REPUBLIC OF THE PHILIPPINES
NATIONAL POWER CORPORATION

FEASIBILITY REPORT ON AGOS RIVER HYDROPOWER PROJECT

APPENDIX E
PROJECT WORKS

MARCH 1981

JAPAN INTERNATIONAL COOPERATION AGENCY

REPUBLIC OF THE PHILIPPINES

NATIONAL POWER CORPORATION

FEASIBILITY REPORT ON AGOS RIVER HYDROPOWER PROJECT

APPENDIX E
PROJECT WORKS

MARCH, 1981

JAPAN INTERNATIONAL COOPERATION AGENCY

AGOS RIVER HYDROPOWER PROJECT

FEASIBILITY REPORT

Main Report		
Appendix	A	Hydrology and Reservoir Operation
Appendix	B	Geology and Construction Materials
Appendix	C	Power Study
Appendix	D	Optimization Study for the Development on the Agos River System
Appendix	E	Project Works
Appendix	F	Computer Outputs
Data Book	I	Topographic Survey
Data Book	II	Meteorology and Hydrology
Data Book	III	Geological Exploration
Data Book	IV	Construction Materials

APPENDIX E PROJECT WORKS

TABLE OF CONTENTS

			PAGE
CHAPTER 1		GENERAL	1-1
CHAPTER 2		DAM	2-1
	2.1	Type of Dam	2-1
	2.2	Dam Axis	2-2
	2.3	Stability Analysis of Agos Dam	2-2
CHAPTER 3		SPILLWAY	3-1
CHAPTER 4		DIVERSION WORKS	4-1
	4.1	Design Flood for Diversion Works	4-1
	4.2	Diversion Tunnels and Cofferdam	4-2
	4.3	Diversion Tunnel Routes	4-3
CHAPTER 5		WATERWAY	5-1
	5.1	Waterway Alternatives	5-1
		5.1.1 Design Criteria	5-1
		5.1.2 Alternative Description	5-2
		5.1.3 Construction Cost for Alternatives	5-3
		5.1.4 Selection of Layout	5-3
	5.2	Economic Diameter of Waterway	5-4
		5.2.1 Diversion Tunnel Utilized for Waterway	5-4
		5.2.2 Economical Diameter of Headrace Tunnel	5-6
		5.2.3 Economical Diameter of Penstock	5-6
CHAPTER 6		AFTERBAY WEIR	6-1

			PAGE
CHAPTER 7		CONSTRUCTION PLAN AND SCHEDULE	7-1
	7.1	Construction Schedule	7-1
	7.2	Mode of Construction	7-4
	7.3	Construction Plan and Method	7-5
CHAPTER 8		COST ESTIMATE	8-1
	8.1	General	8-1
	8.2	Assumptions and Conditions	8-1
	8.3	Cost Estimate	8-3

LIST OF TABLES

		Page
Table 2-1	Cases for Slope Stability Analysis	1
2-2	Material Property	1
2-3	Minimum Factors of Safety	1
Table 3-1	Results of Flood Routing	2
Table 4-1	Main Features of Alternatives	3
4-2	Construction Cost of Alternatives	3
4-3	Summary of Work Quantities for Tunnels Alternatives	3
Table 5-1	Main Features and Head Loss for Waterway Alternatives	4
5-2	Construction Cost of Waterway Alternatives	4
5-3	Economic Diameter of Headrace Tunnel	5
5-4	Economic Diameter of Penstock Pipe	6
Table 8-1	Major Work Quantities of Civil Work	7
8-2	Basic Prices of Material and Wages of Labors	9
8-3	Major Equipment and Plant	10

LIST OF FIGURES

Fig. 2-1 Stability Analysis of Agos Dam

Fig. 4-1 Diversion Tunnel and Cofferdam

4-2 Agos Dam Capacity - Area Curve for Diversion Flood Routing Study

4-3 Optimum Combination of Diversion Tunnel and Cofferdam

Fig. 5-1 Economic Diameter of Waterway

Fig. 7-1 Construction Time Schedule

7-2 Distribution of Excavated and Embankment Materials

7-3 Dam Embankment Volume Curve

LIST OF DRAWINGS

Dwg. 1. Alternatives of Spillway

2. Diversion Tunnels and Cofferdam

3. Alternative Layout of Diversion Tunnel

4. Diversion Tunnels, Left Bank Solution

5. Diversion Tunnels, Right Bank Solution

6. Alternative Layout of Waterway

7. Waterway, Alternative I and II

8. Waterway, Alternative III and IV

9. Afterbay Weir

A N N E X

TABLE OF CONTENTS

		Page
I	DIVERSION TUNNEL AND COFFERDAM	
	Routed Flood Water Level Computation	1
	I.1 Tunnel of 8 m Diameter	2
	I.2 Tunnel of 9 m Diameter	6
	I.3 Tunnel of 10 m Diameter	10
	I.4 Tunnel of 11 m Diameter	14
	I.5 Tunnel of 12 m Diameter	18
II	SPILLWAY ALTERNATIVE DESIGN	
	Routed Flood Water Level Computation	22
	II.1 Alternative I	23
	II.2 Alternative II	27
	II.3 Alternative III	32
	II.4 Alternative IV	36
III	ONE DIVERSION TUNNEL OPERATION	
	Routed Flood Water Level Computation	40
IV	DETAILED COST ESTIMATE	44
	IV.1 Agos Hydropower Project Cost Estimate	45
	IV.2 Summary of Unit Price	62

ABBREVIATIONS AND UNIT

JICA	Japan International Cooperation Agency
NAPOCOR (NPC)	National Power Corporation of Philippines
NK	Nippon Koei Co., Ltd.
PICOREM	Presidencial Inter-Agency Committee for re-study of the Marikina River Multi-purpose Project
NEA	National Electrification Administration
MOE	Ministry of Energy
MERALCO (MECO)	Manila Electric Company
MWSS	Metropolitan Waterworks and Sewerage System
PAGASA	Philippine Atmospheric, Geophysical and Astronomical Services Administration
BPW	Bureau of Public Works
ECAFE	Economic Commission for Asia and the Far East
CDM	Camp, Dresser and McKee International, Inc.
M + E (M & E)	Metcalf and Eddy, Ltd.
$	United States Dollars
₱ (P)	Philippines Pesos
¥	Japanese Yen
FC	Foreign Currency
LC	Local Currency
EIRR	Economic Internal Rate of Return
FIRR	Financial Internal Rate of Return
O & M	Operation and Maintenance
L.F.	Load Factor

AMSL	Above mean sea level
EL.	Elevation in m AMSL
W.L. (WL)	Water level in m AMSL
H.W.L. (HWL)	High water level in m AMSL
L.W.L. (LWL)	Low water level in m AMSL
F.W.L. (FWL)	Flood water level in m AMSL
D.F.W.L. (DFWL)	Design flood water level in m AMSL
P.M.F.W.L. (PMFWL)	Probable maximum flood water level in m AMSL
mm	millimeter(s)
cm	centimeter(s)
m	meter(s)
km	kilometer(s)
m^3	cubic meter
km^2	square kilometer(s)
ha	hectare
m^3/sec (cms)	cubic meter per second
m^3/sec·month	Water volume equivalent to the discharge of 1 m^3/sec for the duration of 1 month
kg	kilogram
t (ton)	metric ton
ℓ	liter
%	percent
^oC	centigrade
o	degree
N	north
rpm	revolution per minute

Hz	Hertz (cycles per second)
kcal	kilocalorie
kV	kilovolt
kVA	kilovolt ampere
MVA	megavolt ampere
W	Watt
kW	kilowatt
MW	megawatt
kWh	kilowatt hour
MWh	megawatt hour
GWh	gigawatt hour
V	volt
BTU	British Thermal Unit

CHAPTER 1

GENERAL

In the optimization study of Agos Hydropower Project, high water level is decided at EL.165 m and the installed capacity is selected to be 140 MW in total. In this Appendix, several alternative layouts for major structures have been studied from economical and technical points of view.

The work is directed towards:

a) Assessing any benefit to be derived from locating the axes of dam.

b) Comparing a series of alternative layouts of diversion tunnels, power facility and spillway and assessing any possibility and practicability of using the diversion tunnels as a part of spillway or power facility.

As the scale of the Project is determined through the project optimization study shown in Chapter 6 of Appendix D using the results of the above-mentioned study, detailed cost estimates are carried out in the following manner.

a) To establish the construction time schedule and analyze the critical path work.

b) To assess the construction method to complete the work within the required time.

c) To estimate the construction cost.

This Appendix presents the whole results of the study mentioned above in detail.

CHAPTER 2

DAM

2.1 Type of Dam

A dam type shall be decided from study results of technical and economical comparison of types prior to selecting an exact dam axis. In the early stage of the study, a concrete dam and a combined type dam were studied as alternatives to the rockfill dam, because

a) relatively big flood discharge may require costly diversion works and huge spillway,

b) the intensity and frequency of rainfall at the damsite may limit the rate of embankment construction,

c) foundation is suitable for both concrete and rockfill dams if excavated until certain depth.

As the field investigation proceeds, it has been made clear that the foundation rock is 40 m deep from the existing river bed.

A rockfill type is finally judged to be the best for Agos dam among those types (concrete, concrete and rock fill combined, and rockfill) from the results as follows:

a) The rockfill dam is the most economical among them. The difference of the construction cost is approximately US$93.3 x 10^6 between the filltype and concrete dams.

b) The concrete dam volume is estimated to be 4.45 x 10^6 m^3 in total. For construction of such big concrete dam in this country, some difficulties are anticipated in construction methods and techniques as follows:

   i) Difficulty of preparation of concrete and aggregate plants of such large scale

   ii) Difficulty of smooth material supply such as cement

   iii) Difficulty of inland transportation of construction plants such as large scaled ones

c) In design, there are many difficult technical problems to be cleared for the concrete dam, as compared with the fill dam, such as foundation treatment and dam cooling during construction.

2.2 Dam Axis

There were three alternative dam axes of Lines A, B and C proposed for Agos fill dam within one kilometer long of the downstream stretch from the confluence as mentioned above. Those three lines were selected by the results of JICA teams reconnaissance and study on the maps of 1/50,000 scale which were only available for study of the project at that time.

Geological investigation has been done on those three lines by seismic exploration and test drilling. On the other hand, topographical survey works have also been made on this dam area for the purpose of preparing maps of 1/1,000 scale, together with the geological investigation.

From results of rough study based on the aforesaid investigation and survey works, Line C was disqualified at the early stage of the investigation because of rather poor geological conditions and larger embankment volume required as compared with those of Lines A and B. Therefore, a dam axis was expected to be selected between both the Lines A and B. The dam axis between both the lines is to be selected owing to the topographical condition and as more upstream side as possible, because of almost the same geological conditions between both the lines.

The dam axis was finally selected at about 400 m downstream from the confluence and to be an alignment so as to close the river at a right angle.

2.3 Stability Analysis of Agos Dam

Slope stability of the dam is checked preliminarily by surface plate sliding method. According to the analysis, the slope 1:2.5 for upstream side and 1:1.9 for downstream side are obtained under the conditions that the factor of safety should be more than 1.2 and the coefficient of earthquake is 0.15.

As the work proceeds in the field, some materials characteristics have been made clear. Together with the values derived from experience, the slope stability was finally examined by slip circle method/1. Conditions of analysis are set up as shown in Table 2-1 and 2-2.

Safety factors computed and the corresponding critical slip circles are illustrated on Fig. 2-1. The minimum safety factors obtained are summarized in Table 2-3.

As shown in Table 2-3, the minimum safety factors of slopes computed are more than 1.5 under the normal condition and more than 1.2

/1: According to "Design Criteria for Dams" by Japanese National Committee on Large Dms.

even the seismic force is taken into account. It is concluded that the safety factor is more than the allowable in all studied cases.

The stability analysis so far carried out by conventional means shows that the proposed dam is structurally safe. The dam is, however, 172 meters in height which indicates that the static analysis by assuming a single coefficient of seismic acceleration to the whole dam body might be unapplicable and the shearing stress in the embankment near the abutment contact may be large. It is recommended that the dynamic stability analysis and stress analysis be performed in the detailed design. Therefore prior to the design, large scale tests should be carried out in order to clarify the design values used for the analysis.

CHAPTER 3

SPILLWAY

The spillway site is selected on the left abutment, from the topographical view point of abutment. The discharge released from spillway flip end is returned to the Agos river where the river course is suitably straight to the center of spillway. The spillway on the right bank requires huge amount of excavation which can not be accommodated into the dam embankment.

Several types of spillway such as full gated spillway, gated spillway with side channels, non gated spillway and morning glory type utilizing diversion tunnels are conceived. Of these non gated spillway and morning glory spillway are discarded, because the former requires long spillway crest and higher dam crest. With respect to the morning glory type, the discharge in tunnel should be of free flow. The capacity of morning glory is limited to the order of only around 500 m^3/sec. As described in later, it is more attractive if the diversion tunnel is used as a part of power tunnel. Hence there is no space for morning glory. The layout study for spillway is carried out these remaining two types, gated spillway and gated spillway with side channels.

For the purpose of comparison, following criteria are applied.

a) the design flood of 10,600 m^3/sec should be discharged without flood routing and the probable maximum flood of 17,300 m^3/sec be passed with the effect of reservoir retardation,

b) the free board is 3.0 m and 2.1 m for design flood and probable maximum flood respectively,

c) the width of side channel outlet is so designed to establish a control section that the flow in channel is sub-critical,

d) the velocity of spillway fore-bay (approach channel) is less than 4 m/sec and the height of spillway weir is more than one fifth of overflow depth.

The optimum size of the spillway is determined from the cost relationship between spillway capacity and dam height. Therefore, four alternatives as described below are studied.

Alternative I

The spillway is equipped with 10 numbers of 12.5 h x 12.0 W m radial gates. This is the original design of spillway used in pre-feasibility design and optimization study.

Alternative II

The spillway is equipped with 6 numbers of 12.5 h x 12.0 W m radial gates and 2 lines of 100 m long side channels at both sides.

Alternative III

The spillway is equipped with 6 numbers of 12.5 h x 12.0 W m radial gates and 2 lines of 185 m long side channels at both sides.

Alternative IV

The spillway is equipped with 4 numbers of 14.0 h x 14.0 W m radial gates and 2 lines of 210 m long side channels at both sides.

The details are shown in Dwg. 1.

The flood and the flood routing studies are carried out for the alternatives. The results are shown in Table 3-1, while the computer output are summarized in Annex II.

The construction costs of alternatives are summarized below,

Construction Cost of Dam and Spillway

Alternative	I	II	III	IV
Dam crest el. (m)	170.3	173.6	171.9	172.0
Spillway gated	12.5Hx12.0Wx10N	12.5x12x6	12.5x12x6	14x14x4
non-gated	-	100m x 2	185 x 2	210 x 2
Cost (million US$)				
Dam	109.81	113.25	111.46	111.73
Spillway (civil)	5.59	6.51	7.56	6.74
" (metal)	4.08	2.45	2.45	2.45
Total	119.48	122.21	121.47	120.92

As having been expected, the least cost alternative is the all gated type spillway, the second one is the alternative IV. The difference in cost is 1.4 million US$. It would be preferable to avoid all gated type spillway for the safety of dam. It is recommended that the spillway is designed to have a part of non-grated portion so that the gate operation, which leads sometimes artificial floods or overtopping caused by failure to open, is minimized according to the recent tendency of spillway in Japan. A computation of reservoir rise has been carried out if, though too severe and too pessimistic, all gates will not open for some reasons. The result of reservoir rise is as follows, in which the flood of 200 year return period is used,

	Reservoir water surface
Alternative I	EL.173.45 m
Alternative IV	EL.169.20 m

The alternative IV can manage to discharge the flood without overtopping the dam crest while the alternative I jeopardizes the dam.

The alternative IV is finally selected.

CHAPTER 4

DIVERSION WORKS

The actual embankment of Agos dam will take about 4 years. Therefore it is necessary to divert the river discharge and to protect the site against floods during the period.

The selection of tunnel route which affects the selection of waterway route will be solved in combination with the waterway.

4.1 Design Flood for Diversion Works

The flood data examined were primarily those available at Banugao gauging station, which has the only and the longest daily discharge records. Of these, the recorded maximum flood on Nov. 20, 1966 has the peak discharge of 6,070 m^3/sec at Banugao (C.A = 911 km^2) which corresponds to once in 50 years frequency. The probability analysis of yearly maximum flood was also carried out. Table below shows the result of the analysis.

Probability flood of the Agos river at Banugao (911 km^2)

Return period in year	Discharge in m^3/sec	Return period in year	Discharge in m^3/sec
5	3,150	30	5,340
10	4,020	50	5,950
20	4,860	100	7,140

It is one of the method to apply the probability analysis for the selection of design flood for diversion work. The probability W which does not occur flood Q during construction period N can be expressed.

$$W = (1 - \frac{1}{T})^N$$

T: Return period of Q

W: The probability W is usually taken more than 85% for fill dam

The result is tabulated as follows

T: Return period N: Const. period	10	15	20	25	30	35	40
4	66	76	82	85	87	90	92
5	59	71	77	82	85	87	88
6	53	66	73	78	82	84	86

Judging from the actual embankment period of 4 years for Agos dam, a 25 years return period may suffice the criteria, however taking into account the magnitude of the work and unforeseenable flood risk exceeds the computed return period which may result in the delay of the work, a 30 years return period is adopted. A 30 years flood of 5,340 m^3/sec at Banugao is naturally converted to the damsite by the square root basin area ratio. Therefore the design flood for diversion work is decided at 5,210 m^3/sec.

4.2 Diversion Tunnels and Cofferdam

The diameter of diversion tunnels is the function of the height of crest elevation of upstream cofferdam. Therefore it is necessary to select the least cost combination of tunnels diameter and the crest of cofferdam. Diameters of tunnels to be compared are D - 8.0 m, 9.0, 10.0, 11.0 and 12.0 m. Two tunnels are selected.

There is no great difference in geological point of view between both banks. So for the purpose of comparison, one tunnel in each bank is temporarily selected.

The upstream cofferdam is mostly incorporated in the main dam, however due to the excavated slope stability of river deposit at dam foundation, the crest of upstream cofferdam is so located that the extended line of downstream slope of cofferdam should not intersect the excavated slope. Hence the less the diameter of tunnel is, the longer the length of tunnel and the higher, of course, the crest of cofferdam are.

With the diameter and the length of tunnel, the computation of flood routing was carried out to obtain the routed flood water level. The crest elevation of cofferdam is decided having the free board more than one meter above FWL. The crest elevation of each alternative is shown in Table 4-1. The results of flood routing in each alternative are shown in Annex I. The embankment volume of cofferdam is read from the interpolation of Fig. 4-1 which is computed for crest EL.80, 90, 100 and 110 m.

The construction cost used for the comparison consists of the cost of diversion tunnels, diversion closure gates and cofferdam. The unit prices derived for pre-feasibility study for Agos Project are used for comparison. The result is shown in Table 4-2 and Fig. 4-3.

Therefore the two-diversion tunnels of 9.0 m diameter with the cofferdam crest elevation 93.0 m is selected for the diversion scheme.

4.3 Diversion Tunnel Routes

There are basically two alternatives for diversion tunnel routes. The one is the right bank route and the other is the left bank. It would be preferable to arrange tunnels at one side for the construction convenience particularly the right bank where existing road is available. However in combination with the power waterway and powerhouse selection, one tunnel each at both banks can be conceived so as to utilize a part of diversion tunnel as the power tunnel. Therefore three alternative routes are examined.

Right bank solution

Inlets are selected just upstream toe of upstream cofferdam. Outlets also selected at downstream of downstream cofferdam. The route was obliged to be shifted to mountain side due to small creeks across the tunnels route. The length of tunnel No.1 is 829 m and No.2 of 934 m. The tunnels have two bends at angles 51 degrees and 47 degrees. The rock is expected to be sound enough for tunnels excavation except the portion of thin earth covering and minor faults.

Left bank solution

Inlets and outlet are provided at the opposite side of the right bank solution. The culvert was adopted for the outlet portion due to thin earth covering. The tunnel No.1 has 755 m in length and No.2 of 705 m both including the culvert portion. The tunnel No.1 has two bends of 10 and 40 degrees. The tunnel No.2 has also two bends of 34 and 40 degrees. As far as the rock condition, no difficulty may be expected except the outlet portion.

Right and left bank solution

The tunnel No.1 (at left bank) is basically the same as the tunnel No.1 of left bank solution, but the route is slightly shifted towards north as to accommodate the powerhouse and to shorten the length of waterway. The length is 816 m. The tunnel No.2 (at right bank) is the same as the tunnel No.1 of right bank solution.

Based on the conditions mentioned above, cost comparison was carried out. Unit prices used for the analysis were of early 1980's price level. As shown below, the cost required for the works is

Reight bank solution	12.49×10^6\$
Left bank solution	10.43×10^6\$
Right/Left bank	11.68×10^6\$

The final selection will be discussed in the next paragraph.

Construction Cost

Unit: million US dollars

	Right	Left	Right/Left
Tunnel excavation (m^3)	5.29	4.38	4.93
Tunnel concrete (m^3)	3.32	2.75	3.09
Plug concrete (m^3)	0.42	0.42	0.42
Reinforcement bars(ton)	1.39	1.15	1.29
Others	2.08	1.74	1.95
Total	12.49	10.43	11.68

CHAPTER 5

WATERWAY

5.1 Waterway Alternatives

Four alternatives are conceived for the waterway route and the powerhouse. Other alternatives such as the powerhouse far downstream from the dam and the underground powerhouse are not cost competitive, judging from the head loss lost in the waterway and the geological condition.

5.1.1 Design Criteria

Dam relevant work

The dam and the spillway are fixed at the location as selected. High water level and low water level are EL.165 m and EL.128 m respectively. The selection of diversion tunnels route varies according to the waterway route.

Waterway relevant work

The installed capacity of power plant is 140 MW. The rated discharge is the function of effective head (rated water level - tail water level - head loss). The rated water level is fixed at one half of available drawdown. Tail water level is fixed at EL.41.5 m for Alt.-I, II and III and EL.39.4 m for Alt.-IV. The lost energy for 50 year operation discounted to the year of commission due to head loss at waterway is evaluated as the cost. The results are shown in Table 5-1.

The sill elevation of intake is decided keeping the water depth twice of headrace tunnel diameter from the LWL so as to avoid the incursion of air into the tunnel. Although each alternative may have the different economical diameter for headrace tunnel, for the purpose of comparison the diameter of tunnel is decided based on the velocity in tunnel 5.5 m/sec. Judging from the elevation on which the headrace tunnels run having deep covering from the surface, there is no difference in tunnel excavation and lining thickness between right and left bank waterway route. The thickness of lining is fixed at 1/12 of tunnel diameter.

As for surge tank, overflow chamber type surge tank is selected so as to avoid the simultaneous occurrence of water hummer and surge rise for Alt.-I, III and IV. Size of structures is derived from Thoma's formula. With respect to Alt.-II, no surge tank is provided. In order to simplify the computation and the comparison, the penstock line is selected of underground type. The penstock diameter is decided according to the water hummer within the range less than 40 % at the closing time of turbine (5 seconds) based on Allievi's formula for Alt.-I, III and IV. As for Alt.-II, the closing time of turbine 7 seconds is selected. The cost incremental of turbine and generator due to extension of closing time is relatively small. The rock cooperation is considered for the computation of thickness of pipe for Alt.-I. Recent drilling work along the penstock route for Alt.-IV revealed that the rock condition is not so favorable. Therefore the rock cooperation is not considered for Alt.-III and IV. The lining thickness of penstock tunnel is fixed at 0.65 m taking the steel pipe installation convenience into account.

5.1.2 Alternative Description

Alternative I

Waterway and powerhouse is located at the left bank. The intake is located on a small ridge in the back of dam abutment. A headrace tunnel of 6.2 m diameter and 270 m in length is connected to the surge tank. Then an underground penstock line of 6.3 m diameter with 435 m in length is provided. A powerhouse is located near the river bed in between diversion tunnel No.1 (next to spillway) and downstream cofferdam. The effect of recurrent flow released from spillway flip end is considered to be small.

Alternative II

Waterway and powerhouse is located at the left bank. The layout is intended to utilize the diversion tunnel as long as possible which will not be used after the completion. The layout of powerhouse and tailrace is the same as that of Alt.-I. The intake of tower type is close to slope end of dam. A headrace tunnel consisting of inclined portion in upstream side and horizontal portion in downstream side of diameter 6.1 m and 180 m in length leads to diversion tunnel No.1. A penstock pipe of diameter 6.1 m is provided from the upstream side of tunnel plug. The downstream side of penstock is embedded in the penstock tunnel. The length of penstock line is 365 m.

Alternative III

Waterway and powerhouse is located at the right bank. The intake is situated in a small creek about 200 m upstream from dam axis. Upstream portion of headrace tunnel about 160 m runs toward south-east direction to maintain sufficient earth covering then bends towards east direction with interangle 50^{o}. A surge tank is provided at EL.175 m. A penstock line of diameter 6.3 m and 420 m in length is of underground type. A powerhouse is located near the existing road, about 100 m downstream of cofferdam and 150 m far from spillway plunge pool. The river bed fluctuation due to spillway released discharge might affect the tailrace.

Alternative IV

The layout is adopted in the prefeasibility study and optimization study. The waterway and a powerhouse is set at right bank. The intake and one half of upstream part of headrace tunnel is the same as those of Alternative III. A headrace tunnel of diameter 6.3 m is 750 m in length. A shaft with chamber type surge tank is provided at the end of headrace tunnel. A penstock line of diameter 5.4 m and 275 m in length is of underground type. A powerhouse is set at the mountain end because the terrace in front of powerhouse requires huge excavation for powerhouse. The river bed fluctuation and flood itself released from spillway flip might be small.

5.1.3 Construction Cost for Alternatives

Unit prices used for the analysis are the ones derived for the optimization study based on early 1980 price level.

The work quantities for intake, powerhouse and tailrace are derived from emperical formulas and graphs, while those for headrace tunnel, surge tank and penstock line are precisely computed from the drawings. Unlisted cost such as grouting ranging from 15 % to 22 % to each work division is added. The results of construction cost for alternatives is shown in Table 5-2.

5.1.4 Selection of Layout

As for the diversion tunnel route, because of small space available for powerhouse between dam and spillway, waterway Alternative I and II be chosen the diversion tunnel at each bank. As for waterway Alternative III and IV can enjoy the shorter tunnel length of left bank solution. The combination of waterway and diversion tunnels is as follows,

Waterway Alternative	I	II	III	IV
Diversion tunnel	Right/Left	Right/Left	Left	Left

The total cost of each alternatives consisting of the cost of waterway, diversion tunnels and the lost energy valued in the present worth cost for 50 years operation is as follows. The cost of energy lost is evaluated in comparison to Alternative I.

Unit: Million US Dollars

Waterway Alternative	I	II	III	IV
Cost of lost energy	0	0.02	0.29	0.47
Cost of D/Tunnels	11.68	11.68	10.43	10.43
Cost of waterway	16.68	14.05	18.17	16.43
Total Cost	28.36	25.75	28.89	27.33

The results show that the minimum cost alternative is the Alt.-II having a small difference to Alt.-IV. However, the Alt.-IV conceives the more cost increasing factor in the headrace tunnel construction, because the fault, though expected thin, is running parallel to the tunnel route. The result of recent drilling work along penstock line may be one of the indication of the unfavorable rock condition along the headrace tunnel. While the Alt.-IV has an advantage of independent construction of waterway. On the other hand the Alt.-II takes a risk of one diversion tunnel during the construction of penstock line. The operation of one diversion tunnel during one year before the completion is carefully studied from the view point of construction schedule as well as the water level risen by the design flood. The resulted flood water level is EL.103 m which is far low from the expected elevation of dam embankment (refer to Annex III). Thus having examined all the factors incorporated in the work, the waterway Alternative II and the diversion tunnel at each bank is selected.

5.2 Economic Diameter of Waterway

In Section 5.1, the alternative layout study has been carried out, then there the alignment of diversion tunnel at each bank and the waterway at left bank of which a part of diversion tunnel utilized for waterway is adopted. At the final stage, the alignment is slightly modified so as to utilize the diversion tunnel as much as possible.

5.2.1 Diversion Tunnel Utilized for Waterway

The headrace tunnel of 146 m from an inlet of intake is connected with the diversion tunnel. Utilizing the diversion tunnel 80 m in length as headrace tunnel, it leads to steel penstock pipe. It is necessary to clarify whether it is beneficial to cover with concrete so that the diameter of headrace tunnel maintains as the same from top to end or to make it remain as it is. The evaluation is made based on the amount of lost energy on both alternatives.

Covering with concrete

The loss head in this 80 m section is only due to friction which is expressed based on Manning's formula as follows,

$$h_f = \frac{124.5 \cdot n^2 \cdot L}{D^{4/3}} \cdot \frac{V^2}{2g} = \frac{124.5 \cdot n^2 \cdot L}{2g \cdot A^2} \cdot \frac{Q^2}{D^{4/3}}$$

where n: 0.014

L: 80 m

g: 9.8 m/sec^2

Q: 163 m^3/sec

D: 6.2 m

Hence h_f = 0.171 m

Gradual expansion and contraction

The transition of 16 m in length from headrace tunnel to diversion tunnel is provided to minimize the loss of head. The transition from diversion tunnel to penstock is also provided. The head loss due to the gradual expansion, gradual contraction and the friction losses is as follows,

$$h_{ge} = 0.06 \cdot \frac{V^2}{2g} = 0.091$$

where 16 m in length transition is required

$$h_{gc} = 0.004 \cdot \frac{V^2}{2g} = 0.005$$

$$h_f = \frac{124.5 \cdot n^2 \cdot L}{D^{4/3}} \cdot \frac{V^2}{2g} = 0.035$$

Total loss is $h_t = h_{ge} + h_{gc} + h_f = 0.131$

As seen from the result, the head loss of the former is bigger than the latter. The cost of the latter is cheaper than that of the former. Therefore the diversion tunnel is utilized for waterway providing the transitions on both sides.

5.2.2 Economical Diameter of Headrace Tunnel

The smaller the diameter of tunnel is, the lower the construction cost is, while the larger the loss of head is. The incremental loss of head 50 year operation discounted to the year of commissioning is valued into the cost. The total cost consisting of construction cost and the present worth cost of lost energy should be minimum.

The result is shown in Table 5-3 and Fig. 5-1 in which the economical diameter of penstock is 6.8 m and its velocity is 4.54 m/sec.

5.2.3 Economical Diameter of Penstock

The procedure of economical diameter of penstock line is principally the same as the one of headrace tunnel.

Penstock line of 350 m in length is designed to be laid between the tunnel plug and the powerhouse, of which upper portion of 50 m is embedded in the tunnel plug, the middle portion of 206 m is placed in the diversion tunnel and the remaining lower portion of 94 m in length is embedded in the penstock tunnel.

For the purpose of comparison, the closing time of turbine is fixed at 7 seconds keeping the pressure rise of water hummer is less than 45% of static pressure. The water hummer is computed based on Alievis formula, but the rated discharge is conservatively adopted for the computation of water hummer. The thickness of pipe shell is determined that the hoop stress should be less than the allowable stress of 2,400 kg/cm^2. The pressure rise due to water hummer is reflected to the increment of thickness of pipe shell. The result is shown in Table 5-4 and Fig. 5-1. The economical diameter of penstock is 6.1 m and its velocity is 5.57 m/sec.

CHAPTER 6

AFTERBAY WEIR

In the pre-feasibility stage, the afterbay weir was designed to generate energy of around 78 GWH by 30 MW installed capacity to harness the remaining head as well as the regulating function of discharge released from Agos powerplant. However the field investigation proceeds, it has been revealed that the foundation rock at the weir site is 40 to 50 m deep from the river bed. So the plan of constructing a high dam equipped with a powerplant was abandoned.

Judging from the population and the habit of the people near-by the river, the necessity of the re-regulating function of discharge released from Agos powerplant may be very little because of the following reasons,

a) As the installed capacity of Agos powerplant is 140 MW, the plant will be operated for more than 12 hrs a day on an average before Kanan water supply project is developed. Even the severest drought year, the plant can be operated for more than 7 hrs a day. So the contineous discharge flows in the day time.

b) Means of transportation is only by canoe across the river between Infanta and General Nakar. A computation of unsteady flow is carried out to verify a rising speed of water level at Banugao (14 km downstream from Agos powerplant) from 0 to 163 m^3/sec within 1 minute without the provision of the afterbay weir. According to the results, the fluctuation of water surface is 1.05 m (EL.6.52 m to EL.7.57) and the rate of rising is 5 cm per minutes. As the annual mean run-off of the Agos river at Banugao is around 120 m^3/sec whose water level is at EL.7.39 m, the difference from annual mean water level is minus 0.85 m and plus 0.18 m which may be tolerable for the transportation across the river. Furthermore, it takes about two and a half hours for the released discharge to reach Banugao. There is an ample time to prepare the change. An alarm system will help this purpose.

c) There are two irrigation intakes supplying water for around 1,450 ha including the area of 200 ha to be extended in future. According to 26 year discharge records at Banugao the second driest nothing discharge in 15 - 52 m^2/sec. As the remaining area downstream of the Agos dam in 44 km^2, the discharge of 0.75 m^3/sec is anticipated.

Although above findings are to be checked in the detailed design stage, the afterbay weir might be excluded in the work.

Followings are the minimum requirement of the after bay weir if the Kanan water supply project is qualified and developed and the water is fully diverted to Metro Manila.

Storage requirement for regulation

After the water at Kaliwa and Kanan is fully diverted to Metro Manila, Agos powerplant is operated for 5.27 hrs/day in case of the severest year during 26 years records. The required regulating capacity is thus obtained to level the discharge to 24 hrs and adding the discharge of 3.4 m^3/sec of remaining area between the dam and afterbay weir. The capacity is 2.26 x 10^6 m^2. No dead space is provided, since the sediment is so designed to be flushed out from gates.

Design site selection

Two (2) sites were studied as the afterbay weir site at the debouchment of the Agos river to the Infanta alluvial plain at village of Magsay-say during the early stage of investigation. One is just near the Magsay-say village (Lower site), the other is about 1.5 km upstream from the lower site (Upper site). Topography and the dam height to store the required capacity preferred the upper site. Furthermore the geological condition of the upper site is also better than that of lower one. Therefore the upper damsite is selected for afterbay weir site.

Judging from the topography, geological condition and the diversion work during construction as well as the magnitude of design flood for spillway, a concrete dam is selected.

Design of structures

The weir consists of two sections, one is overflow section and the other is gate control section. Arrangement of both sections are coinciding with the present riverbed elevation not to reduce the flow area of which the Agos river originally has at the weir site. The weir is designed to have total capacity for the passage of design flood discharge of 8,500 m^3/sec which corresponds to 200 years flood.

The overflow section is of 130 m width for passing the discharge of 3,000 m^3/sec which is 35 % of the design flood. The overflow crest of the weir will be set at EL.25.5 m determined to secure the necessary regulating capacity aforementioned.

A floating type is selected at this section from the economical point of view and it has both rear and fore aprons to secure the stability against seepage flowing under the foundation.

The weir has also a gate control section at leftside half of the river which is provided with 4 gates of vertical roller type of 20 m x 5.5 m and the entrance sill will be set at EL.15.0 m, 2.0 m above present riverbed. Then the gated portion has a capability to flush away 5,500 m^3/sec which is about 65 % of the design flood discharge. (refer to Dwg. 9) These gates will retain water up to EL.25.5 m and control the peak discharge from Agos powerplant by its partial operation.

Piers of gated portion are designed to be supported by concrete bearing piles of cast-in-place. Geological condition of river deposit shows the difficulty of excavation and erection of piles, however it is possible to construct by newest construction method, for instance by a percussion method.

When a flood occurs, control gates shall be open before the flood discharge reaches to the site.

Between gated portion and overflow portion, construction of a training wall of 20 m in height, 100 m in length will be provided for passing the discharge steadily from two portions.

CHAPTER 7

CONSTRUCTION PLAN AND SCHEDULE

7.1 Construction Schedule

The time target of the project completion is planned to be in the end of 1988. In order to achieve this target, the following shall be kept as a basic schedule in the process of the project implementation.

a) Engineering services:

commencement of detailed design: in October 1981

b) Tender call and contract for diversion work: for 10 month from April 1982

c) Preparatory works

commencement: in the end of 1982

d) Main construction works

commencement: in the early part of 1983

completion: in the end of 1988

e) Commissioning of commercial operation of power generation:

in the early part of 1989

During the tender works, a contract for the diversion tunnels and coffer dams shall be concluded so as to start the construction work in the early part of 1983, considering that the main dam works be commenced in the middle of 1984.

Fig. 7-1 shows the construction time schedule.

(1) Commencement of Engineering Services

The engineering services shall be commenced from detailed design and investigation therefor including preparation of tender documents as early as possible. However, its commencement will be very difficult before October 1981, because at least a half year from April 1981 will be required to follow the process of a contract for the engineering services even if a foreign loan be available for the project from that April and an engineering consultant is selected by a direct order.

(2) Works on a Critical Path

As described above, the project works are planned to be started from the detailed design and investigation by an engineering consultant in October 1981 and to be finished by a contractor's test running of the generating equipment in December 1988. The following works are expected to be on a critical path in the project implementation schedule.

a) Preparation of tender documents for diversion tunnels and cofferdams: for 6 months from October '81.

Note: Government's (or a bank's) approval for the tender documents is done during the above period of 6 months. Furthermore, detailed design on the tunnels and cofferdams shall be made prior to the documents preparation. The design work period is also included in this 6 months.

b) Tendering works from tender call to contract for diversion tunnels and cofferdams: for 10 months from April '82

c) The contractor's mobilization and transportation of construction goods for diversion tunnels and cofferdams: for 3 months from February '83

d) Commencement of diversion tunnel No.1 works: in the biginning of May '83

Note: Open excavation for the tunnel No.1 can be done by a local contractor jointed with the main contractor (foreign) for about 3 months from February '83 immediately after the contract. Therefore, the above-mentioned work commencement is for tunnelling operation.

e) Construction works of diversion tunnel No.1:

for 14 month from May '83

f) River diversion: in the end of May '84

g) Main cofferdam: for 6 month from April '84

Note: During the construction period of the main cofferdam, the river diversion of item f) is scheduled to be carried out on the aforesaid date. Diversion tunnel No.2 shall also be completed by the end of September '84 before the rainy season starts.

h) Construction works of inspection gallery at the river bed:

for 7 months from October '84

i) Core embankment of the main dam:

for 41 months from May '85

Note: In the plan, three (3) months from October in wet season for each year are not available for core and filter embankment works.

j) Plug concrete works for diversion tunnel No.1:

for 3 months from April '87

Note: Diversion closure of the tunnel No.1 is planned to be done at the beginning of April '87. The dam embankment shall be heightened up to EL.110.0 m by that closure date, so that a flood of 5,210 m^3/sec can pass only through the diversion tunnel No.2.

k) Installation works for the penstock in the diversion tunnel No.1 including its tests:

for 14 months from July '87

Note: Installation works for the penstock is planned to be commenced at the beginning of June '87, when approximately 50 % of the plug works for the diversion tunnel No.1 is completed for about one (1) month.

l) Tests for the generating equipment:

for 4 months from September '89

Note: Tests for the generating equipment can be made immediately after installation work completion of the penstock including the metal tests.

m) Project work completion: in end of December '88

Other works have sufficient time allowance in their schedules. For instance, a work schedule on the generating equipment has one (1) year allowance, that is, tender call for the generating equipment can be done between January '83 and February '84 after preparation of the tender document.

(3) Preparatory and Main Civil Works

Prior to commencement of the main construction works in early 1983, the preparatory works such as construction of access roads and temporary buildings shall be carried out by local contractors for about two years from September '82.

The main civil works are planned to be carried out from commencement of open excavation of the diversion tunnel No.1 till completion of the main dam embankment for approximately 6 years. Five (5) years are scheduled for the period of the main dam embankment works.

(4) Generating Equipment and Metal Works

The schedules for the generating equipment and metal works have sufficient time allowance for the works from preparation of the tender documents to completion of the installation works as mentioned above.

With regard to the generating equipment, it will take approximately 3.5 years from the contract till completion of the installation works including tests. That installation works including tests will be performed for 1.5 years from the middle of 1987, except for the works for the draft tubes.

Construction of the transmission line and sub-station is scheduled to be carried out for two (2) years from May '86. The work shall be completed by August '88 before starting the generating equipment tests.

As for the metal works, the installation works are scheduled to be started firstly of the spillway gates in July '86. All the installation works including tests are planned to be completed in August '88.

Tender documents for both the electrical and metal works are to be prepared in the detailed design works by September '83. There are, therefore, time allowances of one (1) year or more between dates of tender documents completion and tender work commencement for both the electrical and metal works.

7.2 Mode of Construction

All the project works shall be fundamentally executed by contractors selected by means of open tender. Some contractors for special works shall, however, be selected by selective tender or direct order in view points of limited time schedule and/or special work conditions. The followings are modes of construction for the project works.

	Work item	Contents	Mode of construction
1.	Preparatory work	Access roads, temporary building, water & power supply systems etc.,	Local tender

Work item	Contents	Mode of construction
2. Diversion tunnels & cofferdams	Diversion tunnel Nos. 1 and 2, and upper and downstream coffers	International selective tender
3. Main civil works	Main dam, spillway, power station, and waterway	International open tender
4. Generating equipment	Turbines, generators, draft tubes, overhead crane, switchyard equipment and others	International open tender
5. Transmission line	Transmission line and sub-station	International open tender
6. Metal works	Spillway gates and stop-logs, intake gate and trash-rack, tailrace gates and gantry crane, diversion gates, and penstock	International open tender
7. Engineering services	Detailed design and survey, and construction supervision	Direct order

Concerning the diversion tunnels and cofferdams of item 2, international selective tender is recommended in order to shorten a period of selecting a contractor because of the limited period of only 10 months. At least 5 bidders will be better to be selected for the tender. (Note: International tender including pre-qualification will take more than 6 months in its process than selective tender).

Other construction works except preparatory works are planned to be performed by contractors selected by the international open tenders. The preparatory works will be carried out by some local contractors.

7.3 Construction Plan and Method

(1) Preparatory Works

Access roads is 34 km long in total consisting of main roads of 7 km long and branches of 27 km long. The main roads have 15 m in width and 10 % in the maximum slope and are metaled with gravels. Temporary building of approximately 1,200 m^2 in total floor area is necessary. The above temporary buildings are to be used only for the goverment and engineering consultant staffs and their families. Contractor's temporary buildings are prepared by themselves. Total area for contractor's work shops is estimated to be about 11,000 m^2. Construction plants are mainly concrete aggregate crushing and screening plant with production capacity of 100 t/hr and batcher plant with 30 m^3/hr.

Water supply facility of 6.0 m^3/sec in total capacity is required as follows:

Offices and quaters:	0.5 m^3/sec
Construction plants:	1.9 "
Work sites:	3.0 "
Others:	0.6 "
Total	6.0 m^3/sec

Power supply is planned to be done by diesel generators with total installed capacity of approximately 1,400 kW for the following places.

Construction plants:	120 kW
Repair shop:	150 "
Dam site:	700 "
Water supply facility	100 "
Offices and quarters:	200 "
Others	130 "
Total	1,400 kW

Other facilities such as a telephone system and a clinic are necessary at the project area. A telephone system with 10 circuits is provided at the project area. The National Power Corporation will provided a telecommunication system between Municipality Infanta and the Project site.

(2) Diversion Tunnels and Cofferdams

As mentioned in Paragraph 7.1.(2), the construction works for the division tunnels and cofferdams are very important and on the critical path of all the construction schedule. The following are basic concepts for the construction plan of the diversion tunnels and cofferdams.

i) Construction of the diversion tunnel No.1 is planned to be started in tunnel excavation from April '83 and to be completed by May '84 in the driest season (including curing of concrete), when the river diversion is to be made.

ii) Construction period of the diversion tunnel No.2 is 17 months from April '83 till September '84 just before the flood season starting.

iii) The main cofferdam embankment is planned to be performed for approximately 6 months from March '84. The river diversion from the original river course to the diversion tunnel No.1 shall be done in May '84 (the driest season) when the tunnel No.1 will have been completed. The embankment of the cofferdam shall be completed by the end of September '84 (the flood season) when the tunnel No.2 be completed.

iv) The tunnel No.1 is planned to be closed in April '87 (the dry season) and the installation works for the penstocks is started from May in that year. For the tunnel No.1 closure, the main dam shall be embanked up to EL.110 m by the end of August '87, so that a flood of less than 5,210 m^3/sec can be discharged only through the tunnel No.2. Impounding commencement of the reservoir is scheduled to be done by the tunnel No.2 closure in May '88. Most of the dam works including the spillway works shall be completed by the end of the dry season '88 for the final closure of the diversion tunnel No.2.

A top heading and bench cut method is recommended to be applied for the tunnel excavation. The tunnel excavation in the top heading are done from both the end sides (inlet and outlet sides). To save the cost of main construction equipment such as crawlers mounted drill jumbo and sliding forms for concrete lining, a tunnel construction plan shall be carefully made in consideration of avoiding double preparation of main equipment due to overlapped works of both the tunnels. In this viewpoint, commencement of the tunnel excavation is planned to have 4 months time-lag between the tunnel No.1 and 2.

The cofferdam of about 1,600,000 m^3 is planned to be embanked for 6 months in the dry season as mentioned above. This embankment schedule per month in volume for the cofferdam is approximately 50% of that for the main dam.

(3) Main Dam

All the main dam works are planned to be carried out for about 5 years from the early part of 1984. The following are the main works for the dam.

Excavation	:	2,844,000 m^3
Embankment	:	15,418,000 m^3
Inspection gallery	:	1,000 m
Grouting	:	60,000 m

Fig. 7-2 shows the material sources for the main and cofferdams.

From study results of the existing rainfall data, workable days for embankment works are estimated to be 152 days/year from January to September for core and filter and 273 days/year for rock. The core embankment works cannot be done for about three months of October, November and December because of heavy rainfall in those months. Monthly embankment volumes of the main dam are 350,000 m^3 on an average and 500,000 m^3 at the maximum, assuming that workable hours are 14 hours per day in two shift work for the aforesaid workable days.

Foundation excavation and treatment are planned as follows:

i) Foundation excavation mainly for the core portion is scheduled to be made between February '84 and September '85.

ii) The inspection (and grout) gallery of 1,000 m long will be constructed for 2 years from October '84 and its river bed portion of 160 m long has to be constructed till the end of March '85, just before starting the core embankment.

iii) Grouting works consisting of blanket grout of 24,000 m in total and curtain grout of 34,000 m in total are to be executed for 15 months from September '84 for blanket and for 30 months from January '86 for curtain. Most of all the curtain grout works will be done in the inspection gallery.

(4) Spillway

Excavated rocks of 6.4 million cubic meters from the spillway foundation shall be utilized as the rock embankment materials for the main dam. Total volume of the spillway foundation excavation is approximately 8.4 million cubic meters. For this purpose, the spillway rock excavation is planned, to be done so as to follow the schedule of the main dam embankment and for 5 years from the early part of 1984.

The estimated concrete volume of the spillway will be 195,000 m^3. Its concrete work period is planned for about 3 years from October '85. The concrete (batcher) and aggregate plant capacities are decided to be 30 m^3/hr and 100 t/hr due to the maximum production volume necessary for the spillway concrete works.

Installation works of the spillway gates have the same time allowance in the schedule and is planned to be carried out for 13 months in total.

(5) Waterway

The diversion tunnel No.1 is to be utilized as a part of the waterway after its construction use. The closure of the diversion tunnel No.1 is scheduled in April '87 and the penstock installation works are commenced in May as mentioned above. The following works shall be constructed prior to the aforesaid closure.

i) The inclined tunnel of 110 m long between the intake structure and diversion tunnel No.1.

and

ii) The connection tunnel of 94 m long between the powerhouse and diversion tunnel No.1.

With regard to the penstock, its installation works is scheduled for 14 months from May '87 immediately after the closure of the tunnel No.1. The total length and weight of the penstock is 350 m and 1,450 ton. All the installation works including tests shall be completed by the end of August '88 just before the final tests of the generating equipment.

(6) Powerhouse and Generating Equipment

Three (3) years are scheduled for construction of the powerhouse and installation works of two (2) units of the generating equipment (70 MW each). During this period, total twenty one (21) months are for the generating equipment works consisting of 3 months from the draft tubes, 2 months for the over-head crane, 12 months for the generating equipment and 4 months for the final tests.

(7) Switchyard and Transmission Line

The following electrical works can be executed within the project implementation period of 6 years as follows:

i) Outdoor switchyard: 36 months

ii) Transmission line with sub-station: 41 months

The periods said above are total months from tender call to completion of the installation works including tests.

CHAPTER 8

COST ESTIMATE

8.1 General

The construction of the Agos project is assumed to be carried out during the period of January 1983 to December 1988 and by a competitive contract basis to be awarded to foreign/local contractor.

The costs of the construction works were estimated by work items. The estimated costs of each work item are the accumulated amount of all the necessary costs. The cost includes direct cost such as labor, material, equipment, etc. and indirect cost such as the contractor's overhead and administration expenses. Economic cost is estimated on the assumption that tax and duties to be levied by the Government of the Philippines are exempted from the construction material and equipment.

8.2 Assumptions and Conditions

Cost estimates were prepared on the following basic assumptions and conditions mentioned below:

(1) Prices are based on current prices for labor, materials and equipment as of early 1980.

(2) Ruling exchange rate used is 1.0 US Dollar = 7.5 Pesos = 250 Japanese Yen.

(3) Work quantities are shown in Table 8-1.

(4) Tax and Duties

On the financial cost, unit prices are estimated on the basis of the current price in the Philippines and the prices added tax and duties for imported materials and equipment.

On the economic cost, the tax and duties of local material such as steel bars, cement, fuel and others are exempted at the rate of 30%, 50%, 20% respectively to the current price and the tax and duties of imported equipment and materials are exempted at the rate of 20% to the CIF prices.

Foreign Currency Portion	Local Currency Portion
• Construction plants and equipment including spare parts	• Timber, plywood, log
• Reinforcement bar and steel support	• Sand aggregate
• Fuel, oils, lubricants	• Explosives
• Cement	• Concrete goods
• Salaries and expenses of foreign supervisory personnel employed by contractor	• Miscellaneous hardware
• Contractor's overhead and profit components	• Salaries and wages of construction supervisors and workmen employed by contractor
• Miscellaneous architectural materials, special equipment, etc.	

8.3 Cost Estimate

(1) Civil Works

Cost of civil works was estimated on the base of unit price for each item of schedule of quantities. Unit prices are estimated for the direct cost and the indirect cost according to the above-mentioned assumptions and conditions.

The breakdown and unit prices are shown in the priced bill of quantities in Annex IV. (financial cost)

(2) Generating Equipment and Hydro-mechanical Works

The cost estimate for metal works such as gate, penstock and valve, and for generating equipment such as turbines, generators, transformers and switchgears, were estimated from current contract prices for similar kinds of project.

TABLES

Table 2-1 Cases for Slope Stability Analysis

Case	Condition	Seismic coefficient	Slope
1.	Reservoir water surface at H.W.L. and steady seepage in dam	0.15	upstream & downstream
2.	Immediately after completion (No storage but pore water pressure developed)	0.075	upstream
3.	Rapid reservoir drawdown from H.W.L. to L.W.L. and unsteady seepage in dam	0.15	upstream
4.	Reservoir water surface at probable maximum water level and steady seepage in dam	0	upstream

Table 2-2 Material Property

Zone	Wet density (t/m^3)	Saturated density (t/m^3)	Cohesion (t/m^2)	Friction angle (degree)
Core	1.82	1.91	5.0	18
Filter	2.00	2.23	0	35
Shell (Downstream side)	1.80	2.14	0	43
" (Upstream side)	1.86	2.14	0	45
Alluvial deposit	1.95	2.15	0	30

Table 2-3 Minimum Factors of Safety

Case	Condition	Slope applied	Min. Factors of Safety without earthquake	Min. Factors of Safety with earthquake
1	H.W.L. (EL 165 m)	Upstream	2.64	1.20
		Downstream	1.79	1.24
2	Immediately after completion	Upstream	2.59	1.97
		Downstream	1.77	1.45
3	Rapid drawdown	Upstream	2.28	1.54
4	Max. water level (168m)	Upstream	2.65	1.68

Table 3-1 Results of Flood Routing

Alternative No.	I	II	III	IV
N.H.W.L. (m)	165.000	165.000	165.000	165.000
D.F.W.L. (m)	165.000	169.500	168.000	168.000
P.M.F.W.L. (m)	168.12	171.77	169.76	169.71
PMFWL + 2.1 (m)	170.22	173.82	171.86	171.81
Dam Crest EL. (m)	170.30	173.600	171.90	172.00
D.F. (I=10,600)				
Q gate	10,600	6,490	6,450	5,840
Q nongate	-	4,110	4,150	4,760
Q total	10,600	10,600	10,600	10,600
P.M.F. (I max=17,290)				
Q gate (m^3/sec)	14,700	8,170	7,800	6,880
Q nongate	-	7,460	8,300	9,290
Q total	14,700	15,630	16,100	16,200
Net Crest Length (m)	151.0	198.0	184.0	168.0

N.H.W.L.: Normal high water level
D.F.W.L.: Design flood water level
P.M.F.W.L.: Probable maximum flood water level
Dam Crest EL. D.F.W.L + 3.0 m
or P.M.F.W.L. + 2.1 m
Q gate : Total discharge from gated portion
Q non gate: Total discharge from non-gated portion

Table 4-1 Main Features of Alternatives

Diameter of tunnels	8.0 m	9.0	10.0	11.0	12.0
Length of D/T No.1	800 m	770	760	760	760
Length of D/T No.2	920 m	890	870	865	860
Design flood water level	97 m	92	87	83	79
Cofferdam crest EL.	98 m	93	88	84	80
Embankment volume ($x10^6m^3$)	2.09	1.68	1.33	1.10	0.86

Table 4-2 Construction Cost of Alternatives

Unit: millicn US dollars

Diameter of tunnels	8.0 m	9.0	10.0	11.0	12.0
Diversion tunnels	9.67	11.83	14.36	17.32	20.55
Diversion gates	0.18	0.21	0.27	0.36	0.48
Cofferdam	13.17	10.58	8.38	6.93	5.42
Total cost	23.02	22.62*	23.01	24.61	26.45

* common work be excluded.

Table 4-3 Summary of Work Quantities for Tunnels Alternatives

		Right	Left	Right/Left
Tunnel excavation	(m^3)	155,500	128,770	145,100
Tunnel concrete	(m^3)	43,340	35,890	40,440
Plug concrete	(m^3)	6,360	6,360	6,360
Reinforcement bars	(ton)	1,730	1,435	1,620

Table 5-1 Main Features and Head Loss for Waterway Alternatives

Installed Capacity = 140 MW

Name of Alternatives	I	II	III	IV
Tail Water level (m)	EL.41.5	41.5	41.5	39.4
Gross head (m)	105	105	105	107.1
Length of P. Tunnel (m)	270	233	387	750
" Penstock line (m)	430	365	420	275
Rated Discharge (m^3/sec)	163.3	163.2	163.9	164.2
H loss (m)	3.26	3.24	3.63	5.96
Effective head (m)	101.74	101.76	101.37	101.14

Table 5-2 Construction Cost of Waterway Alternatives

(million US dollars)

Name of Alternative	I	II	III	IV
Intake	3.31	2.91	3.31	3.48
Headrace tunnel	0.81	0.71	1.17	2.48
Surge tank	0.52	-	0.63	0.96
Penstock	6.29	4.68	7.31	3.87
Powerhouse (Civil)	3.85	3.85	3.85	3.89
" (Architech)	0.67	0.67	0.67	0.67
Tailrace	1.23	1.23	1.23	1.23
Total cost	16.68	14.05	18.17	16.68

Table 5-3 Economic Diameter of Headrace Tunnel

Length of headrace (D varies from 6.2 to 7 m)	146 m
" (D = 9.0 m)	80 m

Thickness of lining $t = 1/12.D$

Unit price of energy $0.0234/kWh

The head loss due to friction is computed by

$$hf = \frac{124.5\ n^2.L}{D^{4/3}} \cdot \frac{u^2}{2\ g}$$

Lost energy

Dia	6.2 m	6.4	6.6	6.8	7.0
hf	0.767 m	0.648	0.550	0.469	0.401
Low energy	2.56 GWh	2.17	1.84	1.57	1.34

Construction cost

Dia	6.2	6.4	6.6	6.8	7.0
Excavation (m^3)	9,656	10,290	10,943	11,616	12,310
Concrete (m^3)	2,562	2,730	2,903	3,082	3,266
Reinf. (ton)	102	109	116	123	131
Others (22%)					
Total cost	0.852	0.908	0.965	1.025	1.086
P.W. Lost Energy	0.595	0.502	0.426	0.364	0.311
B + C	1.447	1.410	1.391	1.389*	1.397

Table 5-4 Economic Diameter of Penstock Pipe

Length of water way		
" of headrace	(D = 6.8 m)	146 m
" of "	(D = 9.0 m)	80 m
" of penstock		350 m
Installed capacity		140 MW
Rated head		105 m
Closing time of valve		7 sec
Unit value per kWh		$0.0234/kWh

Variable parameter D of penstock

Lost energy

Dia	6.4 m	6.2	6.1	5.8	5.6
Rated discharge	162.2 m^3/sec	162.5	162.8	163.7	164.4
H loss	2.583 m	2.835	2.979	3.515	3.975
H loss at penstock	1.558 m	1.807	1.949	2.476	2.928
lost energy	5.21 GWh/y	6.04	6.51	8.27	9.79
50^y lost Benefit	$1.208 x 10^6	1.401	1.511	1.92	2.27

Water hammer

Dia	6.4 m	6.2	6.1	5.8	5.6
Water hammer	34.9 %	36.8	37.9	41.4	44.2

Construction cost of Penstock

Dia	6.4 m	6.2	6.1	5.8	5.6
Excavation tunnel (m^3)	4,723	4,486	4,370	4,032	3,814
Concrete tunnel (m^3)	1,699	1,649	1,624	1,549	1,499
Reinforcement bars (t)	68	66	65	62	60
Concrete plug (m^2)	1,573	1,671	1,719	1,860	1,950
Steel pipe (ton)	1,480	1,406	1,370	1,266	1,200
Others (20%)					
Total cost (x$10^6$$)	5.042	4.809	4.697	4.368	4.159
50^y lost Benefit	1.208	1.401	1.511	1.920	2.270
C + B (x$10^6$$)	6.250	6.210	6.208*	6.288	6.429

Economic diameter D = 6.10 m

Velocity in penstock v = 5.57 m/sec

Table 8-1 Major Work Quantities of Civil Work

Civil Works

1) Diversion Tunnel	
Excavation, open	145,700 m^3
Excavation, tunnel	145,300 m^3
Concrete	1,350 m^3
Backfill grout	2,280 m
Grout	
2) Cofferdams	
Embankment, impervious earth	231,000 m^3
" , sand and gravel	232,200 m^3
" , random rock	1,142,250 m^3
3) Main Dam	
Excavation, open	2,855,000 m^3
" , tunnel	1,100 m^3
Embankment, impervious earth	1,941,000 m^3
" , sand and gravel	1,210,300 m^3
" , rock	12,014,700 m^3
" , riprap	252,000 m^3
Concrete	2,475 m^3
Grout	59,590 m
Stripping at material site	1,070,000 m^3
4) Spillway	
Excavation (6,366,000 m^3: dam embankment)	8,407,200 m^3
Backfill	7,000 m^3
Concrete	195,000 m^3
5) Waterway	
a) Power Intake and Intake Tower	
Excavation	82,030 m^3
Concrete	12,375 m^3
b) Headrace Tunnel	
Excavation, tunnel	5,950 m^3
Concrete	2,100 m^3
Backfill grout	40 m^3
Grout	330 m^3
c) Penstock Tunnel	
Excavation, tunnel	4,350 m^3
Concrete	2,000 m^3
Backfill grout	30 m^3
Grout	285 m

6) Power House

Excavation	136,100 m^3
Fill and backfill	7,900 m^3
Concrete	23,500 m^3

7) Tailrace

Excavation	55,900 m^3
Concrete	16,000 m^3

8) Switchyard

Excavation	10,000 m^3
Fill and backfill	9,000 m^3
Concrete	3,000 m^3

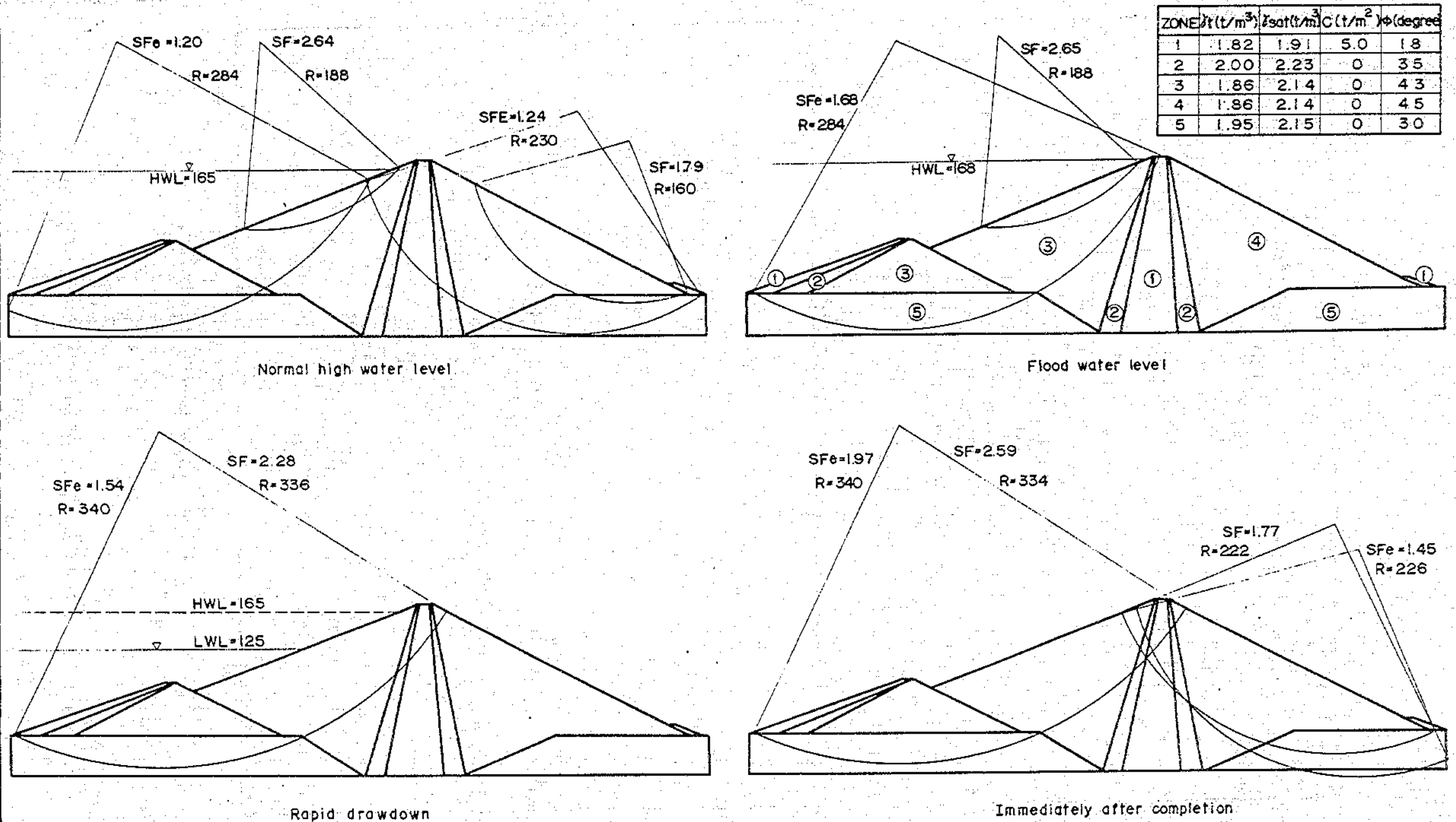
Table 8-2 Basic Prices of Materials and Wages of Labors

Item	Unit	Price ($)	Price (Peso)
Materials			
Reinforcement Bar	ton	533.33	
Steel Support	ton	866.67	
Heavy Oil	ℓ	0.9	
Gasoline	ℓ	0.60	
Lubricant	ℓ	0.91	
Grease	kg	1.24	
Dynamite	kg		22.00
ANFO	kg	1.08	
Portland Cement	ton	100.00	
Timber, Plank	m^3		1,820.00
" , Square	m^3		1,590.00
Foreman	WD		50.00
Mechanic	"		55.00
Electrician	"		24.00
Operator, Heavy	"		30.00
" , Light	"		28.70
Assistant Operator	"		28.70
Carpenter	"		24.00
Mason	"		20.00
Concrete Worker	"		25.00
Reinforcement Worker	"		20.00
Tunnel Worker	"		25.00
Boring Worker	"		25.00
Pipe Filter	"		24.00
Skilled Labour	"		18.00
Common Labour	"		14.00

Table 8-3 Major Equipment and Plant

	Equipment and Plant	Size and Capacity	Nos.
1.	Bulldozer with Ripper	44 ton	7
2.	Bulldozer with Ripper	32 ton	9
3.	Bulldozer	32 ton	17
4.	Bulldozer with Ripper	21 ton	4
5.	Bulldozer	21 ton	5
6.	Bulldozer	15 ton	4
7.	Bulldozer	11 ton	4
8.	Tractor Shovel (for tunnel)	1.2 m^3	2
9.	Tractor Shovel	1.6 m^3	2
10.	Tractor Shovel	2.4 m^3	6
11.	Wheel Loader	3.9 m^3	12
12.	Wheel Loader	7.7 m^3	10
13.	Back Hoe	0.3 m^3	2
14.	Back Hoe	0.6 m^3	2
15.	Back Hoe	1.2 m^3	1
16.	Dump Truck	8 ton	8
17.	Dump Truck	11 ton	23
18.	Dump Truck	20 ton	62
19.	Dump Truck	32 ton	50
20.	Tamping Roller	25 ton	3
21.	Vibration Roller	2 ton	2
22.	Vibration Roller	8 ton	1
23.	Vibration Roller	21 ton	5
24.	Tire Roller	8-15 ton	1
25.	Crawler Drill	23 m^3/min	10
26.	Crawler Drill	10.5 m^3/min	8
27.	Boring Machine	5.5 kW	12
28.	Portable Air Compressor	26 m^3/min	10
29.	Portable Air Compressor	10.5 m^3/min	8
30.	Air Compressor	150 kW	2
31.	Crawler Jumbo	4 booms	2
32.	Raise Climbmer		1
33.	Grout Pump	7.5 kW	6
34.	Grout Pump	11 kW	7
35.	Grout Mixer	2.2 kW	13
36.	Agitator Truck	3.2 m^3	4
37.	Agitator Truck	4.5 m^3	6
38.	Concrete Pump	60 m^3/hr	4
39.	Sprinkler Tank Truck	5.5 kℓ	3
40.	Mortor Grader	3.7 m	3
41.	Concrete Plant	0.75 m^3x2	1
42.	Screening Plant	100 t/hr	1
43.	Truck Crane	30 ton	2
44.	Truck Crane	15 ton	2
45.	Crawler Crane	40 ton	1
46.	Trailer	40 ton	1
47.	Trailer	20 ton	1
48.	Fuel Tank Truck	10 kℓ	5
49.	Diesel Generator	500 kW	2
50.	Diesel Generator	250 kW	2
51.	Lighting Facility		10

F I G U R E S



ZONE	γt (t/m^3)	γsat (t/m^3)	C (t/m^2)	ϕ (degree)
1	1.82	1.91	5.0	18
2	2.00	2.23	0	35
3	1.86	2.14	0	43
4	1.86	2.14	0	45
5	1.95	2.15	0	30

SF : Minimum safety facter
SFe : Minimum safety facter with earthquake
R : Radius of critical slip circle

Fig. 2-1 Stability Analysis of Agos Dam

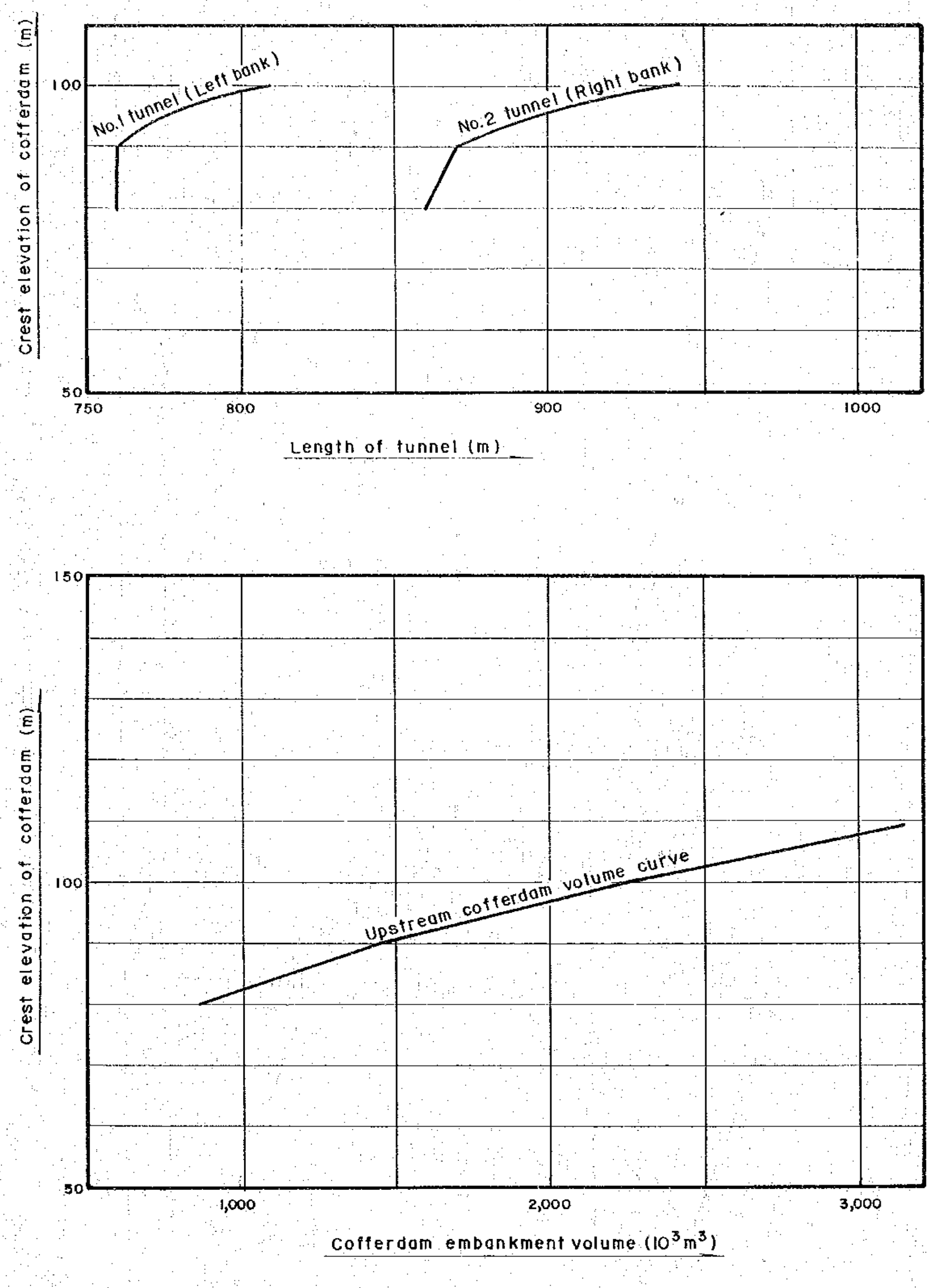
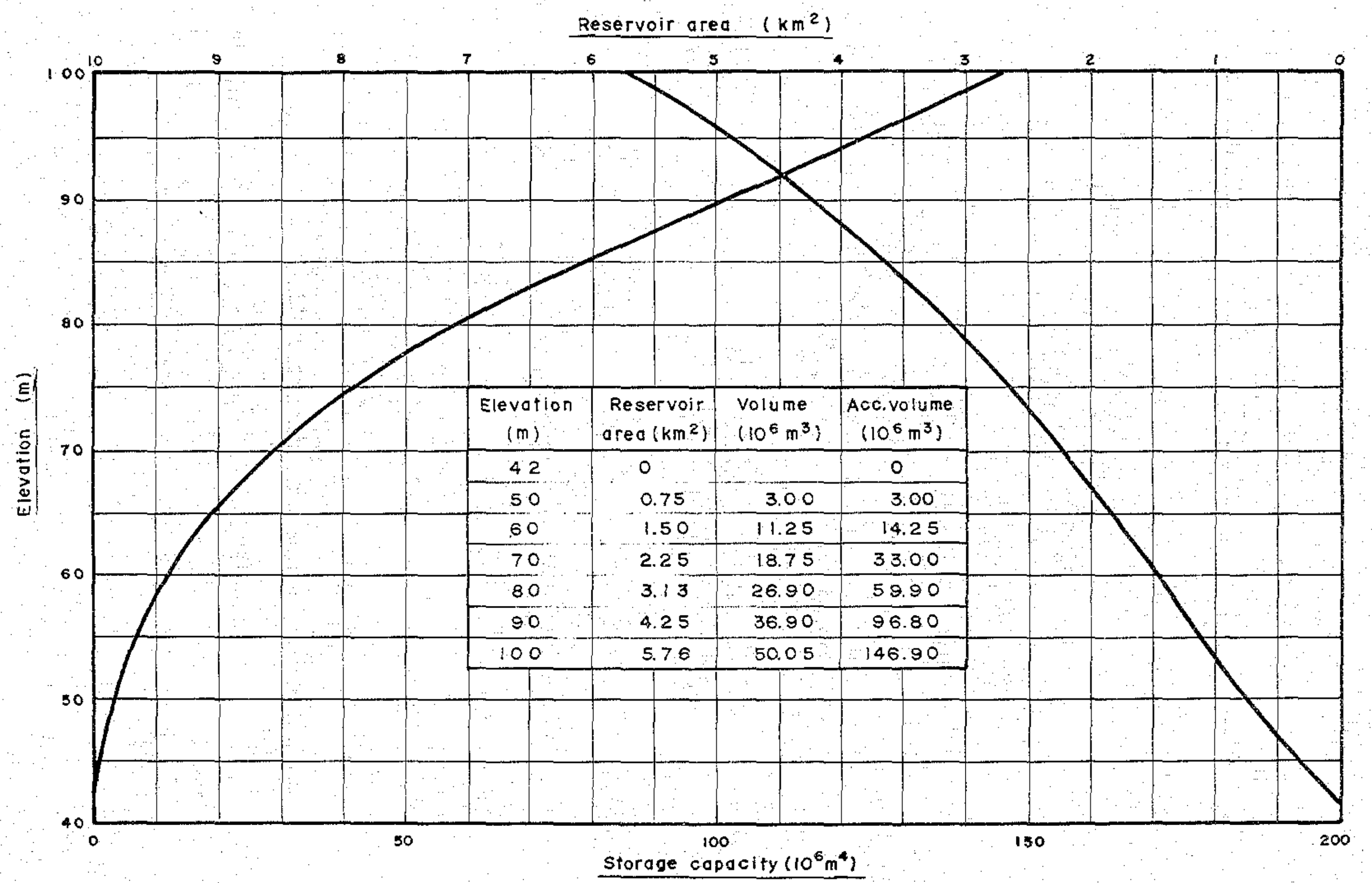


Fig. 4-1 Diversion Tunnel and Cofferdam



Elevation (m)	Reservoir area (km^2)	Volume (10^6 m^3)	Acc. volume (10^6 m^3)
42	0		0
50	0.75	3.00	3.00
60	1.50	11.25	14.25
70	2.25	18.75	33.00
80	3.13	26.90	59.90
90	4.25	36.90	96.80
100	5.76	50.05	146.90

Fig. 4 - 2 Agos Dam Capacity - Area Curve for Diversion Flood Routing Study.

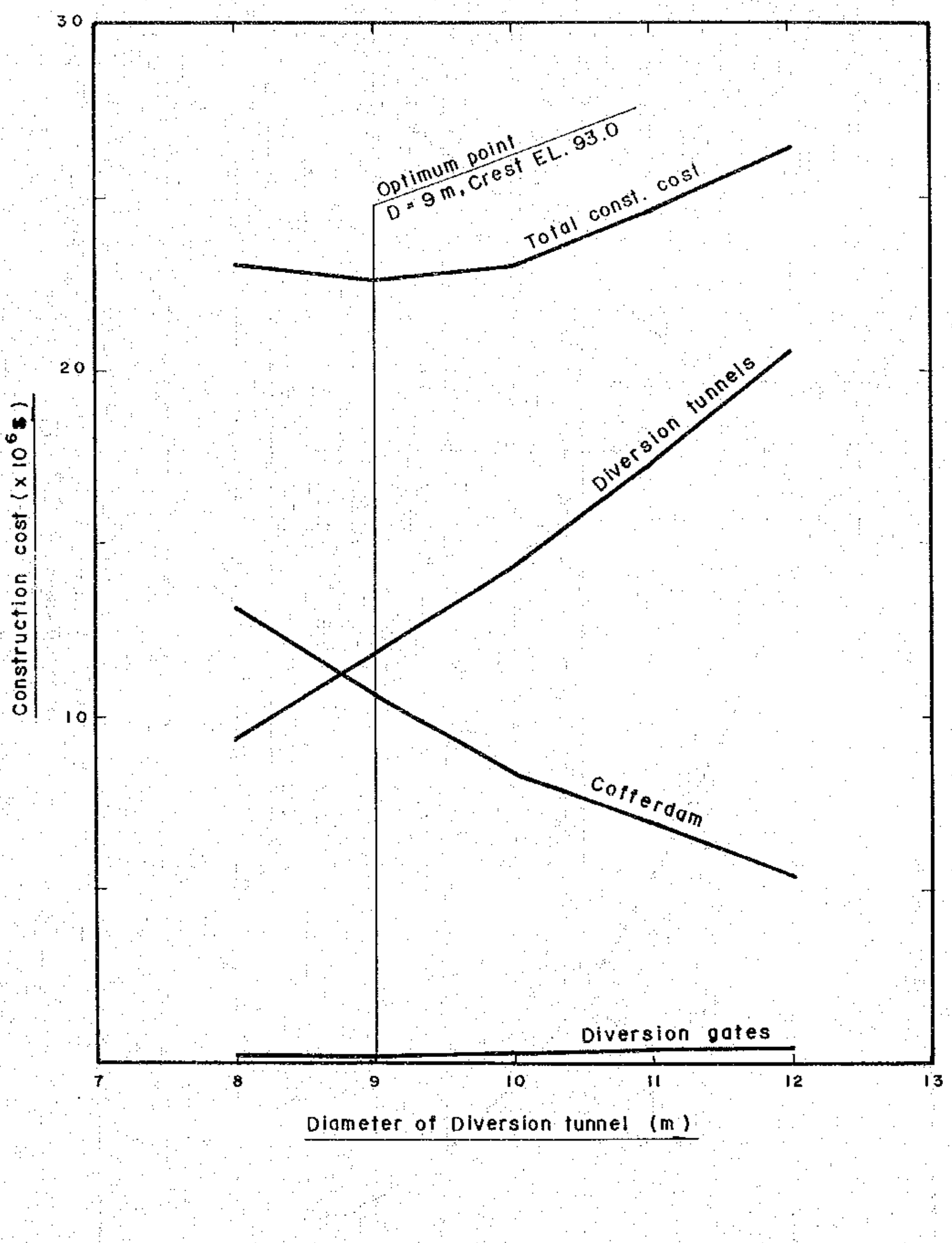


Fig. 4-3 Optimum Combination of Diversion tunnel and Cofferdam.

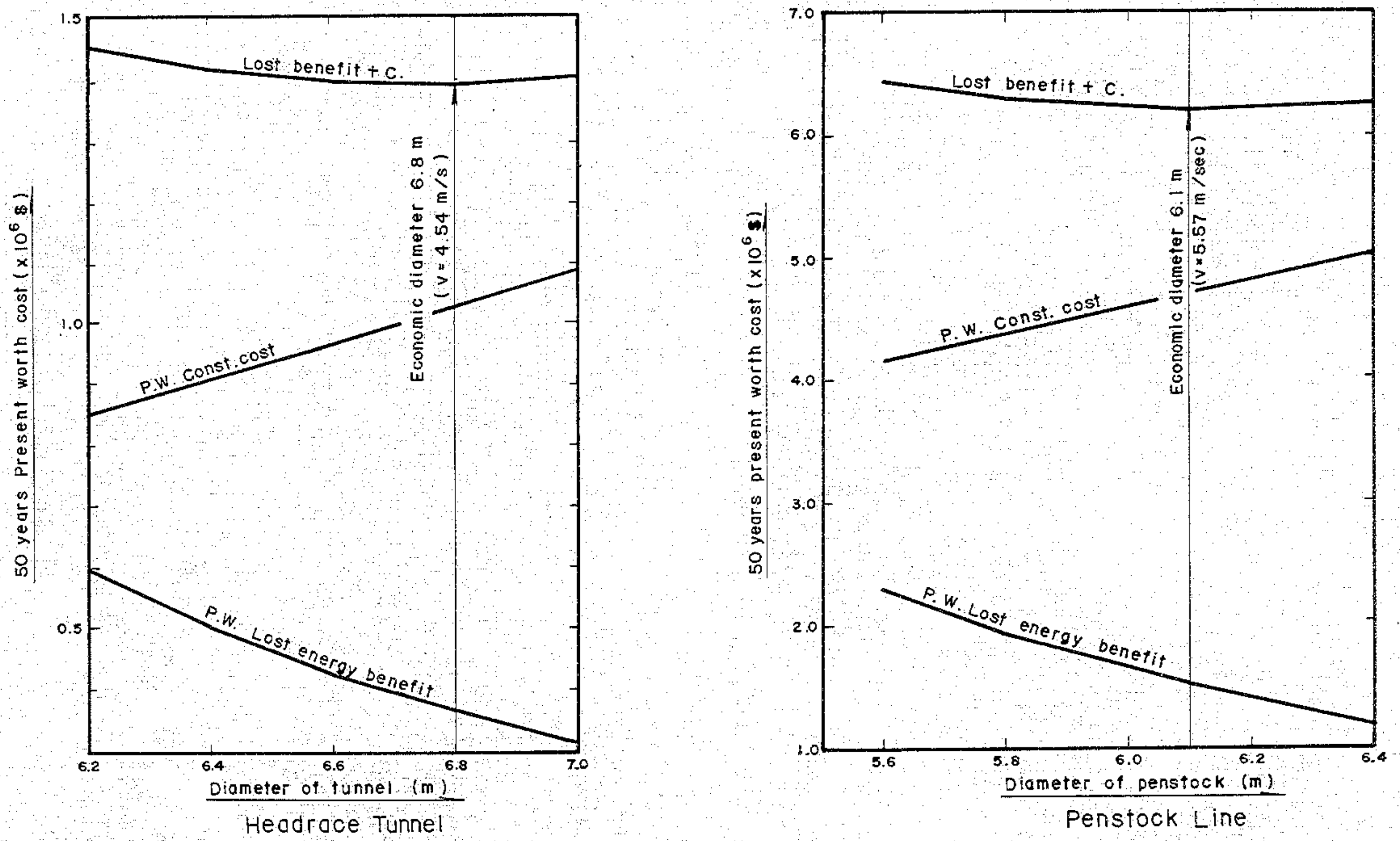


Fig. 5 - 1 Economic Diameter of Waterway.

ITEM	UNIT	WORK QUANTITY
DETAILED DESIGN		
TENDERING AND CONTRACT		
PREPARATORY WORK		
ACCESS AND HAULING ROADS	km	34
TEMPORARY BUILDINGS	m^2	11,300
WATER SUPPLY SYSTEM		60 m^3/min
POWER SUPPLY SYSTEM		1,400 kw
TELECOMMUNICATION SYSTEM		
AGGREGATE AND BUTCHER PLANTS		100 t/hr
		30 m^3/hr
QUARRY AND BORROW PITS (STRIPPING)	m^3	1,070,000
DIVERSION WORKS		
DIVERSION TUNNEL NO.1		
Excavation open	m^3	72,850
tunnel	m^3	72,650
Concrete	m^3	26,600
DIVERSION TUNNEL NO.2		
Excavation open	m^3	72,850
tunnel	m^3	72,650
Concrete	m^3	26,600
UPSTREAM COFFERDAM	m^3	1,512,000
DOWNSTREAM COFFERDAM	m^3	93,000
MAIN DAM		
Excavation	m^3	2,844,000
Inspection galley	m	900
Curtain grouting	m	34,120
Blanket grouting	m	23,720
Embankment core	m^3	1,941,000
filter	m^3	1,210,300
rock and riprap	m^3	12,266,700
SPILLWAY		
Excavation	m^3	8,407,200
Concrete	m^3	195,000
Gates	ton	14m×14m×4nos
		1,460
POWER WATERWAY		
INTAKE		
Excavation	m^3	82,030
Concrete	m^3	12,375
HEADRACE TUNNEL		
Inlet	m	44
Inclined tunnel	m	66
Tunnel	m	36
PENSTOCK TUNNEL		
Tunnel	m	94
Metal works, gates and trashracks	ton	688
Penstock line	m	350
POWER HOUSE AND TAILRACE		
Excavation	m^3	182,000
Concrete in power house, substructure	m^3	23,500
superstructure	m^3	2,720
Concrete in tailrace	m^3	16,000
Generating equipment		2×70,000 kw
Tailrace gates	ton	68
SWITCHYARD		
Excavation and backfill	m^3	19,000
Concrete	m^3	3,000
Electrical equipment	LS	
TRANSMISSION LINE AND SUBSTATION		
Transmission line	km	45
Substation	LS	

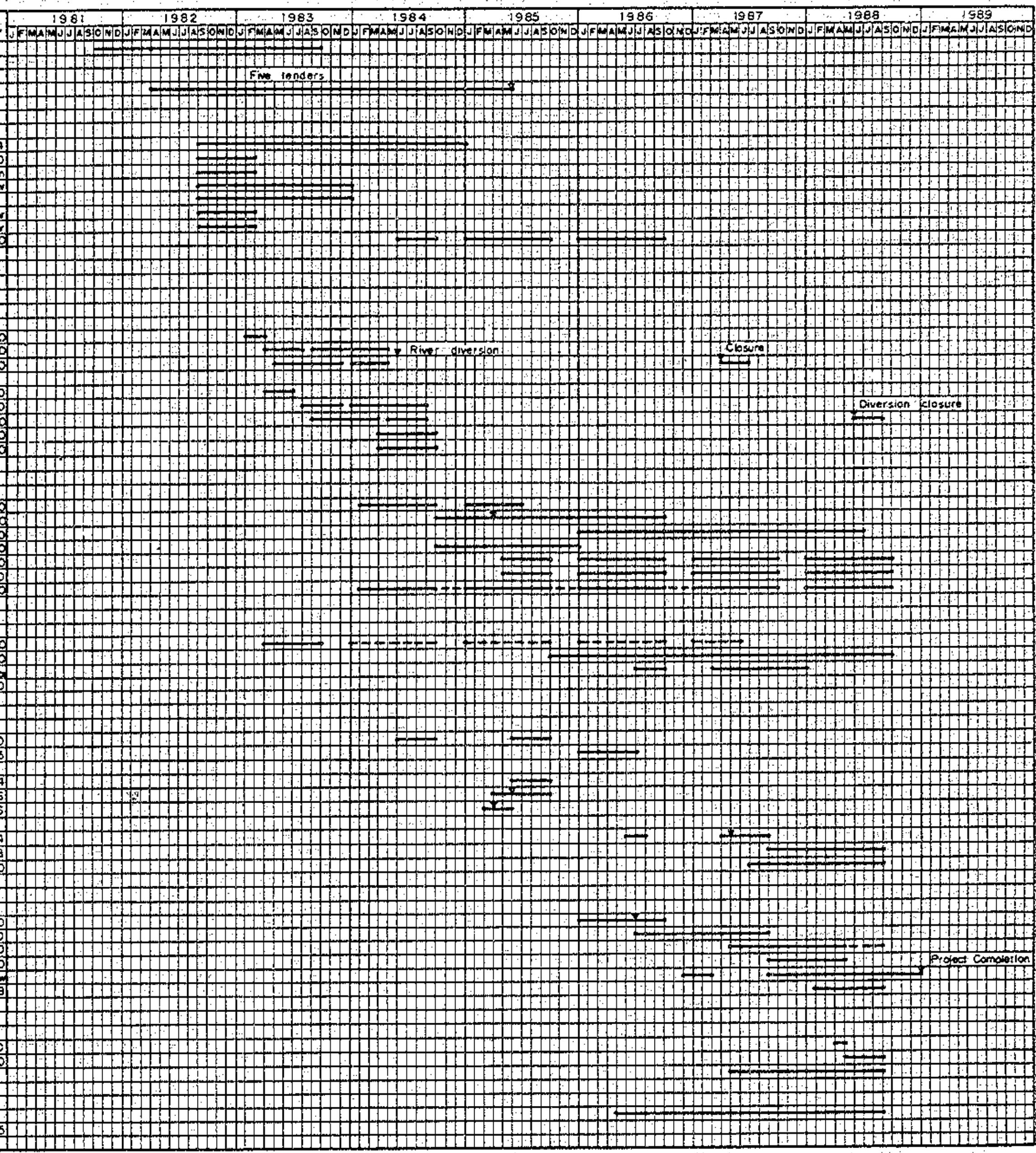


Fig. 7-1 Construction Time Schedule

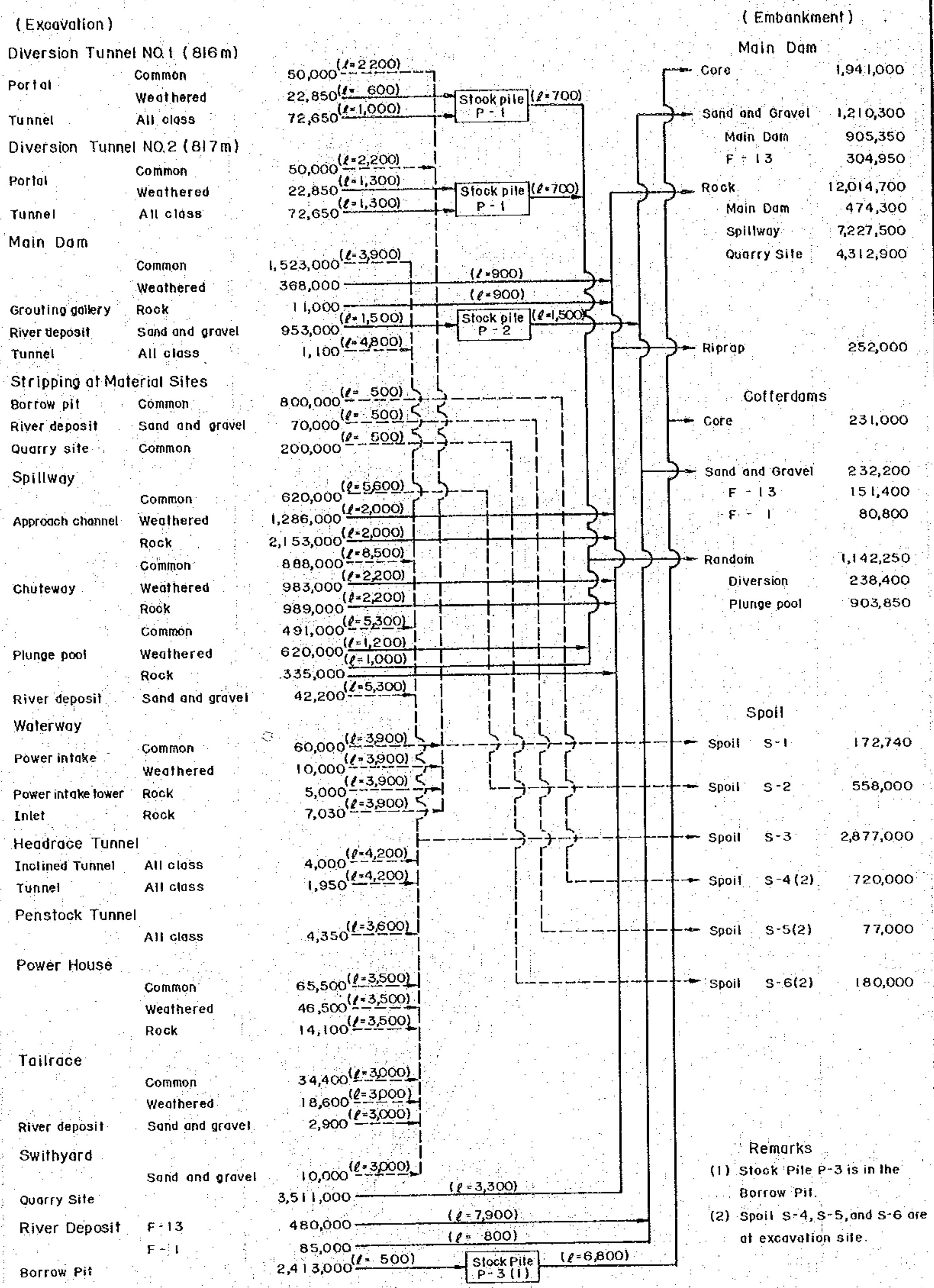


Fig. 7-2 Distribution of Excavated and Embankment Materials

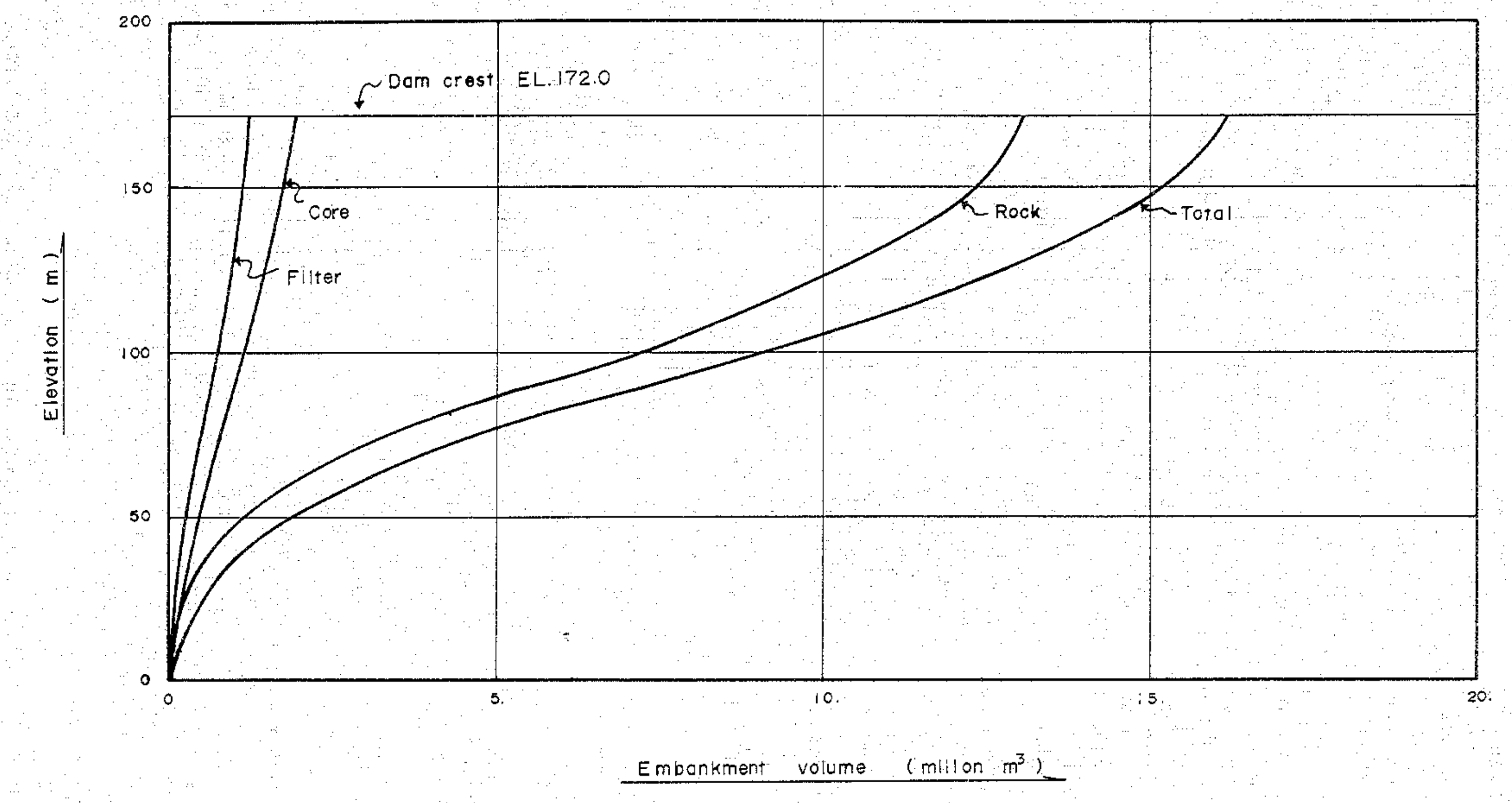
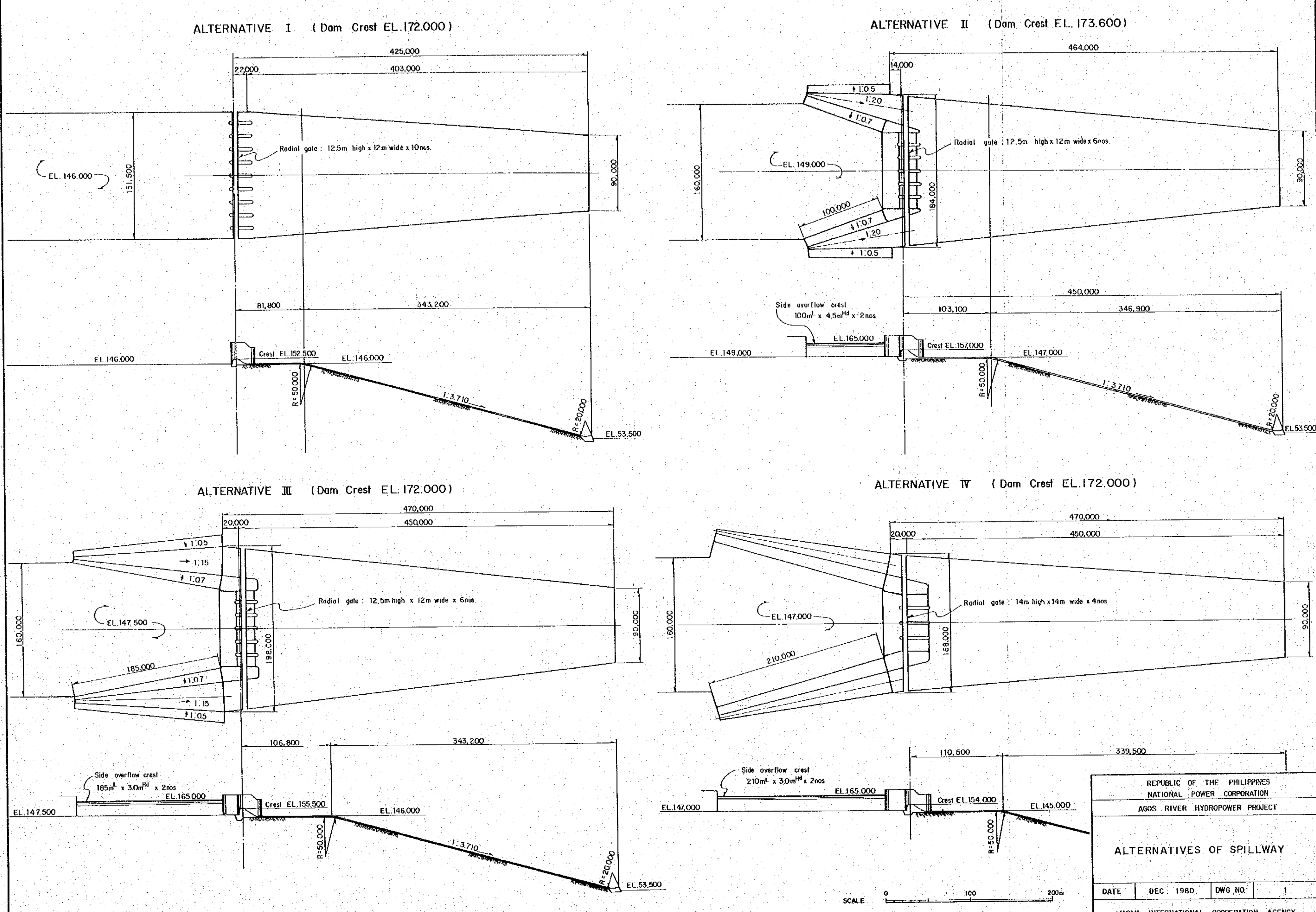


Fig. 7-3 Dam Embankment Volume Curve

D R A W I N G S

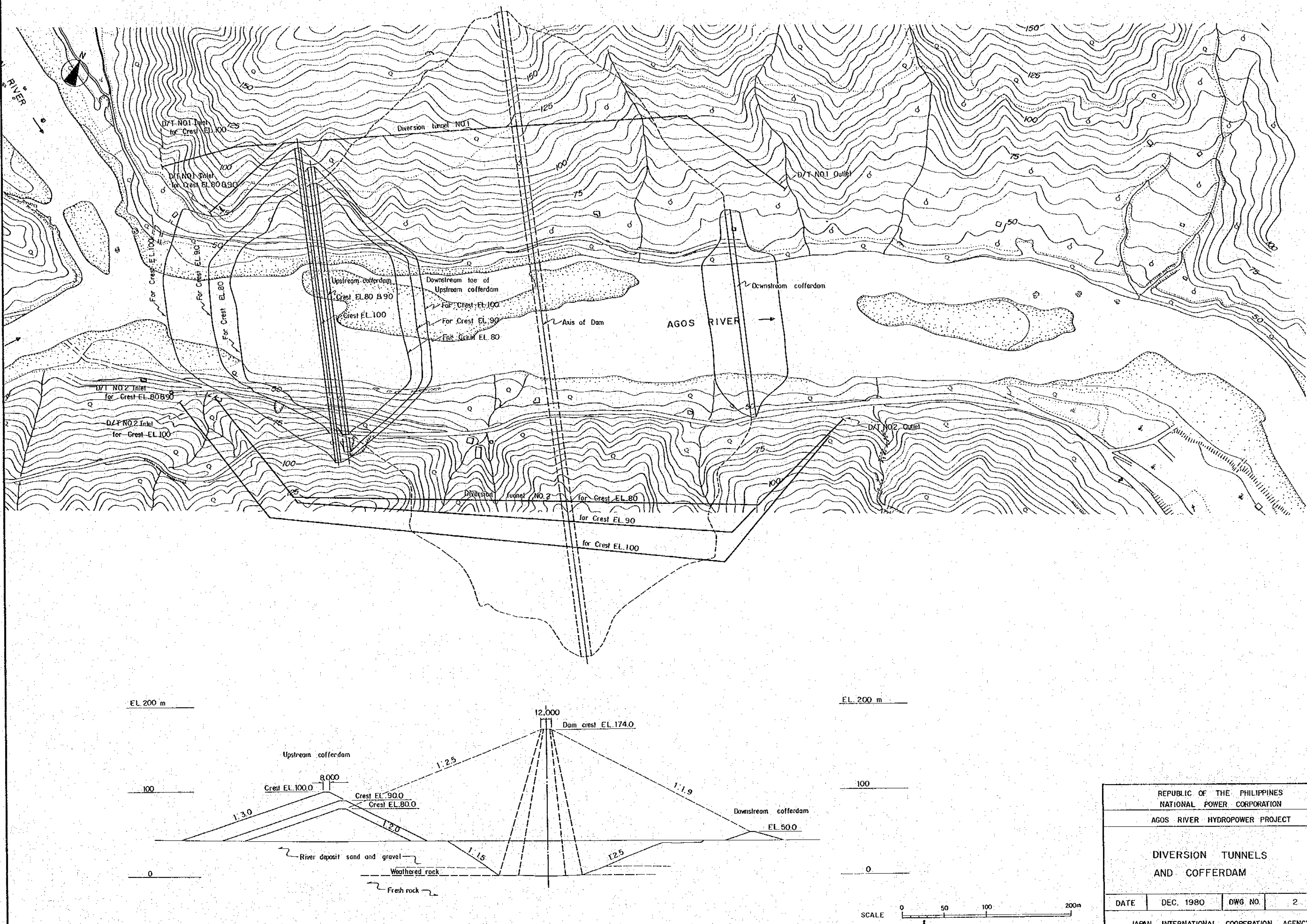


REPUBLIC OF THE PHILIPPINES
NATIONAL POWER CORPORATION
AGOS RIVER HYDROPOWER PROJECT

ALTERNATIVES OF SPILLWAY

DATE	DEC. 1980	DWG NO.	1

JAPAN INTERNATIONAL COOPERATION AGENCY



REPUBLIC OF THE PHILIPPINES
NATIONAL POWER CORPORATION

AGOS RIVER HYDROPOWER PROJECT

DIVERSION TUNNELS
AND COFFERDAM

DATE	DEC. 1980	DWG NO.	2

JAPAN INTERNATIONAL COOPERATION AGENCY