REPUBLIC OF THE PHILIPPINES
NATIONAL IRRIGATION ADMINISTRATION

FEASIBILITY REPORT ON THE GUMAIN RIVER IRRIGATION PROJECT

APPENDIXES VOLUME II

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FEBRUARY 1985

JAPAN INTERNATIONAL COOPERATION AGENCY
TOKYO JAPAN

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ABBREVIATION AND GLOSSARY OF TERMS

Abbreviations used in this report are listed below:

1. Length and Height

mm : millimeter
cm : centimeter
m : meter
km : kilometer
MSL : mean sea level
EL : elevation above MSL

2. Area

cm^2 : square centimeter
m^2 : square meter
km^2 : square kilometer
ha : hectare
MSM : million square meter

3. Volume

lit, ℓ : liter (= 1,000 cm^3)
m^3 : cubic meter
MCM : million cubic meter

4. Weight

mg : milligram
g : gram
kg : kilogram
t (ton) = 1,000 kg

5. Time

sec : second
min : minute
hr : hour
yr : year

6. Electric Measures

kV : kilovolt
kW : kilowatt
kWh : kilowatt-hour
MW : megawatt
MWh : megawatt-hour
GWh : gigawatt-hour

7. Other Measures

% : percent
PS : horse power
°C : centigrade
m^3/sec, m^3/s : cubic meter per second
lit/sec/ha, lit/s/ha : liter per second per hectare
cm/sec, cm/s : centimeter per second
t/ha: ton per hectare
ppm : part per million
No(s), no(s) : number(s)
SPT : standard penetration test

8. Currency

US$: US Dollar
₱ : Philippine Peso (US$1.00 = ₱14.0 = ¥240)

9. Other Abbreviations

(A)

ACA - Agricultural Credit Administration
AD - Agriculture Division
ADB - Asian Development Bank
AMC - Area Marketing Cooperative
APIP - Aurora Peñaranda Irrigation Project
ARBA - Agrarian Reform Beneficiaries Association
AW - Association Worker
AWMT - Assistant Water Management Technician

(B)

BAEcon - Bureau of Agricultural Economic
BAEx - Bureau of Agricultural Extension
BAI - Bureau of Animal Industry
BC - Billing Clerk
BCD - Bureau of Cooperative Development
BFAR - Bureau of Fisheries and Aquatic Resources
BFCD - Bureau of Flood Control and Drainage
BISA - Barangay Irrigation Service Association
BPI - Bureau of Plant Industry
BPW - Bureau of Public Works
BS - Bureau of Soils

(C)

CBP - Central Bank of the Philippines
CDLF - Cooperatives Development Loan Fund
CIA - Communal Irrigators Association
CIS - Communal Irrigation System
CISP - Cooperative Insurance System of the Philippines
CLSU - Central Luzon State University
CMSP - Cooperative Marketing System of the Philippines
COA - Commission of Audit
CRB - Cooperative Rural Bank
CRIS - Caulaman River Irrigation System

(D)

DPB - Development Bank of the Philippines
DT - Ditchtender
DCSPCMAI - Del Carmen Sugar Producer's Cooperation Marketing Association Inc.

(E)

EOD - Engineering and Operations Division

(F)

FACOMA - Farmers' Cooperative Marketing Association
FAD - Farmers' Assistance Division
FAO - Food and Agricultural Organizaiton
FBC - Farmers' Barrio Cooperative
FIA - Farmer-Irrigators' Association or Farmer-Irrigation Association
FIG - Farmer-Irrigators' Group or Farmer-Irrigation Group
FIO - Farmer-Irrigators' Organization
FL - Farmers' Leader
FSDC - Farm Systems Development Corporation

(G)

GK - Gatekeeper
GOP - Government of the Philippines

(I)

IA - Irrigation Association
IAO - Irrigation Association Organizer
IBRD - International Bank for Reconstruction and Development
IGL - Irrigators' Group Leader
IOMP - Input and Output Monitoring Program
IRRI - International Rice Research Institute
ISA - Integrated Services Association
ISF - Irrigation Service Fee

(J)

JICA - Japan International Cooperation Agency

(K)

KAISA - Kalipunan Ng Mga Integrated Service Association
KKK - Kilusang Kabuhayan at Kaunlaran

(L)

LBP - Land Bank of the Philippines
LES - Luzon Experimental Station

(M)

MA - Ministry of Agriculture
MAR - Ministry of Agrarian Reform
MEC - Ministry of Education and Culture
MF - Ministry of Finance
MHS - Ministry of Human Settlements
MITI - Ministry of Industry, Trade and Investment
MLG - Ministry of Local Government
M-99 - Masagana 99 Program (national rice program)
MPWH - Ministry of Public Works and Highways
MRRTC - Maligaya Rice Research and Training Center

(N)

NASUTRA	-	National Sugar Trading Corporation
NASUDECO	-	National Sugar Development Corporation
NCSO	-	National Census and Statistics Office
NEA	-	National Electrification Administration
NEDA	-	National Economic and Development Authority
NFA	-	National Food Authority
NFAC	-	National Food and Agriculture Council
NIA	-	National Irrigation Administration
NIS	-	National Irrigation System
NPC	-	National Power Corporation
NSDB	-	National Science Development Board
NWRC	-	National Water Resources Council

(O)

OECF	-	Overseas Economic Cooperative Fund, Japan

(P)

PAGASA	-	Philippine Atmospheric, Geophysical and Astronomical Services Administration
PATC	-	Philippine Agricultural Training Council
PCARR	-	Philippine Council for Agricultural Research Resources
PCIC	-	Philippine Crop Insurance Corporation
PDA-ADCC	-	Provincial Development Committee - Agricultural Development Coordinating Council
PDSO	-	Provincial Development Staff Office
PELCO-II	-	Pampanga II Electric Cooperative, Inc.
PGRIS	-	Porac Gumain River Irrigation System
PHILSUCOM	-	Philippine Sugar Commission
PIS	-	Pump Irrigation System
PNB	-	Philippine National Bank
PPA	-	Philippine Port Authority
PSPCMAI	-	Porac Sugar Producer's Cooperative Marketing Association Inc.

(R)

RIS	-	River Irrigation System
RP	-	Republic of the Philippines
RUG	-	Rotation Unit Group

(S)

SEC	-	Securities and Exchange Commission

(U)

UNDP	-	United Nations Development Program
UPLB	-	University of the Philippines, Los Baños
UPRIIS	-	Upper Pampanga River Integrated Irrigation System
USAID	-	United States Agency for International Development
USBR	-	United States Bureau of Reclamation

(W)

WCCC	-	Water Control Coordinating Center
WM	-	Watermaster
WMT	-	Water Management Technologist
WMTC	-	Water Management Training Center

(Z)

ZE	-	Zone Engineer

APPENDIX V

OPTIMIZATION OF PROJECT SCALE

APPENDIX V OPTIMIZATION OF PROJECT SCALE

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APPENDIX V OPTIMIZATION OF PROJECT SCALE

CHAPTER 1 BASIC CONCEPT FOR DEVELOPMENT

The objectives of agricultural and irrigation development in the study area are to increase rice production by increasing the unit yield of paddy and to expand the irrigation area especially during the dry season. Also, as there are sugarcane fields in the study area, increase of sugar production through irrigation is included in the basic concept for raising the income of sugar planters.

For the above basic concept, the development plan of the project is formulated as follows:

a) to provide year round irrigation by constructing a storage dam on the Gumain river,

b) to provide new conveyance facilities to supply irrigation water from the Gumain reservoir to the existing irrigation systems as well as to new irrigation areas including the sugarcane fields,

c) to rehabilitate and upgrade the existing irrigation systems,

d) to improve the drainage facilities and road networks including provision of new facilities where necessary,

e) to introduce improved irrigation farming practices,

f) to establish an operation and maintenance system for the irrigation and drainage facilities,

g) to improve the present agricultural support services, and

h) to examine possibility of hydroelectric power generation accompanying release of irrigation water from the Gumain reservoir.

CHAPTER 2 PROJECT FORMULATION

2.1 General

The development plans for water resources, agriculture, irrigation and drainage including a storage dam were studied and formulated in relevant appendixes individually. In this appendix, the optimum project scale was clarified through the comparative studies on overall development plans combinating each development plan.

The process of optimization study is as described below:

1) Assessment of available water resources,

2) Preparation of alternative plans with consideration of the optimization of the irrigation area,

3) Case study for development scale of the proposed Gumain dam,

4) Water balance study for each alternative plan,

5) Cost estimate and benefit analysis for each alternative plan, and

6) Economic evaluation for formulating the optimum development plan.

2.2 Water Balance Study

2.2.1 Assessment of Available Water Resources

The major water sources in the study area are the Gumain, Porac and Caulaman rivers. Although the available discharge records of these three rivers are considerably long, the records of Porac and Caulaman rivers were not to be reliable as mentioned in the APPENDIX I. While the discharge records of the Gumain river well coincide with the rainfall records at the Basa Air Base and it was, therefore, decided that the discharges of the Porac and Caulaman rivers should be estimated based on the discharge records of the Gumain river.

After examining the relation between the discharges of the Gumain river and those of the Porac and Caulaman rivers by the double-mass curve method, the discharges of the Porac and Caulaman rivers could be estimated using the conversion factors for the discharges of the Gumain river of 0.7 and 0.9, respectively as shown below:

$$Qp = 0.7 \times Q_G \times \frac{A_P}{A_G}$$

$$Qc = 0.9 \times Q_G \times \frac{Ac}{A_G}$$

where, Q_G: Runoff of the Gumain river

Q_P: Runoff of the Porac river

Q_C: Runoff of the Caulaman river

A_G: Drainage area of proposed Gumain dam site, 114 km^2

A_P: Drainage area of existing Porac diversion dam site, 111 km^2

A_C: Drainage area of existing Caulaman diversion dam site, 72 km^2

The runoff of three rivers were estimated on 10-days basis for 26 years from 1958 to 1983. The average annual runoff is 248 MCM in the Gumain river, 169 MCM in the Porac river and 141 MCM in the Caulaman river, respectively. The mean monthly runoff of three rivers are summarized as follows:

(Unit: $x10^3\ m^3$)

Month	Gumain R. (C.A = 114 km^2)	Porac R. (C.A = 111 km^2)	Caulaman R. (C.A = 72 km^2)
Jan.	4,838	3,298	2,750
Feb.	3,961	2,699	2,251
Mar.	4,153	2,830	2,360
Apr.	4,088	2,786	2,323
May	12,601	8,589	7,163
Jun.	23,582	16,073	13,404
Jul.	43,080	29,362	24,487
Aug.	60,129	40,983	34,179
Sep.	45,984	31,342	26,138
Oct.	25,705	17,520	14,611
Nov.	12,692	8,650	7,214
Dec.	6,715	4,577	3,817
Annual	247,527	168,709	140,699

C.A: Catchment area at the proposed Gumain dam on the Gumain river and at the existing diversion dams on the Porac and Caulaman rivers.

2.2.2 Methodology of the Water Balance Study

The essence of the Gumain River Irrigation Project is to supply the irrigation water to the project area including the existing irrigation areas, rainfed areas and sugarcane fields by providing a storage dam on the Gumain river.

To clarify the optimum scale of the dam and irrigation area, a water balance study was made on 10-day basis for 25 years from 1958/59 to 1982/83/1 based on the runoff of the Gumain, Porac and Caulaman rivers and the irrigation water demands estimated on the proposed cropping pattern, crop intensity and irrigable area.

The method and steps followed in the water balance study are summarized as follows:

1) Determination of cropping pattern by selecting the crops of paddy, vegetables and sugarcane,

2) Calculation of crop water requirements using the meteorological data in and around the project area,

3) Calculation of effective rainfalls based on the daily rainfall records at the Basa Air Base,

4) Estimation of diversion water requirements,

5) Calculation of creek discharges at the existing checkgate structures for utilization of the return flow,

6) Water balance at the re-use points,

7) Water balance at the existing Porac, Caulaman and Gumain diversion dam sites based on the unregulated flows of the Porac, Caulaman and Gumain rivers, and

8) Reservoir operation study of the Gumain dam based on the inflows of Gumain river.

The above methods and steps are illustrated as the flow chart in Fig. 5.1. And systematic diagram for the water balance study is shown in Fig. 5.2.

In the reservoir operation study, the following criteria concerning limitations for water shortages were utilized to determine the adequacy of project water supplies.

1) Maximum annual shortages should not be greater than 50 percent of the annual irrigation requirements,

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/1: A calculation year was set to be May to April based on the cropping calendar.

2) Maximum combined shortages in any two consecutive years should not be greater than 75 percent of the irrigation water requirements, and

3) The average annual shortage over the 1958/59 - 1982/83 period should not be greater than 7 percent.

2.2.3 Alternative Plans

(1) Irrigable Area

Out of the gross area of 23,700 ha in the study area, the irrigable area was estimated as follows based on the topographic maps with a scale of 1/4,000 taking into account the proposed irrigation canal alignment and topographic conditions.

Total irrigable areas (net)	:	16,750 ha
Paddy fields (net)	:	11,000 ha
Fields provided with irrigation facilities	:	5,970 ha
Fields without irrigation facilities	:	5,030 ha
Sugarcane fields (net)	:	5,750 ha

In the water balance study, the following two alternative plans were taken into consideration for the optimization of the irrigation area.

Alternative 1. To irrigate the existing paddy fields to the maximum extent during both the wet and dry seasons.

Alternative 2. To irrigate the existing paddy fields during the wet season and the sugarcane fields to the maximum extent, in this case to irrigate at least 5,970 ha of the paddy fields provided with the existing irrigation facilities during the dry season.

(2) Gumain Storage Dam

The proposed Gumain dam would be located on the Gumain river just downstream of the confluence of its two tributaries having a catchment area of 114 km^2. A fill type dam was adopted from the view point of the geological conditions of the foundations and abutments at the dam site.

Based on the analysis of geological conditions and availability of dam embankment materials around the dam site, the potential maximum dam scale was considered to be El. 170.0 m of full water level. The water balance study was made for the following six cases of the dam scale to determine the optimum dam scale in relation to the above two alternative plans for the optimization of the irrigation area.

Description	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Dam Crest Elevation (m)	176.5	167.5	160.0	155.0	150.0	140.0
Full Water Level (m)	170.0	161.0	153.5	148.5	143.5	133.5
Gross Storage Capacity (MCM)	176.3	137.6	110.4	94.4	80.2	56.6
Active Storage Capacity (MCM)	164.9	126.2	99.0	83.0	68.8	45.2
Dam Height (m)	124.5	115.5	108.0	103.0	98.0	88.0
Embankment Volume (MCM)	11.15	7.97	5.58	4.79	4.35	3.26

2.2.4 Result of the Water Balance Study

According to the methodology of the water balance study mentioned in Section 2.2.2, the irrigation area for each alternative plan was decided. The results of the reservoir operation study by alternatives are shown in Table 5.1.

The relation between the dam scale and the irrigation area is summarized as shown below from the result of the water balance study.

Dam Scale		Irrigation Area (ha)				
Case	F.W.L. (m)	Paddy (Wet Season)	Paddy (Dry Season)	Vegetables (Dry Season)	Sugarcane	Total/1
Alternative 1						
1	170.0	11,000	9,200	1,800	5,100	16,100
2	161.0	11,000	9,200	1,800	3,000	14,000
3	153.5	11,000	9,200	1,800	400	11,400
4	148.5	11,000	8,300	1,800	0	11,000
5	143.5	11,000	7,200	1,800	0	11,000
6	133.5	11,000	5,970/2	1,100	0	11,000
Alternative 2						
1	170.0	11,000	8,800	1,800	5,750	16,750
2	161.0	11,000	7,500	1,800	5,750	16,750
3	153.5	11,000	6,000	1,800	5,750	16,750
4	148.5	11,000	5,970/2	1,800	4,100	15,100
5	143.5	11,000	5,970/2	1,800	2,400	13,400
6	133.5	11,000	5,970/2	1,100	0	11,000

Remarks: /1: Wet season paddy + Sugarcane
/2: Area provided with the existing irrigation facilities.

2.3 Optimization Study

2.3.1 Cost Estimate and Benefit Analaysis

The optimum development plan was selected based on the result of economic evaluation for each alternative plan. For the evaluation, the project costs were estimated and the benefits also analyzed. The cost estimate and benefit analysis are described in details in Appendix X and Appendix IV, respectively.

(1) Construction Cost

The construction cost comprises the direct construction cost, the cost for operation and maintenance facilities, administration and engineering costs and physical contingency. The construction cost for each alternative plan was estimated as shown in Table 5.2 and is summarized below.

Dam Scale	Construction Cost (P10^6)	
	Alternative 1	Alternative 2
Case 1	2,386.5	2,390.6
Case 2	1,876.0	1,894.1
Case 3	1,491.8	1,530.5
Case 4	1,400.4	1,429.9
Case 5	1,346.3	1,363.8
Case 6	1,255.0	1,255.0

(2) Economic Construction Cost

Economic construction cost for evaluation is obtained by deducting the transfer payment such as direct/indirect taxes and levies from the financial cost. The transfer payment was assumed to be equivalent to 17.3% of the financial cost, which is explained in Appendix XI.

The disbursement schedule of economic construction cost for each alternative plan was estimated based on the tentative construction schedule and is presented in Table 5.3.

(3) Operation and Maintenance Costs

Operation and maintenance costs comprise personal cost, office expenses, operation cost, repairing cost and so on. Annual economic operation and maintenance costs for each alternative plan were estimated as shown in Table 5.4.

(4) Project Benefit

Project benefit to be expected is defined as the difference of primary profit from crops between future with project and without project conditions. On the basis of the estimated production cost and gross income, primary profit for each crop per ha was calculated both on future with and without project conditions. The economic project benefit at the full development stage is summarized as follows:

Dam Scale	Economic Project Benefit (₱10^6) Alternative 1	Alternative 2
Case 1	299.3	307.9
Case 2	256.2	293.1
Case 3	203.0	275.9
Case 4	184.5	241.7
Case 5	169.6	207.0
Case 6	133.2	133.4

The benefit flow for each alternative plan is shown in Table 5.5.

2.3.2 Economic Evaluation

Based on the benefit and economic costs, the Internal Rate of Return (IRR), Benefit Cost Ratio (B/C) and Net Present Value (B-C) were calculated under the assumption which the project life for the development project is 50 years for the evaluation. The results are summarized as follows:

Dam Scale	Alternative 1			Alternative 2		
	IRR (%)	B/C	B - C (₱10^6)	IRR (%)	B/C	B - C (₱10^6)
Case 1	9.8	0.97	-34	10.0	1.00	0
Case 2	10.5	1.07	61	11.4	1.19	207
Case 3	10.6	1.07	69	12.8	1.40	341
Case 4	10.3	1.04	32	12.2	1.30	246
Case 5	10.0	1.00	0	11.3	1.18	139
Case 6	8.8	1.85	-107	8.8	0.85	-107

Remarks: B/C and (B-C) were estimated at a discount rate of 10%.

The relation between dam scale and IRR is illustrated in Fig. 5.3.

2.3.3 Optimum Scale of the Development Plan

From the results of economic evaluation, Case 3 in Alternative 2 was selected as the most economical alternative showing the highest IRR of 12.4%. In determining the dam scale, Case 3 also appeared to be technically more sound than Cases 1 and 2, which would have required a saddle dam on the right bank and special attention to leakage.

Therefore, the optimum scale of the project was determined as follows:

Gumain Storage Dam

Crest elevation	:	El. 160.0 m
Full water level	:	El. 153.5 m
Low water level	:	El. 100.0 m
Gross storage capacity	:	110.4 MCM
Active storage capacity	:	99.0 MCM
Dam height	:	108.0 m
Crest length	:	435.0 m
Embankment volume	:	5.58 MCM

Irrigation Area

Paddy, wet season	:	11,000 ha
" , dry season	:	6,000 ha
Vegetables, dry season	:	1,800 ha
Sugarcane	:	5,750 ha

The results of water balance study at the existing diversion dams in Case 3 of Alternative 2 (final plan) are summarized in Table 5.6 and details are compiled in Data Book of the report.

Table 5.1(1) RESULTS OF RESERVOIR OPERATION STUDY

Alternative I - Case 1

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	9,200	5,100	1,800	16,100

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	176.30	312.46	176.35	4.91	245.16	0	0
1959/60	62.35	108.21	215.59	1.84	0	62.22	28.86
1960/61	15.35	330.35	181.10	3.49	113.83	0	0
1961/62	47.28	157.46	222.04	2.48	0	35.04	15.78
1962/63	15.25	265.90	215.15	3.40	57.35	40.03	18.61
1963/64	45.29	252.30	186.26	3.74	75.79	0	0
1964/65	31.80	201.11	159.10	3.78	0	0	0
1965/66	70.03	260.99	132.79	4.60	108.94	0	0
1966/67	84.70	416.12	160.24	4.60	284.62	0	0
1967/68	51.37	248.17	188.61	3.59	88.45	0	0
1968/69	18.88	191.87	203.17	2.62	0	6.44	3.17
1969/70	11.40	170.11	187.83	2.24	0	19.95	10.62
1970/71	11.40	277.46	141.31	3.86	70.19	12.27	8.69
1971/72	85.29	337.37	106.90	4.98	186.70	0	0
1972/73	124.57	491.22	159.77	4.75	381.50	0	0
1973/74	69.77	210.50	181.80	3.86	45.26	0	0
1974/75	49.36	280.88	174.71	4.38	58.76	0	0
1975/76	92.40	163.63	211.44	3.37	0	0	0
1976/77	41.21	317.15	175.91	4.28	110.07	0	0
1977/78	68.11	251.97	224.45	3.54	62.82	0	0
1978/79	29.28	212.08	215.33	2.93	8.02	5.46	2.54
1979/80	20.54	199.04	230.64	2.54	0	25.00	10.84
1980/81	11.40	172.60	230.58	2.24	0	60.23	26.12
1981/82	11.40	159.08	218.04	2.04	0	61.00	27.98
1982/83	11.46	311.04	216.66	3.54	80.93	40.76	18.81
Average					79.13		6.88

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(2) RESULTS OF RESERVOIR OPERATION STUDY

Alternative I - Case 2

	Paddy (Wet)	Paddy (Dry)	Vegetable (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	9,200	3,000	1,800	14,000

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	137.60	312.46	165.81	3.88	245.60	0	0
1959/69	34.78	108.21	196.63	1.67	0	71.35	36.29
1960/61	16.04	330.35	160.69	2.84	158.44	0	0
1961/62	24.42	157.46	198.78	2.14	0	34.49	17.35
1962/63	15.44	265.90	190.23	2.75	97.27	32.15	16.90
1963/64	23.24	252.30	164.08	2.89	97.26	0.10	0.06
1964/65	11.40	201.11	142.43	3.15	12.73	0.04	0.03
1965/66	54.23	260.99	119.64	3.75	133.42	0	0
1966/67	58.42	416.12	143.08	3.63	300.09	0	0
1967/68	27.74	248.17	167.27	2.77	108.18	13.72	8.20
1968/69	11.40	191.87	181.96	2.35	21.29	13.73	7.55
1969/70	11.40	170.11	166.15	2.41	0	8.04	4.84
1970/71	21.00	277.46	124.06	3.27	111.20	0	0
1971/72	59.94	337.37	95.81	4.08	202.23	0	0
1972/73	95.19	491.22	140.12	3.85	395.33	0	0
1973/74	47.11	210.50	159.39	3.02	69.07	0	0
1974/75	26.13	280.88	153.97	3.49	85.09	0	0
1975/76	64.47	163.63	186.46	2.89	0	0	0
1976/77	38.74	317.15	157.82	3.54	152.66	0	0
1977/78	41.88	251.97	197.27	2.65	85.97	3.44	1.75
1978/79	11.40	212.08	190.81	2.29	47.11	28.13	14.74
1979/80	11.40	199.04	204.72	2.45	12.32	20.46	9.99
1980/81	11.40	172.60	203.18	2.36	0	32.93	16.21
1981/82	11.40	159.08	192.36	2.13	0	35.56	18.49
1982/83	11.54	311.04	188.48	2.91	122.19	31.91	16.93
Average					98.30		6.77

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(3) RESULTS OF RESERVOIR OPERATION STUDY

Alternative I - Case 3

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	9,200	400	1,800	11,400

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	110.40	312.46	153.72	3.14	245.91	0	0
1959/60	20.09	108.21	174.78	1.65	0	64.60	36.96
1960/61	16.47	330.35	138.59	2.43	191.33	0	0
1961/62	14.47	157.46	172.67	2.06	4.64	25.01	14.48
1962/63	17.57	265.90	162.04	2.35	125.63	21.35	13.18
1963/64	14.80	252.30	138.88	2.47	120.95	8.52	6.13
1964/65	13.31	201.11	123.92	2.80	47.10	0	0
1965/66	40.61	260.99	106.18	3.13	148.62	0	0
1966/67	43.67	416.12	124.15	2.99	315.82	0	0
1967/68	16.84	248.17	144.48	2.36	127.20	21.93	15.18
1968/69	12.90	191.87	158.32	2.07	49.84	16.87	10.65
1969/70	11.40	170.11	141.94	2.26	18.95	3.34	2.35
1970/71	21.70	277.46	105.60	2.86	144.26	0	0
1971/72	46.44	337.37	84.78	3.46	218.94	0	0
1972/73	76.64	491.22	119.44	3.27	407.49	0	0
1973/74	37.66	210.50	136.07	2.52	93.50	0	0
1974/75	16.07	280.88	132.40	2.93	113.94	0	0
1975/76	47.69	163.63	159.26	2.52	15.66	0	0
1976/77	33.88	317.15	138.36	2.99	181.42	0	0
1977/78	28.27	251.97	165.97	2.24	111.28	12.33	7.43
1978/79	13.09	212.08	163.19	2.03	74.75	29.76	18.23
1979/80	14.95	199.04	175.99	2.17	42.37	19.32	10.98
1980/81	12.78	172.60	172.09	2.12	31.43	32.75	19.03
1981/82	12.48	159.08	164.83	2.28	0.23	12.07	7.33
1982/83	16.29	311.04	158.29	2.54	152.48	17.72	11.20
Average					119.35		6.93

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(4) RESULTS OF RESERVOIR OPERATION STUDY

Alternative I - Case 4

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	8,300	0	1,800	11,000

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	94.40	312.46	137.34	2.85	247.15	0	0
1959/60	19.52	108.21	158.00	1.71	0	48.49	30.69
1960/61	16.51	330.35	121.58	2.26	208.55	0	0
1961/62	14.47	157.46	153.31	1.96	21.97	23.10	15.07
1962/63	17.79	265.90	144.33	2.19	142.08	19.92	13.80
1963/64	15.01	252.30	121.41	2.31	137.91	8.06	6.64
1964/65	13.73	201.11	109.03	2.60	66.00	0	0
1965/66	37.21	260.99	92.00	2.88	162.12	0	0
1966/67	41.19	416.12	109.26	2.71	330.41	1.46	1.34
1967/68	16.40	248.17	126.53	2.20	143.53	20.92	16.53
1968/69	13.22	191.87	139.34	1.95	66.68	14.65	10.52
1969/70	11.77	170.11	124.51	2.14	35.38	2.50	2.00
1970/71	22.34	277.46	92.11	2.67	162.24	0	0
1971/72	42.79	337.37	73.81	3.16	233.32	0	0
1972/73	69.86	491.22	102.22	3.03	418.14	0	0
1973/74	37.69	210.50	119.86	2.36	110.73	0	0
1974/75	15.25	280.88	119.89	2.69	130.90	0	0
1975/76	42.65	163.63	143.15	2.34	28.66	0	0
1976/77	32.14	317.15	121.50	2.75	197.31	0	0
1977/78	27.73	251.97	147.29	2.09	129.58	12.82	8.71
1978/79	13.57	212.08	144.45	1.90	90.87	26.95	18.66
1979/80	15.38	199.04	157.19	2.05	59.44	17.48	11.12
1980/81	13.22	172.60	153.81	2.00	48.13	30.98	20.14
1981/82	12.87	159.08	146.82	2.18	17.19	10.91	7.43
1982/83	16.66	311.04	138.78	2.41	169.30	16.18	11.66
Average					134.40		6.97

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(5) RESULTS OF RESERVOIR OPERATION STUDY

Alternative I - Case 5

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	7,200	0	1,800	11,000

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	80.20	312.46	118.99	2.63	248.50	0	0
1959/60	22.55	108.21	141.28	1.75	3.21	32.00	22.65
1960/61	16.51	330.35	104.28	2.15	223.46	0	0
1961/62	16.98	157.46	134.15	1.89	39.16	18.56	13.84
1962/63	17.80	265.90	127.31	2.07	156.62	19.91	15.64
1963/64	17.60	252.30	103.95	2.16	155.53	5.47	5.26
1964/65	13.73	201.11	94.31	2.40	83.20	0	0
1965/66	34.93	260.99	77.18	2.67	174.89	0	0
1966/67	41.19	416.12	94.48	2.47	345.20	1.25	1.33
1967/68	16.41	248.17	108.16	2.05	158.28	17.13	15.84
1968/69	13.22	191.87	120.09	1.82	82.44	11.06	9.21
1969/70	11.78	170.11	106.65	2.06	49.89	2.49	2.33
1970/71	25.77	277.46	78.19	2.52	181.42	0	0
1971/72	41.10	337.37	62.62	2.90	247.79	0	0
1972/73	65.16	491.22	85.12	2.82	428.50	0	0
1973/74	39.94	210.50	103.64	2.25	127.78	0	0
1974/75	16.77	280.88	107.94	2.50	147.57	0	0
1975/76	39.65	163.63	128.21	2.21	41.09	0.96	0.75
1976/77	32.72	317.15	104.50	2.56	212.86	0	0
1977/78	29.95	251.97	129.07	1.98	147.81	10.51	8.14
1978/79	13.58	212.08	125.78	1.78	105.15	22.45	17.85
1979/80	15.39	199.04	138.77	1.92	75.08	14.57	10.50
1980/81	13.23	172.60	136.25	1.88	62.48	27.65	20.30
1981/82	12.87	159.08	129.20	2.04	32.33	8.30	6.42
1982/83	16.67	311.04	120.22	2.29	184.84	16.18	13.46
Average					148.60		6.54

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(6) RESULTS OF RESERVOIR OPERATION STUDY

Alternative I - Case 6

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	5,970	0	1,100	11,000

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	56.60	312.46	91.20	2.15	250.08	0	0
1959/60	25.63	108.21	114.96	1.55	26.87	27.05	23.53
1960/61	17.52	330.35	77.41	1.84	250.33	0	0
1961/62	18.30	157.46	104.41	1.67	64.55	13.25	12.69
1962/63	18.38	265.90	100.43	1.79	180.70	19.33	19.25
1963/64	20.68	252.30	75.82	1.83	183.65	3.08	4.06
1964/65	14.76	201.11	70.54	2.01	111.44	0	0
1965/66	31.88	260.99	54.01	2.23	197.36	0	0
1966/67	39.27	416.12	70.62	2.00	367.51	1.89	2.67
1967/68	17.14	248.17	78.54	1.75	183.30	12.75	16.23
1968/69	14.46	191.87	89.94	1.60	107.92	6.06	6.74
1969/70	12.94	170.11	79.35	1.83	74.30	1.34	1.68
1970/71	28.91	277.46	56.14	21.3	213.10	0	0
1971/72	35.00	33.737	44.93	2.38	268.46	0	0
1972/73	56.60	491.22	59.39	2.36	445.24	0	0
1973/74	40.83	210.50	78.28	1.92	153.68	0	0
1974/75	17.45	280.88	88.70	2.03	173.83	0	0
1975/76	33.78	163.63	105.02	1.89	64.97	6.77	6.45
1976/77	32.30	317.15	78.20	2.15	237.28	0	0
1977/78	31.83	251.97	99.48	1.73	175.61	7.90	7.94
1978/79	14.88	212.08	96.19	1.55	128.89	15.83	16.46
1979/80	16.17	199.04	109.73	1.64	101.21	11.94	10.88
1980/81	14.56	172.60	107.25	1.63	87.61	23.47	21.89
1981/82	14.15	159.08	101.71	1.74	57.93	5.61	5.51
1982/83	17.45	311.04	92.94	2.00	210.27	15.40	16.63
Average					172.64		6.90

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(7) RESULTS OF RESERVOIR OPERATION STUDY

Alternative II - Case 1

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	8,800	5,750	1,800	16,750

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	176.30	312.46	172.89	4.97	245.62	0	0
1959/60	65.29	108.21	215.51	1.87	0	58.97	27.36
1960/61	15.09	330.35	181.19	3.51	112.07	0	0
1961/62	48.67	157.46	222.16	2.48	0	33.70	15.17
1962/63	15.19	265.90	216.85	3.43	57.03	42.66	19.67
1963/64	46.43	252.30	186.65	3.77	75.61	0	0
1964/65	32.69	201.11	158.93	3.80	0	0	0
1965/66	71.07	260.99	131.20	4.64	109.84	0	0
1966/67	86.38	416.12	160.11	4.63	285.69	0	0
1967/68	52.07	248.17	188.67	3.63	87.85	0	0
1968/69	20.09	191.87	202.65	2.66	0	4.75	2.34
1969/70	11.40	170.11	187.87	2.26	0	20.01	10.65
1970/71	11.40	277.46	141.62	3.88	70.33	13.68	9.66
1971/72	86.72	337.37	106.25	5.00	187.62	0	0
1972/73	125.21	491.22	159.70	4.79	380.72	0	0
1973/74	71.22	210.50	182.83	3.90	44.49	0	0
1974/75	50.50	280.88	176.83	4.40	56.76	0	0
1975/76	93.39	163.63	213.69	3.36	0	0	0
1976/77	39.97	317.15	175.22	4.31	107.23	0	0
1977/78	70.37	251.97	226.24	3.58	62.50	0	0
1978/79	30.02	212.08	216.24	2.97	7.91	6.57	3.04
1979/80	21.55	199.04	231.80	2.56	0	25.18	10.86
1980/81	11.40	172.60	232.57	2.24	0	62.21	26.75
1981/82	11.40	159.08	219.67	2.02	0	62.62	28.51
1982/83	11.40	311.04	218.46	3.58	80.26	43.45	19.89
Average					78.86		6.96

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(8) RESULTS OF RESERVOIR OPERATION STUDY

Alternative II - Case 2

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	7,500	5,750	1,800	16,750

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Percent (%)
1958/59	137.60	312.46	151.11	4.14	247.52	0	0
1959/60	47.30	108.21	195.72	1.73	0	57.03	29.14
1960/61	15.09	330.35	160.45	2.95	151.59	0	0
1961/62	30.45	157.46	198.93	2.17	0	28.39	14.27
1962/63	15.19	265.90	196.29	2.90	96.32	42.66	21.73
1963/64	28.24	252.30	165.63	3.06	96.70	0	0
1964/65	15.15	201.11	140.79	3.27	12.52	0	0
1965/66	59.68	260.99	112.77	3.93	138.34	0	0
1966/67	65.63	416.12	142.29	3.78	304.68	0	0
1967/68	31.00	248.17	166.82	2.87	106.50	8.43	5.05
1968/69	11.40	191.87	179.45	2.48	20.69	12.36	6.89
1969/70	13.00	170.11	165.82	2.56	0	12.29	7.41
1970/71	27.03	277.46	124.84	3.41	112.10	0	0
1971/72	64.14	337.37	92.16	4.18	206.36	0	0
1972/73	98.81	491.22	137.76	4.02	394.67	0	0
1973/74	53.58	210.50	163.48	3.19	66.22	0	0
1974/75	31.20	280.88	162.55	3.60	76.91	0	0
1975/76	69.03	163.63	195.68	2.84	0	0	0
1976/77	34.14	317.15	154.57	3.67	141.35	0	0
1977/78	51.71	251.97	204.54	2.84	85.12	0.22	0.11
1978/79	11.40	212.08	194.10	2.40	46.69	31.12	16.03
1979/80	11.40	199.04	209.93	2.61	9.86	27.06	12.95
1980/81	16.00	172.60	211.45	2.39	0	36.64	17.33
1981/82	11.40	159.08	198.48	2.10	0	41.50	20.91
1982/83	11.40	311.04	195.59	3.06	119.61	43.45	22.22
Average					97.35		6.96

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(9) RESULTS OF RESERVOIR OPERATION STUDY

Alternative II - Case 3

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	6,000	5,750	1,800	16,750

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/69	110.40	312.46	127.21	3.65	248.96	0	0
1959/60	43.05	108.21	172.87	1.79	0	38.49	22.27
1960/61	15.09	330.35	136.53	2.66	179.67	0	0
1961/62	26.58	157.46	172.13	2.22	0.86	6.36	3.69
1962/63	15.19	265.90	172.62	2.62	124.10	42.66	24.72
1963/64	24.41	252.30	141.40	2.73	120.54	0	0
1964/65	12.04	201.11	120.06	2.92	41.22	0.92	0.76
1965/66	49.87	260.99	91.56	3.49	156.99	0	0
1966/67	58.82	416.12	121.90	3.28	325.89	0	0
1967/68	23.88	248.17	141.67	2.52	127.48	11.02	7.78
1968/69	11.40	191.87	152.67	2.27	47.92	12.36	8.09
1969/70	12.76	170.11	140.71	2.56	17.12	12.53	8.90
1970/71	35.02	277.46	105.73	3.13	150.62	0	0
1971/72	53.00	337.37	76.03	3.66	225.17	0	0
1972/73	85.51	491.22	112.87	3.59	409.86	0	0
1973/74	50.40	210.50	141.23	2.86	90.81	0	0
1974/75	26.01	280.88	146.41	3.14	100.57	0	0
1975/76	56.76	163.63	175.11	2.74	8.66	7.74	4.42
1976/77	41.63	317.15	131.06	3.35	177.02	0	0
1977/78	47.35	251.97	179.50	2.51	110.70	4.78	2.66
1978/79	11.40	212.08	168.56	2.15	74.01	32.65	19.37
1979/80	11.40	199.04	183.29	2.39	38.59	27.06	14.76
1980/81	13.23	172.60	187.09	2.28	21.71	36.65	19.59
1981/82	11.40	159.08	174.03	2.24	0	17.20	9.89
1982/83	11.40	311.04	169.28	2.81	147.42	43.45	25.67
Average					117.84		6.90

Remarks: /1 May to April based on the cropping calendar.

Table 5.1(10) RESULTS OF RESERVOIR OPERATION STUDY

Alternative II - Case 4

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	5,970	4,100	1,800	15,100

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	94.40	312.46	118.52	3.22	249.21	0	0
1959/60	35.91	108.21	157.24	1.81	0	30.69	19.52
1960/61	15.76	330.35	119.87	2.44	200.93	0	0
1961/62	22.87	157.46	153.32	2.05	19.26	9.66	6.30
1962/63	15.35	265.90	152.30	2.38	140.96	36.15	23.74
1963/64	21.76	252.30	123.47	2.47	137.83	1.11	0.90
1964/65	11.40	201.11	106.34	2.66	61.29	0.65	0.61
1965/66	42.86	260.99	81.18	3.13	167.11	0	0
1966/67	52.43	416.12	107.88	2.92	337.76	0	0
1967/68	20.00	248.17	124.37	2.29	143.12	13.01	10.46
1968/69	11.40	191.87	135.32	2.09	65.10	10.89	8.05
1969/70	11.64	170.11	123.57	2.36	34.04	10.14	8.20
1970/71	31.93	277.46	91.86	2.85	168.19	0	0
1971/72	46.50	337.37	66.99	3.30	236.60	0	0
1972/73	76.97	491.22	96.01	3.26	421.55	0	0
1973/74	47.37	210.50	123.06	2.61	109.89	0	0
1974/75	22.31	280.88	129.54	2.84	121.44	0	0
1975/76	49.37	163.63	154.98	2.51	25.40	7.97	5.14
1976/77	38.09	317.15	116.31	3.01	194.45	0	0
1977/78	41.47	251.97	157.52	2.26	129.18	6.92	4.39
1978/79	11.40	212.08	148.58	1.95	90.38	28.82	19.40
1979/80	11.40	199.04	162.89	2.16	57.59	23.60	14.49
1980/81	11.40	172.60	165.31	2.07	41.67	36.45	22.05
1981/82	11.40	159.08	153.16	2.27	9.72	8.79	5.74
1982/83	14.11	311.04	146.56	2.60	165.91	33.91	23.14
Average					133.14		6.59

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(11) RESULTS OF RESERVOIR OPERATION STUDY

Alternative II - Case 5

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	5,970	2,400	1,800	13,400

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	80.20	312.46	110.04	2.81	249.40	0	0
1959/60	30.40	108.21	142.27	1.83	2.34	23.97	16.85
1960/61	16.15	330.35	104.20	2.22	219.91	0	0
1961/62	20.17	157.46	135.23	1.91	36.34	11.74	8.68
1962/63	15.87	265.90	132.62	2.16	155.95	29.60	22.32
1963/64	20.65	252.30	105.91	2.22	155.12	1.71	1.61
1964/65	11.40	201.11	93.63	2.42	79.11	0	0
1965/66	37.35	260.99	71.35	2.79	176.84	0	0
1966/67	47.36	416.12	94.20	2.57	349.14	0	0
1967/68	17.58	248.17	107.69	2.08	157.88	13.59	12.62
1968/69	11.69	191.87	119.24	1.88	81.74	10.71	8.98
1969/70	11.40	170.11	106.70	2.17	49.07	6.85	6.42
1970/71	30.42	277.46	78.39	2.59	185.31	0	0
1971/72	41.59	337.37	59.15	2.96	247.83	0	0
1972/73	69.02	491.22	82.30	2.94	430.24	0	0
1973/74	44.76	210.50	105.62	2.37	127.68	0	0
1974/75	19.60	280.88	113.08	2.56	141.70	0	0
1975/76	43.14	163.63	134.92	2.29	40.31	6.87	5.09
1976/77	36.11	317.15	102.07	2.69	211.60	0	0
1977/78	36.89	251.97	135.66	2.05	147.12	7.37	5.43
1978/79	11.40	212.08	128.89	1.78	104.86	23.46	18.20
1979/80	11.40	199.04	143.11	1.94	74.13	20.14	14.07
1980/81	11.40	172.60	143.49	1.90	59.94	32.73	22.81
1981/82	11.40	159.08	133.03	2.07	27.98	5.75	4.33
1982/83	13.51	311.04	124.42	2.38	182.79	27.83	22.37
Average					147.77		6.79

Remarks: /1: May to April based on the cropping calendar.

Table 5.1(12) RESULTS OF RESERVOIR OPERATION STUDY

Alternative II - Case 6

	Paddy (Wet)	Paddy (Dry)	Vegetables (Dry)	Sugarcane	Total
Irrigation Area (ha)	11,000	5,970	0	1,100	11,000

Year/1	Reservoir Volume (MCM)	Inflow (MCM)	Water Requirement (MCM)	Evapo-ration (MCM)	Spill-out (MCM)	Water Deficit Amount (MCM)	Water Deficit Percent (%)
1958/59	56.60	312.46	91.20	2.15	250.08	0	0
1959/60	25.63	108.21	114.96	1.55	26.87	27.05	23.53
1960/61	17.52	330.35	77.41	1.84	250.33	0	0
1961/62	18.30	157.46	104.41	1.67	64.55	13.25	12.69
1962/63	18.38	265.90	100.43	1.79	180.70	19.33	19.25
1963/64	20.68	252.30	75.82	1.83	183.65	3.08	4.06
1964/65	14.76	201.11	70.54	2.01	111.44	0	0
1965/66	31.88	260.99	54.01	2.23	197.36	0	0
1966/67	39.27	416.12	70.62	2.00	367.51	1.89	2.67
1967/68	17.14	248.17	78.54	1.75	183.30	12.75	16.23
1968/69	14.46	191.87	89.94	1.60	107.92	6.06	6.74
1969/70	12.94	170.11	79.35	1.83	74.30	1.34	1.68
1970/71	28.91	277.46	56.14	2.13	213.10	0	0
1971/72	35.00	337.37	44.93	2.38	268.46	0	0
1972/73	56.60	491.22	59.39	2.36	445.24	0	0
1973/74	40.83	210.50	78.28	1.92	153.68	0	0
1974/75	17.45	280.88	88.70	2.03	173.83	0	0
1975/76	33.78	163.63	105.02	1.89	64.97	6.77	6.45
1976/77	32.30	317.15	78.20	2.15	237.28	0	0
1977/78	31.83	251.97	99.48	1.73	175.61	7.90	7.94
1978/79	14.88	212.08	96.19	1.55	128.89	15.83	16.46
1979/80	16.17	199.04	109.73	1.64	101.21	11.94	10.88
1980/81	14.56	172.60	107.25	1.63	87.61	23.47	21.89
1981/82	14.15	159.08	101.71	1.74	57.93	5.61	5.51
1982/83	17.45	311.04	92.64	2.00	210.27	15.40	16.63
Average					172.64		6.90

Remarks: /1: May to April based on the cropping calendar.

Table 5.2 CONSTRUCTION COSTS FOR ALTERNATIVE PLANS

(Unit: P10^6)

Item	Alternative - I						Alternative - II					
	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
1. Direct Construction Cost												
1.1 Gumain Dam	1,711.6	1,281.2	966.7	889.8	842.9	763.4	1,711.6	1,281.2	966.7	889.8	842.9	763.4
1.2 Diversion Dam	26.8	26.5	24.9	25.5	25.5	25.5	26.6	26.2	26.0	25.7	25.6	25.5
1.3 Irrigation Facilities	141.8	133.7	123.2	120.0	120.0	120.0	142.9	142.9	141.5	135.3	129.1	120.0
1.4 Drainage Facilities	40.0	34.9	27.4	27.4	27.4	27.4	41.7	41.7	41.7	37.6	33.3	27.4
Sub-total:	1,920.2	1,476.3	1,142.2	1,062.7	1,015.8	936.3	1,923.8	1,492.0	1,175.9	1,088.4	1,030.9	936.3
2. Cost for O & M Facilities	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0	11.0
3. Administration and Engineering Costs	144.0	144.0	144.0	144.0	144.0	144.0	144.0	144.0	144.0	144.0	144.0	144.0
4. Physical Contingency	311.3	244.7	194.6	182.7	175.5	163.7	311.8	247.1	199.6	186.5	177.9	163.7
Total	2,386.5	1,876.0	1,491.8	1,400.4	1,346.3	1,255.0	2,390.6	1,894.1	1,530.5	1,429.9	1,363.8	1,255.0

Table 5.3 DISBURSEMENT SCHEDULE OF ECONOMIC CONSTRUCTION COSTS FOR ALTERNATIVE PLANS

Alternative/Case	Total	Year in Order							
		1st	2nd	3rd	4th	5th	6th	7th	8 - 50th
Alternative I									
Case 1	1,964.5	39.9	290.2	347.6	384.2	382.5	324.6	195.5	0
Case 2	1,542.4	39.9	226.8	271.7	299.8	298.0	253.0	153.2	0
Case 3	1,224.5	39.9	179.2	214.5	236.2	234.4	199.0	121.3	0
Case 4	1,148.8	39.9	167.8	200.9	221.1	219.2	186.1	113.8	0
Case 5	1,104.3	39.9	161.1	192.9	212.2	210.4	178.4	109.4	0
Case 6	1,028.7	39.9	149.7	179.2	197.1	195.3	165.2	101.8	0
Alternative II									
Case 1	1,967.7	39.9	290.7	348.2	384.9	383.2	325.2	195.6	0
Case 2	1,557.2	39.9	229.0	274.4	302.8	301.0	255.4	154.7	0
Case 3	1,256.8	39.9	184.0	220.2	242.7	240.9	204.4	124.7	0
Case 4	1,173.3	39.9	171.4	205.3	226.0	224.2	190.2	116.3	0
Case 5	1,118.8	39.9	163.3	195.5	215.0	213.3	181.0	110.8	0
Case 6	1,028.7	39.9	149.7	179.2	197.1	195.3	165.7	101.8	0

Table 5.4 ANNUAL ECONOMIC OPERATION AND MAINTENANCE COSTS FOR ALTERNATIVE PLANS

(Unit: ₱10^6)

Alternative/Case	Year in Order			
	5th	6th	7th	8 - 50th
Alternative I				
Case 1	0.8	1.6	1.6	3.7
Case 2	0.8	1.6	1.6	3.2
Case 3	0.8	1.6	1.6	2.6
Case 4	0.8	1.6	1.6	2.6
Case 5	0.8	1.6	1.6	2.6
Case 6	0.8	1.6	1.6	2.6
Alternative II				
Case 1	0.8	1.6	1.6	3.9
Case 2	0.8	1.6	1.6	3.9
Case 3	0.8	1.6	1.6	3.9
Case 4	0.8	1.6	1.6	3.5
Case 5	0.8	1.6	1.6	3.1
Case 6	0.8	1.6	1.6	2.6

Table 5.5 BENEFIT FLOW FOR ALTERNATIVE PLANS

(Unit: ₱10^6)

Alternative/Case	Year in Order								
	4th	5th	6th	7th	8th	9th	10th	11th	12 - 50th
Alternative I									
Case 1	-	4.5	13.5	22.5	82.3	142.2	197.7	248.5	299.3
Case 2	-	4.5	13.5	22.5	73.7	125.0	171.8	214.0	256.2
Case 3	-	4.5	13.5	22.5	63.1	103.7	139.9	171.4	203.0
Case 4	-	4.3	13.0	21.7	58.6	95.5	128.1	156.3	184.5
Case 5	-	4.1	12.3	20.5	54.4	88.4	118.2	143.9	169.6
Case 6	-	3.9	11.7	19.6	46.2	72.8	95.6	114.4	133.2
Alternative II									
Case 1	-	4.4	13.3	22.2	83.6	145.3	202.4	255.2	307.9
Case 2	-	4.2	12.6	21.0	79.6	138.3	192.7	242.9	293.1
Case 3	-	3.9	11.7	19.5	74.7	129.9	181.2	228.6	275.9
Case 4	-	3.9	11.7	19.6	67.9	116.2	160.7	201.2	241.7
Case 5	-	3.9	11.7	19.6	61.0	102.3	139.9	173.4	207.0
Case 6	-	3.9	11.7	19.6	46.2	72.0	95.8	114.6	133.4

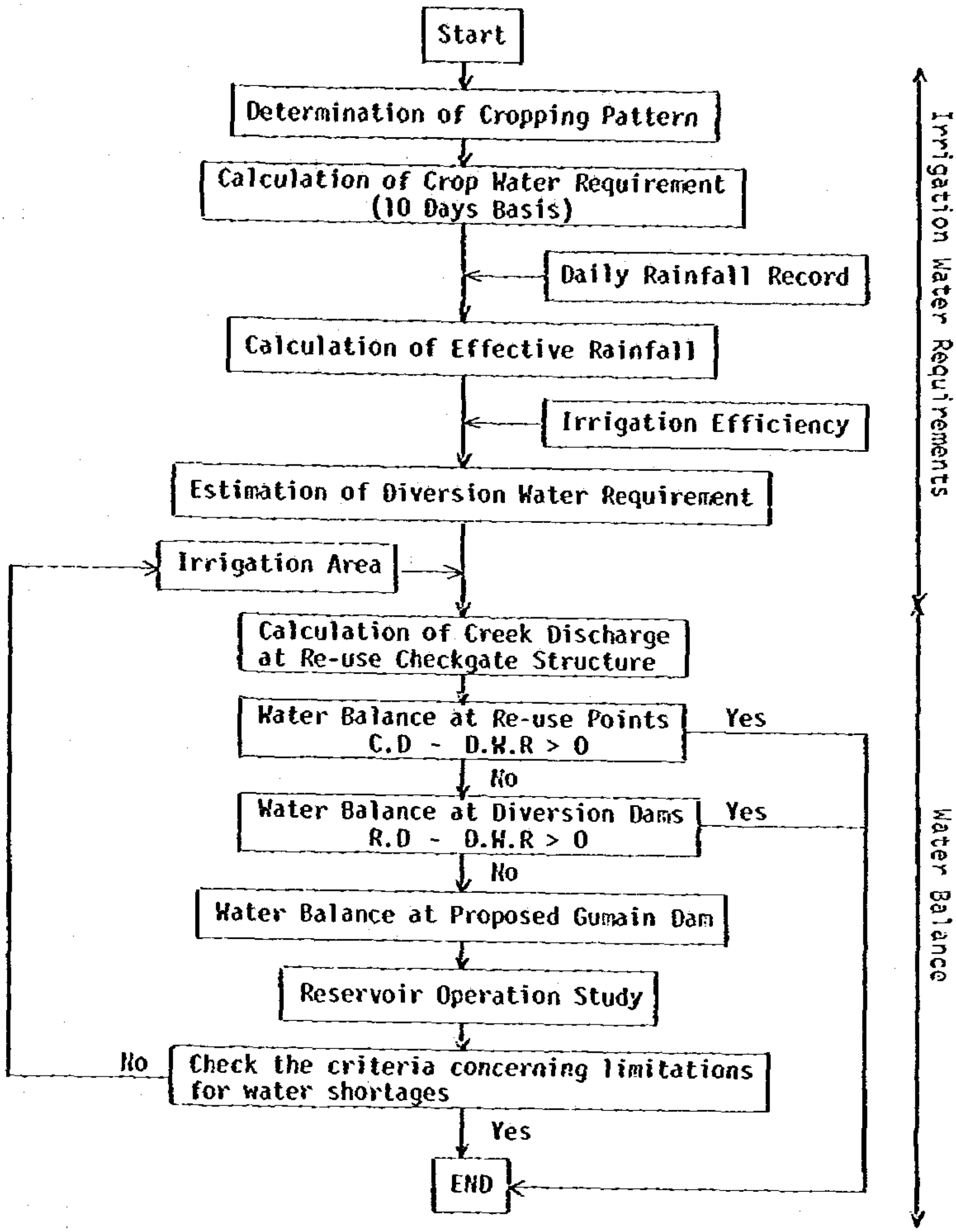
Table 5.6 RESULTS OF WATAR BALANCE STUDY AT THE EXISTING DIVERSION
(ALTERNATIVE 2 - CASE 3)

(Unit: MCM)

Year	Porac Diversion Dam			Caulaman Diversion Dam			Gumain Diversion Dam			Total		
	D.W.R	Deficit	%	D.W.R	Deficit	%	D.W.R	Deficit	%	D.W.R	Deficit	%
1958/59	93.05	-47.64	51.2	44.66	-17.42	39.0	52.52	-40.04	76.2	190.23	-105.10	55.2
1959/60	86.59	-64.19	74.1	56.03	-37.11	66.2	51.47	-45.43	88.3	194.09	-146.73	75.6
1960/61	78.40	-46.43	59.2	56.45	-29.22	51.8	46.21	-35.67	77.2	181.06	-111.32	61.5
1961/62	93.24	-60.51	64.9	63.95	-36.00	56.3	54.80	-46.54	84.9	211.99	-143.05	67.5
1962/63	91.37	-58.90	64.5	66.70	-38.47	57.7	54.48	-45.37	83.3	212.55	-142.74	67.2
1963/64	77.30	-46.62	60.3	58.29	-32.36	55.5	45.54	-36.93	81.1	181.13	-115.91	64.0
1964/65	77.85	-39.89	51.2	51.81	-21.80	42.1	46.15	-34.78	75.4	175.81	-94.47	54.9
1965/66	68.88	-29.29	42.5	44.32	-14.97	33.8	40.22	-27.08	67.3	153.42	-71.43	46.6
1966/67	81.28	-38.45	47.3	58.22	-24.47	42.0	48.62	-32.96	67.8	188.12	-95.88	51.0
1967/68	74.09	-48.92	66.0	54.28	-33.24	61.2	43.34	-35.75	82.5	171.71	-117.91	68.7
1968/69	83.07	-53.79	64.8	55.60	-31.74	57.1	48.22	-41.90	86.9	186.89	-127.43	68.2
1969/70	83.19	-45.73	55.0	57.92	-28.48	49.2	48.74	-40.39	82.9	189.85	-114.60	60.4
1970/71	67.06	-31.73	47.3	48.09	-23.45	48.8	39.73	-29.26	73.6	154.93	-84.44	54.5
1971/72	72.61	-18.93	26.1	48.26	-8.11	16.8	43.22	-26.97	62.4	164.09	-54.01	32.9
1972/73	77.50	-33.85	43.7	57.62	-19.25	33.4	45.61	-34.38	75.4	180.73	-87.48	48.4
1973/74	77.86	-45.11	57.9	58.04	-31.90	55.0	46.47	-38.52	82.9	182.37	-115.53	63.3
1974/75	80.33	-47.53	59.2	57.68	-32.21	55.8	48.96	-40.47	82.7	186.97	-120.21	64.3
1975/76	100.02	-57.54	57.5	70.24	-36.87	52.5	60.05	-48.51	80.8	230.31	-142.92	62.1
1976/77	88.25	-42.17	47.8	60.68	-23.05	38.0	52.36	-38.14	72.8	201.29	-103.36	51.3
1977/78	93.74	-59.77	63.8	71.01	-42.20	59.4	56.28	-46.10	81.9	221.03	-148.07	67.0
1978/79	84.70	-58.44	69.0	61.56	-39.63	64.4	50.09	-43.16	86.2	196.35	-141.23	71.9
1979/80	110.00	-58.52	53.2	75.84	-37.57	49.5	65.49	-52.46	80.1	251.33	-148.55	59.1
1980/81	103.28	-60.69	58.8	74.38	-42.50	57.1	62.26	-50.23	80.7	239.92	-153.42	63.9
1981/82	97.02	-55.72	57.4	68.74	-37.62	54.7	57.51	-49.62	86.3	223.27	-142.96	64.0
1982/83	108.46	-51.22	47.2	80.39	-34.18	42.5	65.01	-47.66	73.3	253.86	-133.06	52.4
Average	85.97	-48.06	55.9	60.03	-30.15	50.2	50.94	-40.33	79.2	196.93	-118.55	60.2

Remarks: D.W.R: Diversion Water Requirement

Fig. 5.1 FLOW CHART OF WATER BALANCE STUDY



C.D: Creek Discharge
R.D: River Discharge
D.W.R: Diversion Water Requirement

Fig. 5.2 SYSTEMATIC DIAGRAM FOR WATER BALANCE STUDY

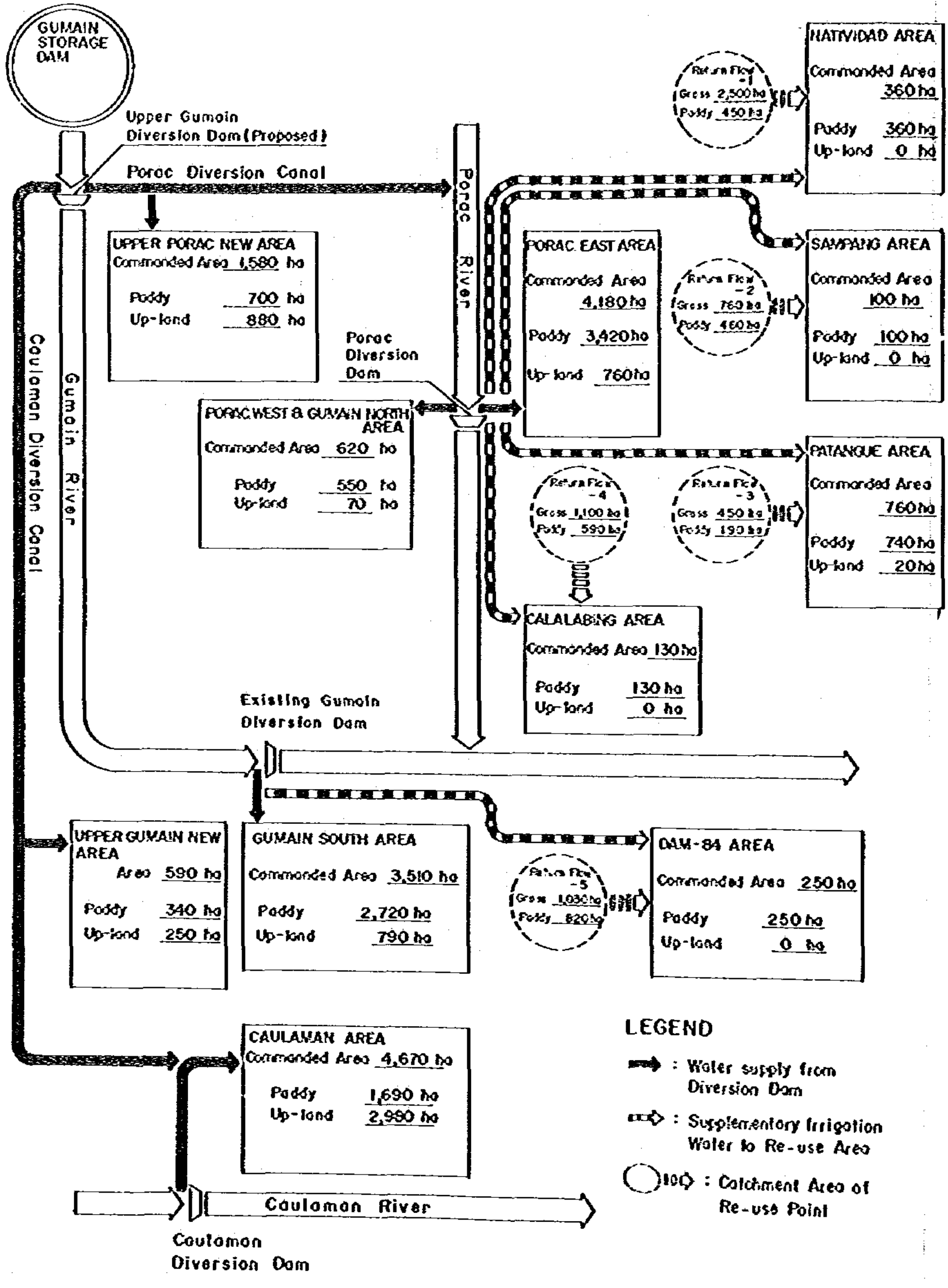
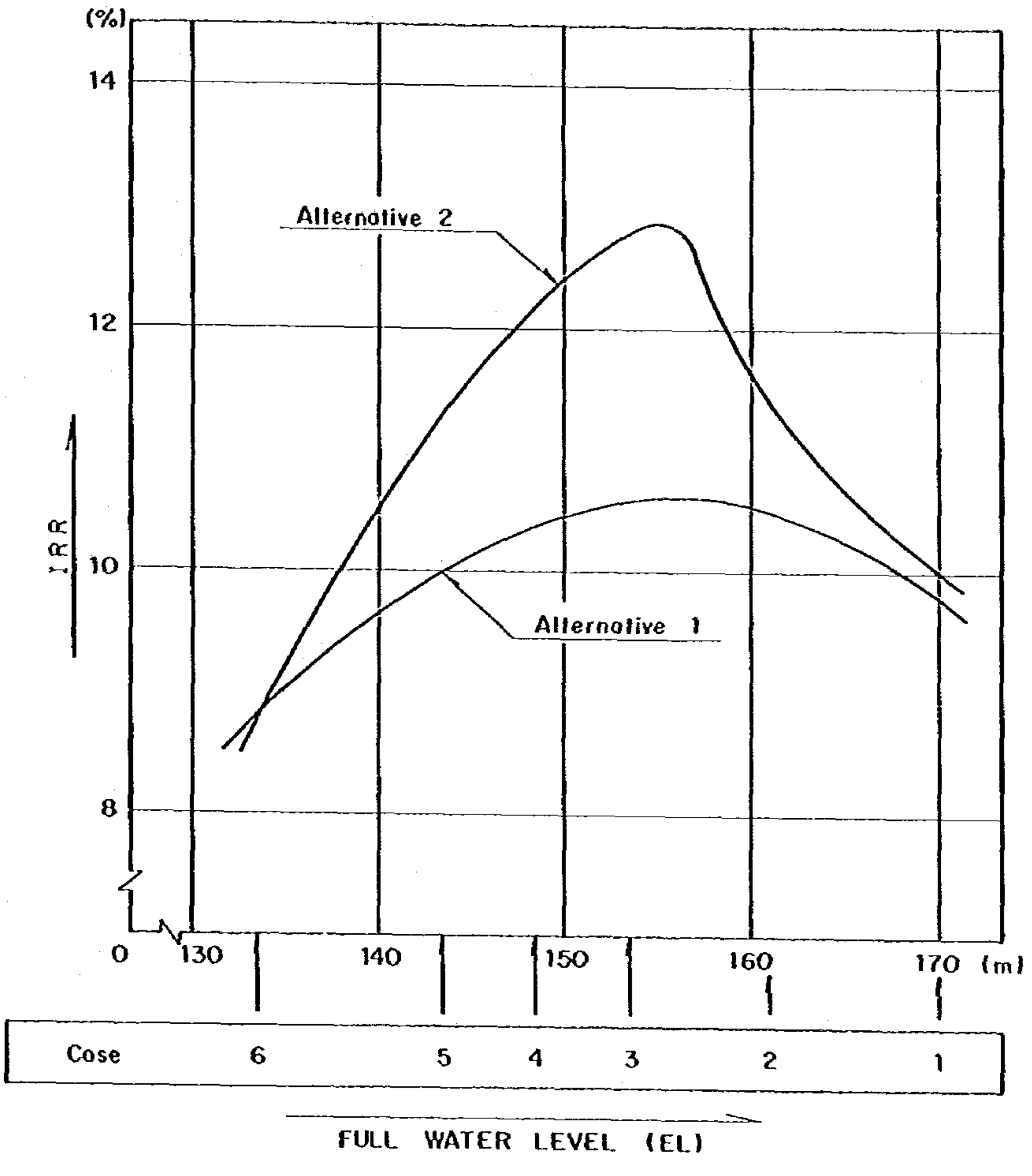


Fig. 5.3 RELATION BETWEEN DAM SCALE AND IRR



APPENDIX VI

DAM AND RESERVOIR

APPENDIX VI DAM AND RESERVOIR

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APPENDIX VI DAM AND RESERVOIR

CHAPTER 1 RESERVOIR PLANNING

1.1 Location of Dam Site

The proposed Gumain dam site is located on the Gumain river just downstream of the confluence of its two tributaries having a catchment area of 114 km^2. The confluence of two tributaries is situated at about 1.5 km upstream from the Summer Place. Each tributary has a catchment area of about 60 km^2 and an average annual runoff of each tributary is about 100 MCM. If a dam site is selected on the tributary, sufficient amount of reservoir water could not be expected due to the small catchment area. On the other hand, a downstream site from the proposed dam site which was investigated during the feasibility study, could not also impound adequate water for the demand of the irrigation due to lack of height of both abutments from the topographical condition. According to the result of the reservoir optimization study, the reservoir is required to have about 110 MCM of the storage capacity for the development plan of the project. Accordingly, the location of dam site was decided to be located at the gorge just downstream of the confluence of two tributaries of the Gumain river.

1.2 Physical Characteristics

1.2.1 Elevation-Area and Elevation-Capacity Curves

In order to obtain the storage volume of the reservoir, the area and capacity versus elevation curves were developed by planimetering using the topographic maps with a scale of 1:4,000. The calculation result and curves are shown in Table 6.1 and Fig. 6.1, respectively.

1.2.2 Water Level and Storage Capacity

(1) Dead Storage Capacity and Low Water Level

The dead storage capacity is defined to be equivalent to the sediment volume deposited in the reservoir. Based on the measurement records and analysis of suspended load in the Gumain river, annual specific yield of suspended load and bed load at the dam site was estimated to be 940 m^3/km^2/year, and considering some safety factors, the design sediment load was determined to be 1,000 m^3/km^2/year (see APPENDIX I). The useful life of the reservoir assumed to be 100 years. As the catchment area at the dam site is 114 km^2, the dead storage capacity (Qs) in the reservoir was estimated as follows:

$$Qs = 1{,}000 \text{ m}^3/\text{km}^2/\text{year} \times 114 \text{ km}^2 \times 100 \text{ years} = 11.4 \text{ MCM}$$

On the assumption that sediment is deposited level with the bottom of the reservoir, the low water level which is equivalent to the deposit level of sediment is given as 100.00 m from Fig. 6.1.

(2) Full Water Level and Active Storage Capacity

The optimum scale of the reservoir was analysed based on the reservoir operation study in connection with the irrigation plan and the full water level of the reservoir was determined to be 153.5 m. The reservoir of this elevation is able to impound the gross storage water of 110.4 MCM and the active storage capacity becomes 99.0 MCM by deducting the dead storage capacity from the gross storage capacity.

(3) High Water Level and Surcharge Storage Capacity

Based on the comparative study on the spillway, the overflow depth at the weir of spillway for the design flood was determined to be 4.0 m as described later, and the high water level of the reservoir was calculated to be 157.5 m. Therefore, the surcharge storage between the full water level and the high water level was estimated to be 14 MCM from Fig. 6.1.

(4) Summary of Water Level and Storage Capacity

The various storage capacities and water levels described before are summarized below:

Water Level		Storage Capacity
Low Water Level	100.0 m	11.4 MCM
Full Water Level	153.5 m	110.4 MCM
High Water Level	157.5 m	124.4 MCM

CHAPTER 2 DAM ENGINEERING

2.1 Selection of Axis and Type of Dam

2.1.1 Axis of Dam

The proposed dam axis was determined to be laid at about 350 m from the confluence of two tributaries of the Gumain river, taking into account the following geological and topographical characteristics at the dam site.

(1) For economical construction of dam, the upstream slope of dam does not cover over the confluence of two tributaries.

(2) Agglomerate with rhyolitic facies in the dam site is fairly weathered and is formed in the range of right abutment. To reduce the quantity of foundation treatment for the above rock, dam is required to be located at the upstream as much as possible.

(3) From the economical viewpoint of spillway construction, the location of spillway is favorable to be selected on the restricted area of the right abutment, and the downstream slope of dam is required to not affect the alignment of spillway.

There is an inferred fault in the dam site, but it runs parallel with the dam axis along the toe of upstream slope. Therefore, it would not affect the selection of dam axis.

2.1.2 Type of Dam

The proposed dam site has massive rock foundations beneath a shallow overburden and a relatively narrow gorge with a steep slope at the left abutment. A width-to-height ratio is about 4. From the above appearances, both a fill dam and a concrete gravity dam were conceived. From the viewpoint of engineering geology, however, the dam site has some weak points for founding a concrete gravity dam as follows:

(1) Unconfined compression strength of foundation rock samples is ranging from 30 to 1,000 kg/cm^2, but 50 to 200 kg/cm^2 predominately. From the results of compression test and seismic exploration, modulus of elasticity of rock foundation is considered to be less than 10,000 kg/cm^2. Above properties of foundation associate a comparatively large and harmful deformation of the foundation for a concrete dam.

(2) Dipping plane of bed rocks is slightly oblique toward the downstream and would coincide with shear plane anticipated in a concrete dam foundation. Since the result of triaxial shearing tests gives initial shearing strength of about 16 kg/cm^2, the shearing strength of interface must be smaller than the above. The foundation with this shear strength is not recommended for the large scale of a concrete dam.

(3) Velocities of foundation layers obtained from seismic exploration are mostly less than 2 km/sec which is generally accepted as a limit for the foundation of a concrete dam.

Accordingly, a concrete gravity dam is not acceptable to the proposed dam site, and a fill dam should be adopted because it has an inherent plastic characteristics to adapt itself smoothly to settlement and deformation of the foundation.

When impervious materials could be obtained easily nearby the dam site, homogeneous earth fill, zone type of earth fill and zone type of rock fill are conceivable. But, it is generally acknowledged that the earth fill dam has lower resistance against shearing than the rock fill dam. When earthquake attacks the dam, the earth fill dam would have possibility to produce the excessive pore pressure due to volumetric change of pores in the impervious or semi-impervious materials. This action will reduce the shearing strength. Intensity of acceleration by earthquake becomes bigger in proportion to the height of dam. Thus, the earth fill type is not nomally adopted for the large scale dam from the earthquake-proof viewpoint.

From the study of seismicity, the proposed dam site would be affected by earthquakes. The proposed dam scale is large with about 100 m in height, and previous materials such as rock and gravelly deposit can be obtained enough around the proposed dam site. Therefore, the zone type of rock fill dam is recommended as the optimum dam type at this site.

2.2 Design of Dam

2.2.1 Fundamental Considerations

(1) Seismic Coefficient

In order to analyze the stability of dam and appurtenant structures, and to estimate the freeboard of dam, etc., seismic coefficient must be decided. The study on earthquake intensity and occurrence was done and the seismic coefficient for design was determined to be 0.12 for the reoccurrence of 100 years (APPENDIX I).

(2) Design Value of Dam Embankment Materials

The design values of dam embankment materials are tabulated below, based on the results and analyses of the laboratory tests. Details are discussed in APPENDIX II.

	Wet Density γ_t (t/m^3)	Saturated Density γ_{sat} (t/m^3)	Submerged Density γ_{sub} (t/m^3)	Cohesion C' (t/m^2)	Angle of Internal Friction ϕ' (°)	Note
Impervious Zone (Zone 1)	1.88	1.94	0.94	9.0	14°00'	Core
Transition Zone (Zone 2)	1.78	1.88	0.88	0	36°00'	Tuff
Rock Zone (Zone 3)	1.80	2.00	1.00	0	38°00'	Aggl.
Rip-rap (Zone 4)	1.82	2.07	1.07	0	40°00'	Cobble - Boulder
Filter	1.78	1.88	0.88	0	35°00'	Sand - Gravel

2.2.2 Zoning of Dam

The following considerations was made for safe and economical construction of the dam, taking into account the availability of embankment materials around the dam site.

(1) It is commonly recognized that impervious core with a width of 30 to 50% of the water head is fairly safe. Since the settlement of the foundation is considered to be relatively large, a width of core would be more than 50% of water head to ensure the safety of dam.

(2) Impervious materials are abundant near the dam site and can be obtained cheaply. To enlarge the core zone is economical way unless it affects the stability against shearing.

(3) The rock materials around the dam site are mostly lappili tuff and agglomerate. Lappili tuff is weaker, but more abundant than agglomerate. To make dam construction economical, rocks of lappili tuff should be utilized predominantly. But it should be placed inner portion and protected by hard agglomerate rocks. Rocks excavated from the foundations of dam and spillway would be comparatively weathered. Most of these rocks should also be placed inner portion.

(4) Gravels and boulders in the riverbed have the high quality and these materials are proposed to be utilized as riprap for the protection of upstream slope.

(5) To reduce a total embankment volume of dam, a cofferdam should be set inside the dam by using riverbed gravels which would be easily obtained at the beginning of construction stage.

Based on the above considerations, a zone type rock fill dam with center core was proposed.

2.2.3 Decision of Dam Crest Elevation

A fill dam should have a safe freeboard over the full water level to prevent from overtopping of embankment. The crest elevation of dam is determined based on the followings:

$$EL. \geq H_f + hw + he + 1 \text{ or } H_f + 3$$
$$\geq H_h + hw + 1 \text{ or } H_h + 2$$

where, EL.: Elevation of dam crest

H_f: Elevation of full water level

H_h: Elevation of high water level

hw: Height of wave due to wind

he: Height of wave due to earthquake

(1) Estimation of hw

The height of wave due to wind (hw) implies the wave uprush on the dam slope including wave height. The wave uprush is able to estimate by the formulas combining the S.M.B (Sherdrup-Munk-Breschneider) method and Saville method. These methods contain some factors such as fetch in the reservoir, velocity of wind (mean velocity per ten minites), gradient of embankment slope and roughness condition of slope surface. To simplify the calculation, iconography is normally used for hw. The relation between hw and above factors is shown in Fig. 6.2.

From this figure, the value of hw was estimated as follows:

hw (for smooth slope) = 2.2 m

hw (for riprap slope) = 0.8 m

when, Fetch of reservoir = 3.0 km

Wind velocity = 30 m/sec

Gradient of slope = 1 : 2.9

The slope condition of Gumain dam is of riprap. But riprap materials will be used riverbed deposits of gravels to boulders which are anticipated to make smoother than usual riprap of rock materials. Therefore, a mean value of riprap slope and smooth slope is recommended. Accordingly, the following was obtained for the design.

$$hw = \frac{2.2 + 0.8}{2} = 1.5 \text{ m}$$

(2) Estimation of he

The wave height due to earthquake (he) is evaluated from the following formula:

$$he = \frac{K \cdot \tau}{2\pi} \cdot \sqrt{g \cdot H_0}$$

where, k: Horizontal seismic coefficient

τ: Period of seismic wave in second

H_0: Depth of reservoir water

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

On the condition of $k = 0.12$, $\tau = 1$ second and H_0 = 88 m, he was estimated to be 0.6 m.

(3) Dam Crest

The crest elevation of dam is given hereunder, considering the wave heights aforementioned.

EL. (based on full water level) $\geq 153.5 + 1.5 + 0.6 + 1.0 = 156.6$ m

EL. (based on high water level) $\geq 157.5 + 1.5 + 1.0 = 160.0$ m

From the above, the dam crest elevation was decided to be 160.0 m appliying the larger one among two values.

2.2.4 Stability Analysis of Dam

Stability of the dam against sliding was analyzed by means of sliced slip circle method. A safety factor obtained by the slip circle method is derived by the following formula:

$$F_s = \frac{\Sigma \{C'\ell + (N - N_e) \cdot \tan \phi'\}}{\Sigma (T + T_e)}$$

where, F_s: Safety factor

N: Normal effective force acting on sliced slip circle

T: Tangental effective force acting on sliced slip circle

N_e: Normal force by seismic load on sliced slip circle

T_e: Tangental force by seismic load on sliced slip circle

C', ϕ': Cohesion and angle of internal friction of materials, respectively on sliced slip circle

ℓ: Arc length of sliced slip circle

Stability analyses against sliding were performed for the following three cases: (i) in case of full water level (Case A), (ii) in case of intermediate water level (Case B), and (iii) in case of rapid drawdown of water level (Case C).

As for cohesionless materials, the slip circle method has characteristics that the safety factor would become smaller when the slip circle becomes shallow. Therefore, the analysis for such case is made by surface plate sliding method as shown below:

$$F_S = \frac{\left(1 - m \cdot k \dfrac{\gamma sat}{\gamma'}\right)}{m + \dfrac{\gamma sat}{\gamma'} . k} \tan \phi'$$

where, F_S: Safety factor

m: Gradient of slope

k: Seismic coefficient

ϕ': Angle of internal friction of materials

γsat: Saturated density of material

γ': Submerged density of material

The above formula was applied to the slope under the water level of reservoir, and for the slope above the water level, the formula can be applied by substituting the wet density (γt) for both γsat and γ'.

The results of analysis of stability are shown in Fig. 6.3 and summarized below:

Method	Case	Pool Level	Portion	Seimic Coefficient	Safety Factor
Slip Circle Sliding	A	EL. 153.5 m	Upstream Slope	0.12	1.228
			Downstream Slope		1.237
	B	EL. 127.0 m	Upstream Slope	0.12	1.267
	C	EL. 153.5 m ↓ EL. 100.0 m	Upstream Slope	0.06/2	1.569
Surface Plate Sliding /1			Upstream Slope	0.12	1.241
			Downstream Slope	0.12	1.225

Remarks: /1: Surface plate sliding method is applied to zone 3 for upstream slope under water level, and zone 2 for downstream slope above water level.

/2: Rapid drawdown of water level is not expected frequently and probability of simultaneous earthquake must be very low. Therefore, a half of seismic coefficient is adopted.

As shown in the above table, the safety factor of critical slip circle for each case exceeds 1.2 which is the minimum allowable safety factor, resulting in the dam being fully stable.

2.2.5 Finite Element Analysis

A finite element analysis is useful for obtaining precise stress and strain, and analyzing the deformation structures. In order to judge the safety of dam and foundation concerned about deformation, inner stress and shearing, the finite element analysis was made:

(1) Parameters for Analysis

Based upon the results of triaxial compression test, unconfined compression test and seismic exploration, parameters of embankment materials and dam foundation were estimated as below:

Foundation

Layer by Seismic Velocity (V_p)	Modulus of Elasticity (kg/cm^2)	Coefficient of Shear Strength C (kg/cm^2)	Coefficient of Shear Strength ϕ(degree)	Poisson's Ratio
$V_p \doteqdot 1.0$ km/s	3,000	3	30	0.25 (assumed)
$V_p \doteqdot 1.8$ km/s	4,000	6	30	
$V_p \doteqdot 2.2$ km/s	5,000	10	35	
$V_p \doteqdot 3.0$ km/s	7,000	15	40	

Embankment

Name of Zone	Initial Modulus of Elasticity (kg/m^2)	Coefficient of N	Failure Ratio (Rf)	Coefficient Shear Strength C (kg/cm^2)	Coefficient Shear Strength ϕ (degree)
Zone 1	320	0.206	0.844	1.0	14
Zone 2	900	0	1	0	35
Zone 3	1,000	0	1	0	38
Zone 4	1,000	0	1	0	40

Remarks: Above parameters are for non-liner analysis.
Parameters of Zone 1 to 3 are assumed.

(2) Results of Analysis

Finite element analyses on both the transverse and longitudinal sections of the dam were made by liner method for foundation and non-liner method for embankment, respectively.

The results of analysis are shown in Fig. 6.5 to Fig. 6.11 and the following considerations were obtained.

1) Maximum settlement of foundation rock was estimated at about 30 cm. This is comparatively large and therefore sufficient grounting to strengthen the foundation would be recommended in the overall core trench.

2) Maximum strain of rock was observed at bottom of core trench. It was about 0.34% in the longitudinal section. Meanwhile, from the results of compression test on foundation rock samples, strain at yielding and failure points are tabulated in Table 6.2 and minimum strain is summarized below:

Compression Test	Strain (%)	
	Yielding	Failure
Unconfined	0.5	0.7
Triaxial	0.5	1.0

In the above strains, the strain of foundation is equivalent of 1/1.5 for the strain of yielding point and 1/2.0 for the strain of failure point. Taking into account reasonable safety factor for of 1.5 against yielding point, the scale of dam designed in the study is considered to be maximum.

3) Critical Ratio of Failure (CRF) implies the reverse number of safety factor against failure by shearing. If CRF is larger than 1.0, it means that there exists local failure. From the figure of CRF distribution (Fig. 6.10 and 6.11), it is founded that the foundation has no local failure and probably safe against shearing, and embankments do not also have any harmful failure.

The above considerations are based on the analysis with many assumed parameters, and therefore, fundamental surveys, especially jack test, are recommended to be made before construction.

2.2.6 Foundation Treatment

(1) Stripping of Foundation

To maintain a stable support against shearing and deformation, stripping would be performed about the following layers and portions.

1) Top soil layer contained organic matter,
2) Weathered layer being less than 50 of N value by standard pentration test,
3) Talus layer and aluvium clayer deposit,
4) Sharply projected rock, and
5) Overhanging portion.

At the left abutment, there exist a cliff between the dam axis and toe of downstream slope. It is suspected to have an experience of moving. Removing of that cliff should be made to avoid rock sliding during the construction.

Sand and gravel layer is formed in the riverbed. Judging by the results of in-site density test and drilling, the sand and gravel layer is fairly firm and could remain as the dam foundation except core zone.

(2) Excavation of Core Trench

In order to obtain a complete contact between impervious core and rock foundation, excavation of core trench would be made until it reachs sound rock. The foundation of core trench should bear the high pressure of grounting. Considering a scale of dam and results of geological survey, excavation line is recommended to mostly set in the rock which would have the seismic velocity of about 2.0 km/s. Thus, the bottom elevation of core trench was planned to be EL. 52.0 m.

(3) Grouting

The foundation of the dam site shows comparatively low permeability, but partially it has a high permeability zone with a Lugeon value of 10 or more. Groundwater of the dam site is existing very deep in the both abutments. Such geological conditions suggest the necessity of careful foundation treatment by grouting against excessive leakage and hydraulic failure.

The grouting would be made to the following extent:

1) The grouting would be made deeper than existing groundwater level.
2) The grouting would be made to cover the zone of higher permeability than 3 Lugeon value.

As for the arrangement of grouting holes, the following considerations are made:

1) The foundation rock is massive and therefore, two rows of grouting are probably enough to improve the deep rocks.

2) But it is commonly known that improvement for shallow rocks is troublesome due to small confined load. Therefore, four rows of subsidiary grouting should be added.

3) To have high resistance against hydraulic pressure and mechanical stress, overall improvement by blanket grouting for core trench should be made.

The effects after grouting would be confirmed by the Lugeon value to be measured at permeability test holes. The improvement index to be stipulated should be one to three Lugeon.

2.3 Design of Appurtenant Structures

2.3.1 Diversion Tunnel

(1) Design Flood for Diversion Tunnel

The peak discharge of the diversion design flood is obtained at the 10-year flood and the cofferdam would have the enough height against overtopping by the 25-year flood. The probable design floods are shown below (Details are given in Appendix I).

Item	Diversion Design Flood	Cofferdam Design Flood
Peak Discharge (m^3/sec)	1,290	1,550
Return Period (year)	10	25

In the design of a diversion tunnel, the free flow condition was applied for 10-year design flood and the pressure tunnel for 25-year design flood.

(2) Diversion Tunnel Routes

The diversion tunnel was selected on the right bank, taking into the consideration the following conditions:

1) Tunnel length on the right bank is shorter than that on the left bank.

2) With regard to geological point of view, there is no big difference between right and left banks, though the geological feature on the right bank is slightly inferior to that on the left bank.

3) One route of the tunnel is chosen considering the economical aspect and the possibility of placing plug concrete during the dry season.

4) Approach channel of downstream from the tunnel is more preferable to arrange at right bank.

(3) Diameter of Diversion Tunnel

The diameter of diversion tunnel for 10-year flood discharge was determined to be free flow condition. The section of tunnel shows in Fig. 6.1 with the selected tunnel route. On the condition that the water depth in tunnel is to be 90% of the tunnel section, the radius of the tunnel was calculated to be 6.0 m by the following formula:

$$Q = \alpha \cdot \beta^{2/3} \cdot I^{1/2} \cdot r^{8/3} \cdot \frac{1}{n}$$

where, Q: Design discharge, 1,290 m^3/sec

$\alpha \cdot \beta^{2/3}$: Coefficient according to the ratio of radius to water depth, 1.85

I: Longitudinal slope of tunnel, 1/128

r: Radius of tunnel, 6.0 m

n: Coefficient of roughness, 0.015

(4) Cofferdam

To estimate the crest elevation of upstream cofferdam against 25-year flood discharge, the following formula was adopted:

$$Q = A \cdot \left(\frac{2gH}{f_i + f_o + \Sigma f_m + f' \frac{L}{R}} \right)^{1/2}$$

where, Q: Design discharge, 1,550 m^3/sec

A: Area of tunnel, A = 104.976 m^2

H: Total head, H = EL.89.2 - EL.63.6 = 25.6 m

f': Coefficient of friction loss, $f' \frac{L}{R} = 0.746$

f_i: Coefficient of entrance loss, $f_i = 0.5$

f_o: Coefficient of outlet loss, $f_o = 1.0$

f_m: Coefficient of other losses, curve loss $f_o = 0.05$

L: Length of tunnel, L = 660 m

R: Hydraulic mean depth

From the above, the water level in the reservoir was estimated to be EL. 89.2 m. Therefore, the crest elevation of upstream cofferdam was decided to be EL. 90.0 m considering some freeboards.

2.3.2 Spillway

(1) Design Flood

The spillway design flood determined in Appendix I is summarized below:

Description	Design Discharge	P.M.F. for Dam Safety
Flood Discharge (m^3/sec)	2,650	2,850
Recurrence Period (year)	1,500	2,800

(2) Type and Route

The spillway site was selected on the right abutment, from the topographical view point of abutment. The discharge released from spillway apron would be returned to the Gumain river where the river course is suitably straight to the center of spillway.

Several types of spillway, i.e. full gated spillway, ungated spillway such as side channel and morning glory type are eligible. The morning glory type, however, seems to be economically and technically disadvantageous at the proposed site, because the diversion tunnel is not available for huge volume of flood discharges. The layout study for spillway was carried out for the remaining two types, gated and side channel spillways as shown in Fig. 6.13. The result of comparison for two types is summarized below:

Description	Gated Spillway	Side Channel Spillway
Elevation of Dam Crest (EL. m)	158.8	160.0
Width of Channel (m)	50	30
Embankment Volume ($10^3 m^3$)	5,393	5,584
Excavation of Spillway ($10^3 m^3$)	1,144	867
Concrete of Spillway ($10^3 m^3$)	110	92
Gate Number	4	-
Cost (₱10^6)	721	646

Remarks: Diversion, intake, outlet and foundation treatment works are excluded from the construction cost.

From the above, the side channel spillway is economically advantageous. It would be also preferable to avoid the troubles for the gate operation.

(3) Optimum Crest Length

The optimum crest length of overflow weir was determined considering the relationship between the crest length of spillway and dam height. Table 6.3 shows the comparison of construction cost for each surcharge head and the results are given as follows:

Surcharge Head (m)	3.0	3.5	4.0	4.5	5.0
Crest Length (m)	239	190	155	130	111
Elevation of Dam Crest (EL. m)	159.0	159.5	160.0	160.5	161.0
Construction Cost (₱10^6)	474	470	469	476	484
Order	3	2	1	4	5

From the cost compared above, the optimum crest length was obtained to be 155 m with the overflow depth at the weir of 4.0 m.

(4) Hydraulic Dimentions for Spillway

Hydraulic calculations on side channel, transition, chute, and energy dissipator were made to determine the hydraulic dimensions of the spillway. The main features of channels are summarized as follows:

(Unit: m)

Structure of Spillway	Width of Channel	Depth of Channel	Elevation of Sill
Side Channel	15.0 - 30.0	16.7 - 24.7	143.5 - 135.5
Transition	30.0	24.7 - 19.0	135.5
Chute	30.0 - 45.0	6.5	138.05 - 43.0
Energy Dissipator	45.0	23.0	43.0

(5) Storage Effect of Reservoir

From the result of flood routing for PMF as described in Appendix I, the Gumain reservoir would mitigate about 418 m^3/sec for the maximum inflow of 2,850 m^3/s as shown below.

(Unit: m^3/sec)

Max. Inflow	2,850
Max. Outflow	2,432
Max. Inflow - Max. Outflow	418
Spillway Design Flood	2,650

2.3.3 Outlet Works

Outlet works comprise intake structure and outlet conduit. Maximum amount of intake water was given to be 15.31 m^3/sec from the irrigation study in Appendix VII. The water taken at the intake would drive through the diversion tunnel up to the point of plug concrete and then flow through outlet conduit as pressure pipe flow.

An inclined intake tower would be of reinforced concrete and equipped with main inlet and emergency gate for intaking water below the low water level during excessively dry season.

The size of conduit pipe was determined to be a diameter of 2.0 m so as to keep the permissible velocity of 5.0 m/sec for the iron pipe.

The hydraulic energy of water would be dissipated at the downstream of outlet conduit by using the jet flow gate with the discharge control. The diameter of the gate was estimated at 1.0 m by the following formula:

$$Q = C \cdot \frac{\pi D_1^2}{4} \sqrt{2g\ (H - H_L)}$$

$$H_L = \frac{8}{\pi^2 g} \left(f_i + f \frac{L}{D_2}\right) \frac{Q^2}{D_2^4}$$

where,
- Q: Offtake discharge, 15.31 m^3/sec
- H: Total head, $H = 100.0m - 60.0m = 40.0m$
- H_L: Total loss head
- D_1: Diameter of gate, 1.0 m
- D_2: Diameter of pipe, 2.0 m
- C: Coefficient of gate loss, 0.80
- f_1: Coefficient of entrance loss, 0.10
- f: Coefficient of friction loss, $124.5\ n^2/D^{1/3}$
- L: Length of pipe, 275.5 m (ϕ2.0 m)

$$H_L = \frac{8}{3.14^2 \times 9.8} \times \left(0.1 + \frac{124.5 \times 0.012^2 \times 275.5}{2.0^{4/3}}\right) \times \frac{15.31^2}{2.0^4} = 2.50$$

$$Q = 0.8 \times \frac{\pi \times 1^2}{4} \sqrt{2 \times 9.8 \times (4.0 - 2.5)} = 17.03 > 15.31$$

In addition, an emergency outlet gate was proposed at just downstream of the plug concrete in parallel with main conduit for supply of irrigation water.

2.4 Work Quantity

On the basis of the preliminary design aforementioned, the drawings of the dam and appurtenant structures were prepared as attached in APPENDIX XII. The work quantity were calculated based on the drawings as shown in Table 6.4.

Table 6.1 ELEVATION-AREA, CAPACITY RELATIONSHIP OF GUMAIN RESERVOIR

Elev.	Elev. Interval Δh (m)	Area A (m^2)	Ave. Area $\bar{A}$ (m^2)	Vol. Interval ΔV (m^3)	Accum. Volume V (m^3)
63.0	-	-	-	-	-
65.0	2.0	26,240	13,120	26.240	26,240
70.0	5.0	83,520	54,880	274,400	300,640
75.0	5.0	161,440	122,480	612,400	913,040
80.0	5.0	250,940	206,190	1,030,950	1,943,990
85.0	5.0	349,760	300,350	1,501,750	3,445,740
90.0	5.0	478,400	414,080	2,070,400	5,516,140
95.0	5.0	628,110	553,255	2,766,275	8,282,415
100.0	5.0	757,280	692,695	3,463,475	11,745,890
105.0	5.0	905,580	831,430	4,157,150	15,903,040
110.0	5.0	1,067,220	986,400	4,932,000	20,835,040
115.0	5.0	1,234,330	1,150,775	5,753,875	26,588,915
120.0	5.0	1,418,710	1,326,520	6,632,600	33,221,515
125.0	5.0	1,625,213	1,521,962	7,609,810	40,831,325
130.0	5.0	1,864,400	1,744,807	8,724,035	49,555,360
135.0	5.0	2,143,920	2,004,160	10,020,800	59,576,160
140.0	5.0	2,446,720	2,295,320	11,476,600	71,052,760
145.0	5.0	2,777,753	2,612,237	13,061,185	84,113,945
150.0	5.0	3,124,160	2,950,957	14,754,785	99,868,730
155.0	5.0	3,459,160	3,291,660	16,458,300	115,327,030
160.0	5.0	3,834,433	3,646,797	18,233,985	133,561,015
165.0	5.0	4,285,020	4,059,727	20,298,635	153,859,650
170.0	5.0	4,708,480	4,496,750	22,483,750	176,343,400
175.0	5.0	5,098,293	4,903,387	24,516,935	200,860,335

Remarks: Area and capacity are obtained from the topographic maps on a scale of 1/4,000 with a contour interval of 5.0 m

Table 6.2 STRAIN BY COMPRESSION TEST

	Sample	Strain (%) Yielding (Ee)	Strain (%) Failure (Ef)	Confined Pressure (kg/cm²)
Unconfined Compression Test	No. 1	0.6	1.0	0
	No. 2	0.5	0.7	
	No. 3	0.9	1.0	
	No. 4	0.6	1.0	
	No. 5	0.5	0.9	
	No. 6	0.9	1.1	
	Average	0.67	0.97	
Triaxial Compression Test	DH-6	3.8	-	3.0
		3.0	-	6.0
		3.1	-	9.0
	DH-9 (21m - 23m)	1.3	2.5	1.0
		1.7	2.2	3.0
		2.0	2.5	6.0
		2.0	3.0	12.0
	DH-9 (26m - 62m)	2.0	2.4	1.0
		2.0	3.0	3.0
		2.3	-	6.0
		2.0	3.5	12.0
	DH-6 (Weathered)	1.0	-	1.0
		1.0	1.8	3.0
		0.8	1.3	6.0
		0.5	0.75	12.0
	Average	1.90	2.30	-

Table 6.3 COMPARISON OF CONSTRUCTION COST FOR EACH SURCHARGE HEAD

Surcharge /1 Head (m)	H.W.L. /2 (EL. m)	Length of Crest (m)	Elevation of Dam Top (EL. m)	Volume of Dam (10^3m^3)	Concrete of Side Channel (10^3m^3)	Excavation of Side Channel (10^3m^3)	Grout (10^3m)	Construction Cost (P10^6)
3.0	156.5	239	159.0	5,420	20.6	194	12.0	474
3.5	157.0	190	159.5	5,500	16.8	164	9.5	470
4.0	157.5	155	160.0	5,580	14.1	143	7.8	469
4.5	158.0	130	160.5	5,740	12.2	126	6.5	476
5.0	158.5	111	161.0	5,900	10.7	113	5.6	484

Remarks: /1: Elevation of weir crest indicates, EL. 153.5 m

/2: Design flood water level

Table 6.4 WORK QUANTITY OF DAM AND APPURTENANT STRUCTURES

I.	Dam		
	1. Excavation		521,660 m^3
	2. Embankment		
	- Main Dam :	Zone 1 (Impervious Zone)	819,000 m^3
		Zone 2 (Transition Zone)	2,536,000 m^3
		Zone 3 (Rock Zone)	1,331,000 m^3
		Zone 4 (Rip-rap)	250,000 m^3
		Filter	476,000 m^3
		Total	5,412,000 m^3
	- Cofferdam :	Zone 1	28,000 m^3
		Sand and Gravel	144,000 m^3
		Total	172,000 m^3
	3. Grouting		65,850 m
II.	Spillway		
	1. Excavation		867,000 m^3
	2. Concrete		92,000 m^3
	3. Reinforced Iron Bar		2,760 ton
	4. Concrete Form		55,200 m^2
III.	Diversion Works		
	1. Excavation		2,000 m
	2. Concrete		13,400 m
	3. Reinforced Iron Bar		590 ton
	4. Concrete Form		16,000 m^2
	5. Tunnel :	Excavation	82,300 m^3
		Concrete	23,800 m^3
	6. Grouting		4,150 m
IV.	Outlet Works		
	1. Concrete		4,300 m^3
	2. Reinforced Iron Bar		40 ton
	3. Concrete Form		4,900 m^2
	4. Steel Pipe :	ø2,000	263 m
		ø1,500	55 m
		ø1,000	12.5 m

Fig.6·1 ELEVATION-AREA, CAPACITY RELATIONSHIP OF GUMAIN RESERVOIR

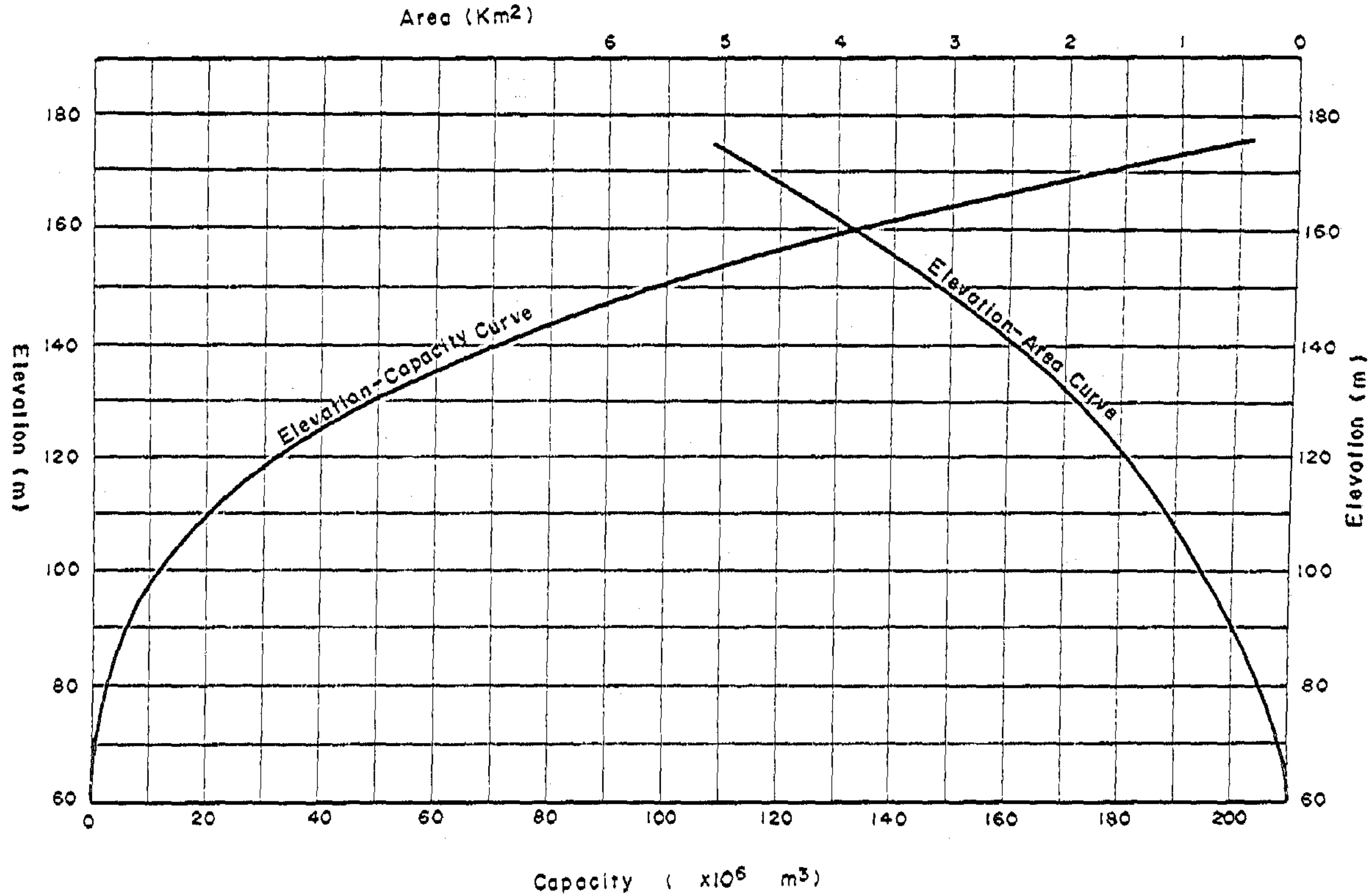


Fig.6·2 WAVE UPRUSH INCLUDING WAVE HEIGHT OBTAINED BY COMBINING THE S.M.B METHOD WITH SAVILLE METHOD

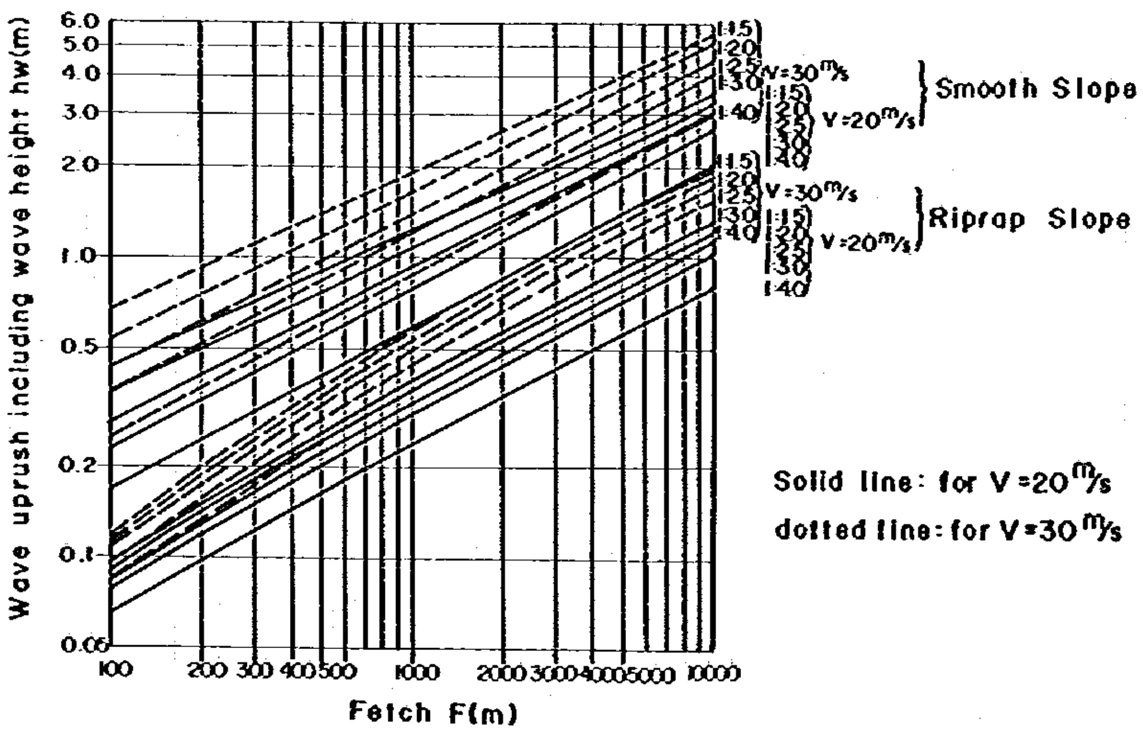
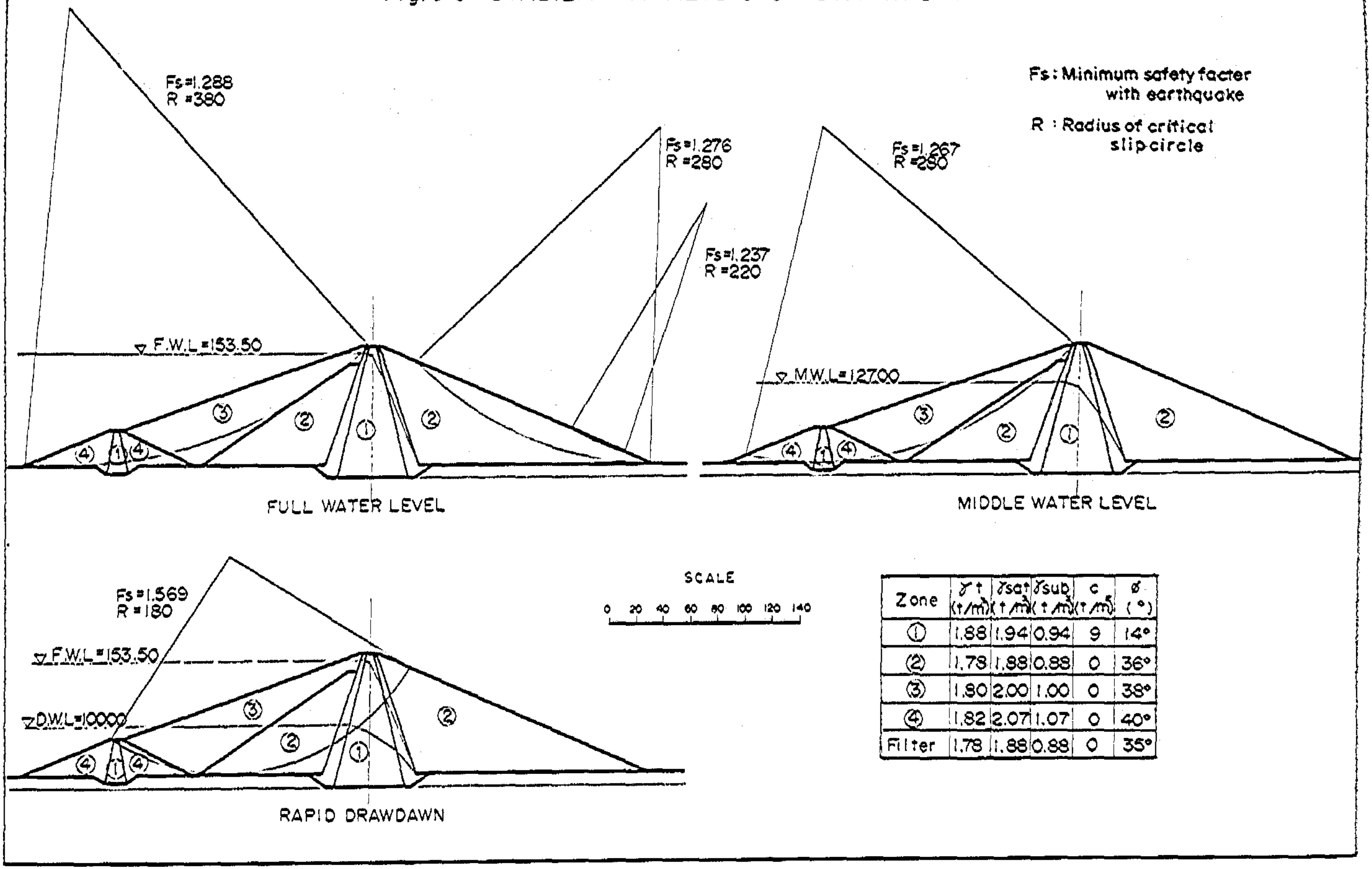
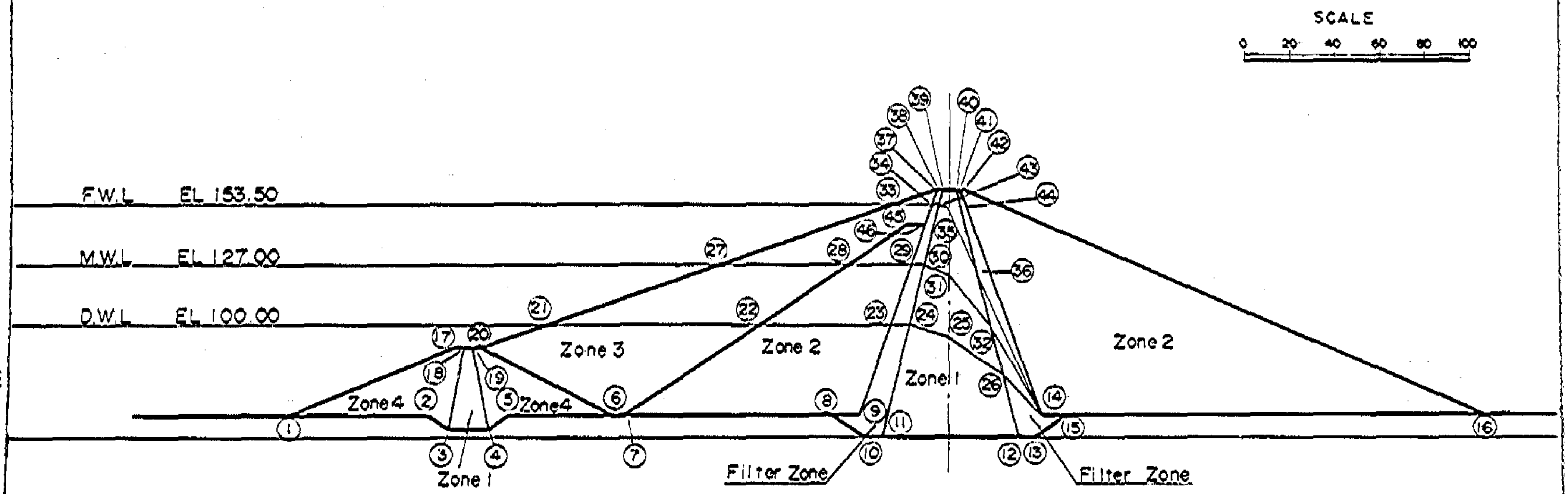


Fig.6·3 STABILITY ANALYSIS OF GUMAIN DAM



Zone	γt (t/m³)	γsat (t/m³)	γsub (t/m³)	c (t/m²)	ϕ (°)
①	1.88	1.94	0.94	9	14°
②	1.78	1.88	0.88	0	36°
③	1.80	2.00	1.00	0	38°
④	1.82	2.07	1.07	0	40°
Filter	1.78	1.88	0.88	0	35°

Fig. 6·4 COORDINATES OF STABILITY ANALYSIS



NO	X	Y	NO	X	Y	NO	X	Y	NO	X	Y	NO	X	Y
1	-294	60	11	-30.5	50	21	-180	100	31	0	122	41	5	160
2	-232.2	60	12	30.5	50	22	-87.35	100	32	18	100	42	6	160
3	-223.2	54	13	37.5	50	23	-27.6	100	33	-24.85	153.5	43	- 4.625	153.5
4	-204.8	54	14	41.6	60	24	-18	100	34	- 8.875	153.5	44	8	156
5	-195.8	60	15	52.5	60	25	0	94	35	0	152	45	-19.85	145
6	-149	60	16	235	60	26	23.5	78	36	12	124	46	-11.85	145
7	-147.35	60	17	-219	90	27	-101.7	127	37	- 6	160			
8	- 52.5	60	18	-216	90	28	- 46.85	127	38	- 5	160			
9	- 41.6	60	19	-212	90	29	- 18.15	127	39	- 3	160			
10	- 37.5	50	20	-209	90	30	- 11.25	127	40	3	160			

Fig. 6.5 MINUTE OF PRINCIPAL STRAIN (LONGITUDE)

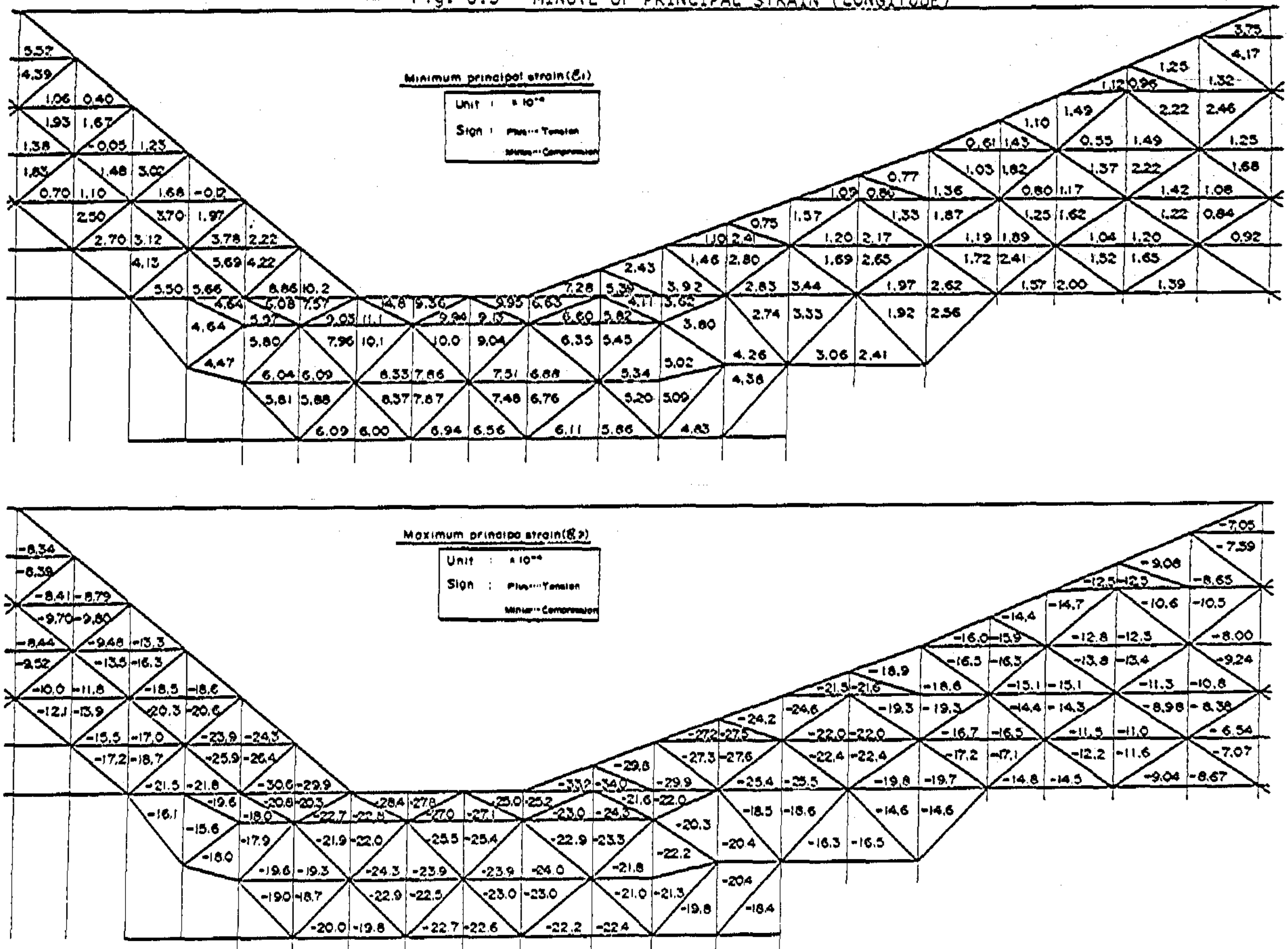


Fig. 9.6 MINUTE OF PRINCIPAL STRAIN (TRANSVERSE)

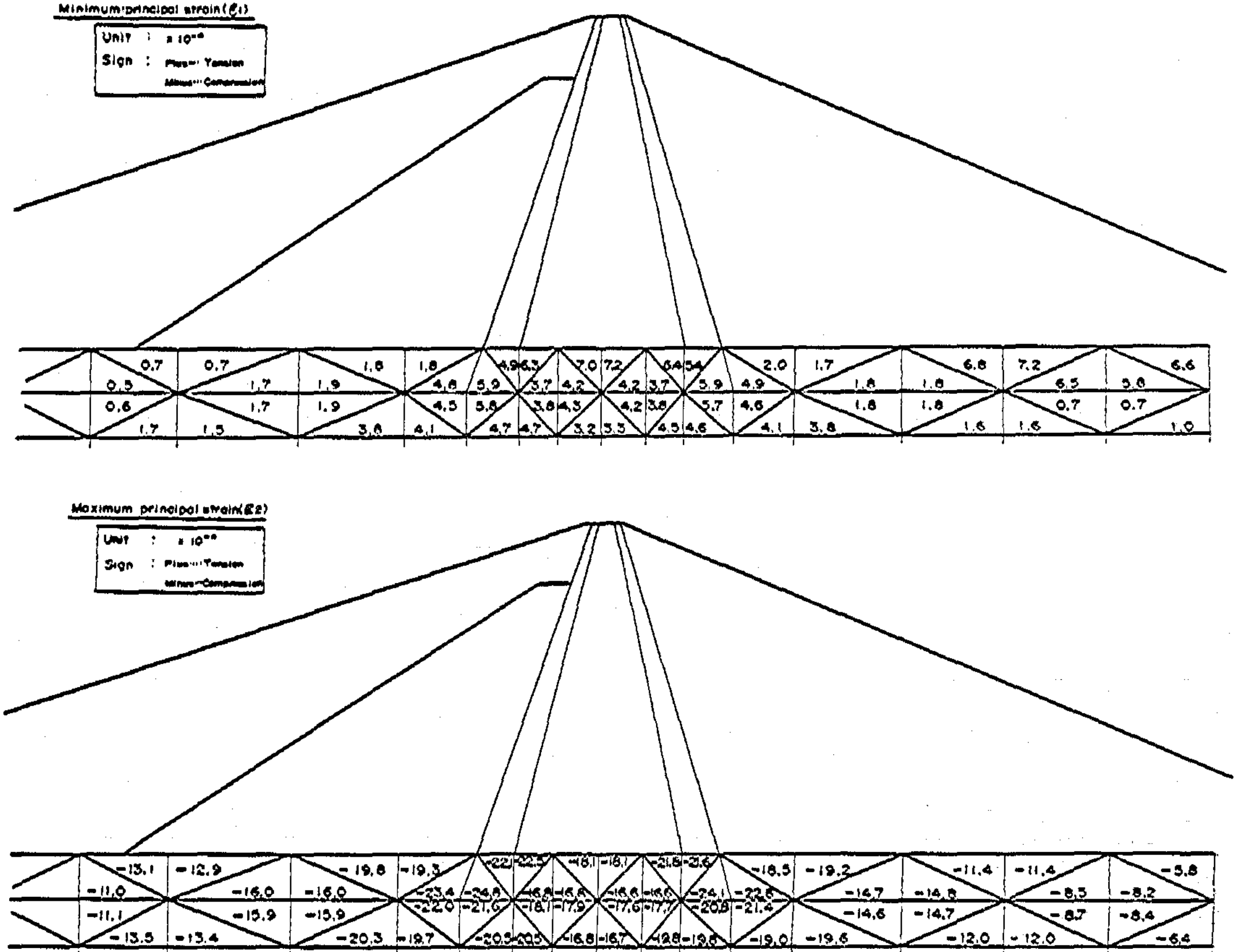


Fig. 6.7 STRAIN AND DISPLACEMENT (LONGITUDE)

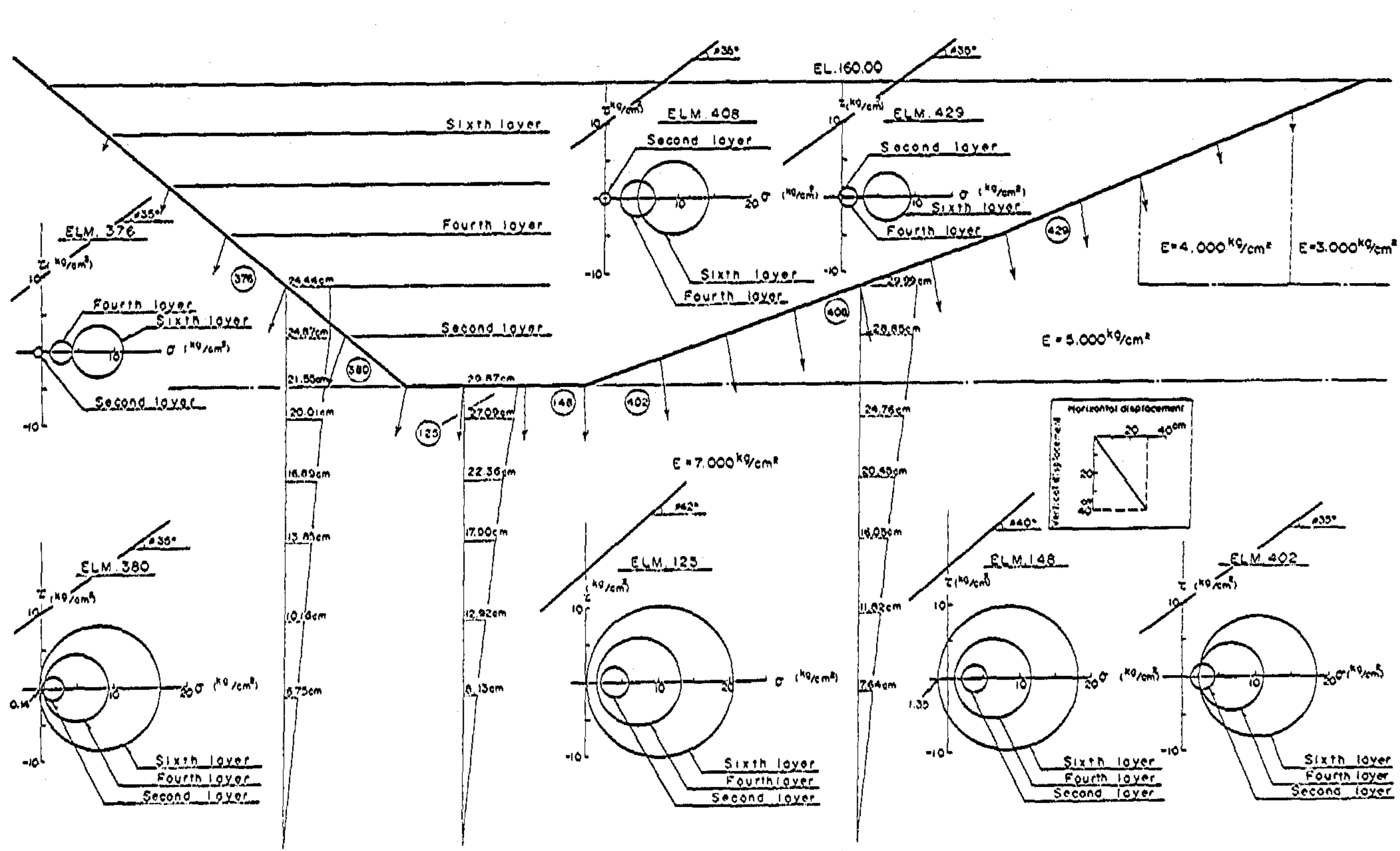
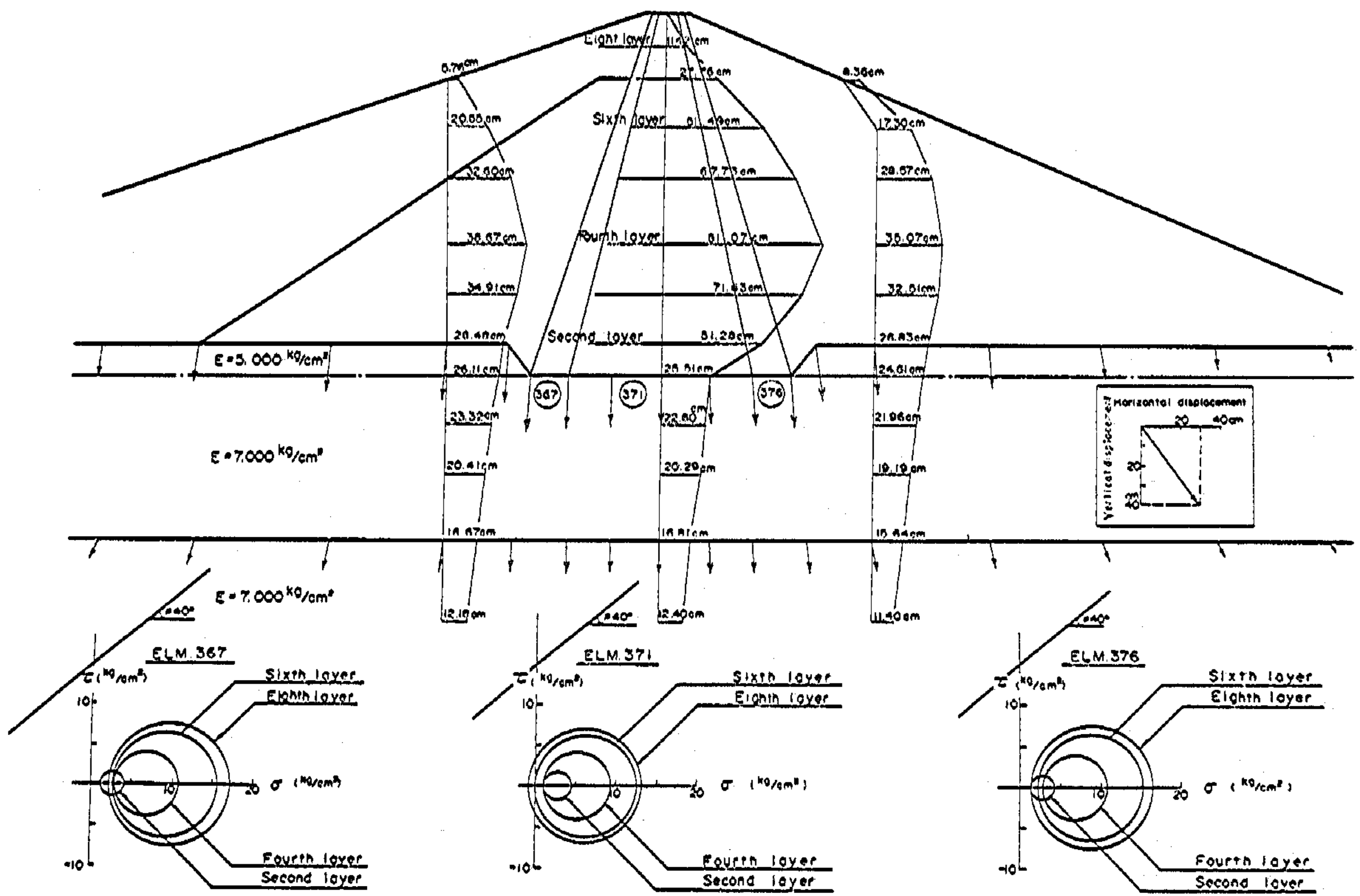


Fig.6.8 STRAIN AND DISPLACEMENT (TRANSVERSE)



SVERSE

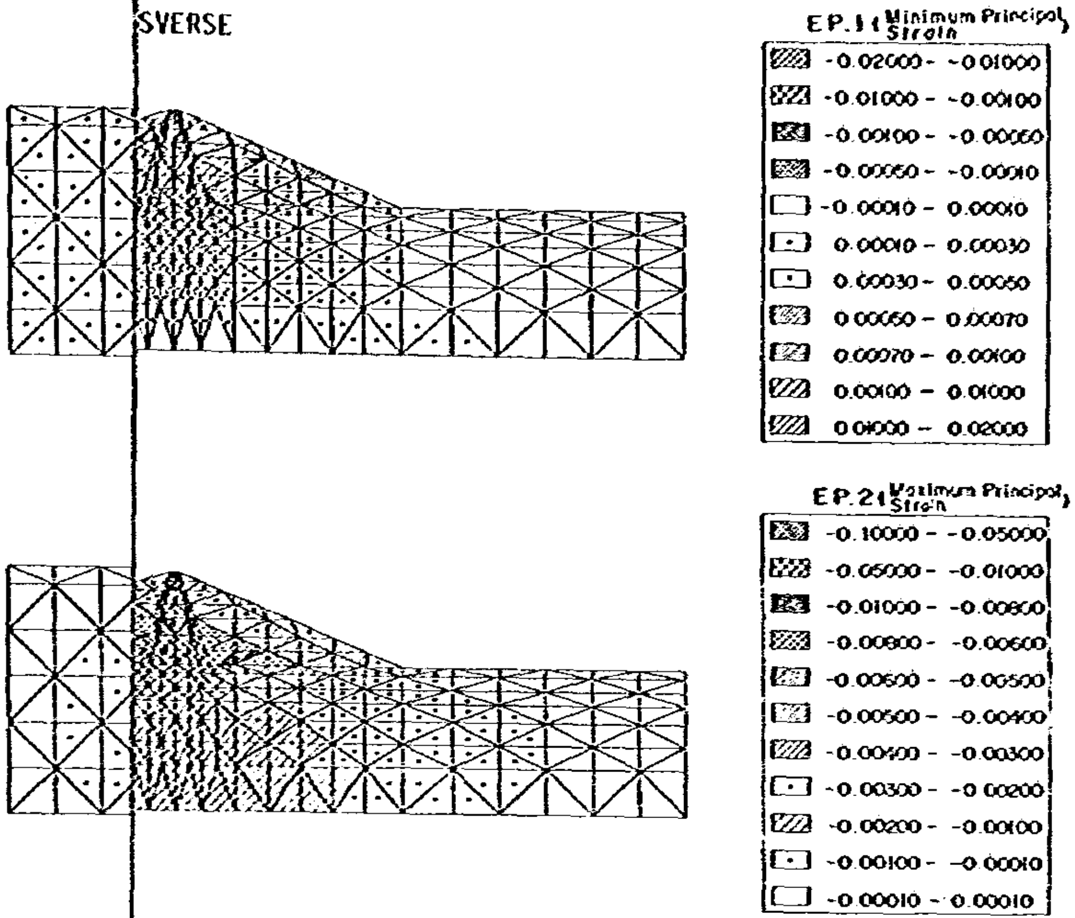

EP.1 (Minimum Principal Strain)
-0.02000 - -0.01000
-0.01000 - -0.00100
-0.00100 - -0.00060
-0.00060 - -0.00010
-0.00010 - 0.00010
0.00010 - 0.00030
0.00030 - 0.00050
0.00060 - 0.00070
0.00070 - 0.00100
0.00100 - 0.01000
0.01000 - 0.02000
EP.2 (Maximum Principal Strain)
-0.10000 - -0.05000
-0.05000 - -0.01000
-0.01000 - -0.00800
-0.00800 - -0.00600
-0.00600 - -0.00500
-0.00500 - -0.00400
-0.00400 - -0.00300
-0.00300 - -0.00200
-0.00200 - -0.00100
-0.00100 - -0.00010
-0.00010 - 0.00010

Fig.6·9 DISTRIBUTION OF PRINCIPAL STRAIN

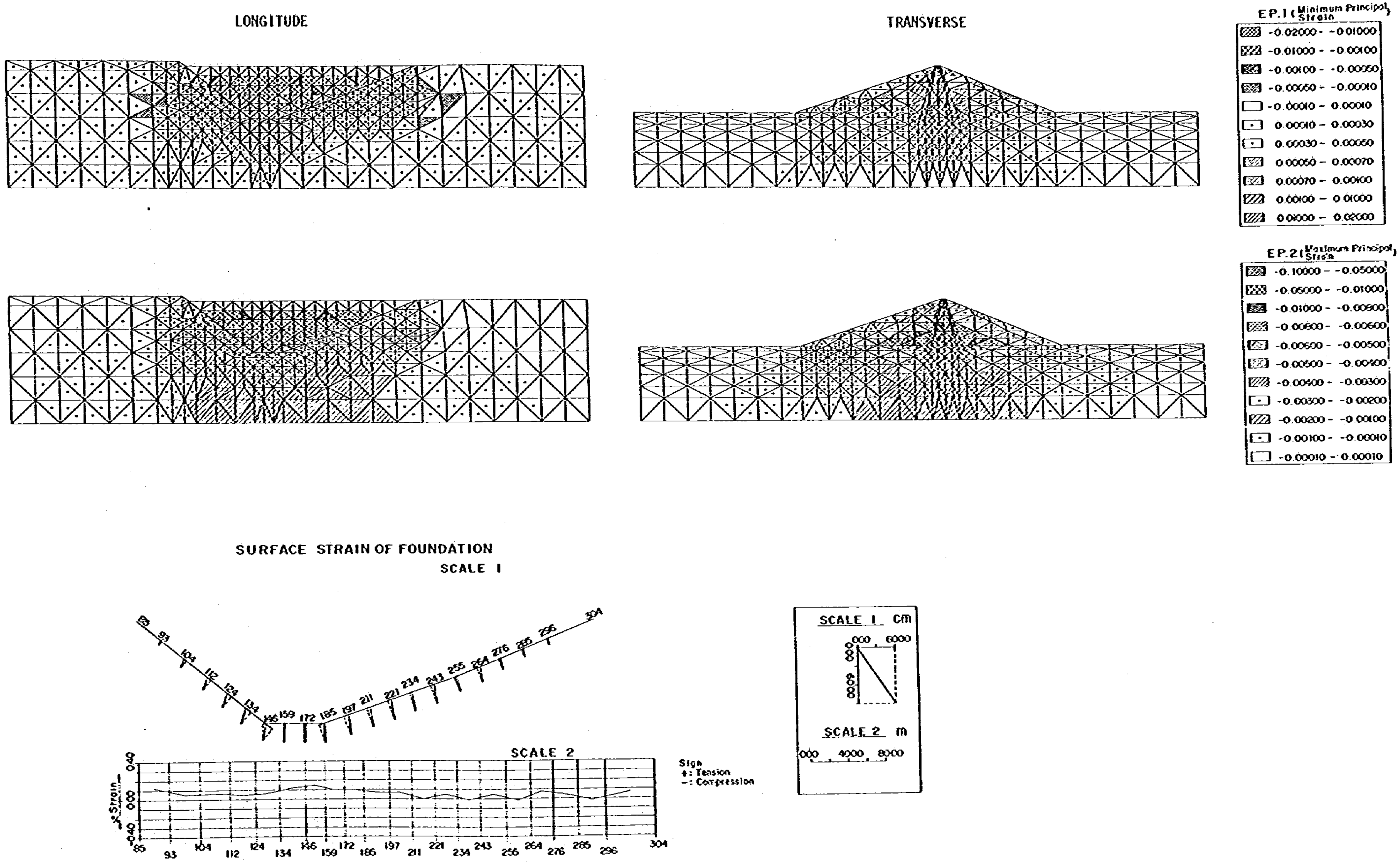


Fig. 6·10 RESULT OF FINITE ELEMENT ANALYSIS (LONGITUDE)

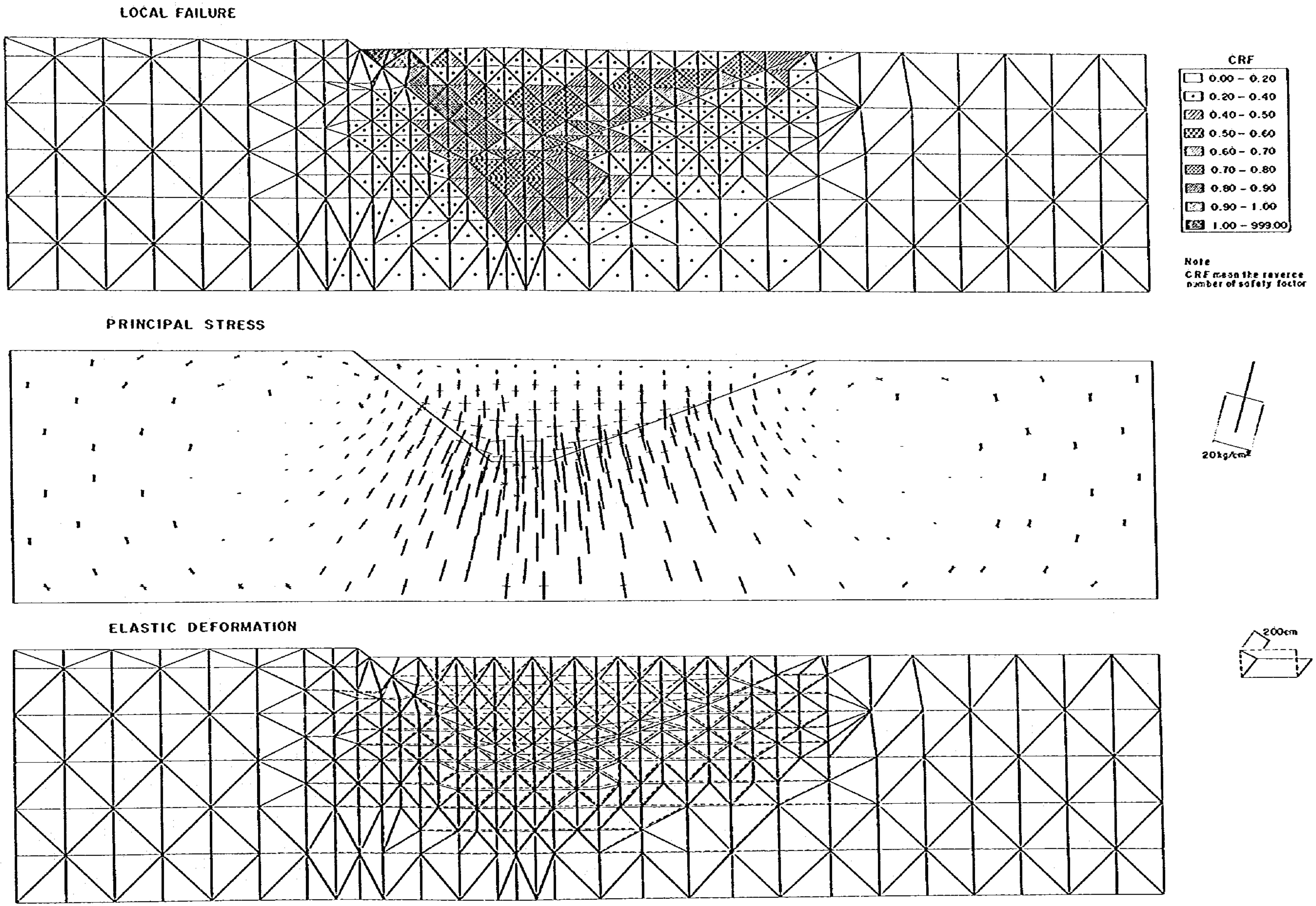


Fig.6·11 RESULT OF FINITE ELEMENT ANALYSIS (TRANSVERSE)

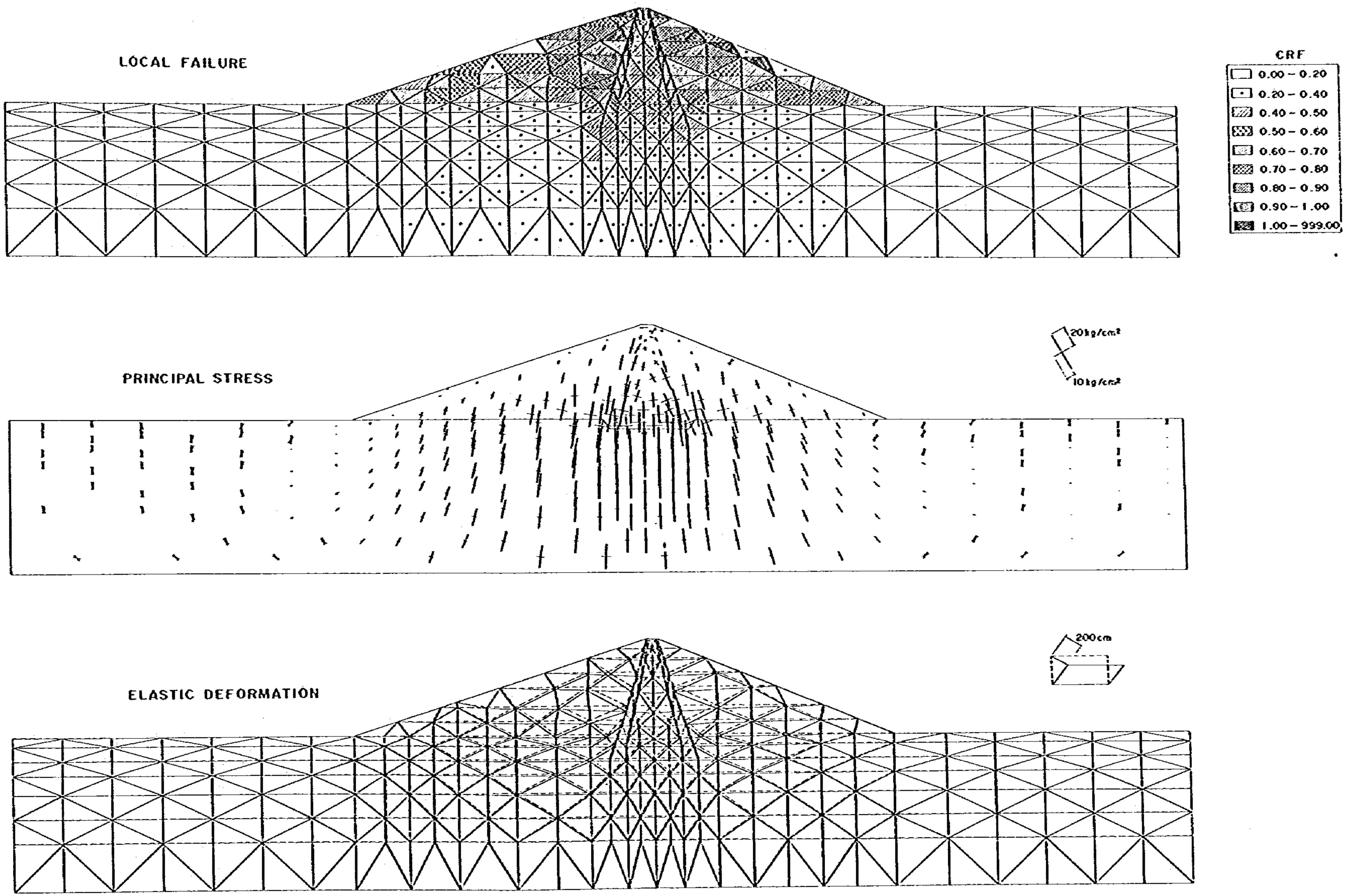
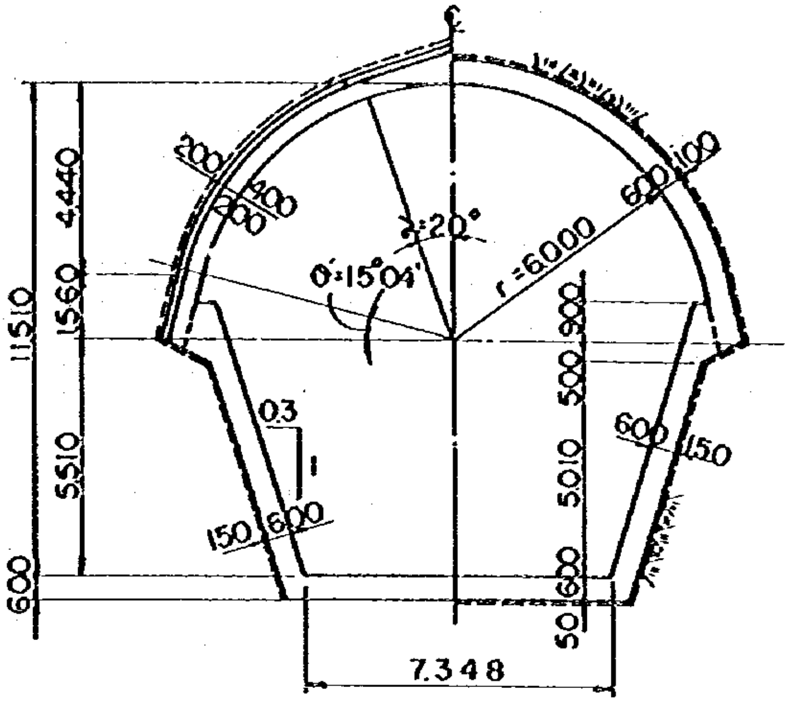


Fig. 6.12 LAYOUT OF DIVERSION TUNNEL



STANDARD SECTION OF TUNNEL

Hydroulic Chorocteristics
$\alpha = A/r^2$, $\beta = k/r$
Open Flow (Woter depth of 90%) $\alpha = 2.762$ $\beta = 0.548$
Pipe Flow (Woter depth of 100%) $\alpha = 2.916$ $\beta = 0.463$

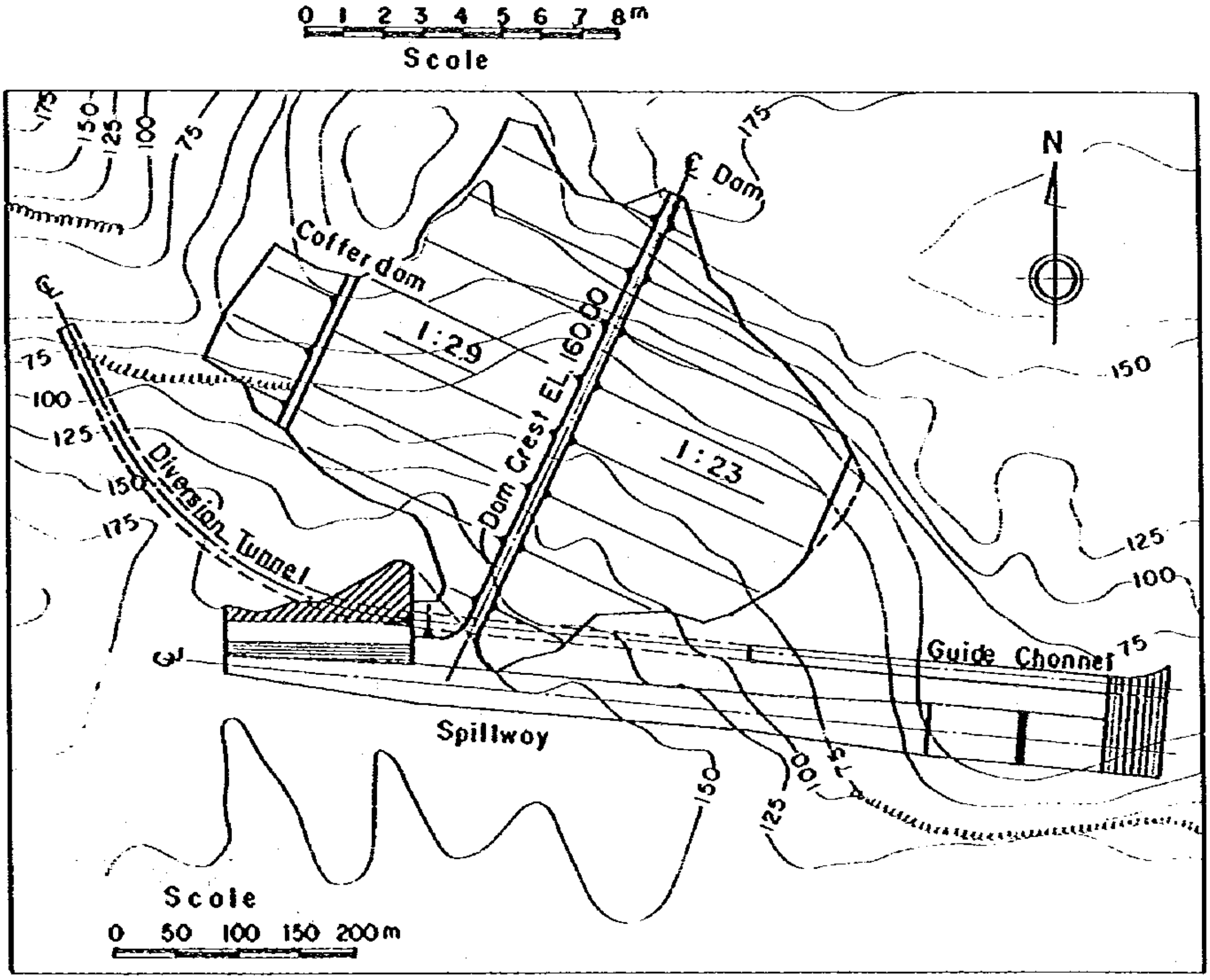
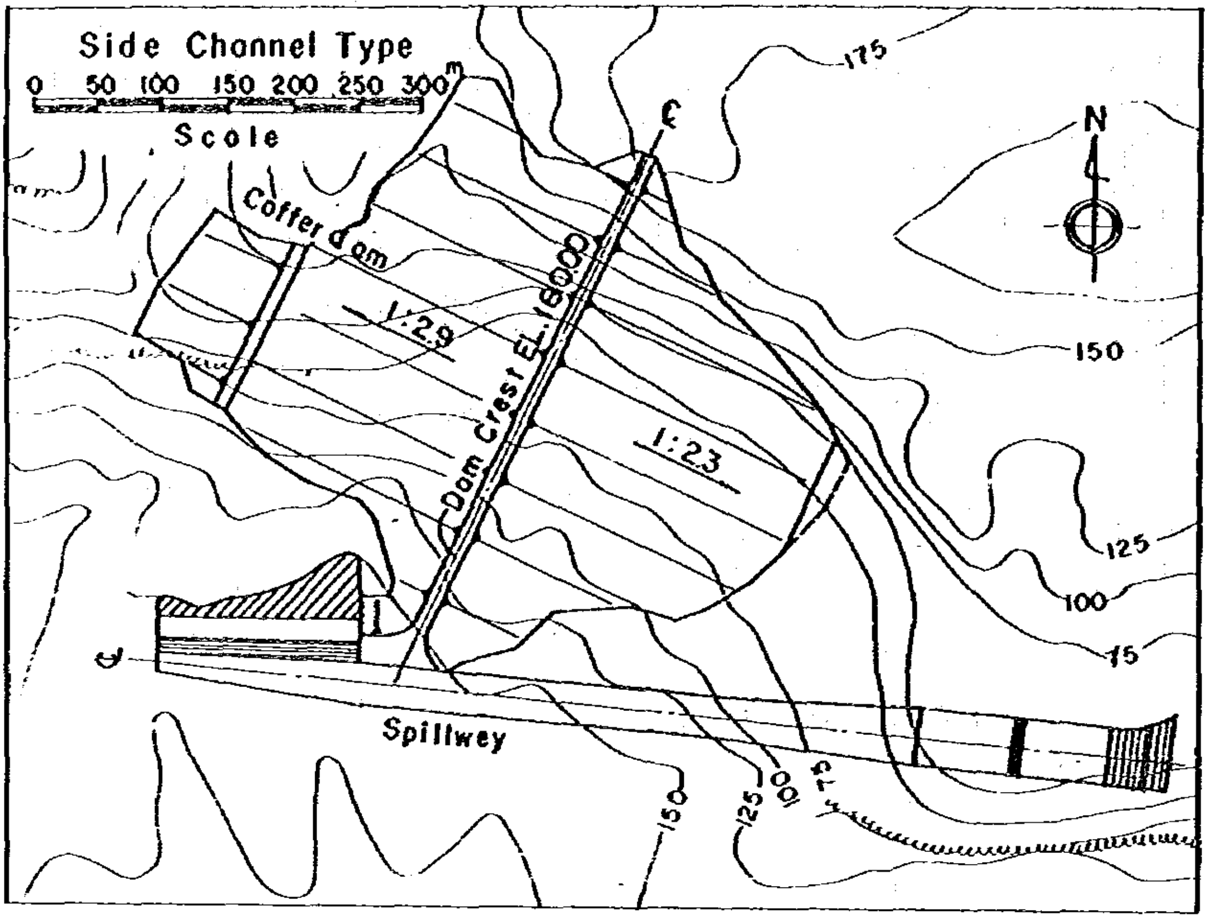
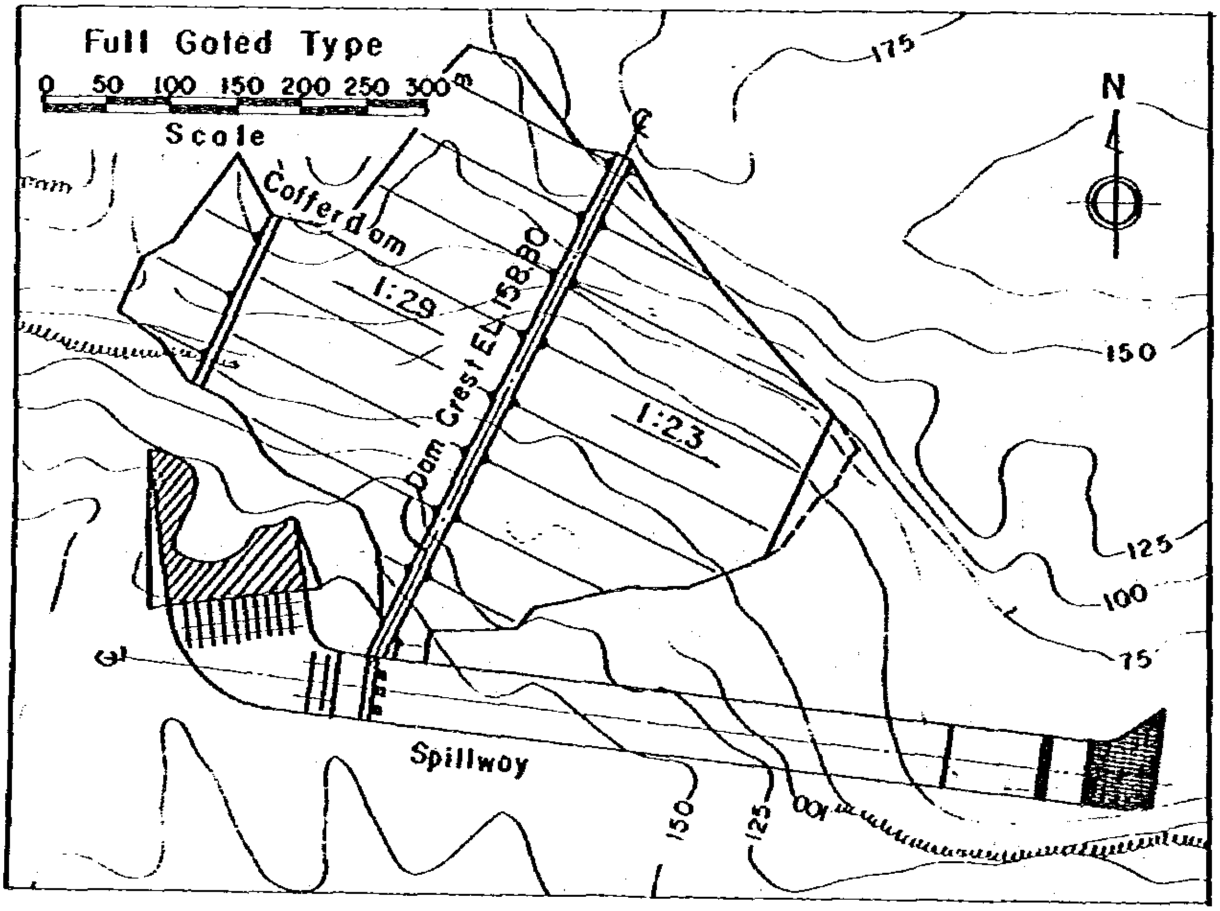


Fig. 6.13 LAYOUT OF SPILLWAY FOR COMPARISON





APPENDIX VII

IRRIGATION
AND
DRAINAGE

APPENDIX VII IRRIGATION AND DRAINAGE

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CHAPTER 1 INTRODUCTION

The irrigation development in Philippines has been promoted in order to raise productivity of agriculture, especially putting a priority on development of irrigation system and expansion of on-farm facilities as well as water resources development.

The study area is well-endowed with land resources suitable for agricultural production, but its present productivity is low because of such various constraints as; (i) uneven seasonal distribution of rainfall, (ii) shortage of irrigation water especially during the dry season, (iii) inadequancy of the perennial irrigation system, (iv) lack of a drainage system, and (v) improper water management.

To resolve these constraints, the optimum irrigation development plan was developed from assessment of the water resources, estimation of irrigation water requirements, and a water balance study. The assessment of water resources was made through hydrological analysis of the three main rivers, Porac, Gumain and Caulaman, and effective use of return flow utilizing the existing checkgate structures. The irrigation water requirements were calculated for the proposed cropping pattern after selecting the most beneficial crops for the project. Based on the available irrigation water sources and irrigation water requirements, the water balance study was made to estimate the guaranteed irrigation area in relation with the scale of the proposed Gumain storage dam.

The proposed irrigation system was determined on the basis of the formulated development plan taking into consideration the best use of the existing irrigation systems in the study area. The drainage system is a important component of the project works as well as irrigation system. The drainage plan was made by means of improving the existing rivers and creeks, and providing new collector drains and farm drains at on-farm level.

CHAPTER 2 PRESENT IRRIGATION AND DRAINAGE SYSTEMS IN THE STUDY AREA

2.1 General

The study area is located in the southwestern part of Pampanga River Basin in the Central Luzon about 70 km northwest of Manila, the capital of Philippines. It is bounded generally in the west by the Cabusilan mountains, both in the east and south by the National Highway Route 7, and in the north by Guagua river. The area extends over the alluvial fans and plains created by the Gumain, Porac and Caulaman Rivers with slops ranging from 0.2% to 2.5%. The elevation of irrigable lands in the study area ranges from 5 m to 40 m. The gross area of the study area is about 23,700 ha, which is covered by new topographic maps on a scale of 1 : 4,000 with 1 m contour interval prepared by JICA on December 1983. In the study area, only one soil group was identified as being derived from recent alluvial deposits and it is suitable for irrigation farming.

2.2 Irrigation Systems

The irrigation systems in the Philippines are classified into the following four categories: a) National Irrigation System, b) Communal Irrigation System (CIS), c) Pump Irrigation System (PIS), and d) Private and other irrigation systems.

In the study area, there are two National Irrigation Systems, the Porac - Gumain River Irrigation System (PGRIS) and the Caulaman River Irrigation System (CRIS), as well as various Communal and Pump Irrigation Systems. The general features of these systems are described as follows:

2.2.1 Porac - Gumain River Irrigation System (PGRIS)

The PGRIS was originally designed to irrigate 6,000 ha of land located in the municipalities of Floridablanca, Lubao, Guagua and Santa Rita, all in the province of Pampanga. This system was completed in 1957. Due to shortage of irrigation water and deterioration of some facilities in the system, it is now only capable of servicing 4,890 ha during the wet season and 3,810 ha during the dry season.

This system has two diversion dams, the Porac Diversion Dam with a falling shutter type located at Pulangmasle, Guagua and the Gumain Diversion Dam with an ogee type located at San Pedro, Floridablanca. These dams contain two main irrigation canals, Porac-East and -West sides and Gumain-North and -South sides. The total length of main irrigaiton canals, laterals and sub-laterals is about 65 km.

The outlines of engineering works in PGRIS are shown below:

1) Year of official opening : 1957

2) Area

a) Original design service area : 6,000 ha

b) Present service area : 5,000 ha

c) Benefited area

Wet season	: 4,890 ha
Dry season	: 3,810 ha

3) Main source of water supply : Porac and Gumain Rivers

4) Diversion Works

a) Porac Diversion Dam

Dam	:	Ogee type with falling shutter	
		Length :	43.40 m
		Height :	2.84 m
Sluiceway	:	Roller gate	
		East :	2.35m x 3.51m x 1-bay
		West :	3.00m x 3.51m x 1-bay
Intake	:	Slide gate	
		East :	1.83m x 1.57m x 3-bays
		West :	1.83m x 1.57m x 1-bay

b) Gumain Diversion Dam

Dam	:	Ogee type	
		Length :	224.0 m
		Height :	2.0 m
Sluiceway	:	Roller gate	
		South :	4.60m x 2.20m x 1-bay
		North :	1.85m x 2.20m x 1-bay
Intake	:	Slide gate	
		South :	1.40m x 1.30m x 4-bays
		North :	1.10m x 1.10m x 1-bay

5) Irrigation Canal Network

a) Main canals : 4 Nos. - 18.3 km

b) Laterals/sub-laterals : 14 Nos. - 46.1 km

c) Farm ditches : 167 Nos. - 157.9 km

6) Canal Structures

a) Control structures : 71 Nos.
(Lateral Headgates, Checks, Drops)

b) Conveyance structures : 63 Nos.
(Flumes, Siphons, Thresher/Road Crossing)

c) Drainage structures : 16 Nos.
(Drainage Flumes/Siphon, Paddy Drains)

d) Terminal structures : 342 Nos.
(Turnout, Divsrion Boxes)

e) Checkgate, Brush Dam for return flow : 26 Nos.

7) Drainage System

a) Farm drains : 28 Nos. - 18.5 km

b) Drainage creeks : 6 Nox. - 52.0 km

8) Service Roads

a) Roads along main canal : 2.3 km

b) Roads along laterals/sub-laterals : 11.5 km

c) Barrio/Municipal roads : 54.5 km

d) Provincial/National roads : 41.0 km

The existing irrigation system in the PGRIS is illustrated in Fig. 7.1 and the result of the inventory survey for existing major facilities is summarized in Table 7.1.

2.2.2 Caulaman River Irrigation System (CRIS)

The CRIS was originally designed to irrigate 2,000 ha of riceland in the municipalities of Floridablanca and Lubao in Pampanga Province and in the municipalities of Dinalupihan and Hermosa in Bataan Province. This system was completed in 1968 and it is now only capable of irrigating 540 ha during the wet season and 480 ha during the dry season. Some parts of the original service area were taken for to residential use or conversion to sugarcane fields, while some are now being served by communal irrigation systems.

The main diversion structure on the Caulaman River consists of an ogee type dam with a length of 72 meters. There are also supplementary dams and checkgates for diverting additional water supply independent of the main diversion dam. Total length of main irrigation canals, laterals and sub-laterals is about 42 km, but some canals and canal structures are in need of repair.

The outlines of engineering works in CRIS are as follows:

1) Year of official opening : 1968

2) Area

   a) Original design service area : 2,000 ha
   b) Present service area : 745 ha
   c) Benefited area

      Wet Season : 540 ha
      Dry Season : 480 ha

3) Main source of water supply : Caulaman River

4) Caulaman Diversion Dam

   Dam : Ogee type
   Length : 72.0 m
   Height : 1.8 m

   Sluiceway : Roller gate
   Left side : 4.30m x 1.90m x 1-bay

   Intake : Slide gate
   Left side : 2.00m x 1.00m x 1-bay

5) Irrigation Canal Network

   a) Main canal : 1 No. - 28.8 km
   b) Laterals/sub-laterals : 6 Nos. - 13.3 km
   c) Farm ditches : 106 Nos. - 43.8 km

6) Canal Structures

   a) Control structures : 16 Nos.
   (Lateral Headgate, Checks, Drops)

   b) Conveyance structures : 94 Nos.
   (Flumes, Siphons, Thresher/Road Crossings)

   c) Drainage structures : 13 Nos.
   (Drainage Flumes/Siphon, Paddy Drains)

   d) Terminal structures : 8 Nos.
   (Turnouts, Division Boxes)

   e) Checkgate, Brush dam for return flow : 11 Nos.

7) Drainage System

a) Farm drains : None

b) Drainage creeks : 6 Nos.

8) Service Roads

a) Roads along main canal : 8.5 km

b) Roads along laterals/sub-laterals : 9.0 km

c) Barrio/Minicipal roads : 12.2 km

d) Provincial/National roads : 21.1 km

The existing irrigation system in the CRIS is illustrated in Fig. 7.2 and the result of the inventory survey for the existing major facilities is summarized in Table 7.1.

2.2.3 Communal Irrigation System (CIS) and Pump Irrigation System (PIS)

The Communal Irrigation Systems (CIS) are operated by gravity with the small scale diversion dams and canal systems to make use of small rivers and streams. CISs have been constructed by NIA on provincial basis, and operation and maintenance are conducted by irrigators associations with guidance of Farm Systems Development Corporation (FSDC) and NIA. There are about seven such CISs in the study area. Although most of these were originally designed as parts of the national irrigation system, they are now operated as the communal irrigation systems.

The small-scale pump irrigation systems (PIS) has been developed by NIA and FSDC to further increase the irrigable area. In the study area, there are many pump irrigation systems for use of surface water and ground water, however these service areas are not always irrigated mainly due to the high cost of operation and maintenance, especially of fuel.

2.3 Drainage Condition

There are four major rivers and many drainage creeks in the study area, namely Gumain, Porac, Caulaman, Guagua rivers and Mababayan, Sapang Matua, Santa Catalina, Santor, Palcarangan, Lauc Pao Creek, etc. The Porac River has been connected with the Gumain River by the Porac - Gumain diversion channel. Flood protection dikes have been constructed for about 8.0 km downstream of the confluence with Porac - Gumain diversion channel, which has a carrying capacity of more than 1,000 m^3/sec. The floods from the upper reaches of the Porac and Gumain Rivers are drawn off throughout the diversion channel without inundation in the cultivated land.

The drainage capacities of the existing rivers and creeks except for the Porac - Gumain diversion channel are in some cases insufficient and the checkgate structures constructed in the creeks are not properly operated. Furthermore, the road crossing structures over rivers and creeks have insufficient carrying capacities, which cause some drainage problems at lower areas along Highway Route 7. The density of farm drains and drainage ditches at on-farm level in most of the study area also remains low.

CHAPTER 3 PROPOSED IRRIGATION AND DRAINAGE SYSTEM

3.1 Development Plan

The irrigation development depends on the water resources endowed in the upper reaches of the rivers in the study area. In this study area, there are three major rivers, the Gumain, Porac and Caulaman Rivers. These rivers are expected as the irrigation water sources. The areas in Porac - Gumain River Irrigation System and Caulaman River Irrigation System have been irrigated with existing diversion dams, the Gumain Diversion Dam, Porac Diversion Dam and Caulaman Diversion Dam.

In order to use the limited water resources to maximum extent, construction of a storage dam is essentially needed. The Gumain River has ample perennial flows and potential damsite was identified in the upper reach of the river. According to the optimization study of the project scale, the proposed active storage in the Gumain reservoir was estimated at about 99 MCM.

The study area is bounded by mountains in west, and the alluvial fans created by three rivers are extending from northwest to southeast with slight undulation in elevation ranging from EL. 40 m to EL. 50 m. Following the results of the soil investigation, present land use and topographic condition, the potential irrigable areas for the development plan were estimated at 11,000 ha of paddy fields and 5,750 ha of sugarcane fields. This potential areas include the existing irrigation systems' areas and are summarized as follows:

(Unit: ha)

Crop	Existing Irrigation Systems					Rainfed Areas	Total
	PGRIS	CRIS	CIS	PIS	Sub-total		
Paddy	4,890	540	540	2,970	8,940	2,060	11,000
Sugarcane	-	-	-	-	-	5,750	5,750
Total	4,890	540	540	2,970	8,940	7,810	16,750

3.2 Irrigation Water Requirement

Rice is the principal agricultural crop in the project area and sugarcane is the second. In addition to these two major crops, vegetables are also recommended as profitable crops for the project. Therefore, the study of irrigation water requirements was made for rice, sugarcane and vegetables.

The irrigation water requirements were calculated for the proposed cropping pattern on 10-day basis for 25 years from 1958/59 to 1982/83.

The basic year for irrigation plan was determined based on the drought year in five year return period with regard to following four items:

- annual rainfall
- dry season rainfall
- annual mean river discharge
- mean river discharge in dry season

The year of 1980/81 was selected as a basic year for irrigation plan because the values in 1980/81 for the above four items are close to values in five-year return period as shown in Table 7.2.

3.2.1 Irrigation Water Requirement for Paddy

(1) Crop Water Requirement

The crop water requirement is defined as the amount of water needed to meet the consumptive demand of the crop for optimum growth from seeding to harvesting. It consists of land soaking, land preparation and field crop requirements as shown below:

$$CWR = (Kls \cdot LS + Klp \cdot LP + Kf_c \cdot FC)\ Tunit$$

where, CWR: Crop water requirement (mm)

Kls: Area factor of land soaking

LS: Land soaking requirement (mm/day)

Klp: Area factor of land preparation

LP: Land preparation requirement (mm/day)

Kf_c: Area factor of field crop requirement

FC: Field crop requirement (mm/day)

Tunit: Number of days for calculation basis

1) Land Soaking Requirement

The land soaking requirement is the amount of water needed to saturate the soil prior to the initial breaking.

$$LS = Sn/t + Ev + P$$

$$Sn = (Sc - Wc \cdot Bd)/_{100} \times Drz$$

where, LS: Land soaking water requirement (mm/day)

Sn: Soil saturation requirement (mm)

t: Number of days to saturate soil (7 days)

Ev: Evaporation (mm/day)

P: Percolation rate (mm/day)

Sc: Soil saturation capacity (%)

Mc: Soil moisture content (%)

Wet season paddy Mc = Pwp
Dry season paddy Mc = (Fc + Pwp)/2

Pwp: Permanent wilting point (%)

Fc: Field capacity (%)

Bd: Bulk density

Drz: Depth of root zone (300 mm)

In the above equation, the rates of Sc, Pwp, Fc and Bd were selected from the following table according to the soil texture. The soil texture in the study area is generally sandy loam.

Soil Texture	Sc (%)	Pwp (%)	Fc (%)	Bd
Sand	38	4	9	1.65
Sandy loam	43	6	14	1.50
Loam	47	10	22	1.40
Clay loam	49	13	27	1.35
Silty clay	51	17	35	1.25

2) Land Preparation Requirement

The land preparation requirement is the amount of water needed to replace the losses due to evaporation and percolation, and to pond for transplanting after land soaking has been satisfied.

$$LP = Ev + P + SP/tsp$$

where, LP: Land preparation requirement (mm/day)

Ev: Evaporation (mm/day)

P: Percolation rate (mm/day)

SP: Depth of ponding for transplanting (25 mm)

tsp: Number of days for land preparation (20 days)

3) Field Crop Requirement

The field crop requirement is amount of water consumed by the crop during the period from transplanting to 15 days before harvesting and the needed percolation in the paddy field.

$$FC = Kc \cdot ETo + P$$

where, FC: Field crop requirement (mm/day)

Kc: Crop coefficient

ETo: Potential evapotranspiration (mm/day)

P: Percolation rate (mm/day)

The crop coefficient of paddy is shown in Fig. 7.3 based on the actual data by NSDB-NIA water management improvement project. The potential evapotranspiration was estimated by the modified Penman method using the meteorological records at Basa Air Base and Hacienda Luisita stations.

The percolation rate was determined on the basis of actual measurement carried out by the drainage section of PDD, NIA in 1983. The results of field tests showed the average percolation rate of 1.85 mm/day, ranging from 0.9 mm/day to 3.2 mm/day at fifteen points in the study area. Considering these results and general soil texture of sandy loam in the area, the percolation rate adapted in the study was 2.0 mm/day.

(2) Farm Water Requirement

The farm water requirement is calculated as follows:

$$FWR = CWR - RE$$

where, FWR: Farm water requirement (mm)

CWR: Crop water requirement (mm)

RE: Effective rainfall (mm)

The effective rainfall was estimated with daily water balance calculation assuming 50 mm high of paddy dike as shown below:

$$PWLf = PWLi - CWR + RA$$

where, PWLf: Final paddy water level after the day's rainfall considered (mm)

PWLi: Initial paddy water level (mm)

CWR: Weighted average crop water requirement (mm)

Ra: Actual depth of rainfall for the day considered (mm)

Using the above equation, an overflow occurs if the computed PWLf exceeds 50 mm limit of paddy spillway height. The amount of effective rainfall was then determined as follows:

Without overflow $RE = Ra$

With overflow $RE = PSH - PWLi + CWR$

where, RE: Effective rainfall (mm)

PSH: Paddy spillway height (50 mm)

In calculating the effective rainfall, the daily rainfall data during 25 years at Basa Air Base station were used.

(3) Diversion Water Requirement

The diversion water requirement is defined as the farm water requirement plus allowances for farm waste, operation losses and conveyance losses.

$DWR = FWR/Ef$

where, DWR: Diversion water requirement (mm)

FWR: Farm water requirement (mm)

EF: Overall irrigation efficiency

The overall irrigation efficiency was assumed as follows:

(Unit: %)

Item	Dry Season	Wet Season
On-farm efficiency	80	75
Conveyance efficiency	80	80
Operational efficiency	85	85
Overall efficiency	55	50

The calculated results of annual diversion water requirements during 25 years are shown in Table 7.3, and the crop water requirement and the diversion water requirement in 1980/81 are shown in Table 7.4 and Table 7.7, respectively.

3.2.2 Irrigation Water Requirement for Sugarcane and Vegetables

(1) Crop Water Requirement

The crop water requirement consists of land preparation for plant cane or vegetables and consumptive use of the crop.

$$CWR = \{(Kc \cdot ETo) \cdot Ksc + LP \cdot Kep\} \cdot Tunit$$

where,
- CWR: Crop water requirement (mm)
- Kc: Crop coefficient
- ETo: Evapotranspiration (mm/day)
- Ksc: Area factor of crop
- LP: Land preparation requirement (mm/day)
- Klp: Area factor of land preparation
- Tunit: Number of days for calculation basis

The crop coefficient of sugarcane and vegetables used for calculation is shown in Fig. 7.4 and Fig. 7.5, where the first month is always for land preparation. Evapotranspiration was estimated by use of the modified Penman method. The requirement for land preparation was assumed to be 50 mm.

(2) Farm Water Requirement

The farm water requirement can be obtained by subtracting the effective rainfall from the crop water requirement as shown below:

$$FWR = CWR - RE$$

where,
- FWR: Farm water requirement (mm)
- CWR: Crop water requirement (mm)
- RE: Effective rainfall (mm)

The effective rainfall was estimated with daily water balance calculation assuming 200 mm of water holding capacity in the root zone as shown below:

$$WHf = WHi - CWR + RA$$

where,
- WHf: Final water height after the day's rainfall considered (mm)
- WHi: Initial water height (mm)
- CWR: Weighted average crop water requirement (mm)
- RA: Actual height of rainfall for the day considered (mm)

Using the above equation, the effective rainfall (RE) was determined as follows:

1) If WHf is more than 200 mm

$$RE = 200 - (WHi - CWR)$$

2) If WHf is less than 200 mm or equal

$$RE - RA$$

(3) Diversion Water Requirement

The diversion water requirement was calculated from the farm water requirement with assumed overall irrigation efficiency of 50%.

The estimated annual diversion requirements during 25 years are shown in Table 7.3, and the crop water requirement and the diversion water requirements in 1980 are shown in Tables 7.5, 7.6, 7.8 and 7.9, respectively.

3.3 Effective Use of Return Flow

3.3.1 Return Flow

There are many checkgate structures on the drainage creeks to intake local water including return flow from the paddy fields. Most of the re-use structures are not functioning normally due to improper operation and maintenance, and lack of control gates. By rehabilitation of these structures, effective use of return flow and internal local flow would be realized.

The available discharge at the re-use structure was estimated on the basis of the following equation which was obtained by analyzing the actual data for creek runoff in UPRIIS.

$$Q = (K \cdot R + a) \cdot A + RF$$

where, Q: Creek discharge (m3/sec/10 days)

K: Coefficient (= 0.00485)

R: 10 days rainfall

a: Constant (= 0.1)

A: Drainage area at re-use structure (km^2)

RF: Return flow (m^3/sec/10 days)

In the dry season, where rainfall in the study area hardly occur and the available discharge at the re-use structure depends on the return flow from the contributing paddy fields. The rate of return flow was judged to be 30% of total irrigation supply to the contributing paddy fields from the results of the study made by Drs. D. Cablayan, W. Ramos and S.I. Bhuiyan, IRRI.

3.3.2 Re-Use Point

Through field investigation, about thirty checkgate structures were found in the drainage creeks. Most of these structures are not functioning normally due to the improper operation and maintenance without control gates. Out of thirty checkgate structures investigated, five structures could be rehabilitated for the effective use of water, taking into consideration the following points:

1) Scale of drainage area

2) Scale of the irrigation area commanded by a creekgate structure

3) Availability of existing leading canal

These five checkgate structures would be used for supplying the intaked water to a part of the proposed irrigation system through leading canals. The supplemental water from the diversion dams would be supplied to the proposed system but not through the checkgate structures at re-use points. The remaining checkgate structures except for the above five structures would be demolished to make smooth flow in the drainage creek.

The total irrigable areas covered by the effective use of return flow were estimated at about 720 ha on an average for 25 years from 1958/59 to 1982/83. Each re-use point and irrigable area are summarized in Table 7.10.

3.4 Proposed Irrigation System

3.4.1 Basic Consideration

In the study area, there are two National Irrigation Systems, the Porac - Gumain RIS and Caulaman RIS, and therefore in the final layout of the definite plan for the irrigation system, these existing facilities were incorporated as much as possible.

The proposed irrigation works in the project area are mainly divided into the following three works:

1) Construction of new facilities for diverting irrigation water from the Gumain reservoir to the existing Porac and Caulaman irrigation systems.

2) Rehabilitation of the existing irrigation facilities.

3) Expansion of irrigation facilities to the existing rainfed paddy fields and sugarcane fields which are proposed to be newly irrigated in the project.

3.4.2 Diversion Method of Irrigation Water

For diverting irrigation water from the Gumain reservoir to the existing Porac - Gumain and Caulaman irrigation systems, two alternatives were considered as shown below:

Alternative 1 : Water stored in the reservoir will be released to the Gumain River and be taken at the Upper Gumain Diversion Dam which will be constructed at about 2.6 km downstream of the Gumain Dam site. From the upper Gumain Diversion Dam, the irrigation water will be diverted to the Porac River by the Porac Diversion canal with 6.9 km in length and to the Caulaman main canal by the Caulaman Diversion canal with 6.7 km in length.

Alternative 2 : Water stored in the reservoir will be released at the outlet of intake structure of the dam without release to the Gumain River and be diverted to the Porac River by the Porac Diversion canal with the length of 9.3 km including the tunnel of about 1.4 km. To divert irrigation water to the Caulaman main canal, one siphon will be newly constructed at about 2.6 km downstream of the Gumain Dam site and then the Caulaman Diversion canal of 6.7 km will convey the irrigation water to the Caulaman main canal.

The above alternatives are illustrated on Fig. 7.6(1) and (2). In order to determine the most suitable diversion method of irrigation water, the comparative study was made, estimating the cost of comparative works. Depending on the cost estimate, Alternative 1 was recommended. The general features are shown in Fig. 7.6(1), (2) and the construction costs are summarized as follows:

(Unit: 10^3 US$)

Alternative 1		Alternative 2	
Work Item	Construction Cost	Work Item	Construction Cost
Preparatory Works	209	Preparatory Works	1,114
Diversion Dam	1,644	Diversion Tunnel	10,875
Intake Structure	317	Diversion Canal	229
Dike	126	Inverted Siphon	39
Total	2,296		12,257

3.4.3 Irrigation System

The proposed irrigation systems would be sub-divided into three river irrigation systems, namely Porac River Irrigation System, Gumain River Irrigation System and Caulaman River Irrigation System. The proposed Porac River Irrigation System consists of the areas commanded by the existing Porac - Gumain River Irrigation System (PGRIS) except for the Gumain South areas, the areas commanded by communal irrigation systems and pump irrigation systems in the north-eastern part of the existing PGRIS and the rainfed areas including the hilly lands around the upper reach of Porac river. The proposed Gumain River Irrigation System consists of the existing Gumain south areas, the areas commanded by the Bodega Earth Dam in the existing Caulaman River Irrigation System (CRIS) and the right bank areas of Gumain River commanded by the communal and pump irrigation systems. The proposed Caulaman River Irrigation System consists of the upper areas of the Gumain River, the existing CRIS's areas except for the areas commanded by the Bodega Earth Dam and the remaining sugarcane areas.

The optimum irrigation areas were determined by the water balance study in relation to the scale of the Gumain storage dam analyzing the available runoffs of Gumain, Porac and Caulaman rivers, irrigation water requirements and effective use of return flow as discussed in the previous paragraphs. Base on the result of the water balance study and economical comparison, the proposed irrigation system and areas are shown in Fig. 7.7 and Table 7.11.

3.5 Drainage Water Requirement

In general, the criteria for calculation of an unit drainage requirement defines as a rainfall intensity with certain probability and a drain period necessary for removal of excess water to an allowable extent.

In this study, the drainage water requirement for the proposed drainage system was estimated separately for removing excess rainfall from the paddy fields and for transporting the runoff from the hilly lands on the basis of a 5-year return period and use of the daily rainfall records at the Basa Air Base.

3.5.1 Drainage Water Requirement for Paddy Field

Generally, on the relation between the yield reduction rate of paddy and, depth and duration of sub-mergence at different growing stages of paddy, the following considerations could be made:

1) The sub-mergence at the growing stage of young panicle formation gives the serious damage to the yield of paddy, on the contrary, damage due to sub-mergence at the maturing stage is insignificant.

2) The duration of sub-mergence within 1 to 3 days is not significant, but damages of paddy remarkably increases due to sub-mergence beyond 3 days.

3) When a part of leaves still remains above water surface, the damage to paddy is decreased as compared with that when leaves are completely sub-merged.

While, the midest rainy season in the study area occurs in August. The growing stage of paddy between middle stage of tillering and beginning stage of panicle formation would correspond to midest rainy season.

Taking into account the above considerations, the drainage requirement for removing the excess rainfall in the paddy field was estimated on the basis of the maximum 3 days consecutive rainfall with 5-year return period. The allowable depth of sub-mergence in the paddy field should be 30 cm depending on the field condition. The equation of requirement is shown below:

$$R = \frac{I - D}{T}$$

where, R: Drainage water requirement for removal of access rainfall in the paddy field (mm/day)

I: 3 days consecutive rainfall (mm)

D: Allowable depth of sub-mergence in the paddy field (mm)

T: Drainage period (day)

The maximum 3 days consecutive rainfall (I) was estimated at 604.1 mm, and the unit drainage water requirement was estimated at 101.4 mm/day.

3.5.2 Drainage Water Requirement for Hilly Land

The estimation of drainage water requirement for transporting the runoff coming from hilly land of catchment area was made by applying the McMatch's formula suggested in DRAINAGE MANUAL, USBR. The design rainfall was estimated from the daily rainfall with 5-year return period.

$$Q_h = 2.3 \times 10^{-3} \cdot C \cdot i \cdot S^{1/5} \cdot A_h^{4/5}$$

$$i = R_{24} \cdot \left(\frac{1}{24}\right)^{1/3}$$

where, Q_h: Peak flow (m^3/sec)

C: Coefficient representing the basin characteristics (= 0.3)

i: Rainfall intensity for the time of flood concentration (mm/hr)

S: Average ground slope (= 1/400)

Ah: Hilly land area of drainage basin (ha)

The design rainfall was estimated at 311.7 mm and the unit drainage water requirement was estimated at 22.5 ℓ/sec/ha.

3.5.3 Design Drainage Water Requirement

The design drainage water requirement was estimated by using the following equation:

$$Q = Q_p + Q_h$$

$$Q_p = (R/8.64 \times 10^3) . A_p$$

where, Q: Design drainage water requirement (m^3/sec)

Q_p: Drainage water requirement for paddy field (m^3/sec)

Q_h: Drainage water requirement for hilly land (m^3/sec)

A_p: Paddy field area of drainage basin (ha)

3.6 Proposed Drainage System

3.6.1 Basic Consideration

The drainage plan in the project area was made by means of improving the existing rivers and creeks as main and secondary drains, and new construction of collector drains so as to minimize the construction cost and the land to be acquired. Further, tertiary drains and drainage ditches both at on-farm level were provided so as to remove excess water in the fields after the heavy rainfall and to create adequate conditions of drawdown for the harvesting of crops.

The improvement paln for the existing rivers and creeks was formulated based on the following basis considerations:

1) The smooth longitudinal profiles of rivers and creeks should be made. Sedimentations are noticeable at downstream reaches of rivers and creeks. Cross sectional flow areas become smaller than upstream and longitudinal profiles are sometimes reversed. In such cases, smooth profiles are first designed regardless of the scale of the rainfall probability.

2) The improvement of the cross sections are to be kept within the existing cross sections as much as possible to minimize the additional land compensation.

3) Widening the existing sections are to be straightened as much as possible except the portion near by community.

3.6.2 Proposed Drainage System

There are many natural rivers and creeks, and they could be used as the main and secondary drainage canals. Out of many rivers and creeks, the Gumain River and the Porac River have been implemented the river training works in the project area, constructing the flood protection dikes and the diversion channel from Porac river to Gumain river. The floods from the upper reaches of the Gumain and Porac rivers are led to lower reaches without inundating to the project area.

The plan of proposed drainage system was mainly made based on the floods from upper reaches of the Caulaman river and the Guagua river, and the drainage water in the project area.

Drainage system consists of the natural rivers, creeks, collector drains, tertiary drains, drainage ditches and catch drains to collect and drain runoffs from outside of the project area. The layout of the drainage system was determined based on the design drainage discharges, layout of the proposed irrigation system and topography. The proposed drainage system is shown in Fig. 7.8.

CHAPTER 4 PRELIMINARY DESIGN OF IRRIGATION AND DRAINAGE FACILITIES

4.1 Design Diversion Water Requirement

According to calculation results of the water requirements for a water balance study, the peak irrigation water requirements for wet season paddy, dry season paddy, sugarcane and diversified crops were found in August, February, May and February, respectively. The unit design irrigation water requirements are defined as the peak irrigation water requirements and are summarized as follows:

(Unit: ℓ/sec/ha)

Crop	Unit Design Irrigation Water Requirement	Peak Period
Wet Season Paddy	1.26	August
Dry Season Paddy	1.95	February
Sugarcane	1.17	May
Diversified Crop	1.51	February

The design diversion water requirements are defined as the peak diversion discharge, which are obtained by multiplying the unit design irrigation water requirement by the irrigation area. The design diversion water requirements calculated for each diversion dam are shown below.

(Unit: m^3/sec)

Irrigation System	Diversion Dam	Design Diversion Water Requirement
PRIS	Upper Gumain Diversion Dam	7.16
	Porac Diversion Dam	
	- East Main Canal	6.09
	- West Main Canal	1.11
GRIS	Gumain Diversion Dam	3.91
CRIS	Upper Gumain Diversion Dam	5.23
	Caulaman Diversion Dam	5.22

4.2 Upper Gumain Diversion Dam

The main function of diversion dam is to introduce the required quantity of irrigation water from the river to the project area at every stage of river water. In order to fulfill this purpose, the structure should be stable against flood and other forces, and should not hamper the river flow.

4.2.1 Basic Design Condition

(1) Location

The proposed dam site is located on the Gumain River about 2.6 km downstream of the proposed Gumain storage dam site. The Gumain river debouches to the downstream area immediately around the proposed diversion dam site. The river width are about 40 m and right bank has steep topography toward upstream reaches. The gradient of river bed is about 1 to 170.

(2) Geology

The geological condition of the diversion dam site is constituted by the sediment deposits consisting of sand and gravel deposits. The low river terrace of gravel and sand has mainly developed on the left bank. The N-value is ranging from 30 to 50. The foundations are expected to have the barring capacity of about 30 t/m^2. The coefficient of permeability (K) is assumed to be around 10^{-3} cm/sec.

(3) Design Flood Discharge

The flood discharge with 100-year return period was taken as the design flood discharge. The estimated design flood discharge is 2,000 m^3/sec which is rounded up the flood discharge with 100-year return period. The design flood water level in the downstream of the dam was calculated to be WL. 38.5 m by applying the manning's formula.

(4) Design Intake Water Level

Based on the ground surface elevation of irrigation area and the topographic condition of the dam site, the design intake water level was determined to be WL. 45.0 m.

4.2.2 Design of Diversion Dam

(1) Type of Diversion Dam

As described in the "Basic Design Condition", the design flood discharge is large and the foundation materials are sediment deposits. The proposed storage dam would be constructed about 2.6 km upstream of the diversion dam site and almost sedimentations would be saved at the storage dam. In such conditions, the ogee-type concrete dam with scouring sluice is proposed. The full width of concrete dam would be constructed on the sediment deposits as a floating type.

(2) Hydraulic Calculation

1) Overflow Depth

Overflow discharge of water under complete overflowing condition can be calculated by:

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

where, Q: Discharge (m^3/sec)

B: Width of crest (m)

H: Upstream water depth above crest (m)

C: Coefficient of discharge (= 1.70)

2) Conjugate Depth and Length of Apron

Conjugate depth and length of apron can be calcualted by applying the standard of USBR. The equations are shown below:

$$h_1 = \frac{q}{\sqrt{2g\,(W + H - h_1 - h_f)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$V_1 = q/h_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

$$F_1 = V_1/\sqrt{gh_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

$$h_2 = \frac{h_1}{2}\left(\sqrt{1 + 8F_1^2} - 1\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

$$L \geqq 4.5 \cdot h_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

where, h_1, h_2: Conjugate depth (m)

q: Unit discharge (m^3/sec)

W: Dam height (m)

H: Overflow depth (m)

h_f: Head loss (m), $h_f = 0.02 \cdot W(W/H + 1)$

V_1: Velocity in the upstream (m/sec)

F_1: Froude number

L: Length of apron (m)

3) Creep Length

Creep length can be calculated by Bligh's formula.

$$L \geqq C \cdot \Delta H$$

where, L: Creep length (m)

c: Coefficient of Bligh's formula which is decided by the foundation materials

ΔH: Difference of water head from upstream to downstream (m)

Typical section of the diversion dam was decided considering such forces as external water pressures, uplift pressures and own weight of the dam. As the foundation of the proposed dam site would have enough bearing capacity for dam, the stability was examined on over turning and sliding. The general features are shown in Table 7.12.

(3) Intake Structures

The intake structures were designed based on the intake water discharges. In order to prevent the sand intrusion from the river, a permissible intake velocity was limited to be about 1.0 m/sec.

(4) Dike

In order to protect the downstream area and the proposed diversion canals against the flood, the dikes are required. Approximate length and height were estimated to be 270 m and 5 m, respectively.

4.3 Diversion Canals

4.3.1 Porac Diversion Canal

(1) Comparative Study of the Construction Method

On the Porac diversion canal route, some portions of the route which are located just at upper portion of Basa Air Base having about 2.0 km length, require deep excavation of about 10 to 30 m due to the topographic condition. However, there is any no possibility of finding the another route to avoid such deep excavation, because the other route will pass through the Basa Air Base and the length becomes longer, about two times of the proposed route. Therefore, the design of the diversion canal should carefully be made taking into account the construction method and cost.

1) Geology

The geological condition of the diversion canal route is constituted by the river terrace deposits consisting of sand and gravel deposits. The N-value was assumed ranging from 30 to 50 and the coefficient of permeability (K) was considered to be around 10^{-3} cm/sec. Therefore for the whole length of the canal, the concrete lining was proposed in order to check seepage from the canal banks and bottom, and to protect the canal section against erosion. The slopes for temporary excavation and permanent excavation were planned to be 1 to 1.2 and 1 to 1.5, respectively.

2) Alternative Plan of the Construction Method

On the above geological conditions, two construction methods were finaly considered as shown below:

Plan-1 : To construct a concrete lining canal with side slope of 1 to 1.5.

Plan-2 : To construct a culvert with side slope of 1 to 1.2 for temporary excavation.

The above alternative plans are shown in Fig. 7.9(1) and (2). In order to determine the most suitable method, the comparative study was made, estimating the cost of comparative works. Depending on the cost estimate, Plan-1 was recommended technically and economically. The construction costs for the portion of comparative works are summarized as follows:

(Unit: 10^3 US$)

Comparative Works	Unit	Q'ty	Cost	Q'ty	Cost
Excavation for Plan-1	m^3	1,620,000	6,430	-	-
Excavation for Plan-2	m^3	-	-	1,440,000	4,450
Backfilling	m^3	-	-	1,410,000	2,440
Reinforcement Concrete	m^3	-	-	16,200	2,240
Concrete Lining	m^3	2,500	190	-	-
Total			6,620		9,110

(2) Design of the Diversion Canal

The diversion canal has a design capacity of 7.16 m^3/sec. The canal would be lined with a 10-cm thick plain concrete for its all reaches. The design velocity is ranging from 0.6 to 1.5 m/sec and the bottom width is 2.0 m. The typical cross section is shown in APPENDIX II "Drawings".

4.3.2 Caulaman Diversion Canal

The geophysical condition for the Caulaman diversion canal is constituted by the river terrace deposits consisting of sand and gravel deposits, as well as the Porac diversion canal.

The Caulaman diversion canal would also require the concrete lining with side slope of 1 to 1.5. The design discharge is 5.23 m^3/sec and the bottom width is 2.0 m. The typical cross section is also shown in APPENDIX XII.

4.4 Rehabilitation of the Existing Diversion Dams

4.4.1 Porac Diversion Dam

According to the inventory survey, the Porac diversion dam is still in good condition. However, the falling shutter and the intake gates at both sides would need replacement and the river banks up and downstream of the dam would need shore protection.

The rehabilitation plan was made based on the following considerations:

- Present intake water level should be kept.
- Present capacity of flood way should be kept.
- Unfunctioning gates should be replaced.
- The sedimentation around the intake should be excluded.
- The eroded bank should be repaired.
- The weathered concrete surface should be treated with plain concrete.
- The rock riprap should be laid at the downstream of apron.

The features of rehabilitated Porac diversion dam are shown in Table 7.13.

4.4.2 Gumain Diversion Dam

According to the inventory survey, the Gumain diversion dam is in good condition. Following the above mentioned considerations, main rehabilitation work would be replacement of the intake gates. The irrigation area commanded by the Gumain North Main Canal would be irrigated from the Porac Diversion Dam on the proposed irrigation system, and therefore, the left side intake for the North Main Canal would not be required. The features of rehabilitated Gumain diversion dam are shown in Table 7.14.

4.4.3 Caulaman Diversion Dam

According to the inventory survey, the Caulaman diversion dam is in good condition as well as the Gumain diversion dam. Following considerations for rehabilitation plan, the intake gates would be required to be replaced. The features of rehabilitated Caulaman diversion dam are shown in Table 7.15.

4.5 Irrigation Canals and Related Structures

4.5.1 Design of Irrigation Canals

All irrigation canals except for the diversion canals were designed as unlined canals with trapezoidal section. The design of the irrigation canals were made based on the basic design criteria described as follows:

(1) Design Discharge

Based on the irrigation water requirement calculated in Section 3.2 and the commanding area, the design discharges for irrigation canals were estimated. Irrigation diagram for the proposed irrigation system is shown in Fig. 7.7.

(2) Velocity

The maximum permissible velocity of the unlined canal was determined so as not to cause scouring of canal. The minimum permissible was determined so as not to induce the growth of aquatic plant and moss, and not to cause the sedimentation in canal. Permissible velocity of each canal was determined as follows:

Canals	Maximum Velocity (m/sec)	Minimum Velocity (m/sec)
Main canals	1.0	0.5
Lateral & sub-lateral canals	0.8	0.3

(3) Roughness Coefficient

Based on the design criteria of NIA, coefficient of roughness was decided as follows:

Canals	Coefficient of Roughness
Main canals	0.0225
Lateral & sub-lateral canals	0.025

(4) Freeboard

The freeboard of the canal was also designed based on the design criteria of NIA as mentioned below.

1) When water depth is 2.0 m or greater : $Fd = 0.25d + 0.30$
2) When water depth is less than 2.0 m : $Fd = 0.4d$

(5) Canal Base Width and Water Depth

The relationship between the canal base width and maximum water depth was decided as follows:

	Main Canals		Lateral & Sub-lateral Canals						
Width	5.0	4.0	3.0	2.5	2.0	1.5	1.0	0.8	0.6
Water Depth	1.8	1.4	1.5	1.5	1.2	1.0	0.8	0.5	0.4

Each canal length is shown in Table 7.16.

4.5.2 Related Structures

Various related structures would be required in conjunction with irrigation canals for conveyance, regulation and measurement of irrigation water and protection of canal system.

The general characteristics and design criteria of these structures are briefed as follows:

(1) Head Gate and Turnout

Head gates would be constructed to divert the required water from main canals to lateral canals, and turnouts would be provided to divert the required water from parent canals, mainly lateral canals, to main farm ditches.

(2) Check Gate

In order to maintain the required water level at the site of offtaking ever during periods of off-peak discharge, check gates would be provided where a number of turnouts are densely provided or where fairly large discharge is diverted. The check gate consists of upstream transition, throat section and downstream transition, and would be equipped with one or several rectangular slide gates and operation deck in the throat.

(3) Bridge and Culvert

Bridges or culverts would be constructed where a road crosses over the canal. These bridges and culverts would be strong enough for the increase of heavy traffic after the project implementation. Bridge would be provided for main canals and culvert would be provided for lateral canals.

(4) Siphon

Siphons would be constructed to convey canal water under rivers and drainage canals. The siphon consists of upstream transition, concrete barrel and downstream transition. Two types of siphon barrel, box type and concrete pipe type would be adopted depending on the design discharge. The box barrel type would be applied for the discharge more than 2.0 m^3/sec.

(5) Drop

The function of drop structure is to dissipate excess energy. Drops are classified into two types depending on canal water discharge, i.e. inclined type and vertical type. The inclined type would be applied in case that canal discharge is more than 2.0 m^3/sec and drop height is more than 1.5 m.

(6) Waste Way and Spillway

Waste ways and spillways would be constructed in the canal system for the purpose of spilling out excess flow or flushing off all water in the canals in case of emergency and clearing and repairing canals. This structure would be provided in the following sites:

1) the upstream of siphon structure.

2) the end of main canal.

3) the site where the canal section is reduced at the downstream of head gate and the canal section does not have a enough capacity to flow the design discharge at the upstream.

All spillways should be connected to the nearby drainage canals.

(7) Cross Drain

Cross drains would be constructed across the irrigation canals at the place where the canals run across depressed lands or natural streams. Cross drains are classified into two types depending on the design discharge. Type-A with a single box barrel would be provided for the design discharge of more than 1.6 m^3/sec. Type-B has a single pipe barrel.

(8) Aqueduct

Aqueducts would be constructed at the crossing point between the proposed canal and the existing canal. Canal bed of the proposed canal should be sufficiently high at the crossing point as to provide enough free board for the existing canal.

The number of structures for the proposed irrigation system are shown in Table 7.16.

4.5.3 Rehabilitation of the Existing Canals and Related Structures

(1) Canals

According to the proposed irrigation system, existing irrigation systems would be required the rehabilitation works. Most existing canals are heavily silted with low and eroded embankments.

The existing canal sections were checked their flow capacities for the proposed design discharges. In order to meet the design discharges, some canals would be necessary to be expanded.

The rehabilitated canal lengths in each proposed irrigation system are shown in Table 7.16.

(2) Related Structures

The existing related structures on the main, lateral and sub-lateral canals would be required the repair or replacement according to expand the canals and to revise the design discharges. The number of rehabilitated structures are shown in Table 7.16.

4.6 Drainage Canals and Related Structures

4.6.1 Design of Drainage Canals

(1) Natural Drainage Canals

The proposed drainage system would consist of natural rivers and creeks, collector drains, tertiary drains, drainage ditches and catch drains to collect and drain runoffs from the outside of the project area.

Many rivers and creeks in the project area would be used for the proposed main and secondary drainage canals with a few expansion around the lower reaches of the area. The design discharges were calculated on the basis of the unit drainage water requirements and the proposed drainage networks. The flow capacities in the major rivers and creeks were checked at several points on each river and creek, and are shown in Table 7.17. Out of rivers in the project area, the old Porac river and the Caulaman river would be required the improvement of the sections around the National Highway No. 7. The general features of natural drainage canals are shown in Table 7.18.

(2) New Constructed Drainage Canals

New constructed drainage canals would consist of collector drains, tertiary drains, drainage ditches and catch drains. The collector drains would be connected into the rivers or creeks.

Tertiary drains are defined as the drainage canals collecting excess water from drainage ditches and leading water to collector drains.

Each length and number of structures of collector drains are shown in Table 7.19 and those of tertiary drains, drainage ditches and catch drains are shown below:

Drain	Length (km)				Related Structures (No.)	
	PRIS	CRIS	GRIS	Total	Bridge	Culvert
Tertiary Drain	25.0	18.0	15.0	58.0	-	116
Drainage Ditch	141.0	105.0	75.0	321.0	-	-
Catch Drain	4.3	5.0	-	9.3	6	10

The proposed drainage layout is shown in Fig. 7.8.

The sections of drainage canals were designed for the following criteria:

Type of channel : Trapezoidal earth canal

Side slope of channel

- Collector drain : 1 : 1.5
- Tertiary drain : 1 : 1.5
- Drainage ditch : 1 : 1
- Catch drain : 1 : 1.5

Coefficient of roughness: 0.03

The hydraulic calculation in the design of canal was made by using the Manning's formula.

4.6.2 Related Structures

The structures related to the drainage network are bridges and culverts. The bridges and culverts were planed and designed with the same principals as mentioned in Section 4.5.2.

The existing checkgates and brush dams in the rivers and creeks would be demolished to secure the flow capacitites except for five checkgates which would be utilized with some rehabilitation works for the effective use of return flow as mentioned in the previous paragraph.

The required number of the structures on the natural drainage canals and new constructed drainage canals are 22 and 67, respectively. The number of the structures on each drainage canal are shown in Table 7.18 and Table 7.19.

4.7 On-Farm Development

4.7.1 On-Farm Development for Paddy Field

A system of farm ditches would be provided to convey water throughout the rotation area of about 50 ha. The main farm ditches would carry irrigation water from turnouts on laterals or main canals. Each supplementary farm ditch receiving water from the main farm ditch through a division box, would serve a rotation unit of about 10 ha. Typical farm layout is shown in Fig. 7.10.

The maximum and minimum permissible velocity in farm ditches is 0.65 m/sec and 0.25 m/sec, respectively. The freeboard of farm ditch was designed based on the design criteria of NIA as shown below:

$$Fd = 0.1d + 0.15$$

where, Fd: Freeboard (m)

d: Water depth (m)

The hydraulic gradient is within the range of 1 in 2,500 to 1 in 1,500. The relationship between the base width and water depth was decided as follows:

(Unit: m)

Base Width	Water Depth
0.5	0.45
0.4	0.30
0.3	0.30

In order to devide irrigation water from main farm ditches to supplementary farm ditches, the division boxes were installed on main farm ditches.

In one lotation unit of 50 ha, one tertiary drain and five drainage ditches were aligned as shown in Fig. 7.10. Those tertiary drains would convey the excess water in the fields to collector drains.

4.7.2 On-Farm Development for Sugarcane Field

(1) Typical Rotation Unit

On farm irrigation system for sugarcane would be applied the furrow irrigation on this project. A system of farm ditches for sugarcane fields would be provided to convey water throughout the rotation area of about 40 ha. The farm ditches would convey irrigation water from the turnouts on laterals or main canals. Each rotation unit of 4 ha would be served irrigation water directly from the farm ditch through a division box. The irrigation water would be applied to furrows between rows of plants. The maximum permissible gradient of field for furrow irrigation would be 1 in 12. The size of rotation unit would be 100 x 400 m. Typical farm layout is shown in Fig. 7.11.

(2) Sample Computations of Irrigation Frequency

Irrigation frequency can be calculated by the following equations.

$$AM = \frac{(FM - WP) \cdot H}{100}$$

$$TAM = \frac{AM}{w} \times 100$$

$$T \leqq \frac{TAM}{CWR}$$

where, AM: Available moisture (mm)

FM: Moisture of field capacity (%)

WP: Moisture at wilting point (%)

H: The first layer depth of root zone (m)

w: Absorption rate of the first layer (%)

TAM: Total available moisture of root zone (mm)

T: Irrigation frequency (day)

CWR: Crop water requirement (mm/day)

In the above equation, the rates of FM and WP were selected from the following table. The type of soil in the project area is generally sandy loam.

Type of Soil	Porosity (%)	% Moisture by Volume	
		at Field Capacity	at Wilting Point
Clay (C)	60	45	27
Silty clay (SC)	60	45	25
Silty clay loam (SCL)	52	39	22
Clay loam (CL)	43	32	19
Sandy loam (SL)	37	28	13
Sandy oag loam (SOL)	33	25	13

On the assumption that the depth of root zone, the first layer depth (H) and absorption rate (w) is 60 cm, 15 cm and 40%, respectively, the total available moisture (TAM) was calculated to be 56.25 mm. The crop water requirement (CWR) was estimated at 5.0 mm/day in the section 3.2. The irrigation frequency (T) was calculated to be 11 days under the above mentioned situations.

4.8 Inspection Road

For proper operation and maintenance of project facilities, well arranged inspection roads are of vital importance. Since these roads would be used as village roads and farm roads after the project implementation, the arrangement of the inspection roads should be made considering the existing road networks.

4.8.1 Main Inspection Road

The main inspection roads would be required for inspection, operation and maintenance of the diversion canals and main canals. Considering future increase of vehicles for the inspection and operation, and heavy construction equipment to be required for the canal maintenance and repair, all the main inspection roads were so designed as to have an effective width of 4 meters and to be gravel-paved. These roads would also be used for movement of agricultural products and equipment, and for day-to-day services between villages and from these to highway.

4.8.2 Lateral Inspection Road

The lateral inspection roads would mainly be provided alongside the lateral canals. All these roads would have an effective width of 3 meters with gravel-paved. These roads would also be used for the purpose of farm operation, particularly for harvesting.

4.9 Work Quantities

On the basis of the preliminary design mentioned above, the drawings of major canals and related structures were prepared as attached in APPENDIX XII. The work quantities of major canals and structures were estimated from these drawings. For rehabilitation works of the existing facilities, the work quantities were obtained considering their present conditions for requirements of the proposed irrigation and drainage systems. The work quantities of the on-farm facilities were estimated based on the sample calculations for a typical area.