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THE REPUBLIC OF INDONESIA

THE FEASIBILITY STUDY ON THE DISASTER PREVENTION PROJECT IN THE SOUTHEASTERN SLOPE OF MT. GALUNGGUNG

FINAL REPORT
SUPPORTING III
DESIGN & CONSTRUCTION SCHEDULE
SUPPORTING IV
COST ESTIMATE
SUPPORTING V
PROJECT EVALUATION

DECEMBER, 1988

JAPAN INTERNATIONAL COOPERATION AGENCY

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S U P P O R T I N G R E P O R T (III)

PRELIMINARY DESIGN

DECEMBER 1988

JAPAN INTERNATIONAL COOPERATION AGENCY

Supporting Report III (Preliminary Design)

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1. Design Criteria

The disaster prevention facilities for the Mt. Galunggung disaster prevention project are consisted of improvement dike, check dams, consolidation dams, revetment works and drainage tunnel. The design criteria in order to practice the outline of the design for these facilities is described below. The design criteria was selected based on the examples constructed in Japan and Indonesia.

(1) Improvement Dike, Revetment Works, Check Dams and Consolidation Dams

The design criteria of the facilities as dike, revetment works, check dams and consolidation dams was on the basis of "Technical Standard of River and Sabo by Ministry of Construction in Japan Tentative" compiled by River Bureau of Ministry of Construction in Japan.

(2) Drainage Tunnel

The design Criteria of the facility of the drainage tunnel was on the basis of "Standard Specifications of Mountains and Tunnels Tentative."

(3) Hydraulic Design

The undermentioned Manning Formula was applied to the hydraulic calculations of the structures as open channels and tunnels.

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

where,

V ; Velocity (m/s)

I ; Gradient

R ; Hydraulic Radius (m)

2. Outline of Disaster Prevention Project

Mt. Galunggung Disaster Prevention is to be put into practice with the following as its objectives: Prevention of disaster arising from removement of Sediment accumulated on Mt. Galunggung's southern slope basin and that arising from reservoir water caused by a rip in the crater lake's wall, sediment runoff, overflow of hot water resulting from eruption, and so on. In implementing this Project, four project units have been designed considering local characteristics, the project's integrity, nature and so on.

To proceed with the preliminary designs for the disaster prevention facilities of the project units, specifications had to be decided.

The outline of the project unit devised in carrying it out is as follow:

(1) Maintenance of Sandpocket (Unit 1)

In order to restore and maintain the function of the sandpockets in the S. Cikunir and S. Ciloseh river basins, dike reinforcement works, sediment management works, the excavation and hauling of deposited sediment, the raising dikes, and the construction of check dams and consolidation dams were carried out.

The 15,432 m dikes of the sandpocket are to be reinforced and raised.

The design management sediment volume for the sandpocket in S. Cikunir for the 10 years work period is $6,141\times10^3$ m^3 and that for the sandpocket in S. Ciloseh is 394×10^3 m^3 making a total of $6,535\times10^3$ m^3.

The following 5 plan were established as alternative plans for the sediment management works.

Alternative A: The amount of design management sediment volume is excavated and hauled to the aggregated plant to be produced into aggregate.

Alternative B: Of the DMSV, 450,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly (unit formly) by the construction machinery. With regard to the raising of the riverbed due to even spreading, existing dikes are to be raised.

Alternative C: Of the DMSV, 319,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly by the construction machinery. With regard to the raising of the riverbed due to even spreading, existing dikes are to be raised.

Alternative D: Of the MDSV, 128,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly by the construction machinery. With regard to the raising of riverbed due to even spreading, existing dikes are to be raised.

Alternative E: The mount of DMSV is spread evenly by the construction machinery. With regard to the raising of riverbed due to this, existing dikes are to be raised.

The sediment that was excavated and hauled from the sandpocket was produced into aggregate the newly constructed aggregate plant near Ciponyo I Dalam. From the standpoint of disaster prevention, the raising of dikes will be carried out on the sandpocket Ciponyo I Dalam and sandpocket Cimampang.

Five check dams are planned for the upper basin of the sandpockets of S. Ciloseh and S. Cimampang. Furthermore, it was judged that in the S. Ciloseh area, the check dam on S. Ciloseh was less urgent than the check dam on S. Cimampang. This was decided based on a study of the riverbed materials deposited and unstable sediment. The S. Ciloseh check dam was therefor removed from the disaster prevention project.

Two check dams are to be constructed in the upstream basin of S. Cimampang. 4 (Four) check dams and 2 (two) consolidation dams are to be constructed in the upper basins of S. Cikunir and S. Cibanjaran.

(2) Stabilization of River course in the Sandpocket Ciponyo II (Unit 2)

With the construction of 4 (four) consolidation dams on S. Cikunir and S. Cibanjaran in sandpocket Ciponyo II, the stabilization of river course will be worked out. Also, by means of constructing a 1.7 Km revetment at the confluence of S. Cikunir and S. Cibanjaran, the prevention works for bank erosion will be carried out, and the flood will be made to flow quickly.

(3) Check Dams on the South Slope of Mt. Galunggung

With a total of 20 check dams to be constructed in the 3 basins of S. Cisaruni, S. Cikupang and S. Cimerah located on the south slope of Mt. Galunggung, the water intake facilities of Cikunten I irrigation channels and the town of Singaparna located in the lower basin, are protected.

(4) Drainage Tunnel of the Crater Lake

With a drainage tunnel (Total length 665 m) for the water stored in the crater lake, the water will be drained into S. Cibanjaran and maintain the design water level of EL 1,082.5 m (water storage volume 750 $\times 10^3$ m^3). The aims of this work is the Prevention of Secondary disaster arising from the action of lake water caused by Mt. Galunggung's eruption in time to come and that arising from the flood flow caused by a break of external wall of the crater lake resulting from the rising of the lake water.

The general locations of each unit project are shown in Fig. - 2.1.

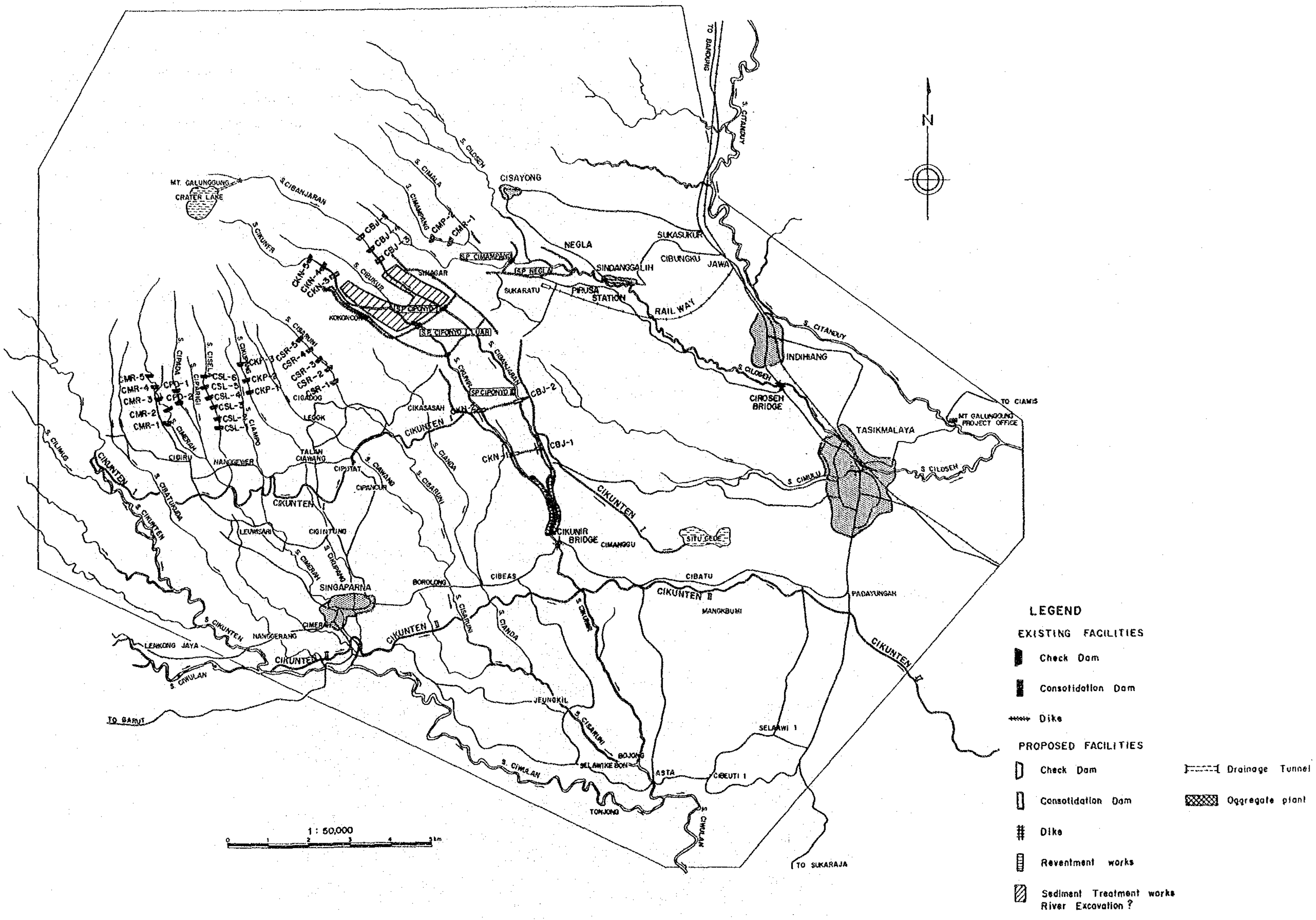


Fig. - 2.1 Location Map of Proposed Facilities of The Disaster Prevention Project

3. Maintenance of Sandpocket (Unit 1)

3.1 Outline

According to the field reconnaisance, the sandpockets of S. Ciloseh basin and S. Cikunir basin are almost full due to the inflow of debris and in a dangerous state from the point of view of disaster prevention, with the dikes insufficiently high and too corroded. Above all, it is estimated that in the next ten years there will be 4,700 m^3 of sediment that will flow into S. Cikunir area, hence the urgent need for countermeasures to be taken. The sandpocket is maintained and its function revived by the following works:

a) Dike improvement works
b) Sediment management works (Excavation and hauling of Sediment or Dike raising)
c) Construction of check dams works

Taking into account the above, the following five alternatives for the sediment management from the sandpocket were established.

After each economic cost is calculated, the final plan is determined by comparing the profit.

Alternative A: The amount of design management sediment volume (DMSV) is excavated and hauled to the aggregate plant to be produced into aggregate.

Alternative B: Of the DMSV, 450,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly (uniformly) by the construction machinery. With regard to the raising of the riverbed due to even spreading,existing dike is raised.

Alternative C: Of the DMSV, 319,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly by the construction machinery. With regard to the raising of the riverbed due to even spreading, existing dike is raised.

Alternative D: Of the DMSV, 128,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly by the construction machinery. With regard to the raising of riverbed due to even speading, existing dike is raised.

Alternative E: The amount of DMSV is spread evenly by the construction machinery. With regard to the raising of riverbed due to this, existing dike is raised.

3.2 Dike Improvement Works

According to the selection of the alternative, the raising of dike will be part of the reinforcement works of dike when the dike in Ciponyo I Dalam is raised. Therefore, as for the dike in Ciponyo I Dalam, the design of reinforcement of dike under the plan to make all the sediment volume aggregate is performed.

3.2.1 Dike Height

The dike height is to ensure more than the height added the free board of 1.0 m to the water level of flowing down of a 50 year probable flood discharge. The gradient of the riverbed is about 1/30 in the upper reach of Ciponyo I Dalam in particular, and so the breaking out of the debris flow is forecasted at flood. For that reason, the dike height should be ensured the height added the free board of 1.0 m to the impact height by debris flow.

Moreover, at the dam site in the middle reach of the sandpocket, where the consolidation dam is planned, it should be more than the height added the free board to the overflow depth of dam.

Generally speaking, lahar is going to flow down in a straight path. The lahar front includes big boulders and gravels and its depth is larger than that at upstream. When the lahar is checked by such a barrier as dike, the depth of lahar front abruptly increase due to the change of dynamic energy to height energy and the lahar overtops the dike if the height of dike is less than that of lahar. The depth of lahar when it is checked by dike can be called overtopping height. The overtopping height varies depending on various factor of lahar such as discharge, velocity, width, density, dynamic friction angle, flowing direction to dike etc.

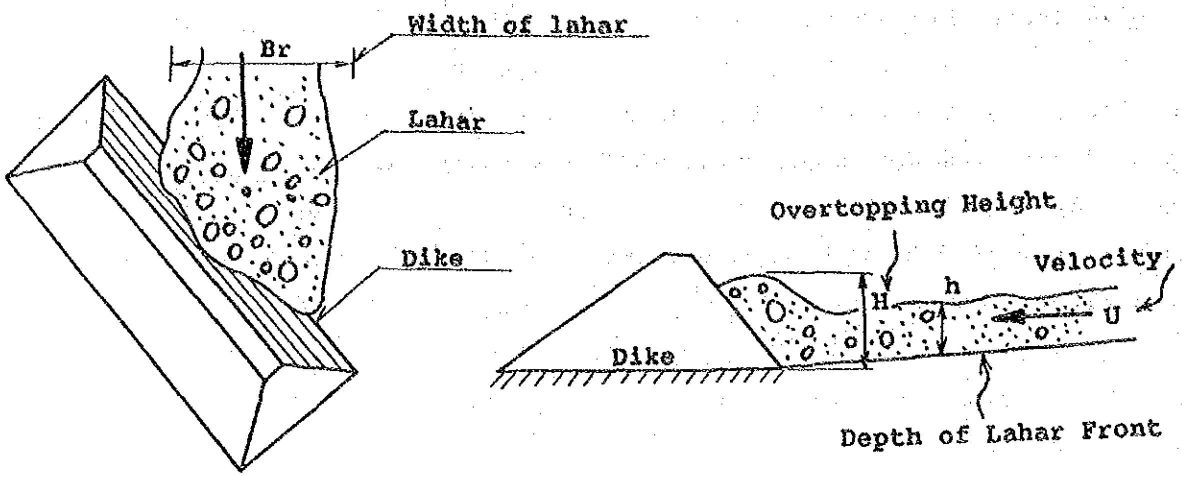


Fig. - 3.1 Scheme of Lahar Overtopping

The formulae for calculation of Overtopping height of lahar are shown below.

$$Cd = \frac{\tan\theta}{(\sigma - \rho)(\tan\alpha - \tan\theta)} \quad \cdots\cdots (3.1)$$

$$\frac{U}{\sqrt{dg}} \Big/ \left(\frac{q_o^2}{gd^3}\right)^{0.3} = \frac{0.693 \sin^{0.2}\theta\{(\frac{C_*}{Cd})^{\frac{1}{3}}-1\}^{0.4} \bullet \{Cd+(1-Cd)\frac{\rho}{\sigma}\}^{0.2}}{(ai \bullet \sin\alpha)^{0.2}(1-Cd)^{0.6}} \quad \cdots\cdots (3.2)$$

$$q = U.h \quad \cdots\cdots (3.4)$$

$$\left(\frac{h}{d}\right) \Big/ \left(\frac{q_o^2}{gd^3}\right)^{0.2} = \frac{1.443\,(ai \sin\alpha)^{0.2}}{\sin^{0.2}\theta\{(\frac{C_*}{Cd})^{1/3}-1\}^{0.4} \bullet \{Cd+(1+Cd)\frac{\rho}{\sigma}\}^{0.2} \bullet (1-Cd)^{0.4}} \quad \cdots\cdots (3.3)$$

$$H = \frac{1}{2}\, h\,(\sqrt{1 + 8Fr \sin\beta} - 1) \quad \cdots\cdots (3.5)$$

$$Er = U/\sqrt{gh} \quad \cdots\cdots (3.6)$$

Where:

Cd : Ratio of grain volume to unit volume of lahar

ρ : Desity of lahar matrix (t/m^3)

σ : Density of grain in lahar (t/m^3)

θ : Riverbed gradient (0)

α : Dynamic friction angle (0)

U : Velocity of lahar (m/sec)

g : Acceleration of gravity (9.8 $m/sec.^2$)

d : Mean diameter of grain in lahar (m)

q_0 : Unit water discharge ($m^3/sec/m$)

ai : Constant (0.042)

C_* : Ratio of grain volume to unit volume of lahar deposition

h : Height of lahar front (m)

q : Unit discharge of lahar ($m^3/sec/m$)

H : Height of overtopping when lahar is checked by dike, dam, bank etc. (m)

F_r : Froude Number

β : Crossing angle between alignment of structure and direction of lahar (0)

The following assumptions are applied to the calculation of overtopping height of lahar.

(i) Water discharge volume is to be the discharge with 50 years return period which was estimated in the supporting report II. (refer to Table - 2.15)

(ii) Width of lahar is assumed by the following formula:
The width in wide area such as alluvial fan, sandpocket etc. is calculated by the following Regime Theory:

$$Br = C \cdot Q^{1/2} \quad \text{.............................} \quad (3.7)$$

Where:

Br : Width of lahar (m)

Q_0 : Water Discharge (m^3/sec)

C : Constant (3 - 7)

(iii) Crossing angle between alignment of dike and direction of lahar is 90° taking into consideration the worst case.

(1) Velocity of Lahar

The result of calculation for velocity of lahar are shown in Table - 3.1.

Table - 3.1 Calculation Result of Velocity of Lahar

unit design peak discharge	cd	A	B	C	D	E	Velocity of Lahar
2 $m^3/s/m$	0.064	4.55	0.373	0.880	0.867	0.444	2.92 m/s
4 $m^3/s/m$	0.064	6.90	0.373	0.880	0.867	0.444	4.91 m/s
6 $m^3/s/m$	0.064	8.80	0.373	0.880	0.867	0.444	6.26 m/s

$A = \sqrt{gd}(\frac{q_0^2}{gd^3})^{0.3}$ $B = 0.693\sin\theta^{0.2}$ $C = \{(\frac{Cx}{Cd})^{1/3}-1\}^{0.4}$

$E = (ai \sin\alpha)^{0.2} * (1-Cd)^{0.6}$ $Cd = \frac{\rho \tan\theta}{(\sigma - \rho)(\tan\alpha - \tan\theta)}$

$D = \{cd+(1-cd)\frac{\rho}{\sigma}\}^{0.2}$ Velocity of Lahar $= \frac{B \times C \times D \times A}{E}$

(2) Height of Lahar Front

The result of calculation for height of lahar front

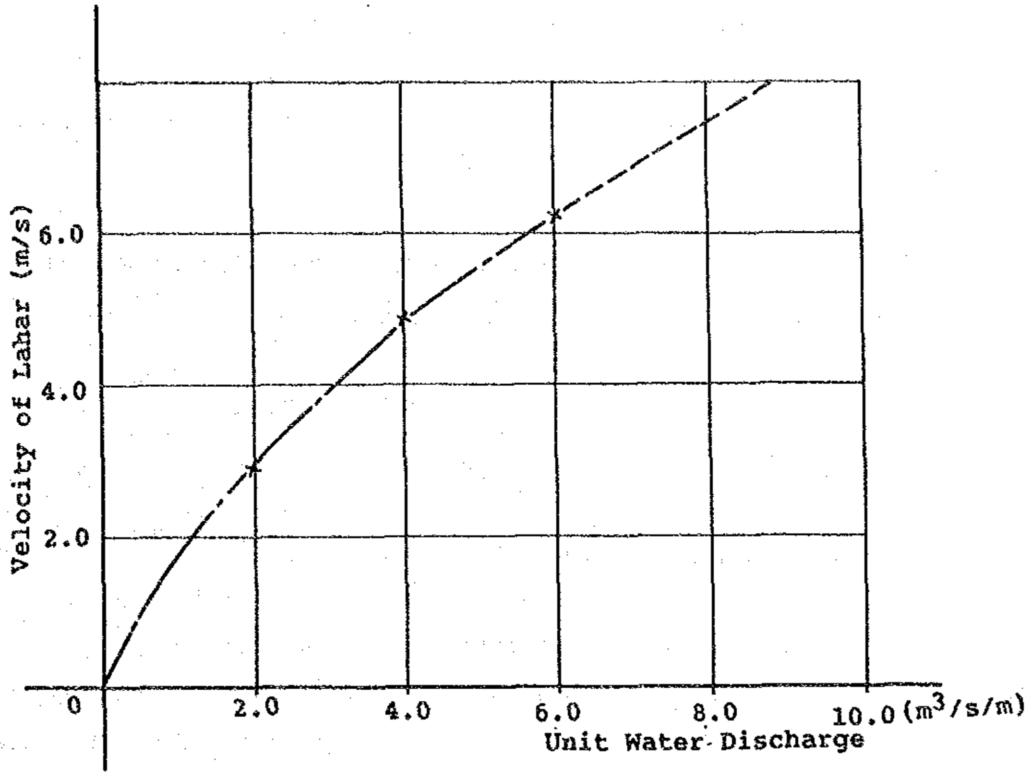
Table - 3.2 Calculation Result of Height of Lahar Front

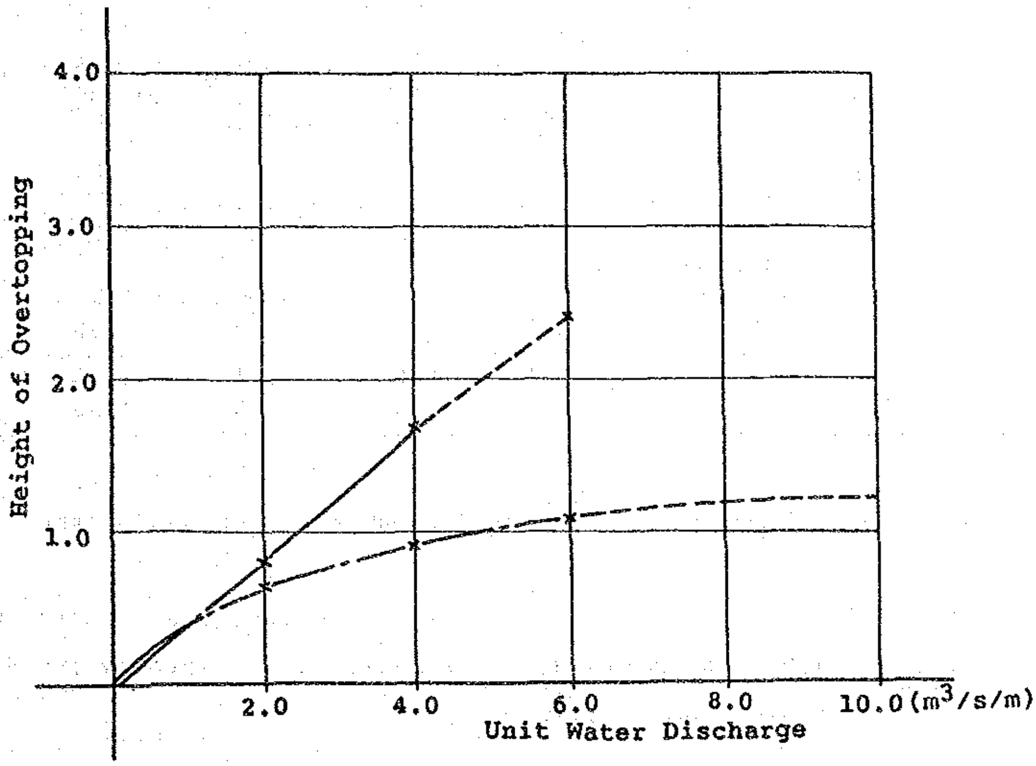
Unit design peak discharge	cd	F	G	H	I	Height of Lahar Front
2 $m^3/s/m$	0.107	0.439	0.618	0.474	0.829	0.69 (m)
4 $m^3/s/m$	0.107	0.579	0.618	0.474	0.829	0.91 (m)
6 $m^3/s/m$	0.107	0.681	0.618	0.474	0.829	1.07 (m)

$F = (\frac{q_0^2}{gd^3})^{0.2} d$ $G = 1.443\ (ai \sin\alpha)^{0.2}$

$H = \sin^{0.2}\theta\{(\frac{C*}{Cd})^{1/3}-1\}^{0.4}$ Height of Lahar Front $= \frac{F \times G}{H \times I}$

$I = \{Cd+(1-Cd)\frac{P}{\alpha}\}^{0.2} x (1-Cd)^{0.4}$


Velocity of Lahar (m/s)
6.0
4.0
2.0
0
2.0
4.0
6.0
8.0
10.0 (m3/s/m)
Unit Water Discharge


Height of Overtopping
4.0
3.0
2.0
1.0
2.0
4.0
6.0
8.0
10.0 (m3/s/m)
Unit Water Discharge

(3) Height of Overtopping

The result of calculation for the Overtopping height are shown in Table - 3.3.

Table - 3.3 Calculation Result of Overtopping Height

	h	V	Fr	H
2 m^3/s	0.69	2.92	1.12	0.80
4 m^3/s	0.91	4.91	1.64	1.70
6 m^3/s	1.07	6.26	1.93	2.43

(4) Required height of dike when lahar is checked by dike.

Adding 1.0 m of free board to the overtopping height calculated in the Table 3.4.

Table - 3.4 Required Height of Dike

		Peak Discharge (m^3/s)	Width of Lahar (m)	Unit Discharge ($m^3/s/m$)	Over-topping Height (m)	Required Height (m)
Cikunir Cibanjaran Basin	Ciponyo I Dalam up stream	177	40	4.4	2.00	3.00
	Ciponyo I Dalam down stream	177	70	2.5	1.20	2.50
	Ciponyo I Luar	177	70	2.5	1.20	2.50
	Ciponyo II	195	100	2.0	0.80	1.80
Cilosen Basin	Negla	558	165	3.4	1.15	2.50
	Cimampang	277	85	3.3	1.15	2.50

(5) Overflow Depth of Consolidation Dams on the each sandpocket

The result of Calculation of Overflow depth on Consolidation Dams are shown in Table - 3.5. The formula for calculation of overflow depth of Consolidation Dams are shown below.

$$Q = \frac{1}{12} \times \alpha \times \sqrt{2 \cdot g} \times (Bx5+h) \times h^{1.5}$$

α : 0.66

Table - 3.5 Required Height of Wing Wall

		Peak Discharge (m^3/s)	Width of Spillway (m)	Over-topping Depth (m)	Height of Wingwall (m)
Cikunir Cibanjaran Basin	Ciponyo I Dalam	177	30.0	2.1	3.1
	Ciponyo I Luar	177	30.0	2.1	3.1
	Ciponyo II	195	40.0	1.7	2.7
Cilosen Basin	Negla	558	40.0	4.65	5.65
	Cimampang	277	40.0	3.10	4.10

(6) Proposed Height of Dike in Sandpocket

The proposed height of dike in the sandpocket from the proposed height of riverbed is determined as follows in consideration of the overtopping height and the overflow depth of the consolidation dam.

a) As S. Ciloseh Basin is the flood flow which does not include the debris, the dike will be made high near by the consolidation dam and it will be 3.0 m added the impact height to the free board in the middle reach.

b) As the interval of the consolidation dam is narrow in S. Cikunir Basin, it will be either height of the overflow depth or the impact height adding the free board to it.

Table - 3.6 Required Height of Dike

		Design peak (m^3/s) Flood Discharge	Riverbed Gradient	Height of Dike (m)
Ciloseh Basin	Negla	558	1/35	3.0
	Cimampang	277	1/25	3.0
Cikunir Basin	Ciponyo I Dalam	177	1/25	3.0
Cibanjaran Basin	Ciponyo I Luar	177	1/55	3.0
	Ciponyo II	195	1/70	2.5

3.2.2 Slope of Dike

(1) Formulae for Stability Analysis

In case that banking material is composed of cohesive soil such as silty sand, calyey sand etc., failure of dike occurs along the slip circle crossing dike or dike and foundation.

However, in case that banking material is composed of cohesionless material such as sand, gravel etc., failure of dike occurs along the sliding line parallel with slope surface. (refer to Fig. - 3.4).

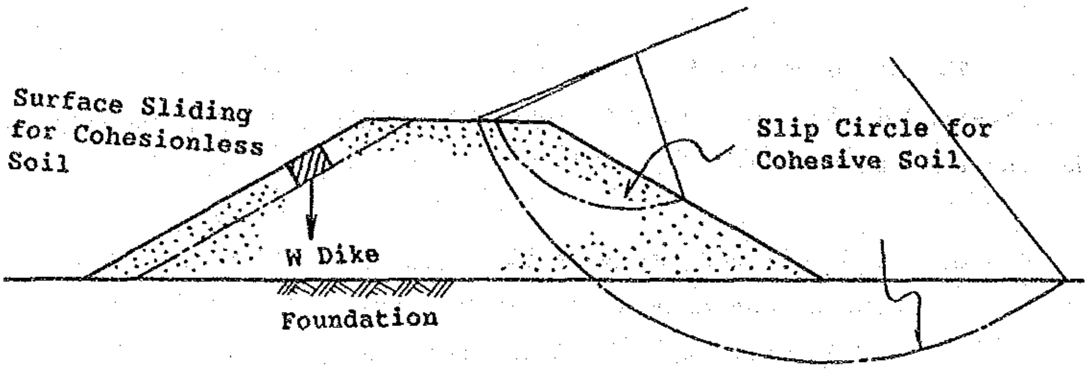


Fig. - 3.4 Pattern of Sliding

The banking material used in the Project is mainly composed of cohesionless soil such as sand, gravel and cobble stone which are classified under SW and SM in the ASTM soil classification, therefore, sliding failure will occur along slope surface.

The formulae for stability along the stability line parallel with slope surface is shown as follows:

$$F = \frac{Pr}{Ps} \quad \text{.......} \quad (3.7)$$

$$Pr = w \cdot d \cdot a \cdot \cos\theta \cdot \tan\theta \quad \text{.......} \quad (3.9)$$

$$Ps = w \cdot d \cdot a \cdot \sin\theta \quad \text{.......} \quad (3.10)$$

$$\theta = \tan^{-1} \left(\frac{\tan\theta}{F}\right) \quad \text{.......} \quad (3.11)$$

Where:

F	:	Factor of Safety
Pr	:	Resisting Force (t)
Ps	:	Sliding Force (t)
w	:	Unit Weight of Material (t/m^3)
d	:	Depth of Sliding Surface (m)
a	:	Unit Area of Soil Block (m^2)
θ	:	Degree of Slope (0)
ϕ	:	Internal Friction Angle (0)

The formula considering earthquake is obtained by adding the coefficient of earthquake (k) into the above formulae as follows:

$$F = \frac{1 - K \tan \theta}{\tan \theta + K} \tan\phi \quad \text{.............................} \quad (3.12)$$

(2) Design Criteria for Slope

(i) Required Factor of Safety

The required factor of safety for sliding is decided based on the KLTUTL Jilid-I* as shown in the Table - 3.7.

Table - 3.7 Factor of Safety for Sliding

In the normal case	In the case of Earthquake
1.25	1.00

The factor of safety in the case of earthquake is reduced from that of KLTUTL Jilid-I by 0.10 because of the following reasons.

* Kestabilan Lereng dengan Tinjavan Utama pada Tanah Lempongon, Jilid I, Pebruar: 1981, Directorat Penyelidikan Masalah Air, Directorat Jenderal Pengairan, Department Pekerjaan Umum, Republic Indonesia.

a) The proposed dike aims at preventing lahar from overtopping and is very different from the dike accommodating river water flowing down. The proposed dike has no usual contact with river water. Therefore, even if a sliding failure will occur in the case of earthquake, the dike can be repaired easily at any time and will not cause any serious damages.

b) The existing dike for sabo work seems not to have such a big factor of safety in Indonesia.

c) Japanese Technical Standard for River and Sabo Works adopts 1.0 in case of earthquake because of the first reason above.

(ii) Coefficient of Earthquake

Coefficient of earthquake is given from "Peta Zona Seismic untuk Perencanaan Bangunan Air Tahan Gempa" as shown in Fig. - 3.5.

Return period	50 years
Zone	IV
Soil Condition	Alluvium deposit
Coefficient	K = 0.12

(iii) Design Criteria of Embankment Material

The design criteria of embankment material are given from "Semeru project soil investigation" as shown in Table - 3.8

Table - 3.8 Design Criteria of Embankment Material

Material	Unit Weight	Internal Friction Angle	(t/m^2)
Soil	$1.80\ t/m^3$	34 degree	0
Gabion	1.49	40 degree	0

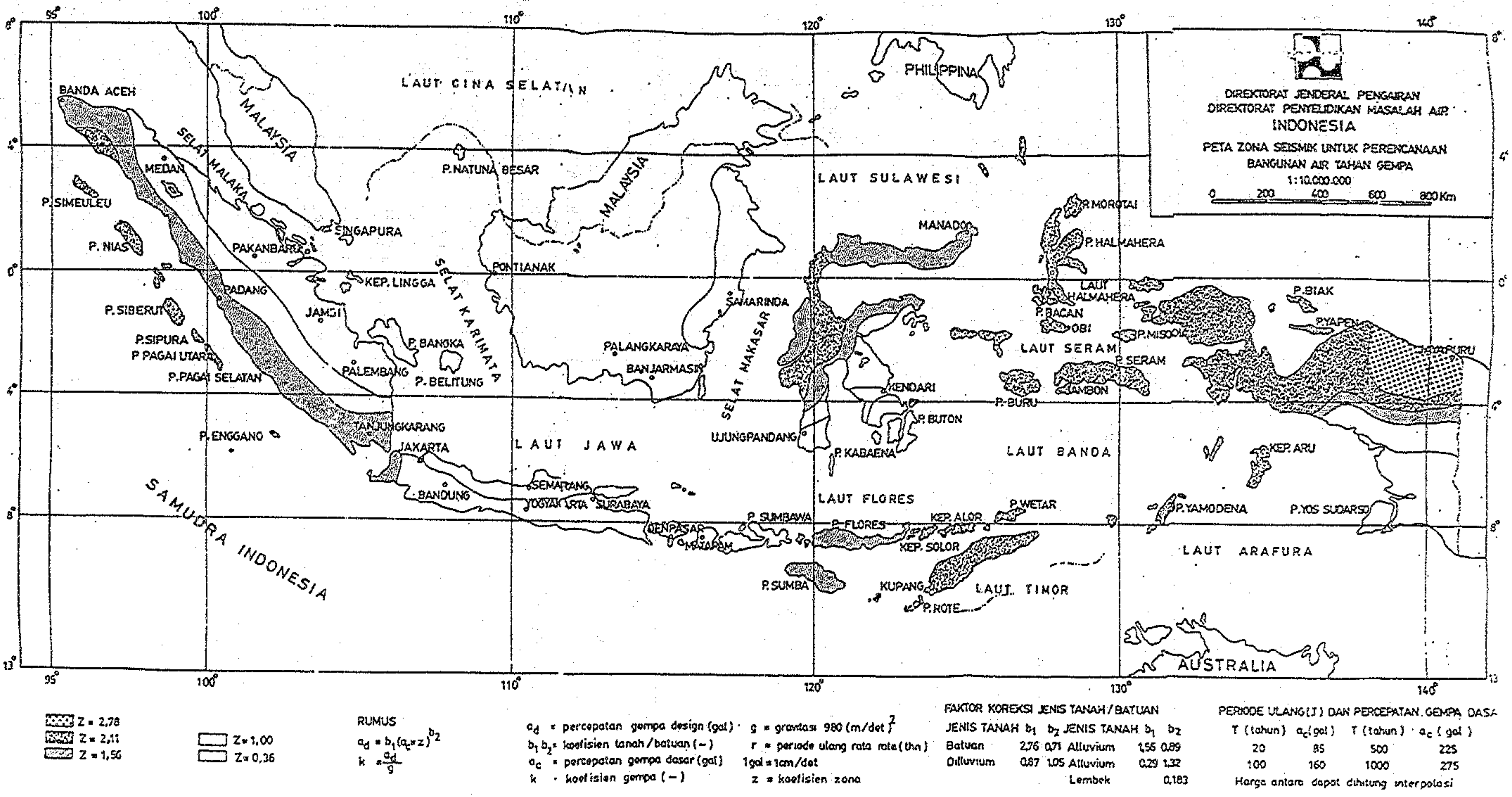


Fig. - 3.5 Coefficient of Earthquake in Indonesia

(3) Required Slope

The result of stability analysis is shown in Fig. - 3.6. The required slope is shown in Table - 3.9.

Table - 3.9 Required Slope of Dike

	Embankment Material at Surface of Dike	Required Slope	
		Normal	Earthquake
Landside slope	Soil	1:1.818	1:1.98
River side slope	Gabion	1:1.5	1:1.5

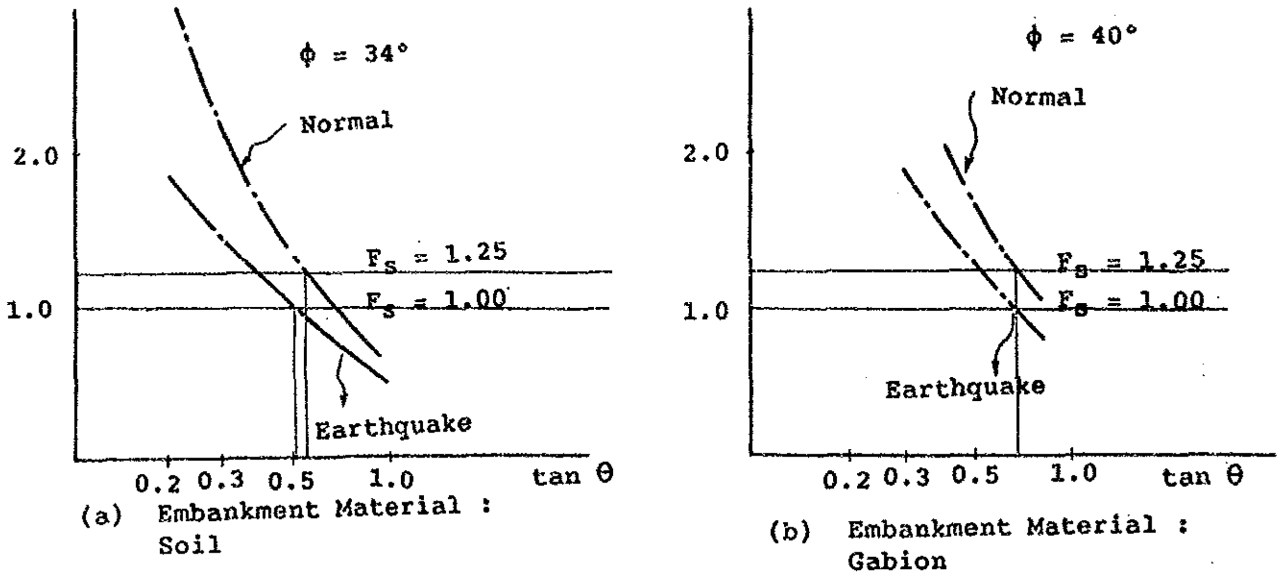


Fig. - 3.6 Results of Stability Analysis

The slope of proposed dike is decided from the Table - 3.11 as follows:

Land side (Soil)	1:2.0
River side (Gagion)	1:1.5

3.2.3 Gabion Work

(1) Revetment

The gabion revetment is to be installed from the base to the top of dike at the river side. The base of dike at the river side is also to be protected by gabion mattress of length 3 m. The thickness of the gabion work is to be 30 cm and the dimension of one unit of the gabion is to be 1 m x 3 m x 0.3 m.

(2) Groyne

According to the analysis of existing dike failure, scouring is one of the causes of failure as well as overtopping. Therefore groyne is to be installed at the riverside of dike to protect the dike from being scoured.

The scouring of dike mainly occurs where the normal flow concentrates, in other words, at dikes which are located at lower ground height and are exposed to river water.

The design criteria of groyne is given by the technical standard of the Ministry of Construction, Japan as follows:

$$L \leq 0.1 \cdot B$$
$$H \leq \text{Low water level} + (0.5\ m - 1.0\ m)$$
$$D = (1.5 - 2.0) \cdot L$$

Where:

L = Length of groyne (m)
B = River width (m)
H = Height of groyne from the river bed (m)
D = Interval of adjoining groyne (m)

The river width on the alluvial fan can be estimated by the Regime Theory as follows:

Discharge width return period of 50 years : $Q_d = 177\ m^3/sec$

River width on the alluvial fan : $B = 5\sqrt{Q_s} = 70\ m$

Length of groyne : $L \leq 0.1\ B = 7\ m \rightarrow 7\ m$

Interval of adjoining groyne : $D = (1.5 - 2.0)\ L = 10.5 - 14\ m$

Low water level : assumed 0.0 m

Height of groyne : $H = (0.5 - 1.0) + 0.0 = 0.5 - 1.0 \rightarrow 0.5\ m$

In spite of the above calculation, the interval of adjoining gryones is proposed to be 20 m for the time being and the interval will be changed into 25 m or 20 m step by step according to the actual river conditions.

Interval of Groyne	: $D = 20\ m$
Length of Groyne	: $L = 7\ m$
Height of Groyne	: $H = 0.5\ m$ (from the riverbed)
Total Height of Groyne	: $H_o = 1.0$ (from the base)

The typical section of groyne is decided based on the stability analysis for base and inner sliding similar to that of dike on the following additional assumptions.

(i) Porosity of gabion stone is 45%.

(ii) Static pressure of lahar is neglected because it acts on the upstream and downstream side of groyne; balances each other.

The material of groyne is to be gabion mattress because of the following reasons.

(i) As groyne is overtopped by lahar, the material can bear overtopping.

(ii) Groyne is to be flexible so as to follow local scouring.

The typical section of groyne is shown in Fig. - 3.7.

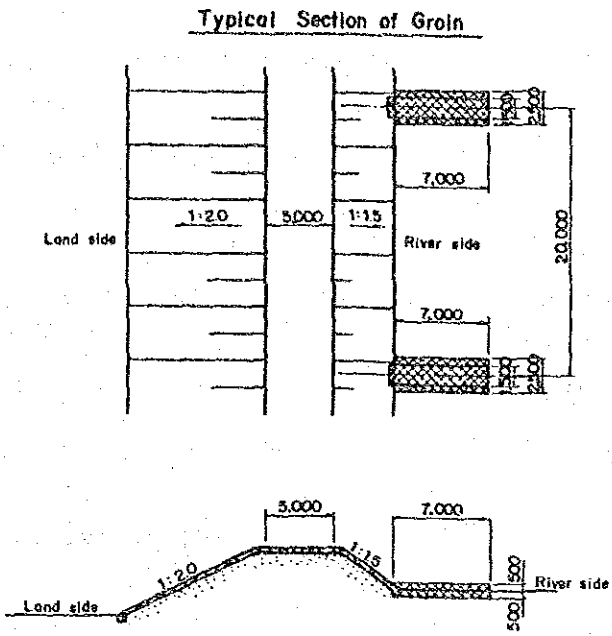


Fig. - 3.7 Typical Section of Groyne (Scale 1:200)

3.2.4 Typical Section of Proposed Dike

The description of improvement dike in each sandpocket is shown in Table - 3.10.

Table - 3.10 Description of Improvement Dike

Name of Dike	Length of Dike (m)	Length of Raising Dike (m)	Groyne (pc)	Dike Type	Remarks
Kokoncong	H=5.0, L=620 m	-	31pc	(D)	
Cimareme	H=3.0, L=718	-	36	(D)	Ciponyo I
Telelor	-	H=3.0, L=570 m	28	(D)	Dalam
Kubang buled	H=3.0, L=426	-	21	(B)	
Kubang martar	H=3.0, L=575	-	28	(B)	
Cibueuk	H=3.0, L=200	-	10	(B)	
Cinagar	-	-	20	(B)	
Sub Total	L=2,539	L=570	174		
Tanggul kanan	H=3.0, L=777	H=1.0, L=400	59	(D)	Ciponyo I
Tanggul kiri	H=3.0, L=413	H=1.0, L=500	41	(E)	Luar
Sub Total	L=1,190	L=900	100		
Kubang econg	H=2.5, L=1,250	-	62	(E)	
Rangapaku	H=2,5, L=1,600	H=1.0, L=500	105	(D)	
Cijuhung	H=2.5, L=250	-	12	(E)	Ciponyo II
Jinkikisik	H=2.5, L=850	-	42	(E)	
Batu beureum	-	-	22	(A)	
Worung sabeulah	H=2.5, L=1,982	-	100	(E)	
Sub Total	L=5,932	L=500	343		
Cimampang	H=3.0, L=750	-	37	(A)	Cimampang
Kiri	H=3.0, L=400	-	20	(A)	
Sub Total	L=1,150		57		
Negla kanan	H=3.0, L=2,073	-	103	(A)	Negla
Kiri	H=3.0, L=578	-	29	(A)	
Sub Total	L=2,651		132		

3.2.5 Construction Work Quantity of Dike Improvement

The construction work Quantity of the dike improvement in each sandpocket is as follows:

The sediment volume required for the dike improvement of dike in each sandpocket without Ciponyo I Dalam is 145,196 m^3. Besides, the aggregate volume for the wire cylinder and the groyne used for the protection of dike by debris flow is 20,900 m^3. These materials are used by excavating the deposited sediment near by the dike in Ciponyo I Dalam where it is in the most dangerous condition among 5 sandpockets.

Table - 3.11 Quantity of Improvement Dike Works

Name of Dike	Dike Improvement (m^3)	Rising Dike (m)	(V)	Gabion
Tanggul kanan	H=3.0, L=777	H=1.0, L=400	10,644	3,354
Tanggul kiri	H=3.0, L=413	H=1.0, L=500	10,841	1,177
Sub Total	L=1,190	L=900	21,485	4,531
Kubang buled	H=2.5, L=1,250	-	24,609	3,188
Rangapaku	H=2.5, L=1,600	H=1.0, L=500	18,469	5,355
Cijuhung	H=2.5, L=250	-	4,922	638
Jinkikisik	H=2.5, L=850	-	16,734	2,168
Batu beureum	-	-		
Worung sabeulah	H=2.5, L=1,982	-	39,021	5,054
Sub Total	L=5,932	L=500	103,755	16,403
Total (Cikunir Area)	L=7,122	L=1,400	125,240	20,934
Cimampang	H=3.0, L=750	-	3,938	
Kiri	H=3.0, L=400	-	2,100	
Sub Total	L=1,150		6,038	0
Negla kanan	H=3.0, L=2,073	-	10,883	
Kiri	H=3.0, L=578	-	3,035	
Sub Total	L=2,651		13,918	0
Total (Ciloseh Area)	L=3,801		19,956	0

3.3 Sediment Management Works

3.3.1 Alternative Plans

The sandpockets in the S. Cikunir and S. Ciloseh areas are almost full from the inflow of volcanic debris from the 1982 Mt. Galunggung eruption. There are still possible deposits in the upper reaches, meaning that sediment inflow into sandpockets will continue in the future.

As the result of the simulation of inflow sediment to the sandpocket, the inflow sediment mostly tends to deposit on the sandpocket in the upper reach. Therefore, the sediment management works will be carried out on the Ciponyo I Dalam. The design management sediment volume for the sandpocket in Cikunir for 10 years of the project executing is shown below.

a)	S. Cikunir Area-Ciponyo I Dalam	- 6,141 x10^3 m^3
	(Deposited Sediment Volume	- 4,741 x10^3 m^3)
	(Spare Capacity	- 1,400 x10^3 m^3)
b)	S. Ciloseh Area-Cimampang	- 394 x10^3 m^3
	(Spare Capacity Volume	- 394 x10^3 m^3)
	Total	6,535 x10^3 m^3 (Average 654 x10^3 m^3/year)

Based on the aggregate production and the raising of dike mentioned in Chapter III, five alternative plans on the sediment management in sandpocket have basically proposed as follows:

Alternative A : The amount of design management sediment volume (DMSV) is excavated and hauled to the aggregate plant to be produced into aggregate.

Alternative B : Of the DMSV, 450,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly (uniformly) by the construction machinery. With regard to the raising of the riverbed due to even spreading, existing dike is raised.

Alternative C : Of the DMSV, 319,000 m3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly by the construction machinery. With regard to the raising of the riverbed due to even spreading, existing dike is raised.

Alternative D : Of the DMSV, 128,000 m^3 is excavated and hauled to the aggregate plant to be produced into aggregate. The remaining volume is spread evenly by the construction machinery. With regard to the raising of riverbed due to even spreading, existing dike is raised.

Alternative E : The amount of DMSV is spread evenly by the construction machinery. With regard to the raising of riverbed due to this, existing dike is raised.

Each plan is referred to as "Alternative A", "Alternative B", "Alternative C", "Alternative D" and "Alternative E" hereafter.

The terms and the suppositions to proposed the alternative are as follows.

(1) The annual sediment management volume of 654 $x10^3$ m^3 is supposed to deposit on the Ciponyo I Dalam. The deposited sediment volume for 10 years to come of 4,741 $x10^3$ m^3 should deposit on Ciponyo I Dalam.

(2) The actual results of the annual aggregate volume which is transported from Pirusa Station to Jakarta as the aggregate is 120 $x10^3$ m^3 at a minimum and 423 $x10^3$ m^3 at a maximum. It is assumed that the transport capability will be able to augment to 600 $x10^3$ m^3 in future.

(3) In case of the raising of dike, S. Cikunir which runs at the center of the sandpocket is needed to shift. The catchment area of S. Cibukur is to be 2.3 km^2 and a 50 year probable flood discharge is to be 65 m^3/s.

(4) There are houses in S. Cikunir basin, so that, to obtain the alternative places for houses and the like is required.

(5) Because the height of existing dike in Ciponyo I Dalam is high, as it is 10 m at the highest, the berm will be provided in order to maintain and control the dike if it is higher than 5 m. The berm width should be 1.0 m.

3.3.2 Sediment Carrying Plan of Each Alternative

The modelling of the sediment carrying plan of each alternative is as shown in Fig. - 3.8.

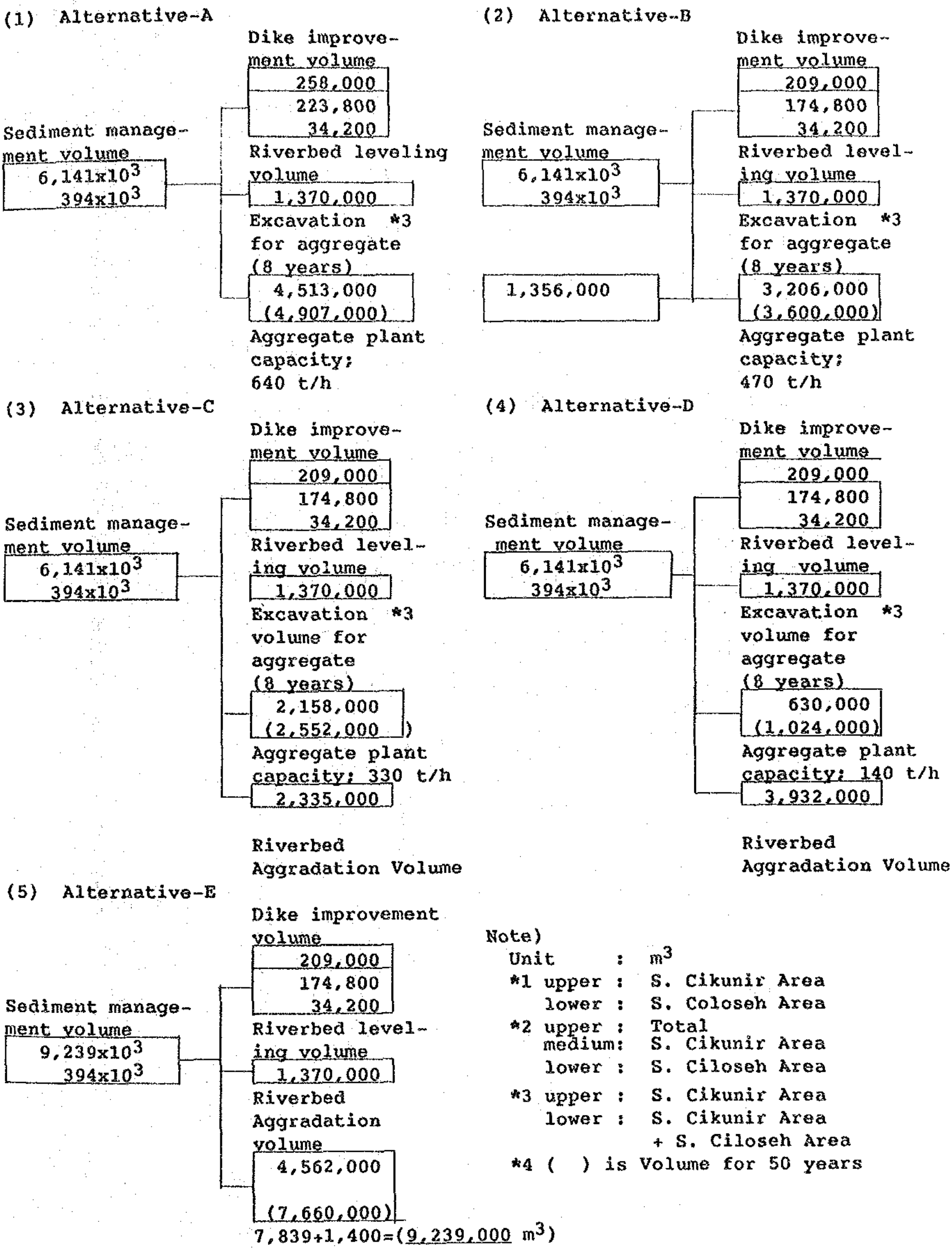
The considered points in the sediment carrying plan of each alternative are as follows.

(1) The deposited sediment being over the design riverbed height which was planned in consideration of the present gradient of riverbed is moved to the dead water zone (refer to Fig. - 3.8, Table - 3.12). This height is supposed to be the design riverbed height of S. Cikunir, S. Cibanjaran and S. Cibukur.

(2) In the alternative with raising dike, all the deposited sediment will be moved by the construction machineries in order to make the deposit shape of the inflow sediment in the sandpocket coincided with the design riverbed gradient (refer to Fig. - 3.9, Table - 3.13). So, the sediment will be deposited with the equal thickness from the design riverbed height in the sandpocket.

(3) In Alternative E, the sediment regulation volume that is able to hoard all the deposited sediment volume of 7,839 $x10^3$ m^3 for 50 years should be established in the sandpocket only by raising dike.

Meanwhile, though Alternative A is considered to be the perpetual countermeasure of the sediment management, Alternative D is not, and so the absolute volume is supposed to be 7,839 $x10^3$ m^3 of the deposited inflow sediment volume for 50 years. As the deposited inflow sediment volume is 30 $x10^3$ m^3 in the 51st year, it is inferred that the volume can be ignored by the scale of the sandpocket.



Note)

Unit	:	m^3
*1 upper	:	S. Cikunir Area
lower	:	S. Coloseh Area
*2 upper	:	Total
medium	:	S. Cikunir Area
lower	:	S. Ciloseh Area
*3 upper	:	S. Cikunir Area
lower	:	S. Cikunir Area + S. Ciloseh Area

*4 ( ) is Volume for 50 years

Fig. - 3.8 Conveyance Plan by Alternative of Sediment Management Works in Sandpocket

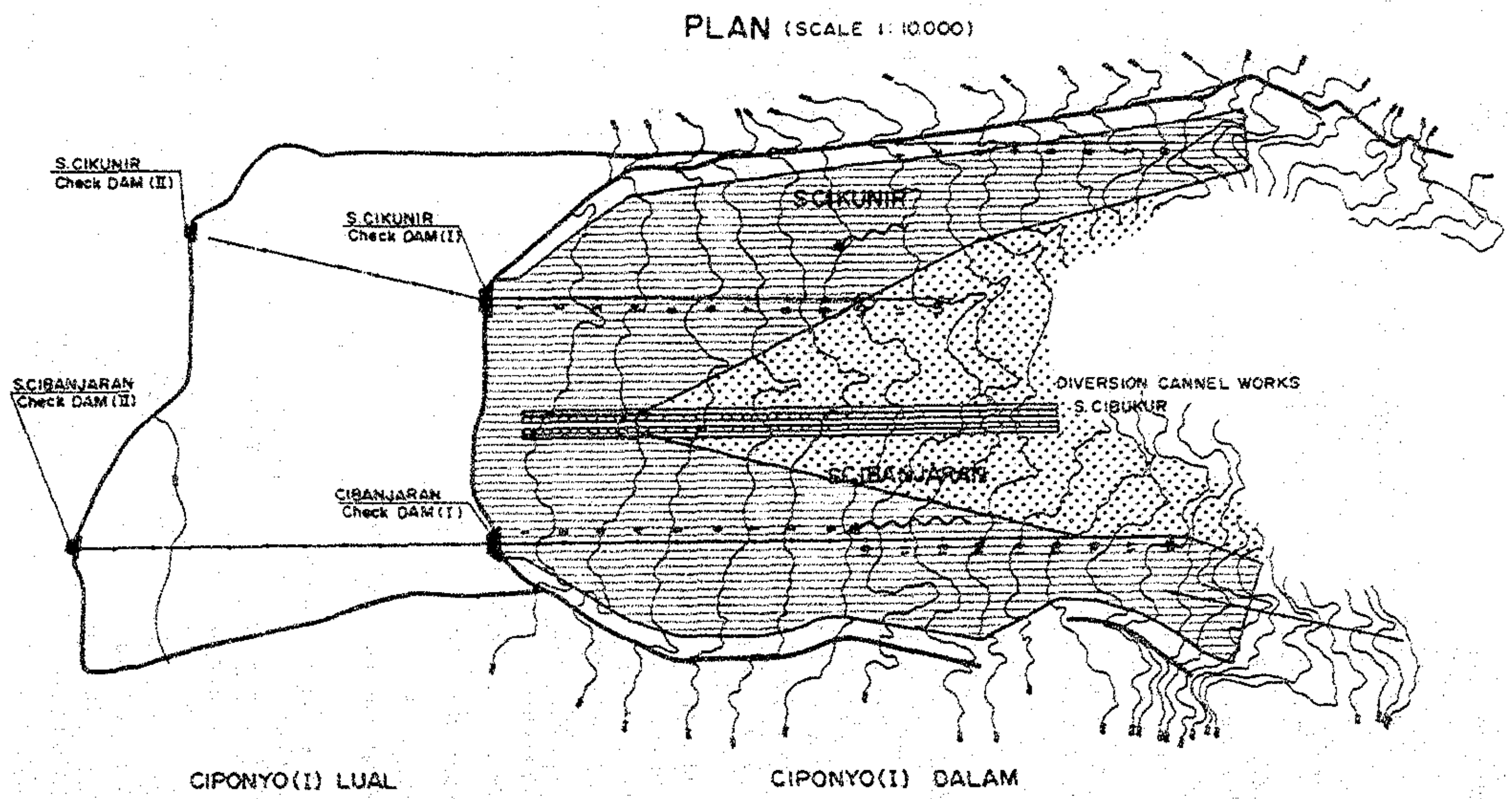


Fig. - 3.9 River Excavation Area in the Ciponyo I Dalam

Table - 3.12 Volume of Stocking Area in Ciponyo I Dalam

	Distance	Area	Average Area	Volume	Cumulative Volume
No. 0				0	0
No. 1	100				
No. 2	100				
No. 3	100				
No. 4	100				
No. 5	100				
No. 6	100	220.44			
No. 7	100	280.48	250.46	25,046	25,046
No.08	100	581.73	421.11	42,111	67,157
No.09	100	844.44	713.09	71,309	138,466
No.10	100	1,314.26	1,079.35	107,935	246,401
No.11	100	2,250.41	1,782.34	178,234	424,635
No.12	100	2,188.91	2,219.66	221,966	646,601
No.13	100	1,735.04	1,961.98	196,198	842,799
No.14	100	1,914.67	1,824.86	182,486	1,025,285
No.15	100	797.22	1,325.95	135,595	1,160,880
No.16	100	538.65	667.94	66,794	1,227,674
No.17	100	823.19	680.92	68,092	1,295,766
No.18	100	646.74	734.97	73,497	1,369,263

Table - 3.13 Volume of Riverbed Excavation in Ciponyo I Dalam

	Distance	Area	Average Area	Volume	Cumulative Volume
No. 0		1,656		0	0
No. 1	100	1,245	1,450.5	145,050	145,050
No. 2	100	957	1,101.0	110,100	255,150
No. 3	100	715	836	83,600	336,750
No. 4	100	725	720	72,000	410,750
No. 5	100	700	712.5	71,250	482,000
No. 6	100	322	511	51,100	533,100
No. 7	100	1,210	766	76,600	609,700
No.08	100	702	756	95,600	705,300
No.09	100	564	633	63,300	768,600
No.10	100	474	519	51,900	820,500
No.11	100	702	588	58,800	879,300
No.12	100	906	804	80,400	959,700
No.13	100	1,614	1,260	126,600	1,085,700
No.14	100	1,457	1,535.5	153,550	1,239,250
No.15	100	1,400	1,428.5	142,850	1,382,100
No.16	100	782	1,091	109,100	1,491,200
No.17	100	1,086	934	93,400	1,584,600
No.18	100	1,014	1,050	105,000	1,689,600

3.3.3 Raising of Dike

Among the annual deposited sediment volume, the volume which correspond to the probable transportation capability by railways of each alternative is carried to the aggregate plant. The rest volume becomes the object of the raising dike.

It is difficult to estimate the deposit condition of the inflow sediment in detail at the sandpocket, however, it is known from experience that a large quantity of the volume deposits in the upper reach there. Consequently, the aggregation at the upper reach becomes remarkable, and then the extent of safety falls relatively as a matter of the disaster prevention of dike.

For that reason, it is desirable to control the design gradient of riverbed to be always fixed moving the sediment deposited in the sandpocket from the upper reach to the lower reach by the construction machineries. Moreover, the sediment will be carried and deposited on the dead water zone of S. Cikunir and S. Cibanjaran.

The area of the sandpocket is 1,747 $x10^3$ m^2. The examined sediment volume, deposit thickness and required dike height for raising dike of each alternative are shown in Table - 3.14.

Table - 3.14 Sediment Volume for Aggradation Riverbed

	Sediment Volume (m^3)	Thickness of Sediment (m)	Proposed height of Dike (m)
Alternative A	0	0	3.00
Alternative B	1,356 $x10^3$	0.8	3.80
Alternative C	2,355 $x10^3$	1.4	4.40
Alternative D	3,932 $x10^3$	2.3	5.30
Alternative E	4,956 $x10^3$	2.9	5.90
	(7,660 $x10^3$)	(4.4)	(7.40)

Note: ( ) is deposited sediment volume for 50 years.

The raising height of dike by spot of each alternative is as shown in the following Table - 3.15.

Table - 3.15 Raising Height of Dike by Spot of Each Alternative

	Dike Length (m)	Alter-native A (m)	Alter-native B (m)	Alter-native C (m)	Alter-native D (m)	Alter-native E (m)
Kokoncong	L=620	-	-	-	1.2	2.3
Cimareme	L=718	-	0.5	1.0	1.7	3.8
Telelor	L=570	3.00	3.80	4.40	5.30	7.30
Kubang buled	L=426	-	-	-	-	1.0
Kubang martar	L=575	-	-	-	-	1.9
Cibueuk	L=200	-	0.7	1.2	1.9	3.9

The result of the table is the raising height of the existing dike and no marked dike is needed the reinforcement and the repair. The sediment volume required for the raising and the repair of each dike is shown in Table - 3.16.

Table - 3.16 Quantity of the Raising Dike

(m^3)

	Alter-native A	Alter-native B	Alter-native C	Alter-native D	Alter-native E
Kokoncong	11,625	11,625	11,625	23,777	57,555
Cimareme	4,847	12,120	19,616	32,095	81,191
Telelor	17,528	25,234	31,852	43,125	81,413
Kubang buled	2,237	3,157	3,936	5,249	27,460
Kubang martar	3,019	4,261	5,313	7,086	48,697
Cibueuk	1,050	7,465	12,000	19,540	49,076
Total	40,306	63,862	84,342	130,872	345,392

(1) Cibanjaran River Side

Table - 3.17 Necessary Height of Each Dike in S. Cibanjaran Side

(m)

No.	Elevation of Existing Dike	Elevation of Riverbed	Height of Existing Dike	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E	H=4.40
CPNVID	523.30	524.30							
1AD	531.45	526.75	4.70	-1.700	-0.900	0.600	2.700	1.300	-0.30
2AD	536.74	529.75	6.99	-3.990	-3.190	-1.690	0.410	-0.990	-2.59
3AD	539.93	532.75	7.13	-4.130	-3.330	-1.830	0.270	-1.130	-2.73
4AD	542.24	535.75	6.49	-3.490	-3.690	-1.190	0.910	-0.490	-2.09
5AD	544.58	538.75	5.83	-2.830	-2.030	-0.530	1.570	0.170	-4.43
6AD	548.60	541.75	6.85	-3.850	-3.050	-1.550	0.550	-0.850	-2.45
7AD	550.87	544.75	6.12	-3.120	-2.320	-0.820	1.280	-0.120	-1.72
8AD	553.77	548.75	5.02	-2.020	-1.220	0.280	2.380	0.980	-0.62
9AD	557.34	552.75	4.59	-1.590	-0.790	0.710	2.810	1.410	-0.19
10AD	560.98	556.75	4.23	-1.230	-0.430	1.070	3.170	1.770	+0.17
11AD	565.09	560.75	4.34	-1.340	-0.540	0.960	3.060	1.660	0.06
12AD	568.07	564.75	3.32	-0.320	0.480	1.980	4.080	2.680	1.08
13AD	571.39	568.75	2.64	0.360	1.160	2.660	4.760	3.360	1.76
14AD	576.28	572.75	3.53	-0.530	0.270	1.770	3.870	2.470	0.87
15AD	583.55	576.75	6.80	-3.800	-3.000	-1.500	0.600	-0.800	-2.40
16AD	590.69	580.75	9.94	-6.940	-6.140	-4.640	-2.540	-3.940	-5.54
17AD	597.62	584.75	12.87	-9.870	-9.070	07.570	-5.470	-6.870	-8.47
18AD	602.13	588.75	13.38	-10.380	-9.580	-8.080	-5.980	-7.380	-8.98

(2) Cikunir Area Side

Table - 3.18 Necessary Height of Each Dike in Cikunir Side

No.	Elevation of Existing Dike	Elevation of Riverbed	Height of Existing Dike	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E	H=4.40
CPNVID	523.30	524.30							
1D	523.86	523.75	0.11	2.890	3.690	5.190	7.190	5.890	4.29
2	530.45	529.75	0.70	2.300	3.100	4.600	6.700	5.300	3.70
3	533.92	532.75	1.17	1.830	2.630	4.130	6.230	4.830	3.23
4	539.27	535.75	3.52	-0.520	0.280	1.780	3.880	2.480	0.88
5	542.74	538.75	3.99	-0.990	-0.190	0.310	3.410	2.010	0.41
6	545.53	541.75	3.78	-0.780	0.020	1.520	3.620	2.220	0.62
7	547.73	544.75	2.98	0.020	0,820	2.320	4.420	3.020	1.42
8	552.40	548.75	3.65	-0.650	0.150	1.650	3.750	2.350	0.75
9	556.01	552.75	3.26	-0.260	0.540	2.040	4.140	2.740	1.14
10	559.86	556.75	3.11	-0.110	0.690	2.190	4.290	2.890	1.29
11	564.72	560.75	3.97	-0.970	-0.170	1.330	3.430	2.030	0.43
2B	568.00	564.75	3.25	-0.250	0.550	2.050	4.150	2.750	1.15
3B	571.72	568.75	2.97	0.030	0.830	2.330	4.430	3.030	1.43
4B	577.67	572.75	4.92	-1.920	-1.120	0.380	2.480	1.080	-0.52
5B	581.79	576.75	5.04	-2.040	1.240	0.260	2.360	0.960	-0.64
6B	587.41	580.75	6.66	-3.660	-2.860	-1.360	0.740	-0.660	-2.26
7B	591.10	584.75	6.35	-3.350	-2.550	-1.050	1.050	-0.350	-1.95
8B	595.04	588.75	6.29	-3.290	-2.490	-0.990	1.110	-0.290	-1.89
1C		592.75							
2C		596.75							

The quantity of masonry for the revetment works of each dike is shown in Table - 3.19.

Table - 3.19 Quantity of Stone Masonry for the Revetment

(m^3)

	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E
Kokoncong	7,192	7,192	7,192	7,465	9,657
Cimareme	6,174	7,037	7,678	8,645	11,184
Telelor	4,903	5,586	6,094	6,864	8,878
Kubang buled	2,045	2,317	2,522	2,829	3,510
Kubang martar	2,760	3,128	3,404	3,818	4,738
Cibueuk	1,920	2,176	2,368	2,656	3,296
Total	24,994	27,436	29,258	32,277	41,263

3.3.4 Groyne

The number of the groyne in Ciponyo I Dalam is 174. As the riverbed aggregates gradually in case of the plan of raising dike, the volume of groyne works is settled accordingly supposing that the sediment deposits to the final shape. The quantity of gabion for Groyne is shown in Table - 3.20.

Table - 3.20 Quantity of Gabion for Groyne

(m^3)

	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E
Number of groyne	174	174	174	174	174
Height	1	3	4	6	10
Section area	14	31.5	40.25	57.75	92.75
Quantity	2,436	5,481	7,004	10,049	16,139

3.3.5 Aggregate Plant

The aggregate plant capability of each alternative differs by the quantity transporting aggregates to Jakarta. The aggregate plant capability, in which the long distance transport capability to Jakarta is to be 428×10^3 m^3 at the maximum and 120×10^3 m^3 at the minimum according to the actual result investigation data of the transportation by railways, is described in the following table. Besides, the conditions in order to estimate the plant capability is as shown below. The location of aggregate plant will be planned being adjacent to Pirusa Station near Sandpocket Negla.

(1)	Number of Annual Working Days of Plant	221 days/year
(2)	Working Hours of Plant	7 hours/day
(3)	unit Deposit Weight of Cast Sediment of Plant	1.60 t/m^3

The capacity of aggregate plant for each alternative are as shown in the following Table - 3.21. The specification of aggregate equipment for alternative-A are as shown in the following Table - 3.22.

Table - 3.21 Capacity of Aggregate Plant

	Annual Dumping volume (m^3)	Workable years	Capacity of aggregate plant (t/h)
Alternative A	613,000	8	630
Alternative B	450,000	8	470
Alternative C	319,000	8	330
Alternative D	128,000	5	140
Alternative E	0	0	0

Table - 3.22 Specification of Aggregate Equipment

Equipment	Quan-tity	Specification	Total Description Power (kw)	Total Weight (t)
1. Feed hopper	2	50 m^3x2	-	
2. Vibrating grizzly	2	160 (cm)x500(cm)	22	22
3. Jaw Crusher	2	500(mm)x750(mm)	90	40
4. Cut gate	4	400 t/h	6	1.0
5. Vibrating screen (I)	4	1,500(mm)x3,000(mm)	30	15.6
6. Vibrating screen (II)	4	2,400(mm)x6,600(mm)	120	40
7. Classifier	2	ϕ1,050(mm)xℓ7,500(mm)	22	28
8. Pump	2	ϕ250mm	110	3.2
9. Belt conveyer	600 m	B=750 mm and 900 mm	(100)	90

The quantity of civil works for aggregate plant are as shown in the following Table - 3.23.

Table - 3.23 Quantity of Civil Works for Aggregate Plant

		Alter-native A	Alter-native B	Alter-native C	Alter-native D	Alter-native E
Civil works	Concrete (m^3)	3,708	2,966	2,225	1,669	0
	Excavation (m^3)	6,648	5318	3,989	2,992	0
Steel (t)		200	160	115	90	0
Placing (t)		335	268	200	134	0
Capacity of aggregate plant (t/h)		630	460	330	140	0

3.3.6 Diversion Channel of S. Cibukur

In case of the plan of raising dike, the lower reach of S. Cikunir is not desirable because it is influenced by afflux of sediment. Therefore, the diversion of S. Cibekur which flows down the center of S. Cikunir and S. Cibanjaran is needed. There is S. Cipanas in the upper stream of S. Cibukur and the warm water flows down as the under flow water. The catchment area is 2.3 km^2 and the design peak discharge of return period 50 years is estimated to be about 65 m^3/s in comparison with the discharge in other basins. The proposed gradient of riverbed is supposed to coincide with the design gradient of riverbed of the sandpocket.

(1) Section of Diversion Channel

The section of the diversion channel is as follows. The water depth the diversion channel is 2.40 m when the design peak discharge of return period 50 years flows down. The height of the channel is supposed to be 3.0 m.

H	A	P	R	$R^{2/3}$	V	Q
1.0	3.00	5.11	0.587	0.701	3.035	9.105
2.0	9.00	8.71	1.033	1.022	4.425	39.825
3.0	18.00	12.317	1.461	1.288	5.577	100.386

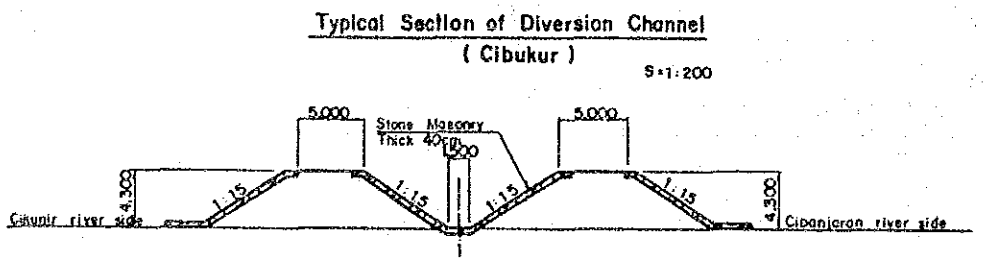


Fig. - 3.10 Typical Section of Diversion Channel to S. Cibukur

(2) Quantity of Diversion Channel

The Quantity of Diversion Channel by spot of each alternative is as shown in the following Table - 3.24.

Table - 3.24 Quantity of Diversion Channel

	Unit	Alter-native A	Alter-native B	Alter-native C	Alter-native D	Alter-native E
Height of bank	(m)	0	3.0	3.40	4.30	6.40
Width of Channel	(m)	0	1.5	1.5	1.5	1.5
Length of Channel	(m)	0	1,500	1,500	1,500	1,500
Slope of dike		0	1:1.5	1:1.5	1:1.5	1:1.5
Area of cross section		0	57.0	68.68	98.47	192.48
Quantity of embankment		0	85,500	103,020	147,705	288,720
Quantity of revetment		0	14,895	16,196	19,125	25,947

3.4 Check Dam Works

Design of the check dam was carried out based on "Ministry of Construction Technical Standard of River and Sabo Engineering (draft) compiled by the River Bureau, Ministry of Construction" and based on Japanese and Indonesian operation cases.

3.4.1 Outline of Facilities

1) S. Cikunir Basin

Check dams, CKN-4, CKN-5, and a consolidation dam, CKN-3, are gravity concrete dams designed for the stabilization of sediment that flows into Ciponyo I Dalam and bank erosion prevention. The base being gravel, energy dissipator pools.

2) S. Cibanjaran Basin

A series of dams constructed with check dams CBJ-4, CBJ-5, and a consolidation dam, CBJ-3. For the design, the same way of thinking was adopted as in the case of S. Cikunir basin.

3) S. Cimampang Basin

Check dams constructed in the upperstream area of sandpocket Negla, CMP-1, CMP-2. The design was the same as in the case of S. Cikunir basin.

3.4.2 Description of the Facility

The structure details of the check dams are shown in Table - 3.25. The quantities of check dam are shown in Table - 3.26.

3.5 Quantity of Construction Work for each Alternative

The quantity of the construction work for each alternative is shown in Table - 3.27.

Table - 3.25 Specification of Check Dams in Cikunir, Ciloseh

Name of River	Name of Facility	Location No.	Design Peak Flood Discharge (m^3/s)	Dam Specification					
				Height (m)	Length (m)	Over Flow Length (m)	Down Stream Slope	Up Stream Slope	Apron Length (m)
S. Cimampang	CMP-1	2/800	226	12.0	41.0	16.0	1:0.20	1:0.60	22.0
	CMP-2	3/800	226	12.0	41.0	16.0	1:0.20	1:0.60	22.0
S. Cikunir	CKN-3	17/510	143	5.0	85.0	30.0	1:0.20	1:0.15	9.0
	CKN-4	17/810	133	5.0	122.0	30.0	1:0.20	1:0.15	9.0
	CKN-5	18/010	123	5.0	104.0	30.0	1:0.20	1:0.15	9.0
S. Cibanjaran	CBJ-3	7/860	209	7.0	213.0	24.0	1:0.20	1:0.25	13.0
	CBJ-4	8/160	196	8.0	54.0	22.0	1:0.20	1:0.45	14.5
	CBJ-5	8/660	187	13.0	104.0	22.0	1:0.20	1:0.55	22.0

Table - 3.26 Quantity of Check Dams

Name of River	Name of Facility	Location No.	Volume			
			Height (m)	Length (m)	Concrete (m^3)	Excavation (m^3)
S. Cimampang	CMP-1	2/800	12.0	41.0	4,400	1,320
	CMP-2	3/800	12.0	41.0	4,400	1,320
S. Cikunir	CKN-3	17/510	5.0	85.0	2,200	660
	CKN-4	17/810	5.0	122.0	3,200	960
	CKN-5	18/01	5.0	104.0	3,200	960
S. Cibanjaran	CBJ-3	7/860	7.0	213.0	8,000	2,400
	CBJ-4	8/160	8.0	54.0	2,700	810
	CBJ-5	8/660	13.0	104.0	8,800	2,640

Table - 3.27 Quantity of Construction for Cikunir Area

I t e m		Alternative A	Alternative B	Alternative C	Alternative D	Alternative E
		1,370,000	1,370,000	1,370,000	1,370,000	1,370,000
Improvement of dike Ciponyo I Dalam	Embankment (m^3)	125,238	125,238	125,238	125,238	125,238
	Gabion(m^3)	20,934	20,934	20,934	20,934	20,934
	Groyne(m^3)	6,202	6,202	6,202	6,202	6,202
Raising of of dike	Embankment (m^3)	40,306	63,862	84,342	123,320	309,358
	Maspnry(m^3)	24,994	27,436	29,258	32,277	41,263
	Groyne(m^3)	2,436	5,481	7,004	10,049	16,139
Aggregate plant civil work	Excavation (m^3)	6,648	5,318	3,989	2,992	0
	Concrete (m^3)	3,708	2,966	2,225	1,669	0
	Steel (m^3)	200	160	115	90	0
	Placing (t)	335	268	200	134	0
Check dam	Excavation (m^3)	8,430	8,430	8,430	8,430	8,430
	Concrete (m^3)	28,100	28,100	28,100	28,100	28,100
Consolidation dam	Embankment (m^3)	34,320	34,320	34,320	34,320	34,320
	Revetment (m^3)	6,990	6,990	6,990	6,990	6,990
	Excavation (m^3)	1,560	1,560	1,560	1,560	1,560
	Concrete (m^3)	5,200	5,200	5,200	5,200	5,200
Revetment works	Excavation (m^3)	10,817	10,817	10,817	10,817	10,817
	Embankment (m^3)	6,490	6,490	6,490	6,490	6,490
	Masonry(m^3)	9,615	9,615	9,615	9,615	9,615
	Concrete (m^3)	2,975	2,975	2,975	2,975	2,975
Excavation for aggregate	(1) (m^3)	4,513,000	3,206,000	2,158,000	630,000	0
Excavation for maintenance	(2) (m^3)	0	1,307,000	2,355,000	3,932,000	4,956,000
Diversion channel	Embankment (m^3)	0	14,895	16,196	19,125	25,947
	Masonry(m^3)	0	85,500	103,020	147,705	288,720

4. Stabilization of River Course in the Sandpocket Ciponyo II

4.1 Outline of Work

In the sandpocket Ciponyo II, there has been great progress in the use of land-like paddy fields and several communities have now formed. At the time of a flood, however, the river channel on the side of S. Cikunir is not stabilized, and it suffers from inundation. The extension of S. Ciponyo II is in the upstream direction covering the distance of 4.5 km and passage between communities is not convenient. Therefore, consolidation dams will be built for the stabilization of river channels, by which to prevent inundation in sandpocket and also to facilitate better communications between communities.

The revetment works to allow for the safe flow of flood water is to be made ground the narrow section of confluence of S. Cibanjaran and S. Cikunir where some sections of the bank are low.

4.2 Consolidation Dam Work

The consolidation work for Ciponyo II is to place CKN-1 AND CBJ-1 AT A SPOT 1.5 km from the confluence, avoiding communities in the sandpocket, and place CKN-2 and CBJ-2 at a spot 3.4 km from the confluence. In the consolidation work, the width of overflow section is to be 40 m. Dikes on both sides of the sandpocket will be connected to the consolidation dam by means of dikes.

The details of the revetment work and the dikes area shown in Table - 4.1 and general sketches are shown at the end of this volume.

Table - 4.1 Consolidation dams

Name of River	Name of Facility	Location No.	Design Peak Flood Discharge (m^3/s)	Dam Specification					
				Height (m)	Length (m)	Over Flow Length (m)	Down Stream Slope	Up Stream Slope	Apron Length (m)
S. Cikunir	CKN-1	11/300	195	4.0	70.0	40.0	1:0.20	1:0.15	8.00
	CKN-2	13/100	193	4.0	70.0	40.0	1:0.20	1:0.15	8.00
S. Cibanjaran	CBJ-1	1/200	192	4.0	70.0	40.0	1:0.20	1:0.15	8.00
	CBJ-2	3/200	187	4.0	70.0	40.0	1:0.20	1:0.15	8.00

Adding 1.0 m of free board to the overflow height calculated the required height of dike at consolidation dam will be 3.0 m. The typical section of dike is shown in Fig. - 4.1.

The length of dike are as follows;

a) Upstream spot 1.5 km from the confluence point 530 m

b) Upstream spot 3.4 km from the confluence point 1,100 m

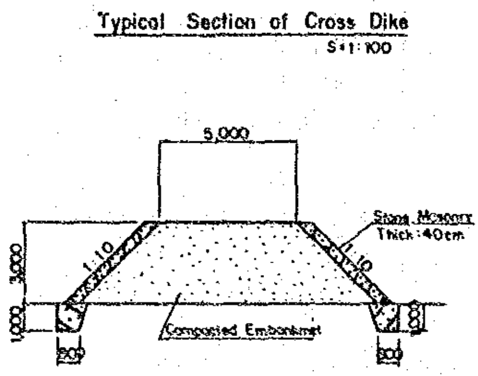


Fig. - 4.1 Typical Section of Dike

The quantity of consolidation dam is shown in Table - 4.2.

Table - 4.2 Quantity of Consolidation Dam

	Consolidation (I)	Consolidation (II)	Total
Embankment	24,000	10,320	34,320
Stone masonry	4,890	2,100	6,990
Excavation	780	780	1,560
Dam masonry	2,600	2,600	5,200

4.3 Revetment Works of Confluence for Cibanjaran and Cikunir

The main purpose of revetment works is to protect the narrow section of dike against flood.

The design criteria of the facilities of revetment works are selected based on the following items.

a)Design Flood Discharge 419 m^3/s

b)Length of Revetment Works station 9K+00 - 10K+700 m

c)Protection Type Stone masonry

d)River Gradient 1/70

e)Area of Channel Section

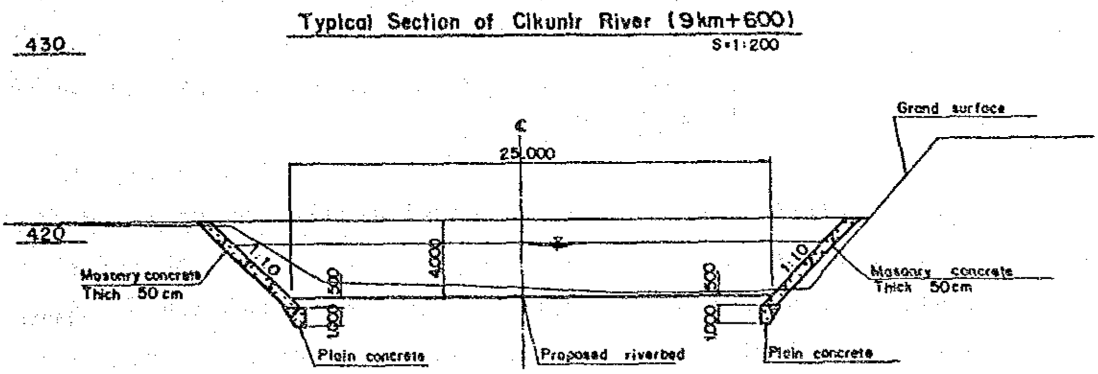


Fig. - 4.2 Typical Cross Section of Revetment Work

The discharge rating curve of the design river cross-section calculated by using uniform flow formula is as follows:

H	A	P	R	$R^{2/3}$	I	V	Q m^3/s
1,000	26.00	27.83	0.934	0.96	0.1195	2.87	74.6
2,000	54.00	30.66	1.761	1.46	0.1195	4.36	235.4
3,000	84.00	33.48	2.509	1.85	0.1195	5.53	464.5
4,000	116.00	36.31	3.195	2.169	0.1195	6.48	751.9

I=1/70=0.01428 n=0.04

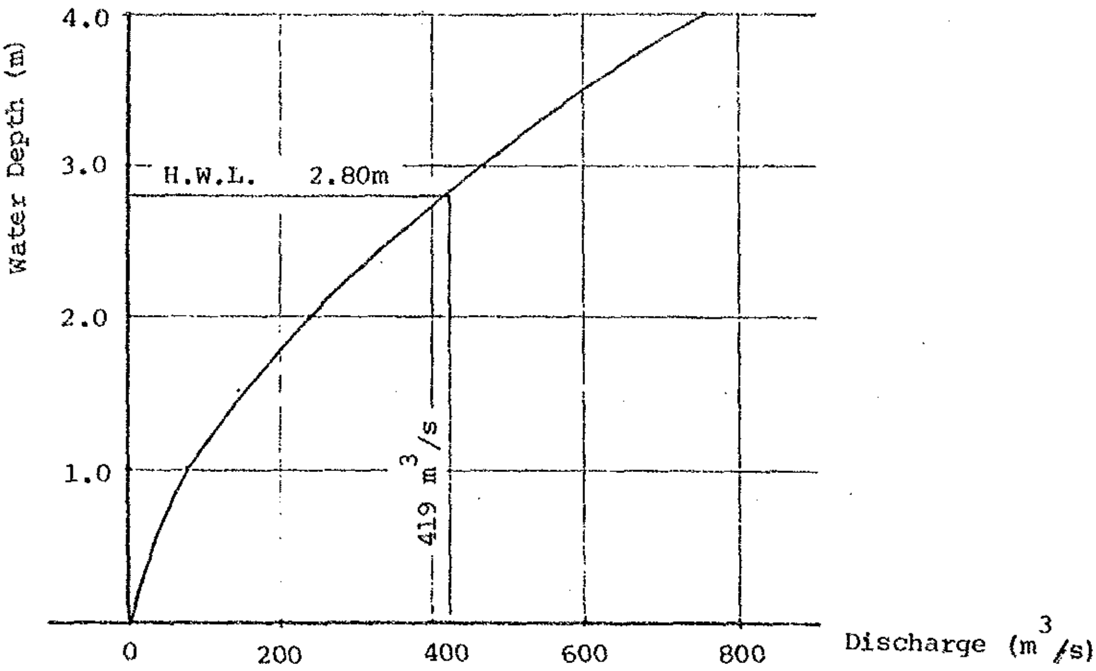


Fig. - 4.3 Discharge Rating Curve of the Design Curve in S. Cikunir

The design flood discharge at the confluence is to be 419 m^3/s. Taking into account the design riverbed gradient and the width of the river, the water level at the time of the flood was estimated at 2.8 m. The revetment works were limited to a 1.7 km long area covering both side of the confluence point. The details of the structure are as shown in Table - 4.3.

Table - 4.3 Specification of Revetment Works

Item	Description	Remarks
Design Flood Discharge	419 m^3/s	
Riverbed Width	25.0 m	
Riverbed Gradient	1/170	
Band Slope	4.0 m	
Slope Protection	Masonry	
Length of Revetment Works	3.4 km	

5. Construction of Check Dams on the South Area Basin (Unit 3)

5.1 General

20 check dams to protect Singaparna against Lahar will be constructed in the following rivers; S. Cisaruni, S. Cikupang and S. Cimerah

The design excess sediment volume of 50 years of return period in three (3) rivers of S. Cisaruni, S. Cikupang and S. Cimerah on the Southern slope of Mt. Galunggung should be controlled by the group of check dams including the existing check dams in upper reaches.

(1) S. Cisaruni Basin

Five (5) check dams will be arranged in order to control the design excess sediment volume of 134 $x10^3$ m^3.

(2) S. Cikupang Basin

Three (3) check dams will be arranged in order to control the design excess sediment volume of 46 $x10^3$ m^3.

(3) S. Cimerah Basin

At the S. Cimerah basin, the check dam to control the sediment runoff volume from the upperstream devastated basin will be arranged.

Thirteen (13) check dams including existing one (1) check dam at S. Cimerah upperstream will be arranged in order to control the design excess volume of 534 $x10^3$ m^3.

5.2 Standard of Design

The design was carried out based on the following technical standards.

(i) Technical standards for River and Sabo Engineering, River Bureau, Ministry of Construction Japan 1985

(ii) Design Manual for Sabo Engineering, Flood Control and Sabo Engineering Associations, Japan 1986

5.2.1 Design Load and Constants Applied to Stability Analysis

(1) Design Load

The following loads are considered in stability analysis of check dam. (Refer to Fig. - 5.1)

a) Dam weight : W

b) Hydro-pressure : Pw

Case : Flood Time

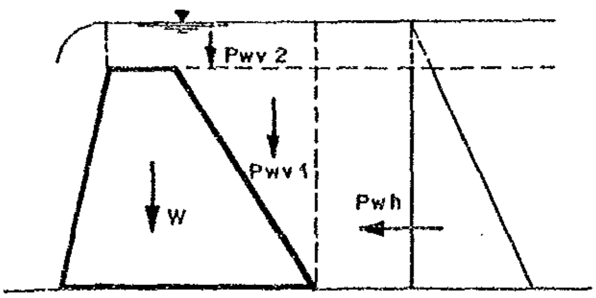


Fig. - 5.1 Loads for Stability Analysis of Check Dam

1) Dam Weight

The dam weight is given with the following formula:

$$W = \gamma m.\ Am \quad \ldots\ldots\ldots\ldots \quad (5.1)$$

Where,

W : Dam weight expressed in ton (t)/unit width (m)

γm : Unit weight of masonry (t/m^3)

Am : Volume (m^3) per unit width (m) of masonry

2) Hydro-pressure

The hydro-pressure against the dam body consists of the horizontal load of hydrostatic pressure and vertical load of flowing water weight. This pressure is expressed below:

$$Pwh = \int_0^{hw} Wo.\ hw.\ dhw = \frac{1}{2}\ Wo.\ hw^2 \quad \ldots\ldots\ldots\ldots \quad (5.2)$$

$$Pwv = Wo.\ Aw \quad \ldots\ldots\ldots\ldots \quad (5.3)$$

Where,

Pwh : Hydrostatic pressure expressed in ton (t)/unit width (m)

Wo : Unit weight of water (t/m^3)

hw : Water depth (m)

Pwv : Water weight expressed in ton (t)/unit width (m)

Aw : Volume (m') per unit width (m) of water.

(2) Constant Applied to Stability Analysis

The constants numbers to be applied to stability analysis of check dam can be set in the following way:

1) Unit weight

a) Water (Wo)

- Flood water — $Wo = 1.00\ t/m^3$
- Underground water — $Wo = 1.20\ t/m^3$

b) Concrete (γs) — $\gamma s = 2.35\ t/m^3$

c) Masonry (γm) — $\gamma m = 2.20\ t/m^3$

2) Coefficient of Friction between Dam Base and Foundation Subgrade — $f = 0.60$

5.2.2 Overflow Section and Wing Wall

(1) Overflow Section

To determine the width (B1) of overflow section, the following regime formula is applied. Refer to Fig. - 5.3.

$$B1 = \alpha r \cdot Qp^{*1/2} \quad \text{.......} \quad (5.6)$$

Where,

B1 : Overflow section width (m)

αr : Constant (=4.0)

Qp^* : Design of peak discharge (Qp) plus 10% sediment of Qp^* ($Qp^* = Qp + 0.1\ Qp^*$)

The design of overflow is calculated by the following formula.

$$Qp^* = \frac{2}{15} \cdot C \cdot \sqrt{2g}\ (3B_1 + 2B_2)\ h_1^{3/2} \quad \text{.......} \quad (5.7)$$

Where,

C : Discharge coefficient (0.60)

B : Acceleration of gravity (9.8 m/s^2)

B_1 : Bottom width of the overflow (m)

B_2 : Water surface width (m)

h_1 : Overflow depth (m)

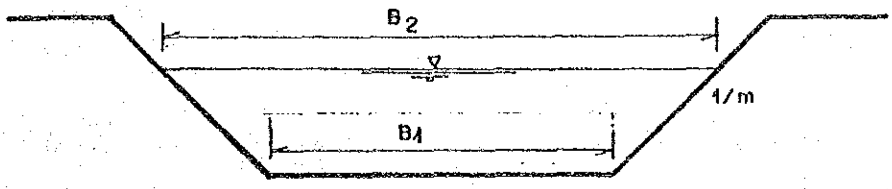


Fig. - 5.2 Overflow Section Consolidation Dam

(2) Wing Wall

The wing wall shall be so designed that the flow direction of flood is correctly controlled by avoiding the overtopping from the wing wall. Therefore, more than 1 m freeboard is included in the height of wing wall. The width of wing wall shall be designed to bear the impact of design debris flow. The wing wall shall also be penetrated into the river bank, because the floods may erode this part.

5.2.3 Main Dam

(1) Stability Condition

The following conditions shall be satisfied in each of load condition for the stability of check dam:

a) The resultant force shall be acted on the middle 1/3 of dam base, in order to prevent the creation of tensile stress at the bottom.
b) Sliding does not occur between the dam base and the foundation subgrade.
c) Maximum stress at the foundation subgrade shall be less than the allowable bearing stress of the subgrade.

(2) Stability against Overturning and Tensile Stress

To avoid tensile stress at the base of the dam foundation, the resultant force of the dam's weight and external forces shall be acted on the middle of 1/3 dam base. On the other hand the eccentricity of resultant force is less than 1/6 of dam base. In this way, the dam's stability against overturning can also be secured.

(3) Stability against Sliding

The sliding safety of the dam base and the subgrade can be confirmed using the following formula:

$$NS = \frac{f \bullet V}{H} \geq 1.2 \quad \text{.......} \quad (5.8)$$

Where,

NS : Safety factor for sliding

f : Friction coefficient

V : Total vertical load per unit width (t)

H : Total horizontal load per unit width (t)

(4) Stability against Settlement of Foundation

The maximum stress at the base of dam shall not exceed the allowable stress of the subgrade:

$$\sigma = \frac{V}{b_2} (1 + \frac{6 \bullet e}{b_2}) \geq qa$$

$$e = \frac{b_2}{2} - \frac{M}{V} \geq b_2/6 \quad \text{.......} \quad (5.10)$$

Where,

σ : The maximum vertical stress at the base of dam (t/m^2)

V : Total vertical load per unit width (t)

b_b : The width of the dam base (m)

e : The eccentricity of resultant force (m)

M : Total moment per unit width (tm)

qa : Allowable stress of the subgrade (t/m^2)

(5) Summary of Loads and Stability Conditions

The loads and stability conditions of consolidation dam can be summarized as in Table - 5.1.

Table - 5.1 Summary of Loads and Stability Conditions

Loads & Stability Condition	Items	Formula	Flood Condition
Loads & condition	Dam weight	$W = \gamma m. Am$	0
	Hydro pressure	$\frac{1}{2} Pwh = b \cdot Wo \cdot hw^2$	0
	Overturning	$e = \frac{b_2}{2} - \frac{M}{V}$	$\geq b_2/6$
	Sliding	$NS = \frac{f \cdot v}{H}$	≥ 1.2
	Settlement	$\sigma = \frac{V}{b_2}(1 + \frac{6 \cdot e}{b_2})$	$\geq qa$

5.3 Description of Check Dams

As for sabo facilities on the southern slope basin of Mt. Galunggung, check dam work is the main work. In determining the specifications of the sabo dam, the following items were taken into consideration.

a) Height of Check Dams

The foundation of check dams is mostly composed of unconsolidated lahar deposits, weak in soil bearing capacity and high in permeability, and therefore not adequate as foundation. For this reason, it is desirous that the height of the check dam should be lower than 15 m. If it has to be higher than 15 m, the dam should be firmly placed to rocks.

b) Free Board of Check Dams

Free board is determined in consideration of design flood discharge, riverbed gradient, and so forth. Here in this region, however, riverbed fluctuations being intense, it should be higher than 1 m.

c) Countermeasures for Prevention of Scouring in the Apron Sections of Check Dams and Consolidation Dams. The Apron Section of check dams is subject to scouring due to the change of hydraulic conditions such as the decrease of runoff sediment caused by the construction of the check dam. Therefore, the material for the construction of apron should be made by concrete.

d) Materials and operation

In view of the economical and employment conditions, one should consider adopting the full use of stone sediment, bamboo, wood and the like, as well as considering construction by manpower. Items planned for check dams along S. Cikunir, S. Cikupang and S. Cimelah area shown in Table - 4.8.

Table - 5.2 Description of Check Dam Works in Southern Slope of Mt. Galunggung

Name of River	Name of Facility	Location No.	Design Peak Flood Discharge (m^3/s)	Dam Specification					
				Height (m)	Length (m)	Over Flow Length (m)	Down Stream Slope	Up Stream Slope	Apron Length (m)
S. Cisaruni	CSR-1	2/100	110	7.0	30.5	12.0	1:0.2	1:0.40	13.0
	CSR-2	2/600	104	11.5	31.0	12.0	1:0.2	1:0.50	19.5
	CSR-3	3/100	99	22.0	56.0	12.0	1:0.2	1:0.85	34.5
	CSR-4	3/500	76	22.0	55.0	8.0	1:0.2	1:0.85	34.5
	CSR-5	3/900	51	18.0	51.0	6.0	1:0.2	1:0.85	28.5
S. Cikupang	CKP-1	2/200	82	7.0	27.0	8.0	1:0.2	1:0.40	13.0
	CKP-2	3/200	67	12.0	39.0	8.0	1:0.2	1:0.55	20.5
	CKP-3	3/800	51	17.0	36.0	8.0	1:0.2	1:0.85	27.0
S. Cimerah	CKP-1	2/095	150	7.0	28.5	10.0	1:0.2	1:0.45	14.05
	CKP-2	3/110	150	9.5	30.0	10.0	1:0.2	1:0.55	18.0
	CKP-4	5/000	143	22.0	63.0	10.0	1:0.2	1:0.90	36.0
	CKP-5	6/300	57	17.0	53.0	6.0	1:0.2	1:0.85	27.0
S. Cisera	CSL-1	2/000	106	7.0	31.0	12.0	1:0.2	1:0.40	13.0
	CSL-2	2/400	94	7.0	28.0	10.0	1:0.2	1:0.40	13.0
	CSL-3	3/000	82	12.0	36.0	8.0	1:0.2	1:0.60	20.5
	CSL-4	3/400	74	22.0	51.0	8.0	1:0.2	1:0.85	34.5
	CSL-5	4/000	54	22.0	62.0	5.0	1:0.2	1:0.85	34.
	CSL-6	4/600	27	17.0	52.0	5.0	1:0.2	1:0.80	25.5
S. Cipada	CPD-1	2/100	44	15.0	51.0	4.0	1:0.2	1:0.85	24.0
	CPD-2	2/800	31	15.0	53.0	4.0	1:0.2	1:0.75	22.5

5.4 Quantity of Check Dam

The quantity of check dam on the southern slope basin of Mt. Galunggung are shown in Table - 5.3.

Table - 5.3 Quantity of Check Dams

Name of River	Name of Facility	Location No.	Height (m)	Length (m)	Volume Concrete	Volume Excavation
S. Cisaruni	CSR - 1	2/100	7.0	30.5	1,400	420
	CSR - 2	2/600	11.0	31.0	3,200	960
	CSR - 3	3/100	22.0	56.0	17,900	5,370
	CSR - 4	3/500	22.0	55.0	14,900	4,470
	CSR - 5	3/900	18.0	51.0	9,300	2,790
Sub Total					46,700	
S. Cikupang	CKP - 1	2/200	7.0	27.0	1,300	390
	CKP - 2	3/200	12.0	39.0	3,500	1,050
	CKP - 3	3/800	17.0	36.0	7,100	2,130
Sub Total					11,800	
S. Cimerah	CMR - 1	2/095	7.0	28.5	1,400	420
	CMR - 2	3/110	9.5	30.0	2,300	690
	CMR - 4	5/000	22.0	63.0	18,000	5,400
	CMR - 5	6/300	17.0	53.0	8,700	2,610
Sub Total					30,400	
S. Cisera	CSL - 1	2/000	7.0	31.0	1,400	420
	CSL - 2	2/400	7.0	28.0	1,300	390
	CSL - 3	3/000	12.0	36.0	3,300	990
	CSL - 4	3/400	22.0	51.0	13,400	4,020
	CSL - 5	4/000	22.0	62.0	15,300	4,590
	CSL - 6	4/600	17.0	52.0	10,000	3,000
Sub Total					44,700	
S. Cipada	CPD - 1	2/100	15.0	51.0	5,500	1,650
	CPD - 2	2/800	15.0	53.0	5,900	1,770
Sub Total					11,400	

6. Drainage Tunnel Project

6.1 Outline of Drainage Tunnel Project for Crater Lake

3.75 km^2 of the upper area of the S. Cibanjaran was blocked when Mt. Galunggung erupted in 1982. Ever since, rainwater has accumulated in the crater and formed a crater lake.

The maximum water level ever is estimated to be 1,107.6 m, at which point the total volume of water is estimated to have been 7,500,000 m^3. The water level of the crater lake fluctuates between an elevation of 1,090 m to 1,099 m and the water volume of stored at an elevation 1,095 m is 4,000,000 m^3.

The simulated results of the amount of rain and the water level of the crater lake for the survey period are examined in Chapter 3. The results show that the water level rose rapidly but there was no collapsing of the crater wall. However, because the crater lake catchment is waste land it is clear that sediment will flow into the crater lake, and a rise in the water level of the crater lake, however gradual, is anticipated.

The average eruption of Mt. Galunggung is short, being approximately 52 years. Therefore, flooding due to the collapsing of the crater wall caused by a rise in the water level of the crater lake or collapsing of the crater wall due to an eruption of the Mt. Galunggung will most likely cause extensive damage to the city of Tasikmalaya located downstream. Setting up a drainage tunnel in the crater lake to lower the water level would be very beneficial. Construction of the drainage tunnel while the water level is low is economical and technically easy. It is advisable to start construction immediately.

6.2 Location of Drainage Tunnel

As for the locaition of the drainage tunnel, the place where the crater lake wall of S. Cibanjaran side was narrowest and the withdrawing to S. Cibanjaran was possible was selected by the following reasons.

1) The riverbed elevation of S. Cibanjaran is lower than that of S. Cikunir.

2) When the center line is decided at the same river elevation, the water lake wall of S. Cibanjaran narrower.

3) The basin of the crater lake belonged to S. Cibanjaran before eruption and the changing basin has much influence to the lower reach.

4) The geology of the crater wall does not have much difference at any places.

6.3 Alternative Plan for Drainage

6.3.1 Alternative Plan

(1) General

It is necessary that the water level of the crater lake be lowered at all times if the destruction of its walls and direct damage from volcanic eruptions are to be reduced.

The following points should be considered when planning facilities to drain water from the crater lake.

(i) There is a high temperature zone (Approx. 150 m sector)

(ii) The outer wall of the crater lake is loose.

(iii) There are cracks and traces of damage in the rock of the inner wall of the crater lake.

(iv) A discharge f gas is expected.

(v) Transportation of Construction Materials.

The conditions described above were taken into consideration and an alternative for the facilities to drain the crater lake was selected.

(2) Alternative I - A

By constructing a tunnel in the wall of the crater, the water in the crater lake can be drained. This method has been used before at Kelud volcano. The execution procedures for the drainage tunnel are as follows:

(i) A vertical shaft of 4.00 m is constructed in the inner wall of the crater lake. Several horizontal tunnels will be necessary to drain the lake water with syphons, and the vertical tunnel is to facilitate this work.

(ii) The main tunnel is constructed from the outer wall of the crater lake. The extension of this tunnel is approximately 300 m. The high temperature zone is distributed from the center to the crater lake side.

(iii) It is thought that it will be possible for the normal temperature zone to be executed using conventional methods. The execution in the high temperature zone will be performed using both the freezing method and the fan method (in which cold area is sent in).

(iv) When the execution of the tunnel is completed the water level of the crater lake will be lowered using syphons. The drain capabilities of various bores of syphon are shown in Table - .

(v) Because the drain capability of the syphon is the 8 m of the water pillar, several horizontal tunnels will be constructed and the water drained to the design level.

The typical section of drainage tunnel are as shown in Fig. - 6.1. Brief descriptions for Alternative II - A are as follows;

Table - 6.1 Specification of Alternative II - A facilities

Facilities		Specification	
Horizontal tunnels	No. 1	2,000	Corrugate pipe - 25.0 m
	No. 2		Corrugate pipe - 35.0 m
	No. 3		Corrugate pipe - 45.0 m
	No. 4		Corrugate pipe - 378.0 m
Vertical shaft		4.000	Corrugate pipe - 90.0 m
Cooling plant*)			2 units

Note *) Capacity of refrigerating plant is 192,000 kcal/unit.

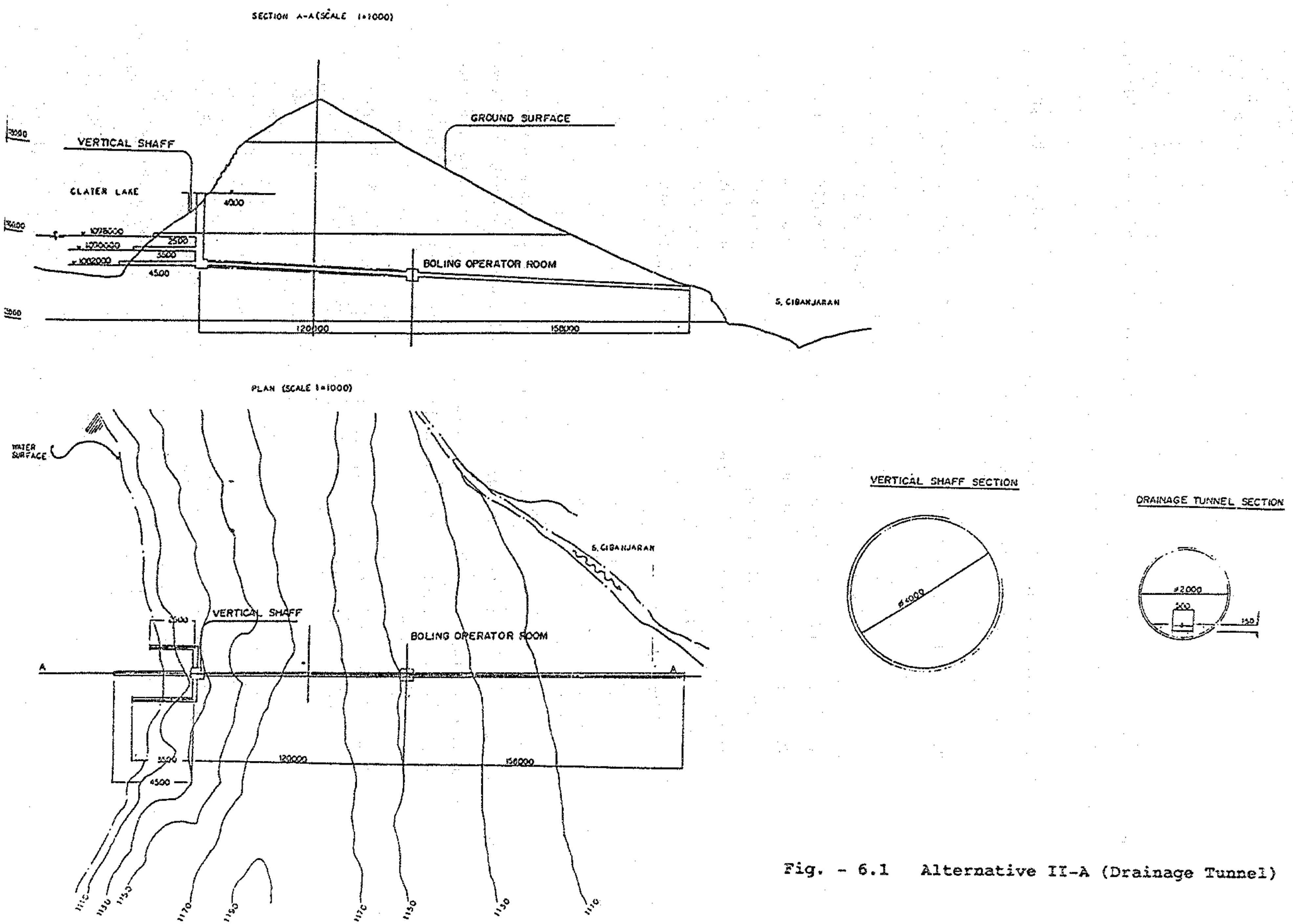


Fig. - 6.1 Alternative II-A (Drainage Tunnel)

(3) Alternative II - B

Excavating is performed to the elevation of the drain channel. Execution will be performed by machines or using cooling. Thus excavation may be easily executed in the high temperature zone.

The typical section of open cut is as shown in Fig. - 6.2. Brief description for Alternative II - B facilities

Table - 6.2 Specification of Alternative II - B Facilities

I t e m	Quantity (m)
Excavation of sand and gravel by manpower	336,800
Excavation of sand and gravel by machine	505,000
Excavation of rock by manpower	144,300
Excavation of rock by machine	216,400
Total	1,202,400
Transportation	1,442,900

(4) Alternative II - C

Drainage is performed using a pump, necessitating no excavation of the high temperature zone crater lake wall. If the discharge is to be the same as in syphoning, 6 pumps will be needed. Adding 3 reserves, this makes a total of 9 pumps. In this alternative it will be possible to drain 0.23 m^3/s.

The typical section of pump up is as shown in Fig. - 6.3. Brief description for Alternative II - C is as shown in Table - 6.3.

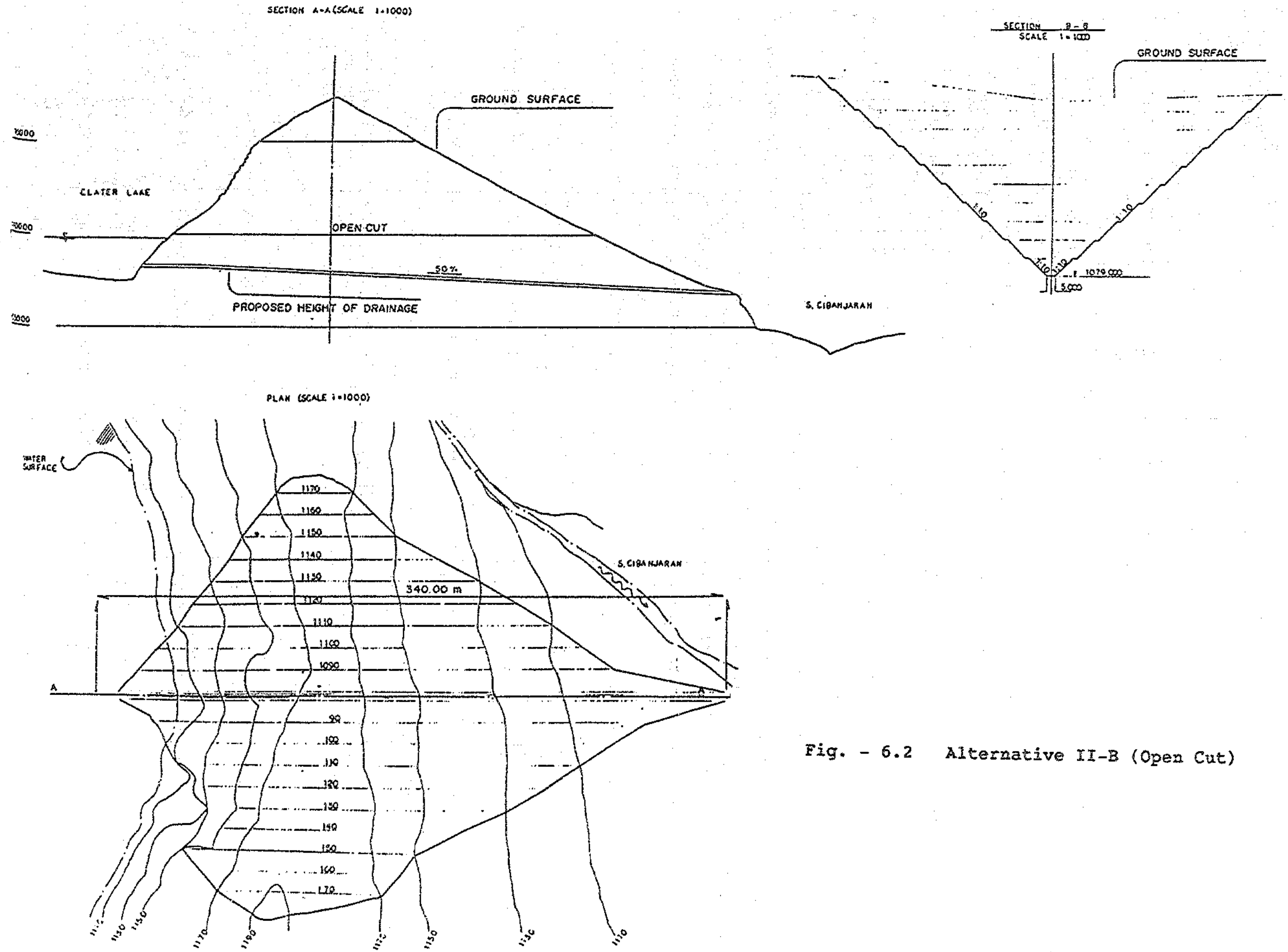


Fig. - 6.2 Alternative II-B (Open Cut)

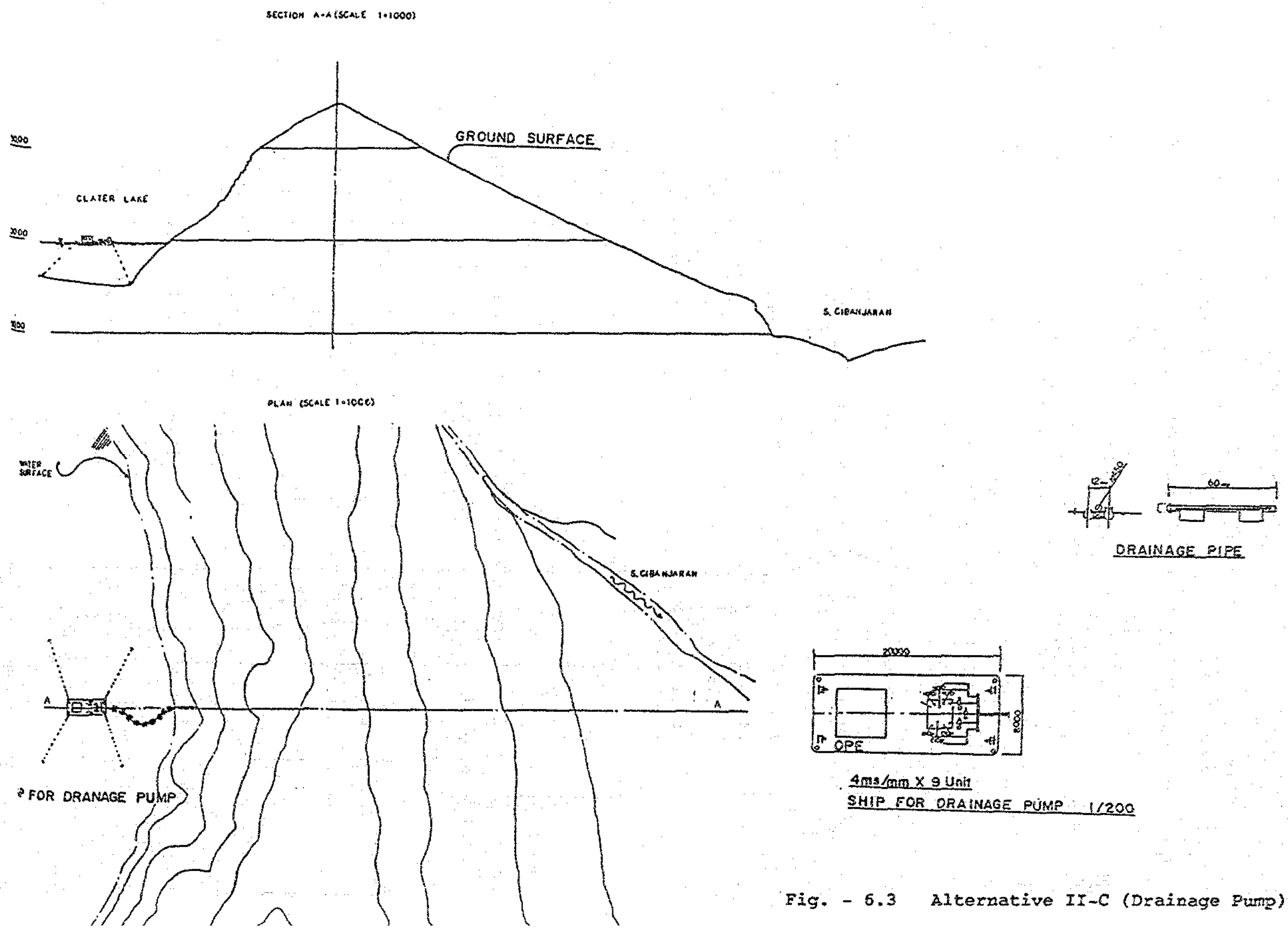


Fig. - 6.3 Alternative II-C (Drainage Pump)

Table - 6.3 Specification of Alternative II - C Facilities

I t e m	Specification	Quantity
Distributing material	Head 100 m capacity 132 kw diagram 20 mm	9 units
Control board	3.6 kv 132 kw	9 units
Uncoming panel	7.2 kv	1 unit
Transpower board	1,000 kvA	1 unit
Piping material	600 mm	1 unit
Distributing material		

6.3.2 Preliminary Cost Estimate

The preliminary cost estimate were made for major works such as excavation of tunnel, tunnel materials, open cut, transportation of material. The estimate costs included costs for construction works and procurement of materials and equipment. For the sake of comparative study of alternative, the expenses needed for preconstruction works are not include in the cost estimate; they are costs for land aquisition, government administration, engineering service, physical contingency, price escalation and so on.

Cost summary of alternatives are shown in Table - 6.4. Breakdown cost of construction are indicated in Annexs.

Table - 6.4 Construction Cost of Alternatives

($Rp \times 10^6$)

Construction	Alternative II - A	Alternative II - B	Alternative II - C
Construction	2,640.6	7,370.6	4,630.5

6.3.3 Selection of Alternatives

The selection comparison of three alternatives are shown in Table - 6.5.

The countermeasure of crater lake is alternative II - A to be adopted for drainage of crater lake because of the following reason.

Table - 6.5 Alternative of Countermeasure for Crater Lake

Specification	Alternative II-A	Alternative II-B	Alternative II-C
Specification	Tunnel (I) = 2.0 m L = 383.0 m Vertical tunnel (II) = 4.0 mm L = 43.0 m Cooling plant 2 units	Open channel W = 1.5 m	Drainage pump 4 mm/min 132 kw Pipe line 30 cm - 500 m Electric Equipment
Quantity	Excavation 2,483.0 m^3 Corrugate pipe 2.0 m 383.0 m Corrugate pipe 4.0 m 43.0 m Cooling time 4,600 hour	Excavation 1,202,000 m^3 Slope protection 40,700 m^3 Transportation of Sand & Gravel 1,443,000 m^3	Excavation 800 m^3 Drainage pump 9 units Electric control board 9 units Pipe Line 500 m
Cost of Construction (Rp x 10^6)	2,640.6	7,370.7	4,630.5
Selective Comparison of Alternative			

6.4 Layout of Drainage Tunnel

(1) Proposed Inlet Height of Drainage Tunnel

The proposed inlet height of Drainage Tunnel is to be 1,082.5 m. This is the lowest elevation considering the sediment inflow volume and the capacity of the crater lake at this elevation is about 750 $\times 10^3$ m^3. With this capacity, it is judged that the secondary disaster by re-eruption will be small.

(2) Longitudinal Gradient

The longitudinal gradient of the drainage tunnel is to be 5% in consideration of carrying out of excavated material and the drainage tunnel.

(3) Proposed Height of Main Drainage Tunnel

The proposed height of main drainage tunnel is estimated by the following formula in order to drainaged water of crater lake by the syphon.

$$\frac{P}{W} = H' - \frac{1 + \xi_e + \xi_b + f\frac{\ell_1}{D}}{1 + \xi_e + \xi_b + f\frac{\ell_1 + \ell_2}{D}} H \quad \text{.............. (1)}$$

$$f - 12.7\ gn^2/D^{1/3}$$

where:

- P : hydrostatic pressure (t/m^3)
- W : unit weight of water (t/m^3)
- ξ_e : roughness of inlet
- ξ_b : roughness of outlet
- n : manning roughness
- ℓ_1 : upstream length of syphon
- ℓ_2 : downstream length of syphon
- D : diameter of syphon

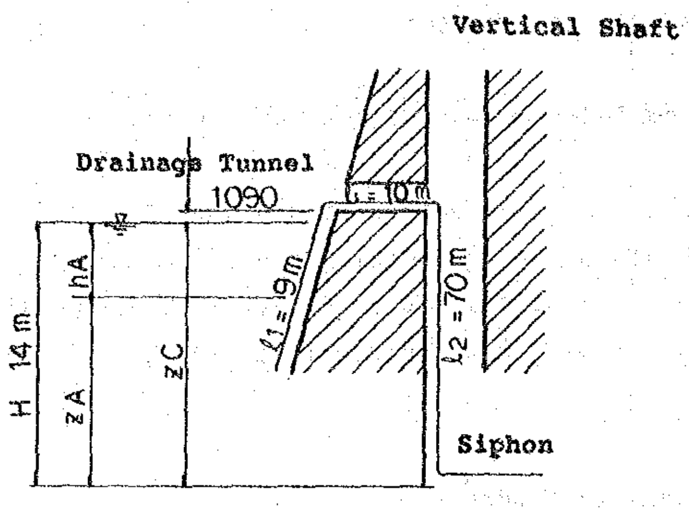


Fig. - 6.4 Layout of Drainage Siphon

Table - 6.6 Calculation Result of Hydrostatic Pressure of Siphon

	H	ZC	H'	N.H	Pc	
1090	15.0	15.0 m	0	4.64	-4.64	OK
1089	14.0	"	-1.0	4.33	-5.33	OK
1087	13.0	"	-2.0	4.02	-6.02	OK
1089	12.0	"	-3.0	-3.71	-6.71	OK
1086	11.0	"	-4.0	-3.40	-7.40	OK
1085	10.0	"	-5.0	-3.09	-8.09	OK
1084	9.0	"	-6.0	-2.781	-8.781	OK
1083	8.0	"	-7.0	-2.472	-9.472	OUT

Through the siphon process, the water level attainable by water drainage is approximately 6.2 m for the 1,090 m planned for the operations tunnel. Because 1.8 m remains until the elevation of the design pipe-end is reached, from the siphon exit an auxiliary pump will be utilized in order to expel the water. Accordingly, if the design elevation for the main drainage tunnel is 1,075.0 m, it will be possible to drain the crater lake.

6.5 Typical Section of Drainage Tunnel

(1) The diameter of the drainage tunnel section is to be 2.0 m in consideration of the possible width of the carrying out works of excavated material by trolley.

(2) The lining of the drainage tunnel should be performed by the lining plate. Because the lining plate is light, it is easy to carry and to perform works in the narrow tunnel, besides, it is possible to carry by manpower to the crater lake of Mt. Galunggung.

(3) The drainage capacity by tunnel is 10 m^3/s. As the result of the storage reservoir simulation in the case of drainage by tunnel after the inflow of a 50 year probable flood wave to the crater lake, the crater wall is not in danger of the collapse though the water level rises 3.00 m from the drainage tunnel elevation.

Upon consideration of the economic aspects and the actual installation execution of the standard cross section of the drainage tunnel, two proposals - 1) use of concrete and 2) use of a liner plate were compared in the decision.

6.6 Construction Method of Drainage Tunnel

As the water level in the crater lake is higher than the design tunnel elevation, it is difficult to enforce the drainage tunnel and the inlet at the crater wall because they are in water.

Besides, it is also difficult to construct the temporary coffering dike at the water lake side because it is in water. So, in order to drain the water lake, the method of syphon practiced at Mt. Kerut will be used. The method of construction is as follows. The process of work is as shown in figure.

The following describes the conditions necessary for the refrigeration process.

(1) Vertical Shaft Construction

In order to lower the water level of the crater lake by the siphon process, a vertical shaft will be constructed for work use. The walls of the crater lake are sharply sloped, so the position of the shaft has been chosen at the most gentle diagonal slope. From the entrance of the vertical shaft to the elevation of the Main Tunnel, there will be a distance of 90 m.

(2) Main Tunnel Construction

The Main Tunnel will be constructed from the Cibanjaran river to the vertical shaft. The construction of the tunnel will be conducted by drilling from the exit of the drainage tunnel. This will be an extension of 270 m.

(3) Construction of the First Stage Work Use Tunnel

From a position at the water level of the crater lake, the work use drainage tunnel will be constructed from the (vertical) shaft.

(4) Drainage by the Siphon Process

The thickness of the siphon pipe at the position of the work use drainage tunnel will be determined at 300 mm. The water will pour into the siphon pipe and by the removal of the air from the inside of the pipe, the lake will be drained through the siphon pipe.

(5) Construction of the Second Stage Work Use Tunnel

In the case that the water level of the Crater Lake reaches a level where it is no longer draining through the First Stage Work Use Tunnel, the construction of the Second Stage Work Use Tunnel will be undertaken, and the water level of the Crater Lake will be lowered through a siphon process.

(5) Installation of the Pipe End

When the water level of the Crater Lake reaches the level of the design pipe end, installation of the Pipe End segment will be undertaken, and the construction of the drainage tunnel will be completed.

6.7 Countermeasures for Construction of Drainage Tunnel

6.7.1 Construction in Intense Heat Zone

The temperature in the intense heat zone was about 90°C or so according to the result of boring. It is considered that the highest temperature is about 30°C to work in the tunnel by manpower. There were the examples of the construction in the intense heat tunnel in Japan, however, they were considerably. By these actual results, the temperature in the pit is required to be lowered in order to work in the tunnel. Though the ventilation by the cool is considered to lower the temperature in the pit, it cannot be lowered very rapidly by ventilation and the like, so that the way to lower the rock bed temperature by force is proposed. There is the freezing method commonly used for the example of enforcement to lower the rock bed temperature. This is the method to heighten the ground strength by freezing the weak layer. In this enforcement, the rock bed near 100°C will be cooled to 40°C - 50°C using this method to execute the works in tunnel. The capacity of the cooling machine is 70,000 kcal and 2 machines will be needed to use them by turns at construction. Besides, the more detailed investigation is required for the construction.

As a result of boring procedures, the internal ground temperature was found to reach approximately 90°C. Through the refrigeration process, the temperature will be lowered to 30°C, such that work operations will be possible, and the tunnel will be drilled.

The following considerations should be given for the design of freezing method.

(1) Design Conditions

1) Specification of freezing pipe
SGP 90A (diameter 101.6 mm thick of pipe 4.2 mm)

2) Specification of brine
liquid of chloridate calcium -30°C

(2) Calculation of Heat Quantity

The arrangement of same freezing pipe connected at the same spacing are shown as Fig. - 6.5.
The pict of freezing pipe is to be 80 cm.
The member of freezing pipe is to be 13.

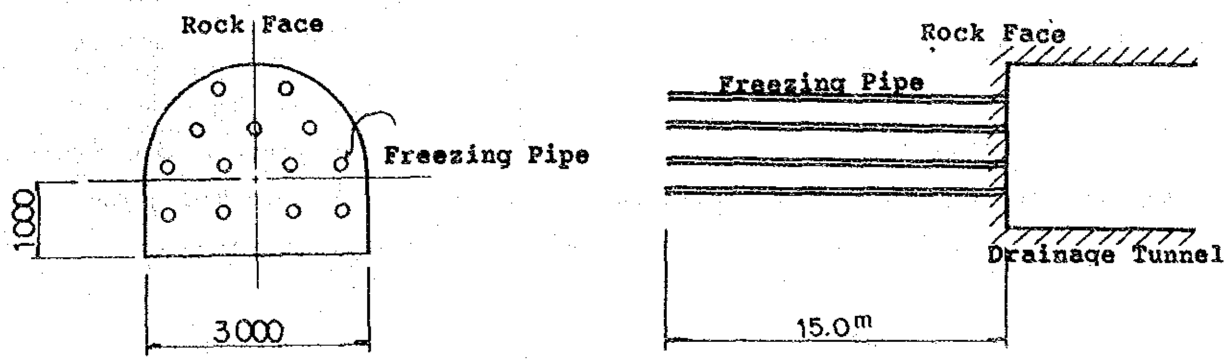


Fig. - 6.5 Arrangement of Freezing Pipe in Rock Face

The required capacity of cooling machine for freezing method can be expressed by the following formula:

$$\theta = AU\,\Delta\theta$$

where,

θ : capacity of cooling machine (Kcal/h)

A : surface of freezing pipe (m^2)

U : Overroll heat transfer coefficient ($Kcal/m^2h°C$)

$\Delta\theta$: variation of temperature freezing pipe

The overroll heat transfer coefficient change with time. The relation of the overroll heat transfer coefficient and time is shown in Table - 6.7.

Table - 6.7 Overroll Heat Transfer Coefficient

Time	U ($Kcal/m^2h°C$)	Heat Load (%)	Overroll Heat Transfer	Capacity of Plant
2 h	37.63	100	37.63	640,000
12 h	23.45	"	23.45	398,000
24 h	19.27	"	19.27	327,500
2 day	16.45	"	16.45	279,000
4	14.89	90	13.40	227,500
8	13.17	85	11.20	190,000
16	11.49	80	9.20	156,400
24	10.90	75	8.15	128,500
30	10.50	75	7.88	128,900
50	9.98	70	6.98	118,700

Upon consideration of the construction cost and construction period, the overroll heat transfer coefficient is to be 8.15 $Kcal/m^2h°C$. The capacity of cooling machine is estimated by the following formula:

$$\theta = A \cdot U \, \Delta\theta$$
$$= 62.25 \, m^2 \times 8.15 \times 120$$
$$= 60{,}880.5 \, Kcal/h \doteq 70{,}000 \, Kcal$$

where:

A : surface of freezing

$A = 13 \times \pi \times 0.1016 \times 15 = 62.25 \, m^2$

$\Delta\theta$: 90°C - (-30°C) = 120°C

The capacity of cooling machine is 70,000 Kcal and machine will be needed to use them by turns at construction. The flow of cooling machine in Fig. - 6.6.

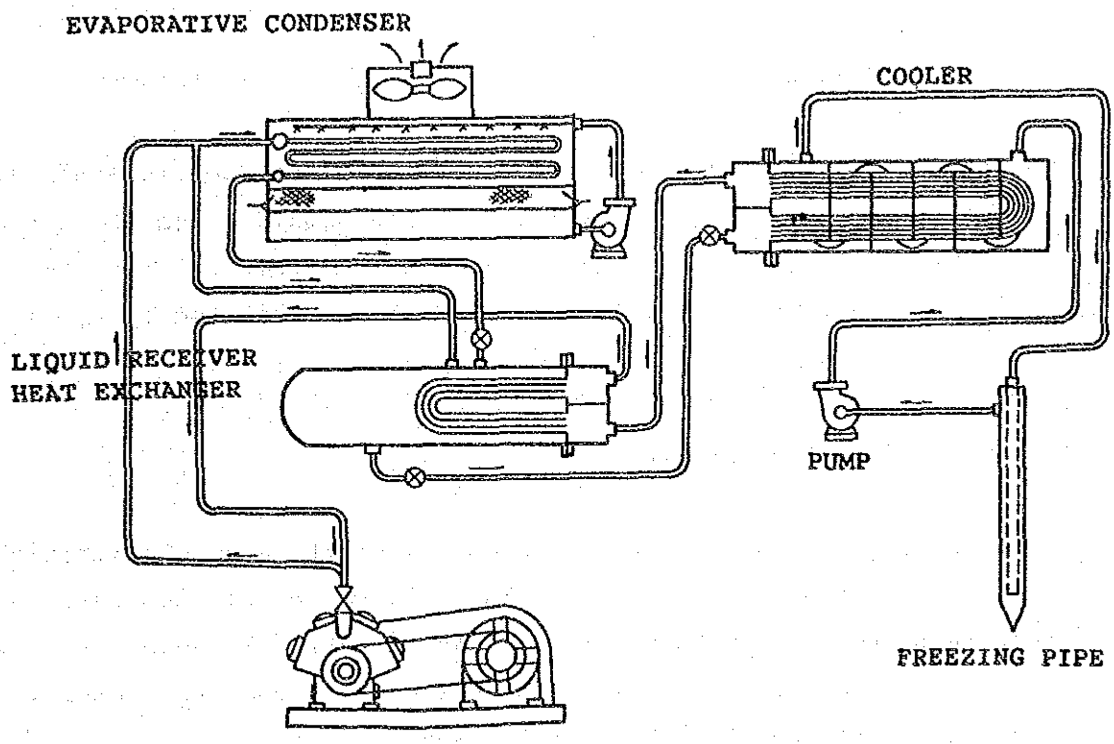


Fig. - 6.6 Flow of Cooling Machine

6.7.2 Countermeaure for Gas

The gas of Co_2 was found by the effluent gas analysis at the crater lake on the spot. The works in tunnel is discontinued when these gas spring up. The countermeasure for it is the ventilation. Fan is used for the ventilation. As for the presence of the gas layer as well as the leaky layer, it is desirable to certify previously by an advanced boring at rock face.

6.7.3 Excavation of Tunnel

The tunnel will be excavated by manpower and blast and materials will be carried out by manpower. The excavating section should be reinforced by the trench timbering as the crater wall is the loose sediment. When the excavation of the selected section is over, it will be executed by the lining plate.

6.7.4 Transportation of Materials

The site road will be constructed from the point where the materials can be carried in by truck at present to the point where the site road can be constructed. From that point to the crater lake at Mt. Galunggung, they will be carried in by manpower. Besides, as for the heavy goods, the materials will be transported to the site by the small-sized bulldozer.

7. Quantity of Construction Works

Each project unit from project unit 1 to project unit 4 was classified into the following facilities and quality of construction work was calculated for each facility.

1) Dike improvements
2) Maintenance of sandpocket (excavation, hauling and aggregate production)
3) Check dams
4) Consolidation dams
5) Revetment works
6) Crater lake drainage tunnel

Quantity of construction work for each alternative is shown in Table - 7.1 and Table - 7.2.

Table - 7.1 Quantity of Construction Works for each Project Area

Description	Unit	S.Ciloseh Area	S.Cikunir Area *1	Southern Slope Area	Crater Lake	Total
(1) Dike Improvement & Raising Lenth	m	3,801	11,631	-	-	15,432
Embankment Volume	m^3	19,956	256,110	-	-	276,066
(2) Riverbed Leveling						
Leveling Volume	m^3	-	1,370,000	-	-	1,370,000
(3) Riverbed Aggragation						
Aggradation Volume	m^3	-	3,932,000	-	-	3,932,000
(4) Excavation & Hauling						
Hauling Volume	m^3	394,000	630,000	-	-	1,024,000
(5) Aggregate Plant						
Number	site	-	1	-	-	1
(Manufacture Capacity)	ton/h	-	(140)	-	-	(140)
(6) Diversion Channel						
Length	m	-	1,500	-	-	1,500
Embankment Volume	m^3	-	147,705	-	-	147,705
Masonry Volume	m^3	-	19,125	-	-	19,125
(7) Check Dam						
Number	site	2	4	20	-	26
Excavation Volume	m^3	2,640	5,370	43,530	-	51,540
Masonry Volume	m^3	8,800	17,900	135,100	-	161,800
(8) Consolidation Dam						
Number	site	-	6	-	-	6
Dike Length	m	-	1,400	-	-	1,400
Embankment Volume	m^3	-	34,320	-	-	34,320
Excavation Volume	m^3	-	4,620	-	-	4,620
Masonry Volume	m^3	-	15,400	-	-	15,400
(9) Revetment						
Length	m	-	1,700	-	-	1,700
Excavation Volume	m^3	-	10,817	-	-	10,817
Masonry Volume	m^3	-	9,615	-	-	9,615
(10) Drainage Tunnel						
Length	m	-	-	-	655	655

Note)

*1 Alternative D for the sediment management works in Ciponyo I Dalam

Table - 7.2 Quantity of Construction Works for each Alternatives

Description	Unit	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E
(1) Dike Improvement & Raising Lenth	m	11,631	11,631	11,631	11,631	11,631
Embankment Volume	m^3	165,544	189,100	209,580	256,110	470,630
(2) Riverbed Leveling						
Leveling Volume	m^3	1,370,000	1,370,000	1,370,000	1,370,000	1,370,000
(3) Riverbed Aggragation						
Aggradation Volume	m^3	0	1,356,000	2,355,000	3,932,000	4,956,000
(4) Excavation & Hauling						
Hauling Volume	m^3	4,513,000	3,206,000	2,158,000	630,000	0
(5) Aggregate Plant						
Number	site	1	1	1	1	1
(Manufacture Capacity)	ton/h	(640)	(470)	(330)	(140)	
(6) Diversion Channel						
Length	m	0	1,500	1,500	1,500	1,500
Embankment Volume	m^3	0	85,500	103,020	147,705	288,720
Masonry Volume	m^3	0	14,895	16,196	19,125	25,947
(7) Check Dam						
Number	site	4	4	4	4	4
Excavation Volume	m^3	5,370	5,370	5,370	5,370	5,370
Masonry Volume	m^3	17,900	17,900	17,900	17,900	17,900
(8) Consolidation Dam						
Number	site	6	6	6	6	6
Dike Length	m	1,400	1,400	1,400	1,400	1,400
Embankment Volume	m^3	34,430	34,320	34,320	34,320	34,320
Excavation Volume	m^3	4,620	4,620	4,620	4,620	4,620
Masonry Volume	m^3	15,400	15,400	15,400	15,400	15,400
(9) Revetment						
Length	m	1,700	1,700	1,700	1,700	1,700
Excavation Volume	m^3	1,0817	10,817	10,817	10,817	10,817
Masonry Volume	m^3	9,615	9,615	9,615	9,615	9,615

8 Construction Schedule

8.1 Outline of Construction Schedule

As for disaster prevention facilities, there are facilities which must be operated immediately judging from the past disasters and those which may more expediently be operated extending over a long period of time, or, gradually, paying attention to the changing situation of runoff sediment and that of riverbeds. The period of implementation of this project was set for ten years hence, during which inflow sediment will be high in volume. The first five years were planned for the first stage of the work and the remaining five years for the second.

In the first stage, facilities urgently needed for disaster prevention and essentially important in terms of sediment disposal are to be adopted. In the second stage, the remaining facilities are to worked on. The contents of each period of work are shown below:

1) The first stage (5 years)

a) Raising and repair work for the dike at the sand pocket for the length of 15.5 km.
b) Sediment management works at the sand pocket.
c) Repair works for aggregate plant indispensable to sediment management.
d) Drainage works at crater lake (655 m).
e) Check dam work (15 sites).
(S. Cimampang .... 2, S. Cikunir .... 2, S. Cibanjaran .... 2, South Slope ... 9)
f) Ciponyo II consolidation dam works (4 sites), revetment works (1.7 km).

2) The second stage (5 years)

a) Sediment management works at sand pocket.
b) Construction of check dam on southern slope (11 sites).

8.2 Construction Plans

In laying out the construction plan, the availability of machinery and materials for construction at the site, their prices, builders' operational abilities, insurance for machinery, transport of materials into the jobsite, and other matters related to construction work were taken into account.

General construction materials, such as cement, timber, brick, stone, fuel, oil, are all available at the sight. Aggregate plant materials, tunnel lining and the like, however, have to be imported.

Workable days were decided to be 207 for earth work, 221 for aggregate plants, 300 for aggregate transportation. As regards construction methods for structures, in view of economical and employment conditions, the full use of stone, sediment, water and manpower were conclusively adopted.

With all of the above considered, the construction schedule was designed as shown in Fig. - 8.1.

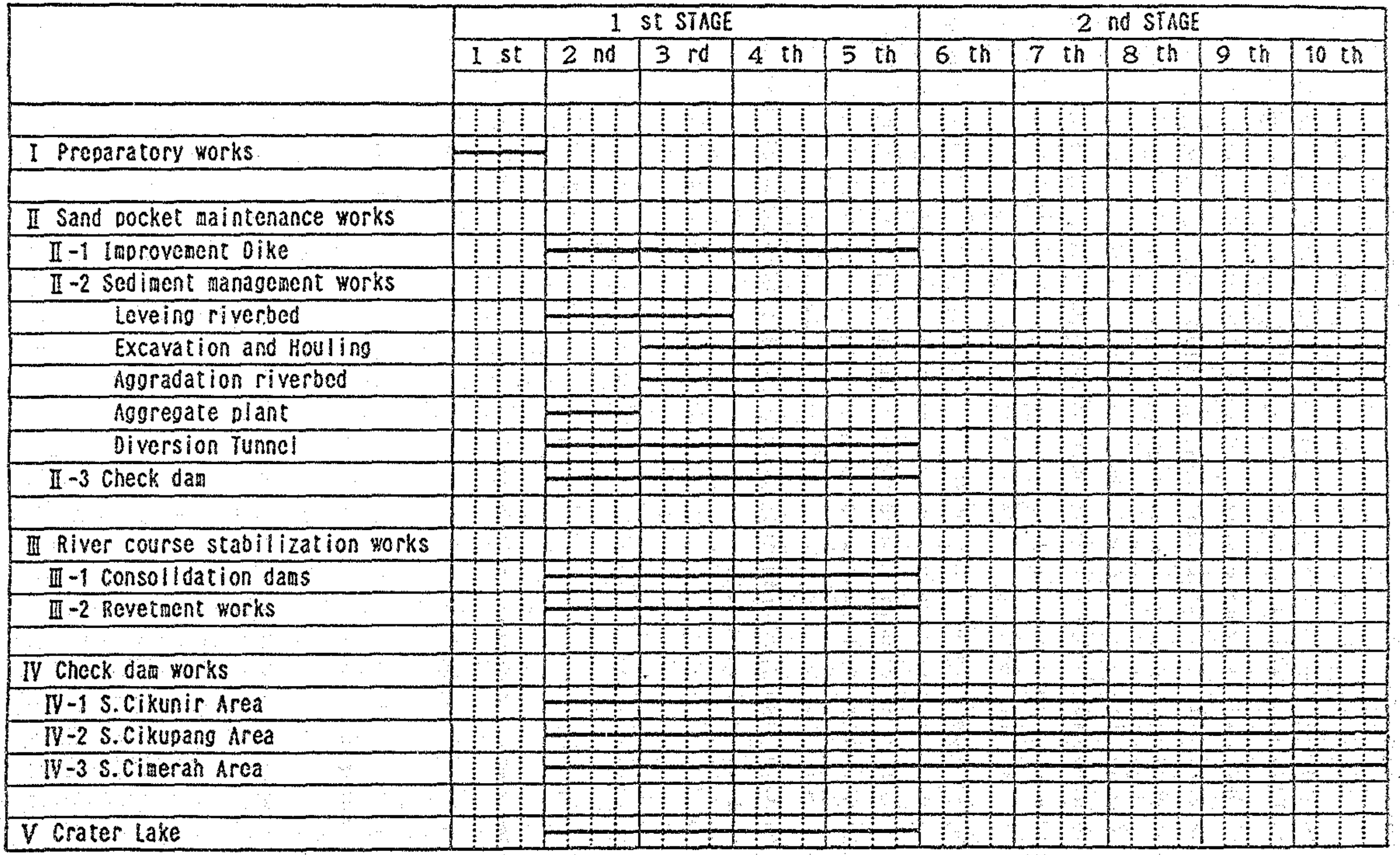


Fig. - 8.1 Construction Plan

THE REPUBLIC OF INDONESIA

THE FEASIBILITY STUDY OF THE DISASTER PREVENTION PROJECT IN THE SOUTHEASTERN SLOPE OF MT. GALUNGGUNG

S U P P O R T I N G   R E P O R T   (IV)

COST ESTIMATE

DECEMBER 1988

JAPAN INTERNATIONAL COOPERATION AGENCY

Supporting Report IV (Cost Estimate)

Contents

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1. General Conditions

1.1 General

The basic conditions and methods of cost estimate must be established in prior to the actual calculation for cost estimate.

The conditions for cost estimate in regard to the following items are described in the succeeding sections:

(1) Quantity of work and time schedule

(2) Constitution of cost estimate

(3) Construction cost
- 1) Direct cost
- 2) Indirect cost

(4) Government Cost
- 1) Land acquisition cost
- 2) Government administration cost
- 3) Engineering service cost
- 4) Contingency

(5) Price level and price escalation

(6) Exchange rate