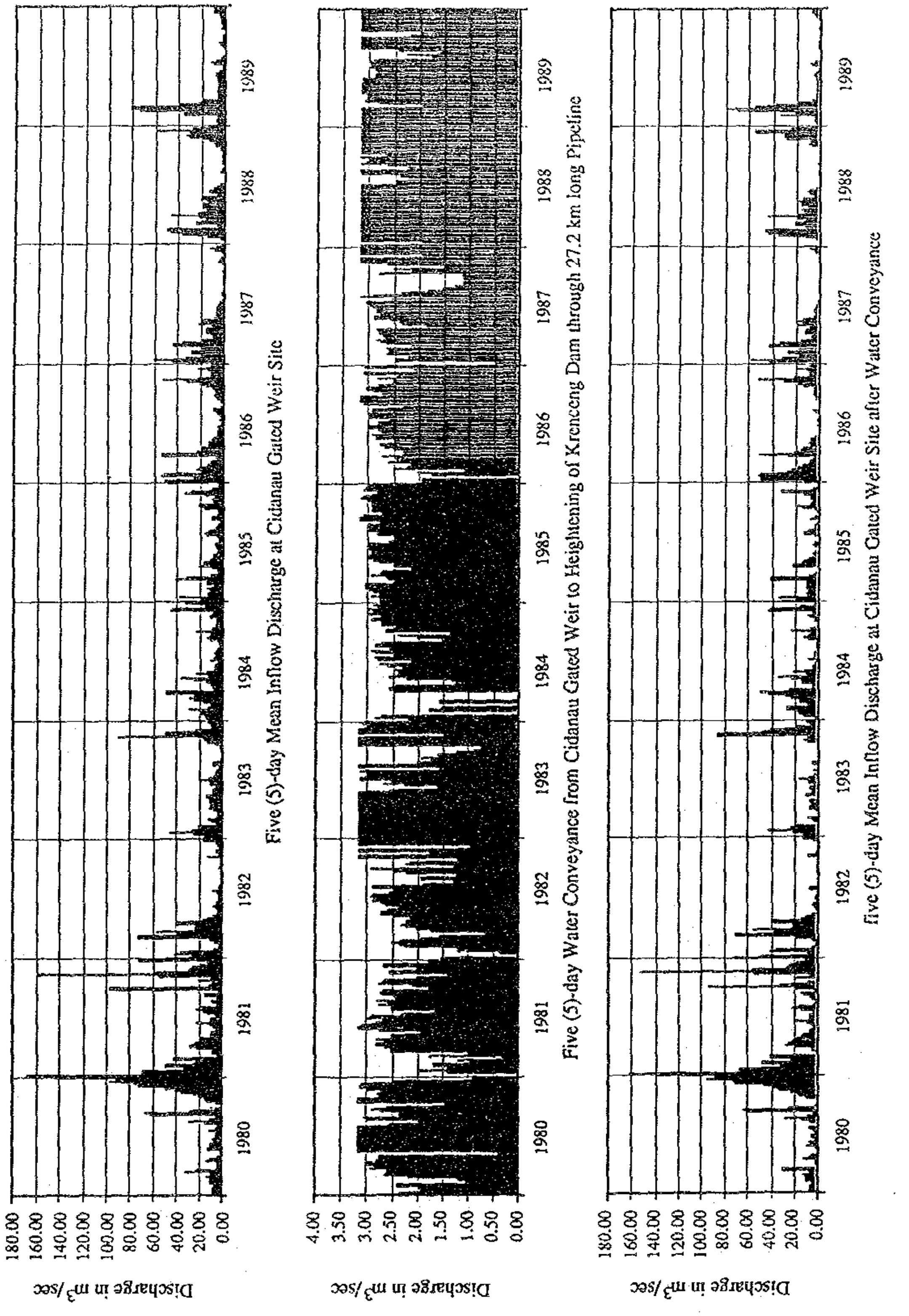
Fig. F-61

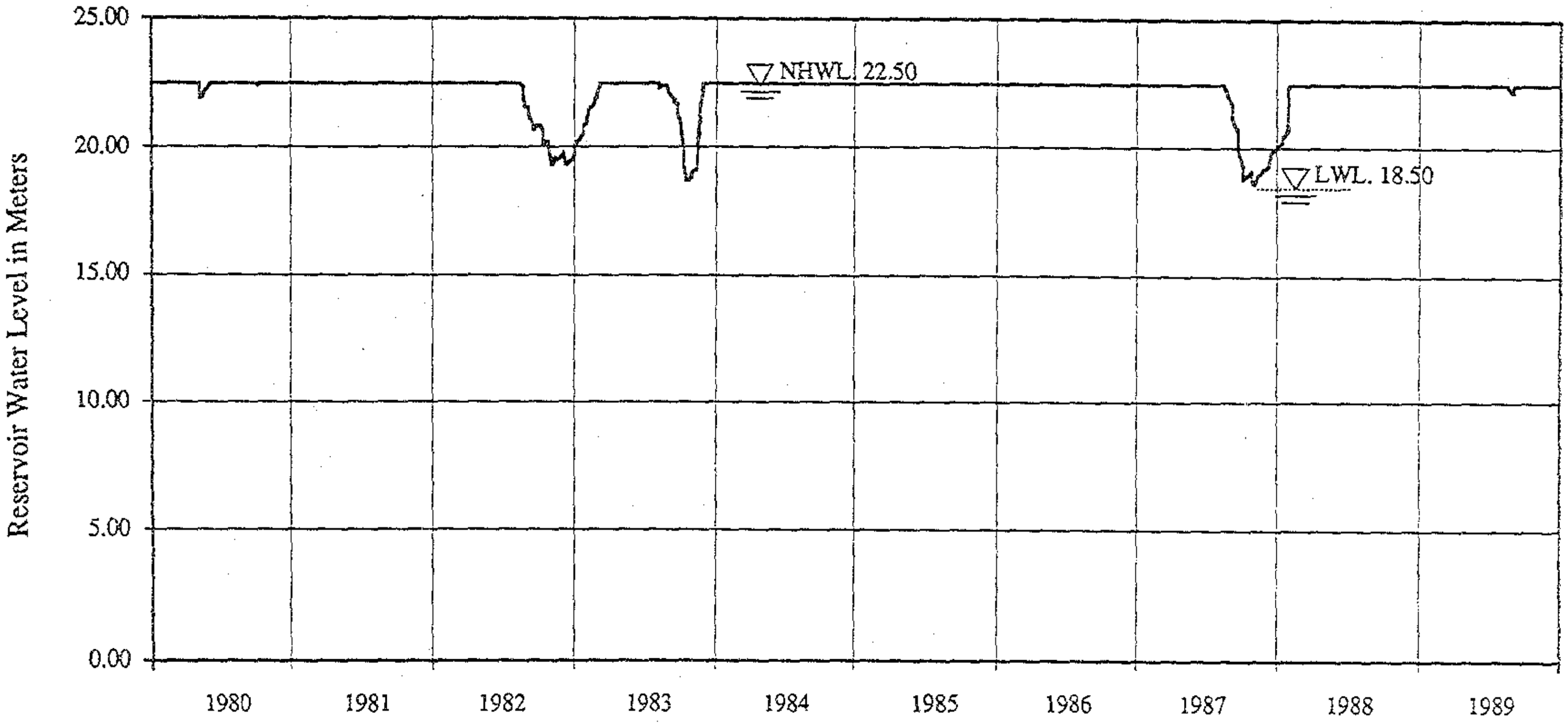


INFLOW DISCHARGE FOR HEIGHTENING OF KRENCENG DAM WITH BEROENG DIVERSION TUNNEL AND CIDANAU GATED WEIR

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-62



Reservoir Operation for Existing Krenceng Dam
(Safe yield: 1.94 m3/sec after evaporation from reservoir)

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
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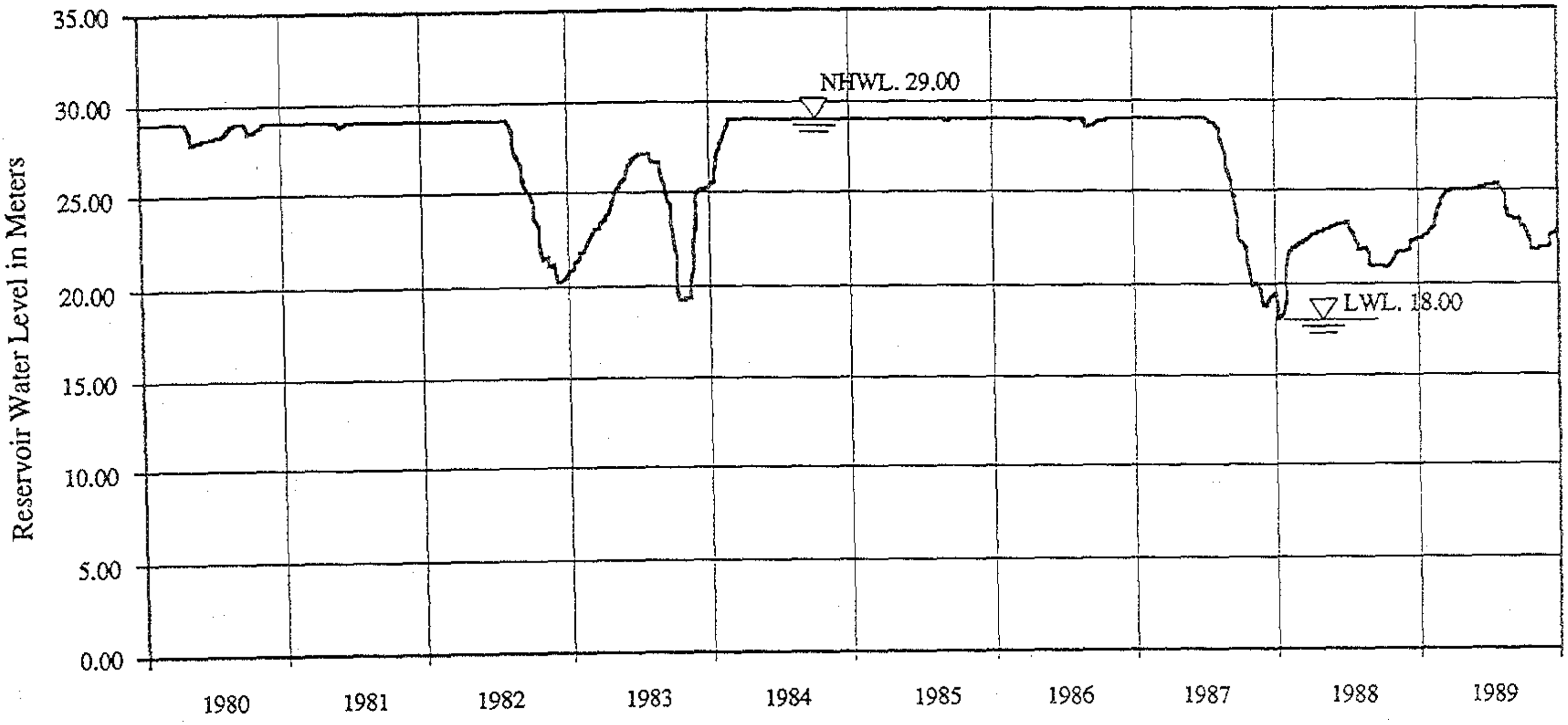
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-63

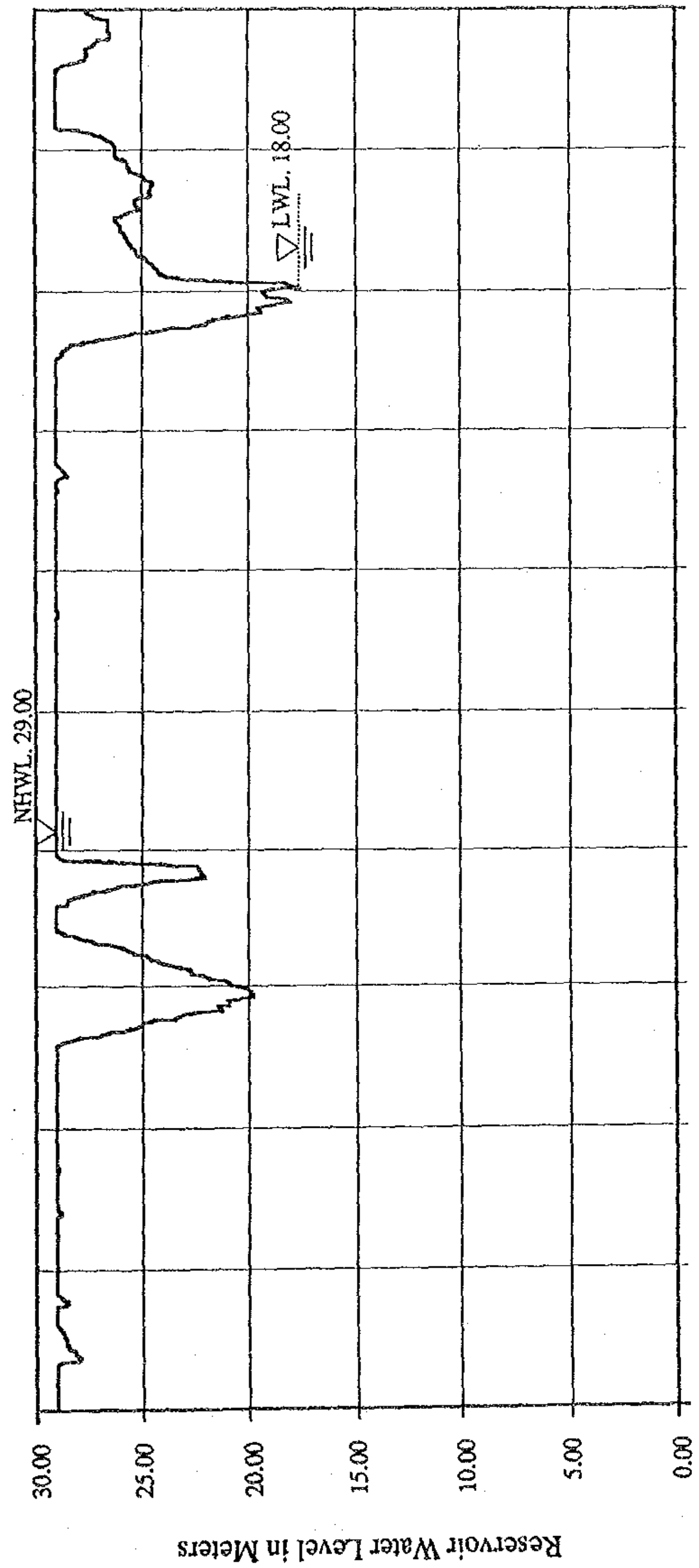


Reservoir Operation for Heightening of Krenceng Dam (without Beroeng diversion Tunnel)
(Safe Yield: 3.05 m^3 /sec after evaporation from reservoir)

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU - CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-64

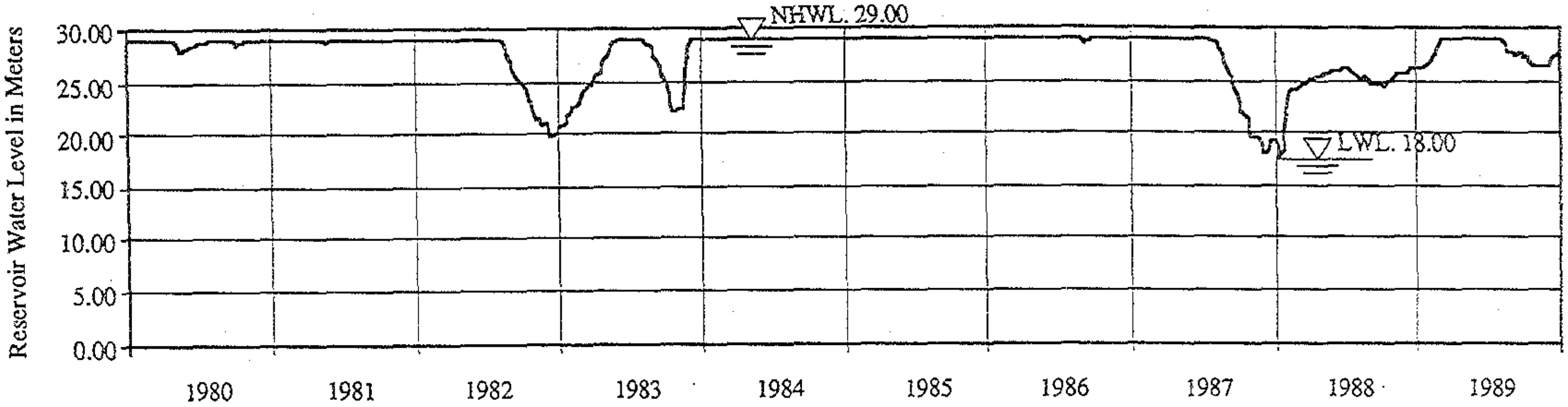


Reservoir Operation for Heightening of Krenceng Dam with Beroeng Diversion Tunnel
(Safe Yield: 3.11 m^3/sec after evaporation from reservoir)

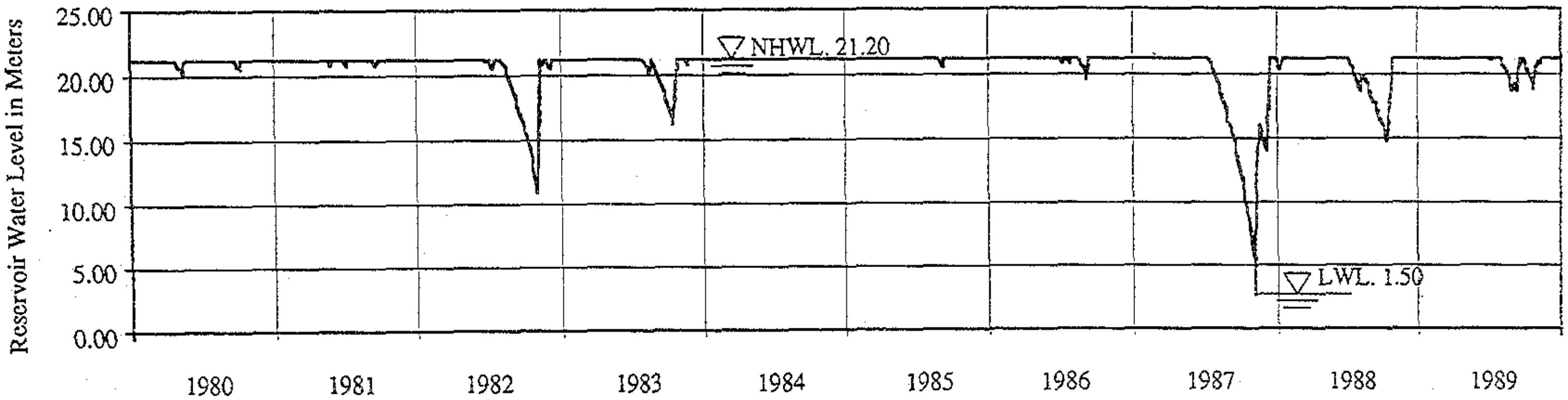
MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN
WATER RESOURCES DEVELOPMENT PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. F-65



Reservoir Operation for Heightening of Krenceng Dam with Beroeng Diversion Tunnel
(Safe yield: 3.11 m^3/sec after evaporation from reservoir)



Reservoir Operation for Cidanau Gated Weir
(Safe yield: 3.325m^3/sec after evaporation from reservoir)

Reservoir Operation for Heightening of Krenceng Dam with Beroeng Diversion Tunnel and Cidanau Gated Weir

MINISTRY OF PUBLIC WORKS
DIRECTORATE GENERAL OF WATER RESOURCES DEVELOPMENT
FEASIBILITY STUDY ON CIDANAU-CIBANTEN WATER RESOURCES DEVELOPMENT PROJECT

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APPENDIX - G
PRELIMINARY DESIGN

APPENDIX - G
PRELIMINARY DESIGN

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1. General

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i) Scheme K-1: Heightening of Krenceng dam (without Beroeng diversion tunnel)

ii) Scheme K-2: Heightening of Krenceng dam (with Beroeng diversion tunnel)

iii) Scheme C-3: Heightening of Krenceng dam (with Beroeng diversion tunnel) and Cidanau gated weir

2 Heightening of Krenceng Dam

2.1 Design Flood

1) Discharge capacity of spillway

Design flood discharge of spillway is adopted to be 225 m^3/sec which is 200-year probable flood plus 20% of the peak discharge.

This flood is designed to be discharged through spillway without flood regulation in the reservoir.

2) The design discharge of the energy dissipator is adopted to be 171 m^3/sec which is the peak discharge of 100-year probable flood.

3) The design discharge of the river diversion is adopted to be which is the peak discharge of 25-year probable flood. However, the river diversion during the construction is not required because the embankment work for the downstream of the existing spillway will be carried out during the final dry season after the completion of the construction work of relocated spillway for the heightening of Krenceng dam.

4) The design flood water level is determined by routing the PMF assuming an initial reservoir water level at NHWL.29.00 m.

2.2 Optimization of Gated Spillway

The flood attacked the heightening of Krenceng reservoir is controlled by the gated spillway to keep NHWL.29.00 m as far as possible and save the dam construction cost for the heightening due to excess freeboard above the rising flood water surface.

1) Alternative gated spillways

The optimization of gated spillway was studied for 3 alternatives by fixing the normal high water level,NHWL.29.00 m and satisfying the design discharge capacity of spillway,1.2 times of 200-year flood peak (Fig. G-1).

Case No.	B (m)	H (m)	Weir Crest (El-m)	Nos.	Net width (m)
Case 1	7.75	4.30	25.00	2	15.5
Case 2	6.00	3.30	26.00	4	24.0
Case 3	5.00	2.80	26.50	6	30.0

Notes: B denotes gate width, H gate height.

2) Flood operation rule

The flood routing study of PMF was made assuming the initial reservoir water level at NHWL.29.00. The flood inflow is released in accordance with the inflow to be equal to outflow rule in principle, or the gates will be operated to keep NHWL as far as possible.

3) Flood routing study

The maximum flood water level was obtained for each alternative gated spillways as shown below:

Description		Unit	Alternative No. of gates		
			Case 1	Case 2	Case 3
i)	Max.water level	El-m	30.15	30.08	29.96
ii)	Max.rise above NHWL	m	1.15	1.08	1.96
iii)	Max.outflow	m^3/sec	345	360	370

4) Economic comparison

The economic comparison for the alternatives was made by the hydraulic jump type spillway.

	Description	Unit	Alternative		
			Case 1	Case 2	Case 3
1)	NHWL	EL-m	29.00	29.00	29.00
2)	Weir crest	El-m	25.00	26.00	26.50
3)	Net width at crest	m	15.50	24.00	30.00
4)	Total width	m	18.00	30.00	37.50
5)	Slope of chuteway		1: 3.0	1:3.0	1:3.0
6)	Bottom El of basin	El-m	7.00	7.00	7.00
7)	Length Chuteway	m	56.0	59.0	60.5
8)	Basin	m	31.5	25.0	23.0
9)	Excavation	m^3	49769	60798	69360
10)	Construction cost [1]				
	Civil work	10^6Rp	6552	8236	9179
	Metal work	10^6Rp	1404	1797	2190
	Total	10^6Rp	7956	10033	11369

Note: The construction cost for the heightening of Krenceng dam is excluded because the difference of flood water level is negligible small considering the results of flood routing study and thus the dam height is same at any alternative.

As shown in the table above, Case 1 yields the lowest cost among the 3 alternatives.

2.3 Dam Crest Elevation

1) Dam design freeboard

The freeboard which provides the highest crest elevation of non-overflow section of a main dam is adopted from the following alternative combination of freeboard and the maximum design water surface.

Maximum design water surface	Freeboard requirement
Normal high water level	Hf(1) = hw +he+ha+hi or 3 m for fill type and 2 m for concrete type
Design flood water level	Hf(2) = hw+ha+hi or 2 m for fill type and 1 m for concrete type

where, hw : Wave height due to wind

he : Wave height due to earthquake

ha : Rise of water level due to unexpected accident in operating spillway gates (0.5 m for a gated type and 0 for a non-gated type) hi:Addition of allowance for safety according to type and importance of dams (1 m for fill dams and 0 m for concrete dams)

2) Dam crest elevation

The crest elevation of the non-overflow section of dam which corresponds to the crest elevation of the impervious core of a fill dam, is the sum of the maximum design water level and the freeboard.

From the results of flood routing study and the above standard, the crest elevation for the heightening of Krenceng dam is set at El.32.00 m considering the flood routing against the PMF (Fig. G-2).

Normal condition : NHWL.29.00 + hw + he + ha + hi= El.31.30 m
NHWL.29.00 + 2.0 +hi = El.32.00 m (hw +he <1.5)

Flood condition : Max.Wl.30.15 + hw + ha + hi = El.32.15 m

Where, hw = $0.00086 \times V^{1.1} \times F^{0.45}$

v = 20 m/s wind speed on the 10 minutes average

F = 1000 m distance between dam and opposite bank

he = $kT (gH)^{1/2}/2/3.14$

k = 0.15 seismic coefficient

T = 1.0

2.4 Type of Heightening of Krenceng Dam

The foundation rock of the heightening of Krenceng dam is composed of welded pumice tuff. Geologically,this foundation is not so hard but the ordinary grout curtain could be technically conducted. The dam height is limited in 20-25 m. Accordingly,filltype dam is conceived suitable to the site.

The cost comparison was made for the following 2 alternatives:

Type A : Zoned rockfill dam with the centre core
(U/S=1.30, D/S=1:2.0)

Type B : Impervious random earthfill
(U/S=1:3.0, D/S=1:3.0)

Typical sections for the above two types of dams are shown in Fig G-3.

Item	Unit	Type A	Type B
Dam			
1) Excavation	m^3	230506	255991
2) Embankment			
Core	m^3	285224	1356001
Filter	m^3	95200	-
Random	m^3	273989	-
Rock	m^3	465751	-
3) Grouting			
Blanket	t	4109	4109
Curtain	t	3881	3881
Spillway			
1) Excavation	m^3	48424	49769
2) Concrete	m^3	17629	17866
Spillway gates			
No.		2	2
Dimension (BxH)		7.75 x 4.30	7.75 x 4.30
Direct const. cost			
1) Civil work			
Dam	10^6Rp	26251.6	19124.0
Spillway	10^6Rp	6893.6	6990.3
2) Metal work	10^6Rp	1404	1404
Total	10^6Rp	33727	26605.8

As shown in the table above, Type B, the impervious random earthfill type is economically adopted compared with the zoned rockfill type.

2.5 Slope of Heightened Dam

1) Design criteria

The design criteria adopted for justified the slope stability of the dam is to have the safety factor of more than 1.2 in any case including seismic condition in accordance with Japanese dam design practice. The slip-circle method is used for the stability analysis.

The stability of heightening of Krenceng dam including the existing dam and bottom foundation which consists of weakly welded pumice tuff, D-CL class is checked against the sliding.

2) Condition of stability analysis

Case	Condition	Slope to be examined	Seismic coefficient	Pore pressure condition	Minimum safety factor	
					Normal	Seismic
(1)	Reservoir full	U/S, D/S	Kh = 0.15 Kv = 0	Steady seepage	1.5	1.2
(2)	Reservoir empty	U/S, D/S	Kh = 0.75 Kv = 0	50% weight remain	1.5	1.2

3) Design value of dam embankment materials and foundation

	Item	Unit	Impervious random	Filter	Rock	Existing Krenceng dam	Foundation (welded tuff)
1)	Specific gravity	t/m^3	2.492	2.768	2.30	-	-
2)	Water content	%	23.9	7.293	6.0	-	-
3)	Void ratio		0.72	0.73	0.40	-	-
4)	Dry density	t/m^3	1.468	1.60	1.643	-	-
5)	Wet density	t/m^3	1.728	1.717	1.742	1.70	
6)	Saturated density	t/m^3	1.857	2.021	1.928	1.80	
7)	Friction angle	degree	26	33	40	20	32
8)	Cohesion	t/m^2	3	0	0	2	5
9)	Permeability	cm/sec	1×10^{-6}	1×10^{-3}	1×10^{0}-10^{-1}	1×10^{-5}	1×10^{-4}

4) Analyzed section

The stability analysis was carried out for two (2) sections;

(i) Maximum height section : 20 m right side of existing spillway

(ii) Middle height section : 50 m upstream of Krenceng intake

5) Results of stability analysis

The results of stability analysis are shown in Figs. G-4 to G-11 and summarized as below.

Section	Case		Slope condition applied	Location of slip circle	Safety Factor	
					Normal	Seismic
1) Maximum	(i)	Reservoir full	U/S=1:3 D/S=1:3	Dam body " Foundation "	3.436(U/S) 2.738(D/S) 7.295 (U/S) 2.772 (D/S)	1.541 (U/S) 1.737 (D/S) 2.200 (U/S) 1.686 (D/S)
			U/S=1:3 D/S-1:2.5	Dam body " Foundation "	3.436 (U/S) 2.369 (D/S) 7.125 (U/S) 2.480 (D/S)	1.541 (U/S) 1.593 (D/S) 2.191 (U/S) 1.546 (D/S)
	(ii)	Reservoir empty	U/S=1:3 D/S=1:3	Dam body " Foundation "	2.569 (U/S) 2.738 (D/S) 6.246 (U/S) 4.283 (D/S)	1.931 (U/S) 2.138 (D/S) 3.946 (U/S) 3.194 (D/S)
			U/S=1:3 D/S=1:2.5	Dam body " Foundation "	2.574 (U/S) 2.369 (D/S) 6.246 (U/S) 3.571 (D/S)	1.933 (U/S) 1.916 (D/S) 3.943 (U/S) 2.764 (D/S)
2) Middle	(i)	Reservoir full	U/S=1:3 D/S=1:2.5	Dam Body " Foundation "	4.628 (U/S) 3.423 (D/S) 5.830 (U/S) 3.518 (D/S)	2.553 (U/S) 2.549 (D/S) 2.441 (U/S) 1.913 (D/S)
	(ii)	Reservoir empty	U/S=1:3 D/S=1:2.5	Dam body " Foundation	3.443 (U/S) 3.423 (D/S) 4.994 (U/S) 4.751 (D/S)	2.949 (U/S) 2.755 (D/S) 3.852 (U/S) 3.815 (D/S)

6) Conclusions

From the above results,the slope of homogeneous earthfill dam is conservatively decided to 1:3.0 in upstream and 1:2.5 in downstream.

The bottom foundation and existing dam are judged to be stable in normal and seismic conditions.

2.6 Stability Against Piping for Heightening of Krenceng Dam

1) Design criteria

The stability against piping is checked by comparing the critical velocity to unstable soil particles with the seepage velocity to be obtained from the seepage analysis by FEM.

2) Condition of seepage analysis

The seepage analysis including the existing dam and bottom foundation was made under the full reservoir, NHWL.29.00 m.

3) Design value

The adopted design value is summarized as below.

Item		Permeability coefficient	
		Horizontal Kx (cm/sec)	Vertical Kv (cm/sec)
1)	Existing dam body	10^{-5}	10^{-5}
2)	Heightening of Krenceng dam	10^{-5}	10^{-6}
3)	Bottom foundation		
	D-CL class above El.8.00 m	10^{-4}	10^{-4}
	CM class below El.8.00 m	10^{-5}	10^{-5}
4)	Curtain grouting zone	10^{-6}	10^{-6}

4) Analyzed section

The seepage analysis was carried out for two (2) sections;

(i) Maximum height section : 20 m right side of existing spillway
(ii) Middle height section : 50 m upstream of Krenceng intake

5) Results

The results for the seepage analysis by FEM are shown in Figs. G-12 and G-13.

The maximum velocity in the heightened embankment body and bottom foundation is summarized as below.

(unit:cm/sec)

Case		Max. velocity in heightened embankment body	Max. velocity in bottom foundation
i)	Maximum height section	0.000005	0.00002
ii)	Middle height section	0.000005	0.00002

6) Critical velocity

Justin's formula for critical velocity is expressed as below.

$$Vc = \{2/3 * (Gs - 1)d\ g\}^{0.5}$$

where,
- Vc : critical velocity (cm/sec)
- Gs : Specific gravity of grain(2.492 g/cm3)
- g : acceleration of gravity (980 cm/sec2)
- d : diameter of grain (cm)

From the results of construction material investigation, d10 is 0.01 mm.

The critical velocity is to be 0.99 cm/sec.

7) Actual velocity

The seepage velocity obtained by FEM is based on Darcy's formula, so that the actual velocity through the void is obtained as below.

$$V = Vd/b$$

$$b = 1 - (1-n)^{2/3}$$

$$n = e/(1+e)$$

where,
- V : actual seepage velocity (cm/s)
- Vd : mean velocity (cm/sec)
- b : area void ratio
- n : porosity (0.425)
- e : void ratio (0.74)

Therefore,

$$V = Vd/0.308$$

Accordingly, the actual seepage velocity is obtained as below.

Case		Max. actual seepage velocity (cm/sec)	
		Embankment body	Bottom foundation
i)	Maximum section	0.00002	0.00006
ii)	Middle section	0.00002	0.00006

The above results show that the actual seepage velocity in the embankment body and foundation is far less than the critical velocity obtained by Justin's formula.

2.7 Type of Spillway

The spillway for the heightening of Krenceng dam is relocated at the left side of the existing spillway taking into account topographical and geological conditions.

The comparative study for the type of spillway was studied for 2 alternatives by fixing the two gated spillways (2 nos.x 7.75 m wide x 4.3 m high) on the weir crest El.25.00 m and dam slope with 1:3.0 in upstream and 1:2.5 in downstream ;

Type A : Flip bucket with plunge pool
Type B : Hydraulic jump with horizontal stilling basin

Typical sections for the above two types of spillway are shown in Fig. G-14.

Item			Type A	Type B
1)	Chuteway	length (m)	51.748	46.128
2)	"	slope	1:3.5	1:2.5
3)	"	width(m)	18	18
4)	Energy dissipator	length (m)	50	31.50
5)	"	depth (m)	7.6	7.00
6)	"	bottom (El-m)	6.00	7.00

The work quantity and its construction cost are summarized as below.

	Item	Unit	Type A	Type B
1)	Excavation	m^3	51168	48123
2)	Concrete	m^3	13846	15865
3)	Rein bar	t	210	179
4)	Grouting	t	54	54
5)	Spillway gate		2 Nos.x7.75Bx4.3H	2 Nos.x7.75Bx4.3H
6)	Direct const.cost			
	Civil work	10^6Rp	6400	6230
	Metal work	10^6Rp	1404	1404
	Total	10^6Rp	7804	7634

As shown in the above, Type B, the hydraulic jump type is economical more than Type A.

The optimized structural components for the heightening of Krenceng dam are shown in Figs. G-15 to G-17.

3 Cidanau Gated Weir

3.1 Design Flood

1) Design flood

The design discharges for the spillway,energy dissipator and river diversion are specified in accordance with the design criteria described in Chapter 2.1.

2) Design peak discharge

<u>Hydrological characteristics in upstream Cidanau basin</u>

The records for the flood hydrograph and hourly duration rainfall in the upstream Cidanau basin were not obtained.

However,as seen in the hydrological data including the Kubang Baros water level gauging station and Padarincang rainfall gauging station, the average daily discharge at Kubang Baros is very small comparing to the flood estimated by the actual daily rainfall.

This difference is caused by the following reasons;

(i) There is a vast and nature reserve area,Rawa Danau having more 70 km^2 in flat upstream of Kubang Baros.

(ii) The discharge capacity in the river course through the Rawa Danau will be limited less 100 m^3/sec.

(iii) The flood beyond the discharge capacity will be inundated surrounding the Rawa Danau.

(iv) The peak flood will retard for the reach between upstream Rawa Danau and Kubang Baros due to the inundation and storage in the Rawa Danau.

Adopted design peak discharge

In this study, the probable design peak discharge against the Cidanau gated weir was tentatively obtained by the flood routing at the Kubang Baros because no data for topography survey in the river course of Rawa Danau and its inundation area. The flood due to the remaining basin between Kubang Baros and Cidanau gated weir site is neglected because the peak in both hydrographs is different.

The rating curve at the Kubang Baros was enlarged by the map of 1:5000 adding to that of gauging station.

Flood routing

The base flow during the flood was assumed at 30 m^3/sec considering the average discharge in wet season.

The results of flood routing are shown in Figs. G-18 to G-20 .

Probable flood (year)	Peak discharge at Cidanau gated weir (m^3/sec)
25	346
1.2 x200	535
PMF	907

3.2 Spillway Gate

1) Sizing of spillway gate

The Cidanau gated weir was designed as an alternative storage plan by provision of a weir with high gate located at 200 m upstream of the existing Krakatau intake weir which may possibly flush the sediment.

From the topographic condition of river course and maximum exploitable gate scale, the size of gate was determined by 3 nos. x 17 m (width) x 20 m (height).

The crest of weir is set at El.1.50 m considering the tidal level condition.

2) Gate operation

The gates will be operated to keep NHWL.21.20 m as far as possible.

The discharge capacity of gate opening is sufficient for the regulated of PMF.

3.3 Dam Crest Elevation

From the design criteria described in Chapter 2.3, the crest elevation of Cidanau gated weir is set at El.24.20 m adding to the freeboard, 3.0 m.

3.4 Type of Cidanau Gated Weir

From the structural requirement for the spillway gate,the type of overflow section is designed by the concrete gravity.

The comparative study for type of both abutments was made for the following 2 alternatives ;

Type A : Impervious random earthfill
Type B : Concrete gravity

Typical sections for the above two types of abutments are shown in Figs. G-21.

Item		Unit	Type A	Type B
Dam(Non-overflow)				
1)	Excavation	m^3	33320	21324
2)	Concrete	m^3	-	23519
3)	Embankment	m^3	112831	-
4)	Grouting	t	571	571
Spillway(Overflow)				
1)	Dental work			
	Excavation	m^3	22389	22387
	Concrete	m^3	8379	8379
2)	Weir&pier			
	Excavation	m^3	1670	1670
	Concrete	m^3	10761	10761
3)	Hoist&bridge			
	Concrete	m^3	1200	1200
4)	Abutment wall			
	Concrete	m^3	31175	-
5)	Spillway gate			
	Roller gate		3 Nos.x17Bx20H	3 Nos.17Bx20H
6)	Direct const.cost			
	Civil work	10^6Rp	19188	7569
	Metal work	10^6Rp	25272	25272
	Total	10^6Rp	44460	32841

As shown in the table above, the concrete gravity type is economically adopted compare with the impervious random earthfill type.

3.5 Outlet Facility

The water stored in the Cidanau gated weir is conveyed to the existing trap basin through 1400 steel conduit, 1400 hollow jet valve and the energy dissipator.

1) Inlet

The inlet structure which consist of 1400 steel conduit is embedded in the non-overflow concrete dam body and the centre of steel conduit is set at El.2.20 m.

2) Pipe line between dam and valve house for hollow jet valve

The pipe line, 150 m long between the dam toe and the valve house is embedded along the downstream right bank.

3) Valve house

The valve house for the guard valve and hollow jet valve is located at the end of 1400 steel conduit just upstream of existing Krakatau intake weir.

The stored water is released by the guard valve and hollow jet valve.

4) Energy dissipator

The energy dissipator for the outlet is provided at the downstream of hollow jet valve so as to make the stable flow condition into the trap basin.

The size of energy dissipator is designed by 3.5 m wide x 5.5 m depth x 39.4 m length.

The optimized structural components of the Cidanau gated weir are shown in Figs. G-22 and G-23.

4 Beroeng Diversion Tunnel

4.1 Diversion Tunnel

Based on the results of optimization study,the internal diameter of the diversion tunnel is fixed at 1.50 m. The diversion tunnel is of pressure type. The tunnel length is about 280 m with no slope.

The bottom elevation excavated at both inlet and outlet portals is set at El.36.80 m which corresponds to the river bed in the outlet side because the river bed in the Beroeng river at the inlet portal is about 5 m lower than the river bed of Krenceng river at the outlet portal.

4.2 Maximum Discharge Capacity

The outflow discharge into the diversion tunnel is determined by combination of the overflow water depth and width at the control weir to be provided in front of tunnel satisfying the pressure tunnel flow condition. The water surface level at the inlet portal is set at the elevation adding to the water depth of one (1) diameter of the tunnel above the elevation of tunnel crown at inlet.

El.of tunnel invert at inlet	:	El.37.10 m
El.of tunnel crown at inlet	:	El.38.60 m
El.of crest for control weir	:	El.39.35 m
Width of control weir	:	3 m
Maximum discharge	:	4 m^3/sec
Water level at maximum discharge	:	Wl.40.10 m

4.3 Intake Weir

The intake weir is provided crossing the Beroeng river,just downstream of tunnel portal. The intake weir consist of non-overflow section and overflow section. The crest elevation of overflow section is set at El.40.10 m which corresponds to the water surface level on the control weir at the maximum discharge, 4 m^3/sec. The discharge beyond 4 m^3/sec is released from the overflow section to the downstream Beroeng.

The crest elevation of non-overflow section is set at El.44.10 m adding to the freeboard, 1.0 m plus the overflow depth, 3.0 m against the 200-year probable flood, 170 m^3/sec above El.40.10 m.

4.4 Outlet Facility

The outlet facilities which consist of 800 Howell-bunger and 2000x2000 outlet gate are provided in the concrete non-overflow section of intake weir so as to release the downstream water requirement such as irrigation and river maintenance flow and to flush the sediment.

The optimized structural components of the Beroeng diversion tunnel are shown in Figs. G-23 to G-26.

5. Water Conveyance Facilities

5.1 Basic Design Criteria

The basic design criteria for the water conveyance facilities is described as below.

5.1.1 Study Cases

According to the selected schemes as the feasibility study, the following three cases are studied;

Scheme		Name of development	Net safe yield (m^3/sec)
1)	K-1	Heightening of Krenceng dam without diversion	3.05
2)	K-2	Heightening of Krenceng dam with Beroeng diversion	3.11
3)	C-3	Heightening of Krenceng dam with Beroeng diversion and Cidanau gated weir	3.435

5.1.2 Design Conditions

(1) In principle,the incremental development due to the Project above the design capacity of existing Krakatau water conveyance and treatment facilities is additionally provided for the intake, sand trap basin and Cidanau pump station and the water treatment plant.

(2) The existing water conveyance facility which consists of Cidanau pump station and 27.2 km long pipe line having the conveyance capacity of 2 m^3/sec at the maximum pumping head of 67.1 m is operated so as not to exceed the design capacity of the existing.

(3) The additional pumps to be provided at Cidanau pump station should be designed at the total head of 67.1 m.

(4) In case that the development yield exceeds the existing, full water cannot be conveyed upto the Krenceng reservoir by using the design head of Cidanau pumps. Therefore, the booster pump station should be provided at the intermediate point, about 14.25 km from the Cidanau pump station so as to convey the water up to the Krenceng reservoir after receiving of water conveyed from the Cidanau pump station.

(5) According to the booster pump station, the surge tank is additionally provided at the place between the booster pump station and Krenceng receiving well.

(6) The existing Krenceng pump station is replaced in accordance with the proposed heightening of Krenceng dam.

5.2 Intake Facilities

5.2.1 Intake

Assuming the flow velocity in the intake of 0.5 m/sec and effective water depth of 1.6 m which are the same as the existing,the additional intake facility is estimated as below.

$$B = Q/v/H \quad (m)$$

where, B : width of intake (m)
v : flow velocity (0.5 m/sec)
H : water depth (1.6 m)

Scheme	Additional discharge (cms)	Width of additional intake, B (m)
K-1	1.05	1.50
K-2	1.11	1.50
C-3	1.435	2.00

5.2.2 Sand Trap Basin

The sand trap basin is added by one lane, 77.6 m length x 6.5 m wide x 1.8 m depth which is same as the dimension of existing trap basin. The settling velocity and flow velocity of the additional sand trap basin is estimated as below.

$$F=Q/As$$
$$V=Q/A$$

where, F : settling velocity (cm/sec)
Q : discharge (m^3/day)
As : surface area of trap basin (77.6 m length x 6.5 m wide)
A : sectional area of flow (6.5 m wide x 1.8 m water depth)
V : flow velocity (m/sec)

	Scheme			Allowable design in Japan 1)
	K-1	K-2	K-3	
Settling velocity (cm/sec)	0.21	0.22	0.28	< 0.3-0.8
Flow velocity (m/sec)	0.09	0.095	0.123	> 0.02-0.07

As seen in the above,the dimension of additional sand trap basin is satisfied for surface area but flow velocity exceeds the allowable one.

The sand trap basin is additionally provided by one lane of the existing one considering the allowance of settling velocity of the existing.

5.3 Water Conveyance Facilities

5.3.1 Design of Water Conveyance Facilities

The existing Cidanau pump station is designed with the discharge of 3000 m^3/hr and the total head, H=67.1 m.

Assuming that the development yield including the existing is directly conveyed from the Cidanau pump station to the Krenceng reservoir through a 27.2 km long pipe line, the required total head is estimated as below.

$$H = Ha + Hf + alfa \quad (m)$$

where,
- H : total head (m)
- Ha : static head (31.6 m=33 m at Krenceng - 1.4 m at Cidanau)
- Hf : friction head
- alfa : residual head(1.0 m)

$$Hf = I*L \quad (m)$$

$$I = 10.666^{-1.85}*D^{-4.87}*Q^{1.85}$$

- I : hydraulic gradient
- L : length of pipe line (27200 m)
- C : velocity coefficient (100)
- Q : discharge (m^3/sec)

Scheme	Q (m^3/sec)	Ha (m)	Hf (m)	alfa (m)	H (m)
K-1	3.05	31.6	88.7	1.0	121.3
K-2	3.11	31.6	91.9	1.0	124.8
C-3	3.435	31.6	110.4	1.0	143.0

The results of the mentioned above reveals that;

1) The total head will reach to 120-140 m which corresponds to two times of design head, 67.1 m for the existing pump. This will bring the instability operation due to the difference of capacity between additional and existing pumps and the structural damages for the existing pipe line at the bend and joint portions although the working stress is lower than the allowable stress.

2) The pumping head due to the development yield including the existing should be designed so as not to exceed the design head of existing pumps and thus available conveyance distance is determined.

3) The booster pump station should be provided at the intermediate point of 27.2 km pipe line so as to convey the water upto the Krenceng reservoir after receiving of water conveyed from the Cidanau pump station.

5.3.2 Cidanau Pump Station

From the check calculation of actual pumping head, the water is conveyed from the Cidanau pump station upto 14.25 km by using the Cidanau pump station (Fig. G-27).

Scheme	Q (cms)	I (10^{-3})	Ha (m)	Hf (m)	alfa (m)	H (m)
K-1	3.05	3.26	1.0 1]	46.5	1.0	48.5
K-2	3.11	3.38	1.0	48.2	1.0	50.2
C-3	3.435	4.06	1.0	57.9	1.0	59.9

Note: 1] Booster pump station El.2.40 minus Cidanau pump station El.1.40 m

As seen in the above, total head is smaller than the design head, 67.1 m.

<u>Pump & motor output</u>

P = 0.163*Q*H*(1+ k)/eta (kw)

where, Q : discharge (m^3/min)
H : total head (m)
eta : Pump efficiency (0.8)
k : allowance (0.25)

Scheme	Q (cms)	H (m)	P (kW)	Unit	Pump output (kW)	Total output (kW)
K-1	1.05	67.1	1078	2	550	1100
K-2	1.11	67.1	1138	2	580	1160
C-3	1.435	67.1	1471	2	740	1480
Existing	2.00	67.1	2050	4 1]	1000 2]	4000

Note: 1] included one standby.
2) Actual capacity of the existing because the pump is designed at 2.5 m^3/sec.

5.3.3 Booster Pump Station

Water is conveyed from the intermediate point upto the Krenceng receiving well, about 12.95 km by the booster pump station.

Scheme	Q (cms)	I (10^{-3})	Ha (m)	Hf (m)	alfa (m)	H (m)
K-1	3.05	3.26	30.6 1]	42.2	1.0	73.8
K-2	3.11	3.38	30.6	43.8	1.0	75.4
C-3	3.435	4.06	30.6	52.6	1.0	84.2

Note: 1] Receiving well El.33.00 minus booster pump station El.2.40 m

In principle,the quantity of booster pump station is required for the purpose of regular operation and one standby so as to avoid the trouble due to the operation under the high pressure condition.

Scheme	Q (cms)	H (m)	P (kW)	Unit	Pump output (kW)	Total output (kW)
K-1	3.05	73.8	3440	4 1]	1150	4600
K-2	3.11	75.4	3584	4	1200	4800
C-3	3.435	84.2	4420	4	1500	6000

Note: 1] included one standby.

5.3.4 Surge Tank

The existing one-way surge tank is located at about 4 km point from the Cidanau pump station so as to reflect the pressure wave and protect the pipe line against water hammer overpressures due to pump-trip.

Owing to the addition of booster pump, the surge tank is added to the existing one and provided at the place between the booster pump station and Krenceng receiving well.

In this study,the volume of surge tank was tentatively estimated according to the water conveyance of existing, 2.0 m^3/sec and its surge tank, 575 m^3. The existing surge tank is also replaced as below.

Scheme	Volume of surge tank (m^3)	Nos. of surge tank
K-1	915	2
K-2	933	2
C-3	1030	2

5.4 Treatment Facilities

The existing water treatment facilities which are designed at 2.0 m^3/sec consist of the high rate coagulation basin,rapid sand filtration, sludge treatment,purified water reservoir and water tower (Fig. G-28).

5.4.1 Krenceng Pump Station

The storaged water in the Krenceng reservoir is conveyed to the water treatment facilities by using the Krenceng pump station.

Owing to the heightening of Krenceng dam, the Krenceng pump station is replaced. The quantity of pumps is provided for regular operation and one standby in an emergency.

Scheme	Q (cms)	H (m)	P (kW)	Unit	Pump output (kW)	Total output (kW)
K-1	3.05	20 [2]	932	4 [1]	310	1240
K-2	3.11	20	950	4	320	1280
C-3	3.435	20	1050	4	350	1400
Existing	2	13	410	5 [1]	110 [4]	550

Note: 1] included one standby.
2] Ha=El.33-LWL.18.5=14.5 m, Hf=3.0 m, alfa=1.0 m
3] Ha = EL.28.2 - LWL.18.0 = 10.2 m, Hf = 2.0 m, alfa = 1.0 m
4] Actual capacity of the existing.

5.4.2 Receiving Well

The dimension of receiving well is estimated by the storaged volume in 5 minutes according to the existing.

Scheme	Volume of receiving well (m^3)
K-1	275
K-2	280
C-3	310

5.4.3 High-rate Coagulation Basin

The existing facilities consist of three (3) units x 0.7 m^3/sec (=1,500 m^2 x settling rate 1.6 m/hr/3,600 sec). It includes the chemical feeding and associated equipment. The facility is added by incremental yield due to the project, 3-4 units x 0.5 m^3/sec.

5.4.4 Rapid Sand Filtration

The existing facilities are designed at 0.5 m^3/sec per unit. The following units are additionally required.

Scheme	Required units
K-1	3
K-2	3
C-3	4

Note: 1] included one standby.

5.4.5 Purified Water Reservoir

The existing facility is designed at 7200 m^3 (= 2 m^3/sec x 3600 sec) which corresponds to the capacity-to-supply of purified water in 1 hour.

The following is added.

Scheme	Additional purified water reservoir (m^3)
K-1	3780
K-2	3966
C-3	5170

5.4.6 Water Tower

The existing water tower is designed by the capacity-to-supply of water in 15 minutes.

The following is added.

Scheme	Volume of receiving well (m^3)
K-1	945
K-2	999
C-3	1290

5.4.7 Sludge Treatment

The sludge treatment will be designed according to the existing.

The optimized structural components of water conveyance and treatment facilities are shown in Figs. G-29 to G-33 and summarized in Table G-1.

The principal features for selected development schemes are summarized in Table G-2.

TABLES

Table G-1 Water Conveyance and Treatment Facilities

	Scheme		
	K-1	K-2	C-3
<u>Intake Facilities (Cidanau)</u>			
Intake			
Width	1.5 m	1.5 m	2.0 m
Depth	2.2 m	2.2 m	2.2 m
Length	40 m	40 m	40 m
Sand trap basin			
Width	6.5 m	6.5 m	6.5 m
Depth	1.8 m	1.8 m	1.8 m
Length	77.6 m	77.6 m	77.6 m
<u>Water Conveyance Facilities</u>			
Cidanau pump station			
Additional capacity	1.05 m^3/s	1.11 m^3/s	1.435 m^3/s
Total head	67.1 m	67.1 m	67.1 m
Flow rate for unit	0.53 m^3/s	0.56 m^3/s	0.72 m^3/s
Number of units	2	2	2
Pump motor output	550 kW/unit	580 kW/unit	740 kW/unit
Booster pump station			
Total capacity	3.05 m^3/s	3.11 m^3/s	3.435 m^3/s
Total head	73.8 m	75.4 m	84.2 m
Flow rate per unit	1.02 m^3/s	1.04 m^3/s	1.15 m^3/s
Number of units	4 units 1]	4 units 1]	4 units 1]
Pump motor output	1,150 kW/unit	1,200 kW/unit	1,500 kW/unit
Surge tank			
Capacity pre unit	915 m^3/s	933 m^3/s	1,010 m^3/s
Number of units	2	2	2
<u>Purification Facilities</u>			
Receiving well			
Well capacity	275 m^3	280 m^3	310 m^3
High-rate coagulation basin			
Treatment capacity	0.7 m^3/s/unit	0.7 m^3/s/unit	0.7 m^3/s/unit
Number of units	2	2	2
Rapid sand filter			
Treatment capacity	0.5 m^3/s/unit	0.5 m^3/s/unit	0.5 m^3/s/unit
Number of units	3	3	4
Purified reservoir			
Capacity	3,800 m^3	4,000 m^3	5,200 m^3
Water tower			
Capacity	950 m^3	1,000 m^3	1,300 m^3
Krenceng pump station			
Total capacity	3.05 m^3/s	3.11 m^3/s	3.435 m^3/s
Total head	20 m	20 m	20 m
Flow rate per unit	1.02 m^3/s	1.04 m^3/s	1.15 m^3/s
Number of units	4 units 1]	4 units 1]	4 units 1]
Power	310 kW/unit	320 kW/unit	350 kW/unit

Note: 1] Included one standby

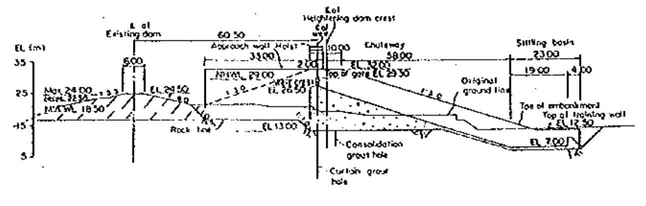
Table G-2 Principal Features for Priority Development Schemes

		K-1	K-2	C-3	
		Heightening of Krenceng Dam without Diversion	Heightening of Krenceng Dam with One Diversion	Heightening of Krenceng Dam with One Diversion	Cidanau Gated Weir
Reservoir					
Name of river		Krenceng	Krenceng	Krenceng	Cidanau
Catchment area	km^2	13.3	13.3	13.3	214.95
Reservoir surface area	km^2	1.8	1.8	1.8	0.41
Gross capacity	10^6m^3	14.1	14.1	14.1	3.44
Effective capacity	10^6m^3	12.9	12.9	12.9	3.44
Development yield	m^3/s	3.05	3.11	3.11	0.325
High water level	EL-m	29.0	29.0	29.0	21.2
Low water level	EL-m	18.0	18.0	18.0	0
Annual rainfall	mm/yr	2,250	2,250	2,250	3,000
Mean runoff	m^3/sec	0.43	0.43	0.43	0.43
Design peak flood	m^3/sec				
25 yrs		128	128	128	346[1/]
100 yrs		171	171	171	444[1/]
1.2 x 200 yrs		225	225	225	535[1/]
Dam and Rated Facility					
Diversion Work					
River diversion		Multi-stage diversion	Multi-stage diversion	Multi-stage diversion	Multi-stage diversion
Diversion tunnel, L	m	-	-	-	-
D	m	-	-	-	-
Diversion gate	Nos.	-	-	-	-
Dam					
Type		Impervious random-fill	Impervious random-fill	Impervious random-fill	Gravity
Crest elevation	EL-m	32	32	32	24.2
Height (from river bed)	m	16	16	16	24.2
Crest length	m	2,911	2,911	2,911	299
Embankment/Conc.volume	10^3m^3	1,270	1,270	1,270	43
Spillway					
Type		Roller gate	Roller gate	Roller gate	Roller gate
Crest elevation of weir	EL-m	25	25	25	1.5
Width of weir	m	18	18	18	61
Gate (wide x height x Nos.)		7.75x4.3x2	7.75x4.3x2	7.75x4.3x2	17x20x3
Outlet Works					
Intake type		-	-	-	Horizontal
Steel conduit, L	m	-	-	-	200
Guard valve	Nos.	-	-	-	1
Hollow jet valve	Nos.	-	-	-	1
Diversion Tunnel					
Name of river		-	Beroeng	Beroeng	
Catchment area at weir	km^2	-	12.1	12.1	-
Mean runoff	m^3/sec	-	0.39	0.39	-
Maximum discharge capacity	m^3/sec	-	4.0	4.0	-
Diverted tunnel, L	m	-	280	280	-
D	m	-	1.5	1.5	-
Water Transmission Facility					
Transmission pipeline, L	km	Existing	Existing	Existing	Existing
D	m	Existing	Existing	Existing	Existing
Krakatau pump station [3/]					
Pump discharge	m^3/s	1.05	1.11	1.435	
Pump head	m	67.1	67.1	67.1	
Additional pumps	kW	2unitsx550	2unitsx580	2unitsx740	
Booster pump station [3/]					
Pump discharge	m^3/s	3.05	3.11	3.435	
Pump head	m	73.8	75.4	84.2	
Pump capacity	kW	4unitsx1150	4unitsx1200	4unitx1500	
Krenceng pump station [2/]					
Pump discharge	m^3/s	3.05	3.11	3.435	
Pump head	m	20	20	20	
Pump capacity [4/]	kW	4unitsx310	4unisx320	4unitsx350	
Water treatment plant [3/]	m^3/hr	5400	5400	7200	

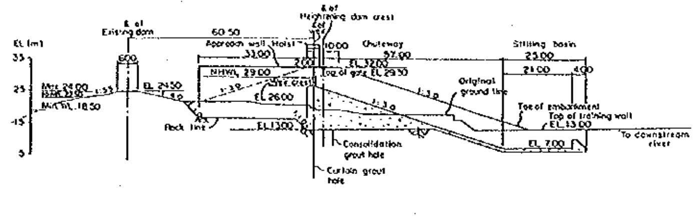
Notes:
1/ means regulated peak outflow at the outlet of Rawa Danau.
2/ Facility replaced due to development scheme
3/ Facility added due to development scheme
4/ Included one standby.

FIGURES

Fig. G-1



PROFILE OF SPILLWAY



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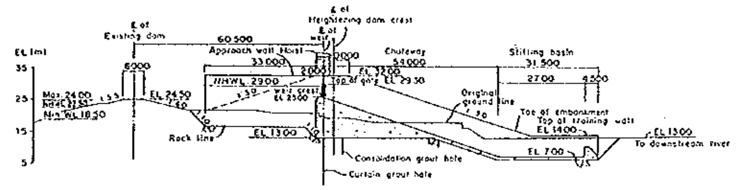

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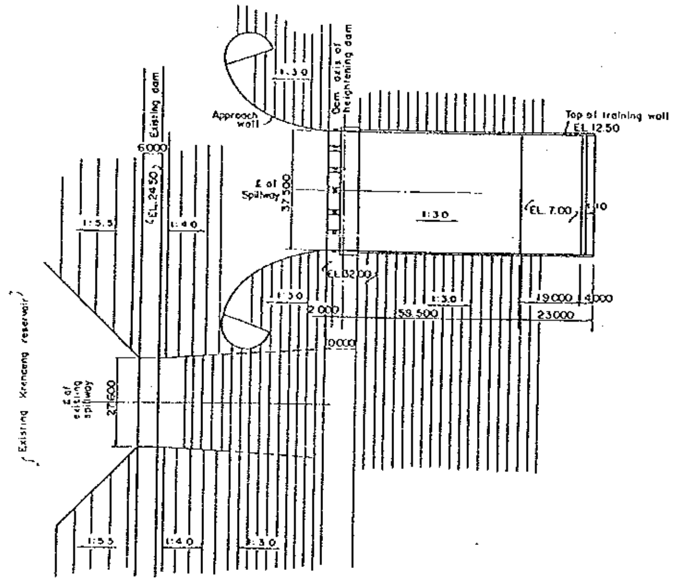
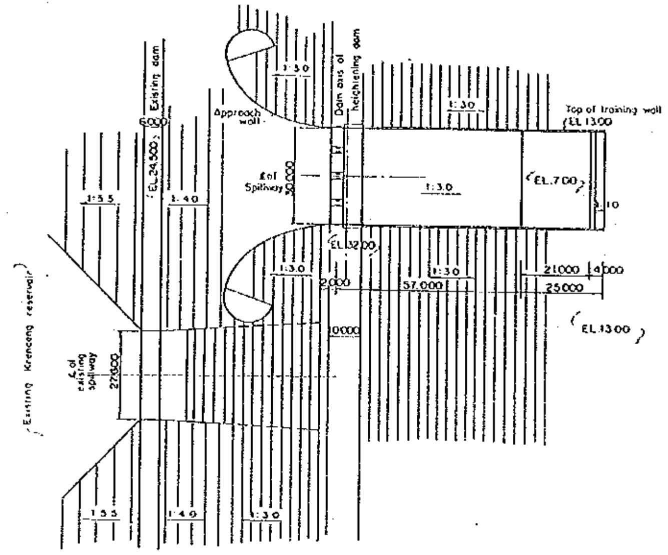
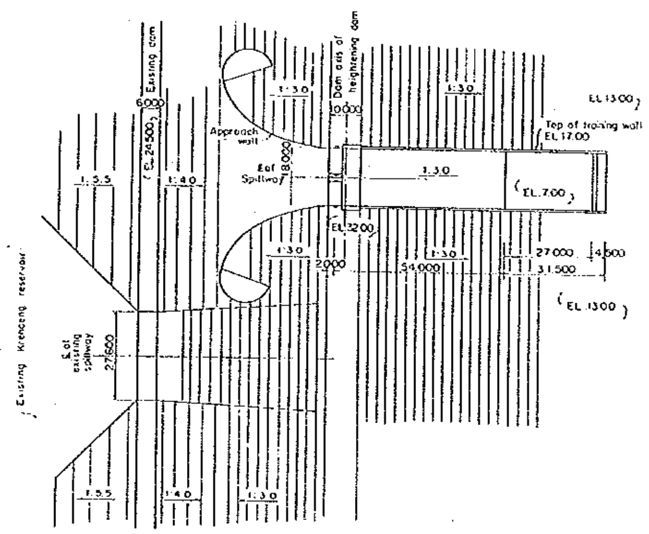


PLAN OF SPILLWAY



PLAN OF SPILLWAY



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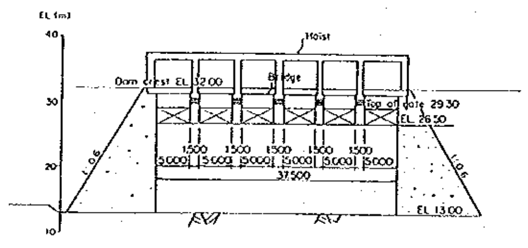
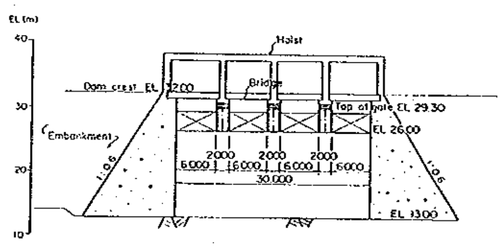
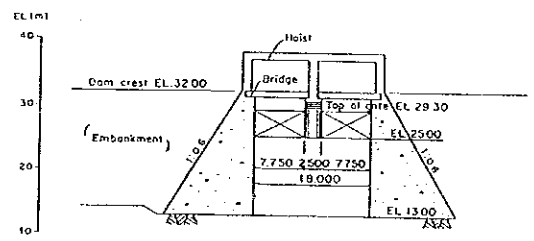


UPSTREAM VIEW OF SPILLWAY



UPSTREAM VIEW OF SPILLWAY



UPSTREAM VIEW OF SPILLWAY

Alternative Gated Spillways for Heightening of Krenceng Dam

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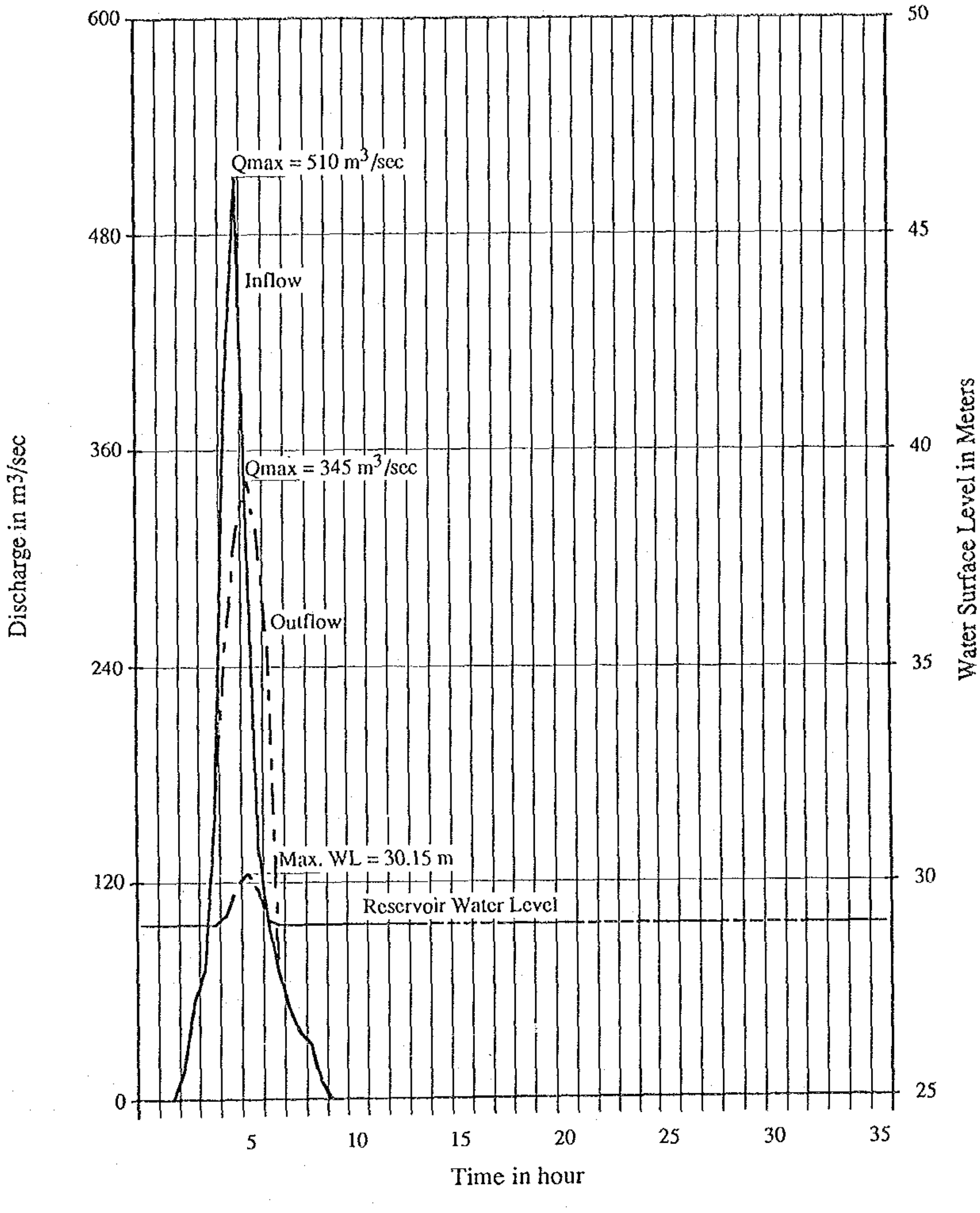

Flood Routing by Krenceng Spillway against PMF

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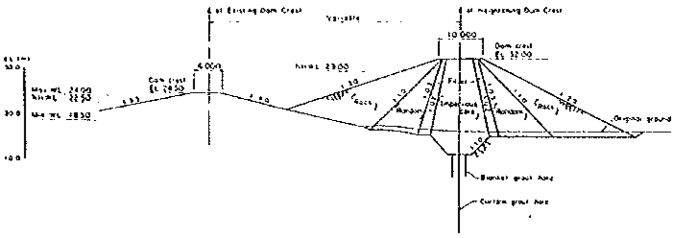
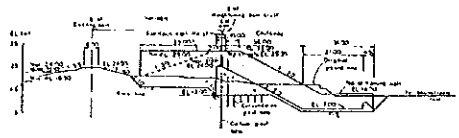
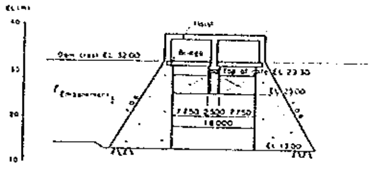
Fig. G-3



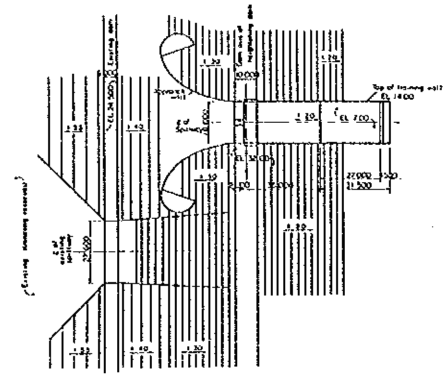
PROFILE OF HEIGHTENING DAM



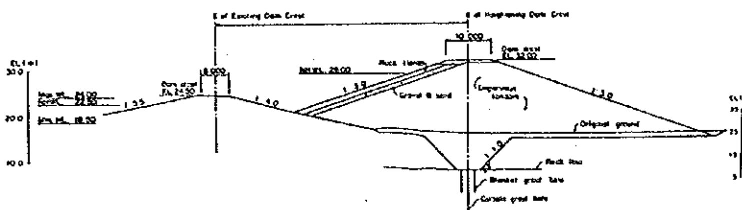
PROFILE OF SPILLWAY



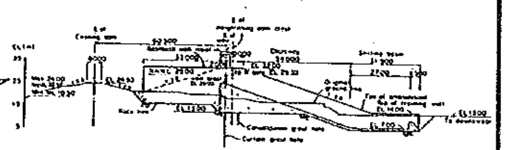
UPSTREAM VIEW OF SPILLWAY



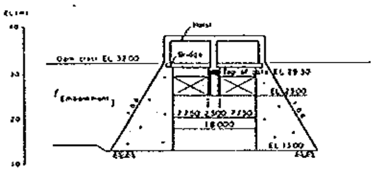
PLAN OF SPILLWAY



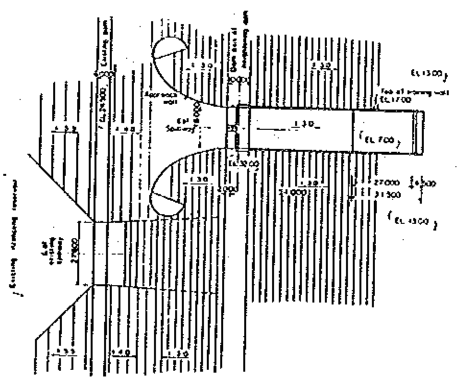
PROFILE OF HEIGHTENING DAM



PROFILE OF SPILLWAY



UPSTREAM VIEW OF SPILLWAY



PLAN OF SPILLWAY

Alternative Dam Types for Heightening of Krenceng Dam

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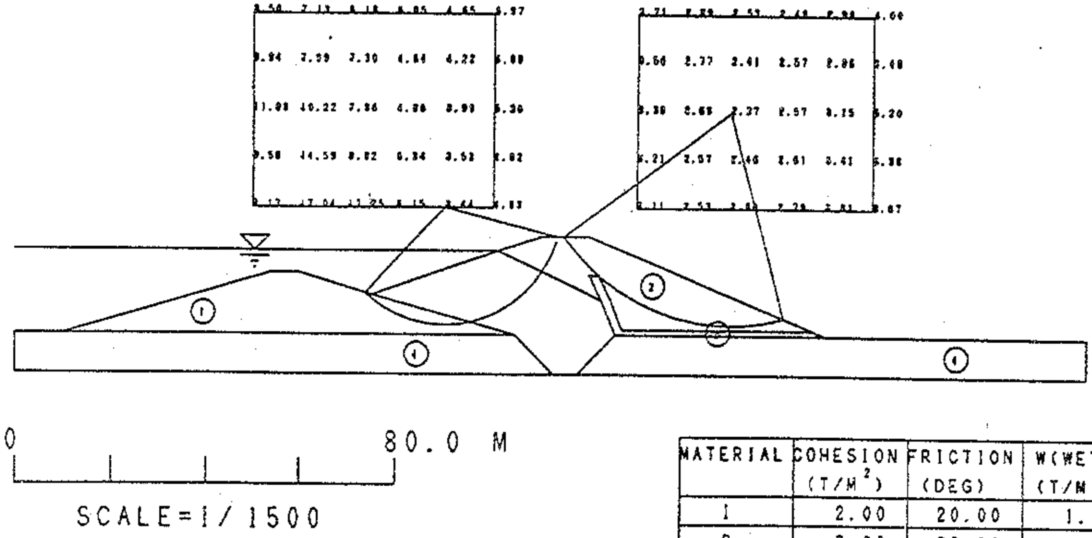
Fig. G-4



MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	2.00	20.00	1.70	1.80	0.80
2	3.00	26.00	1.73	1.86	0.86
3	0.00	33.00	1.72	2.02	1.02
4	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.150	

MINIMUM SAFETY FACTOR (NORMAL)	
UP STREAM SIDE	DOWN STREAM SIDE
3.436	2.369

Stability Analysis of Heightening of Krenceng Dam
(Maximum Section, Reservoir Full, Normal Condition)

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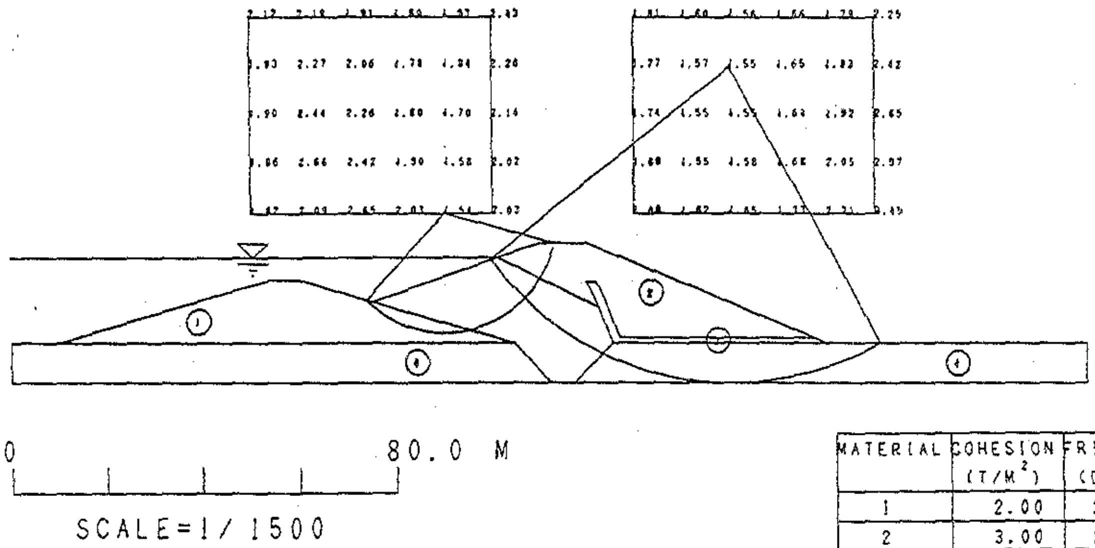
Fig. G-5



MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	2.00	20.00	1.70	1.80	0.80
2	3.00	26.00	1.73	1.86	0.86
3	0.00	33.00	1.72	2.02	1.02
4	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.150	

MINIMUM SAFETY FACTOR (SEISMIC)	
UP STREAM SIDE	DOWN STREAM SIDE
1.541	1.546

Stability Analysis of Heightening of Krenceng Dam
(Maximum Section, Reservoir Full, Seismic Condition)

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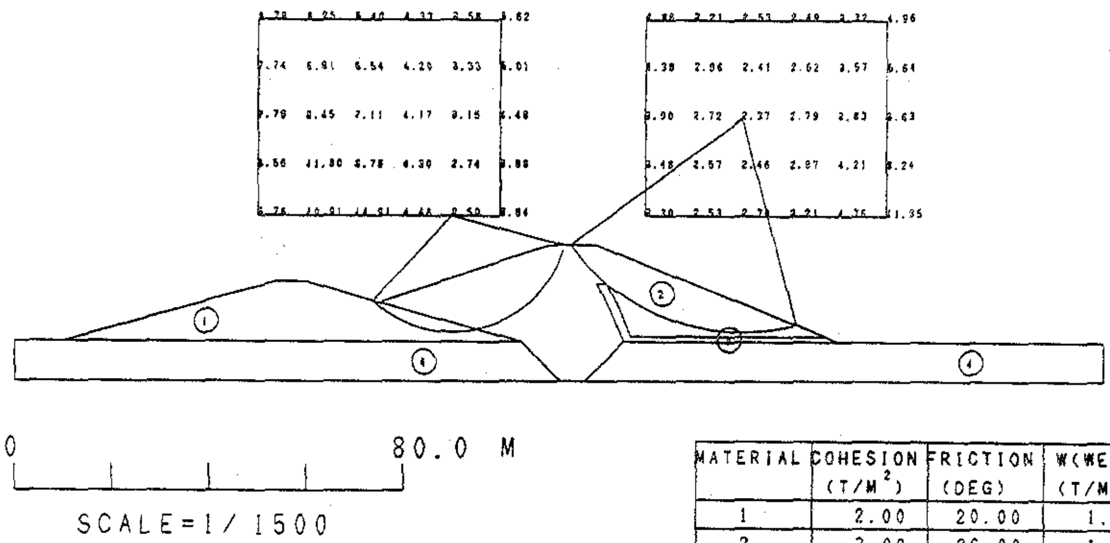

MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	2.00	20.00	1.70	1.80	0.80
2	3.00	26.00	1.73	1.86	0.86
3	0.00	33.00	1.72	2.02	1.02
4	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.075	

MINIMUM SAFETY FACTOR (NORMAL)	
UP STREAM SIDE	DOWN STREAM SIDE
2.590	2.369

Stability Analysis of Heightening of Krenceng Dam
(Maximum Section, Reservoir Empty, Normal Condition)

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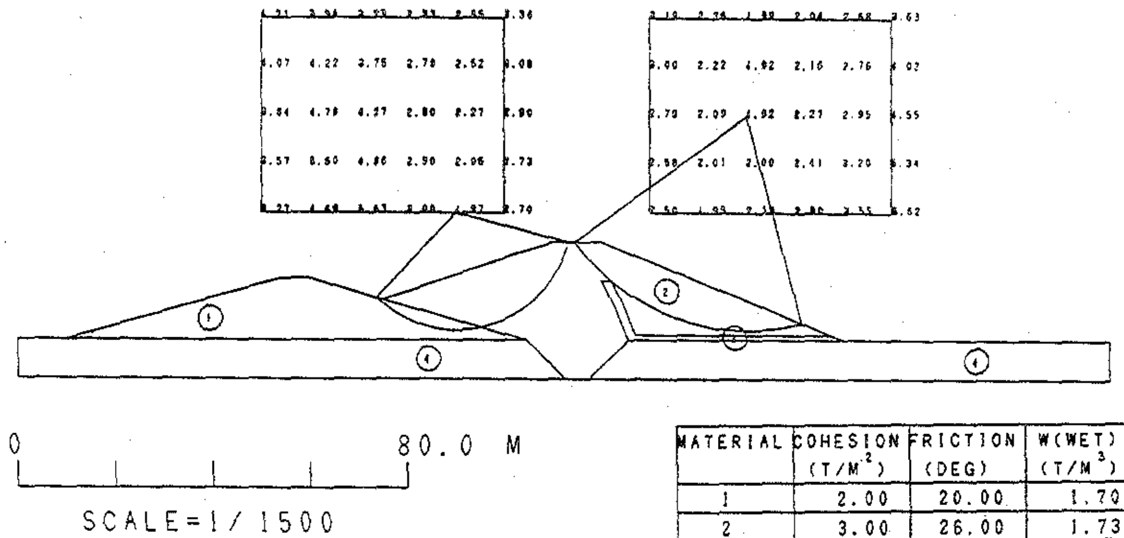
Fig. G-7



MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	2.00	20.00	1.70	1.80	0.80
2	3.00	26.00	1.73	1.86	0.86
3	0.00	33.00	1.72	2.02	1.02
4	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.075	

MINIMUM SAFETY FACTOR (SEISMIC)	
UP STREAM SIDE	DOWN STREAM SIDE
1.971	1.916

Stability Analysis of Heightening of Krenceng Dam
(Maximum Section, Reservoir Empty, Seismic Condition)

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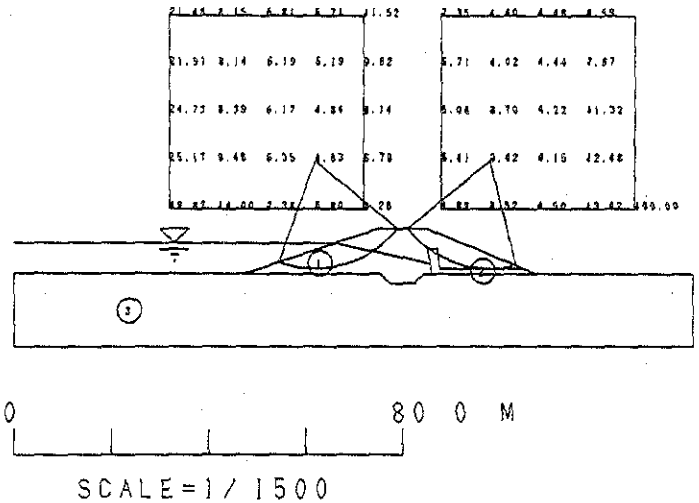
Fig. G-8



MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	3.00	26.00	1.73	1.86	0.85
2	0.00	33.00	1.72	2.02	1.02
3	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.150	

MINIMUM SAFETY FACTOR (NORMAL)	
UP STREAM SIDE	DOWN STREAM SIDE
4.628	3.423

Stability Analysis of Heightening of Krenceng Dam
(Middle Section, Reservoir Full, Normal Condition)

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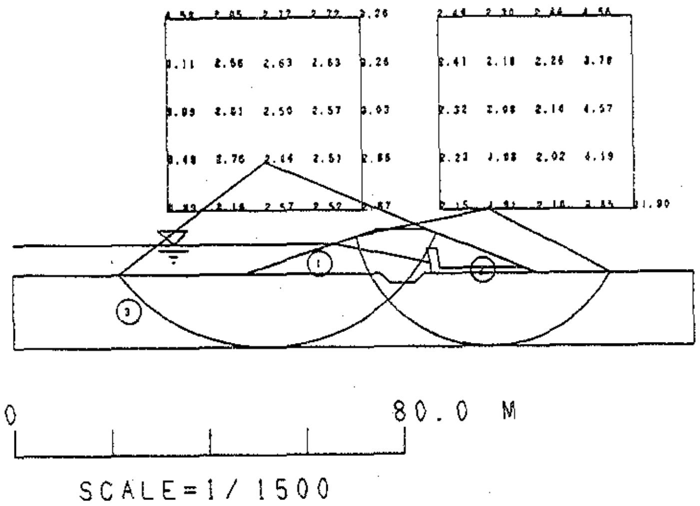
Fig. G-9

MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	3.00	26.00	1.73	1.86	0.86
2	0.00	33.00	1.72	2.02	1.02
3	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.150	

MINIMUM SAFETY FACTOR (SEISMIC)	
UP STREAM SIDE	DOWN STREAM SIDE
2.441	1.913



Stability Analysis of Heightening of Krenceng Dam
(Middle Section, Reservoir Full, Seismic Condition)

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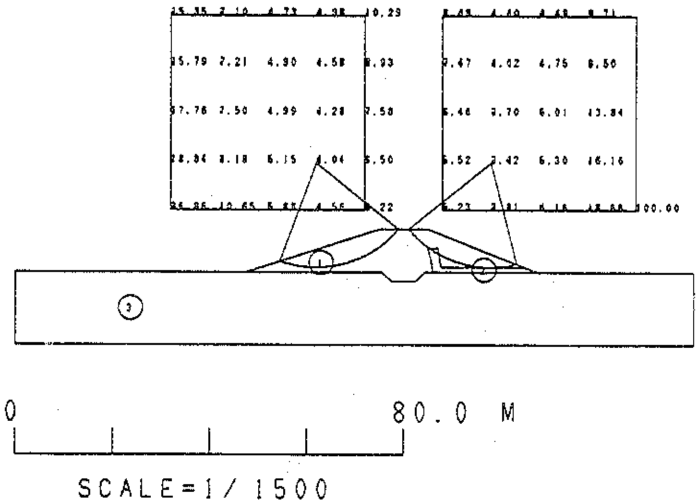
Fig. G-10



MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	3.00	26.00	1.73	1.86	0.86
2	0.00	33.00	1.72	2.02	1.02
3	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.075	

MINIMUM SAFETY FACTOR (NORMAL)	
UP STREAM SIDE	DOWN STREAM SIDE
4.043	3.423

Stability Analysis of Heightening of Krenceng Dam
(Middle Section, Reservoir Empty, Normal Condition)

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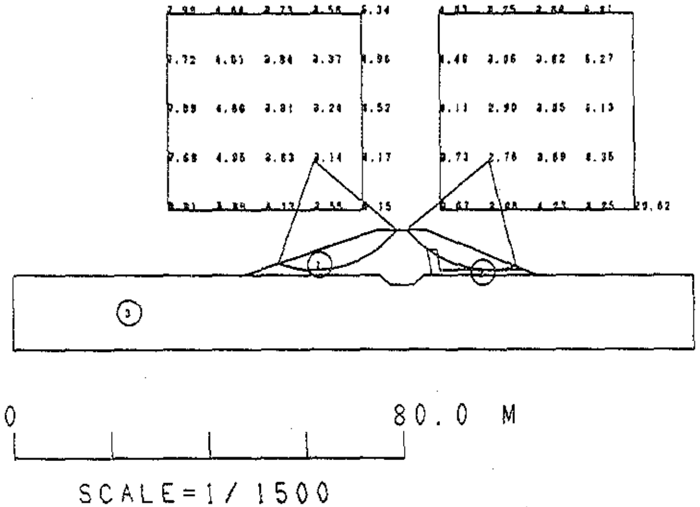
Fig. G-11



MATERIAL	COHESION (T/M^2)	FRICTION (DEG)	W(WET) (T/M^3)	W(SAT) (T/M^3)	W(SUB) (T/M^3)
1	3.00	26.00	1.73	1.86	0.86
2	0.00	33.00	1.72	2.02	1.02
3	5.00	32.00	1.70	1.80	0.80
ACCELERATION OF EARTHQUAKE				0.075	

MINIMUM SAFETY FACTOR (SEISMIC)	
UP STREAM SIDE	DOWN STREAM SIDE
3.143	2.755

Stability Analysis of Heightening of Krenceng Dam
(Middle Section, Reservoir Empty, Seismic Condition)

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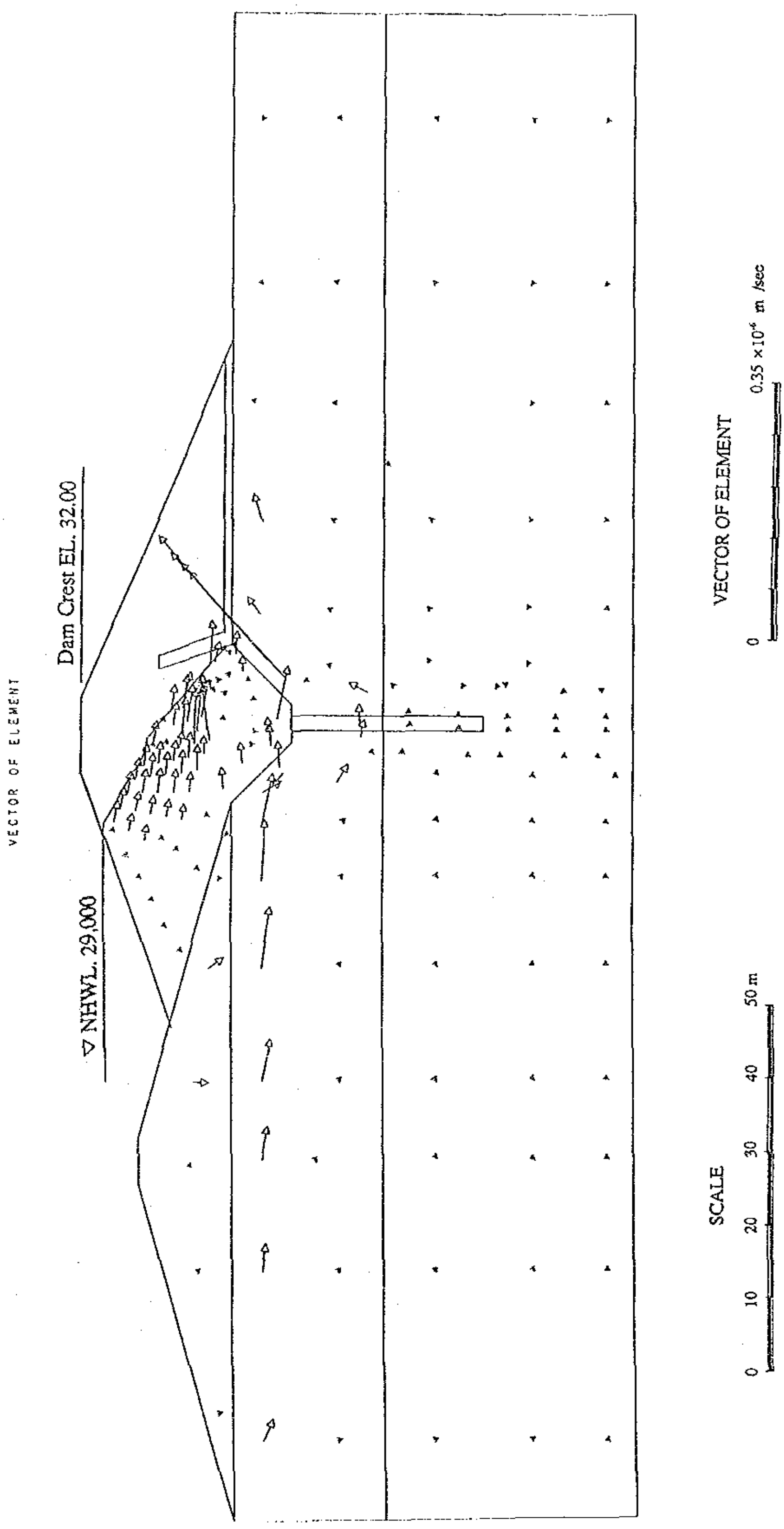
Fig. G-12



Seepage Analysis of Heightening of Krenceng Dam
(Maximum Section)

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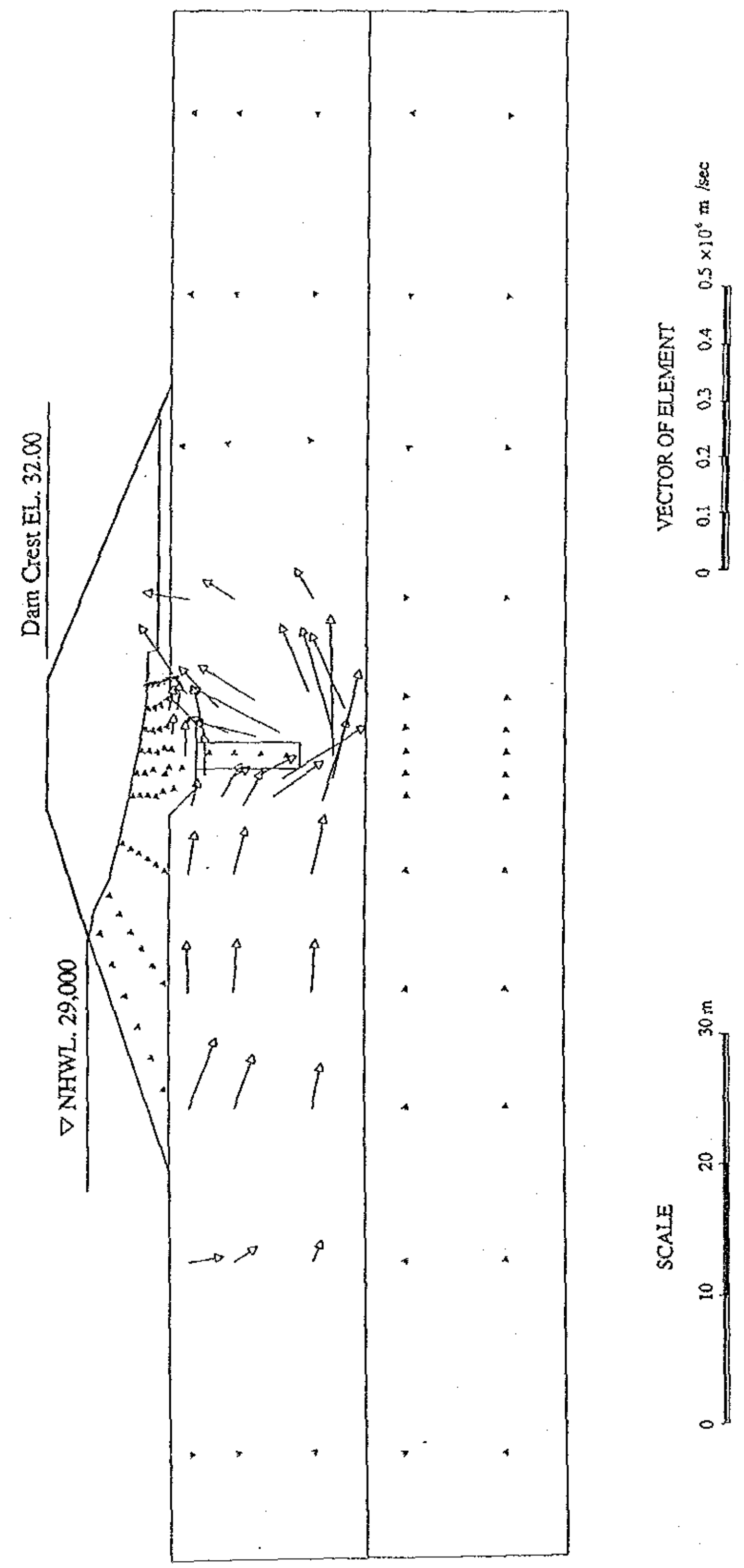
Fig. G-13



Seepage Analysis of Heightening of Krenceng Dam
(Middle Section)

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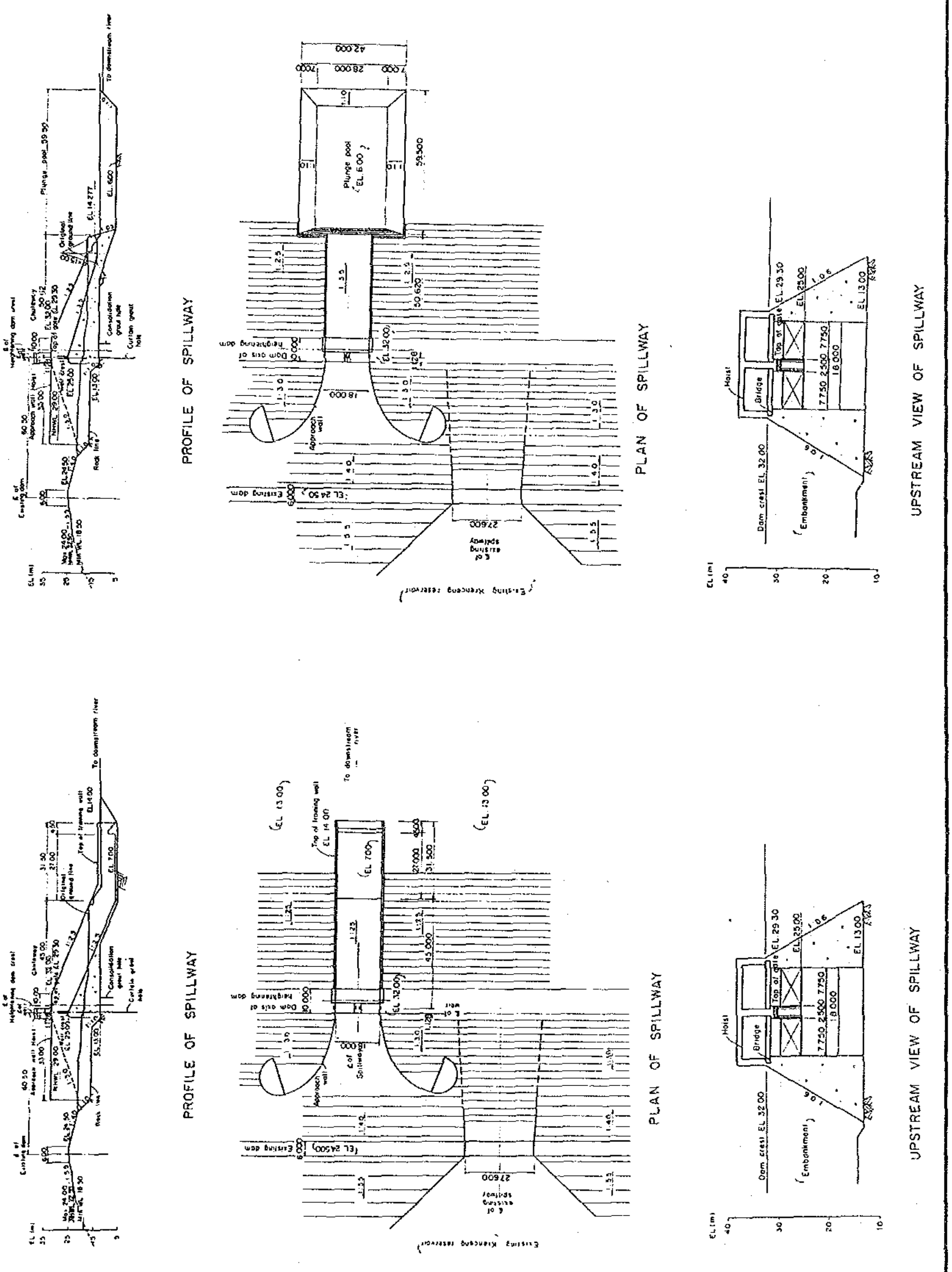

Alternative Spillway for Heightening of Krenceng Dam

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