PT. PERUSAHAAN LISTRIK NEGARA
(PERSERO)

KOTAPANJANG HYDROELECTRIC POWER AND ASSOCIATED TRANSMISSION LINE PROJECT

PROJECT COMPLETION REPORT
ON
KOTAPANJANG HYDROELECTRIC POWER PROJECT

MAIN REPORT

NOVEMBER 1999

TOKYO ELECTRIC POWER SERVICES CO., LTD.
in association with
PT. YODYA KARYA & PT. TRIMITRA NUSA ENGINEERING

TEPSCO TOKYO ELECTRIC POWER SERVICES CO., LTD.
IN ASSOCIATION WITH
P.T. YODYA KARYA AND P.T. TRIMITRA NUSA ENGINEERING
KOTAPANJANG HYDROELECTRIC POWER AND ASSOCIATED TRANSMISSION LINE PROJECT

No. KTPJ/350/99

Jakarta, November 19th, 1999

To : Mr. Sartam Suhartama
General Project Manager
PT. PLN (PERSERO)
Proyek Induk Pembangkit dan Jaringan
Sumatera Barat dan Riau
Jl. Batang Agam No.5
Bukittinggi

Contract : Loan OECF No. IP - 374 & IP - 358
Contract No.060.PJ/070/1991/M dated 03 June 1991
Contract No.123.PJ/070/1991/M dated 03 June 1991
Engineering Services I & II

Subject : Submission of the Project Completion Report

Dear Sir,

We are pleased to submit to you thirty (30) copies of the "Project Completion Report" for the Kotapanjang Hydroelectric Power Project. The report consists of the following volumes.

- Summary Report
- Main Report
  - Part 1 Project Planning and Design of Civil Works
  - Part 2 Construction of Civil Works
  - Part 3 Architectural Works
  - Part 4 Telemetering and Flood Warning System
  - Part 5 Metal Works (Hydro Mechanical Works)
  - Part 6 Generating Equipment (Powerstation and Switchyard)
  - Part 7 Transmission Line
  - Part 8 Substation
  - Part 9 Road Relocation
  - Part 10 Project Cost
  - Part 11 Contract Administration
  - Part 12 Training and Transfer of Knowledge
  - Part 13 Environmental Aspects
- Photo Record

Site Office : C/O PLN Rantau Berangin Base Camp., Kampar Regency, Riau Tel. (0762) 21272-6 ext. 6202, 6203, Fax. ext. 6202
Jakarta Office : Summitmas Building II, 4th Fl. Jl. Jend. Sudirman Kav. 61-62, Jakarta 12190, Indonesia Tel. (021)-2523335 (Hunting), Fax. (021)-2525193
Head Office : Hibiya Chunichi Bldg. 1-4 Uchisaiwai 2-chome, Chiyoda-ku, Tokyo 100, Japan
Tel. (03) 5510-6000, Telex : TEPSCO.J25674, Fax. : (03) 3501-8919

No. KTPJ/350/99 Jakarta, November 19th, 1999

This report has been prepared in accordance with the Engineering Services Agreement No.060.PJ/070/1991/M which was concluded between PERUSAHAAN LISTRIK NEGARA and TOKYO ELECTRIC POWER SERVICES CO., LTD. in association with PT. YODYA KARYA AND PT. TRIMITRA NUSA ENGINEERING on June 3, 1991.

All the Engineering Services for this project performed by us are now complete. In this regard, we should note that the enclosed Project Completion Report is the culmination of our effort during the period from June 1991 to date for implementation of the project and covers all survey, investigation, design, construction supervision, etc. of the project.

In closing, we wish to take this opportunity to express our sincere gratitude to you and your staff for the kind assistance and courtesy you extended to us during the entire project.

Yours faithfully,

TOKYO ELECTRIC POWER SERVICES CO., LTD. TEPSCO K H P P

Y. Yoshida
Resident Manager

C.C :
1. KDIVPKIT
2. KDIVPRING
3. KDLH I
4. KDLJ I
5. KAPRO TEAM
6. KSOP
7. KAPRO
8. TEPSCO JKT OFFICE
9. TEPSCO TOKYO

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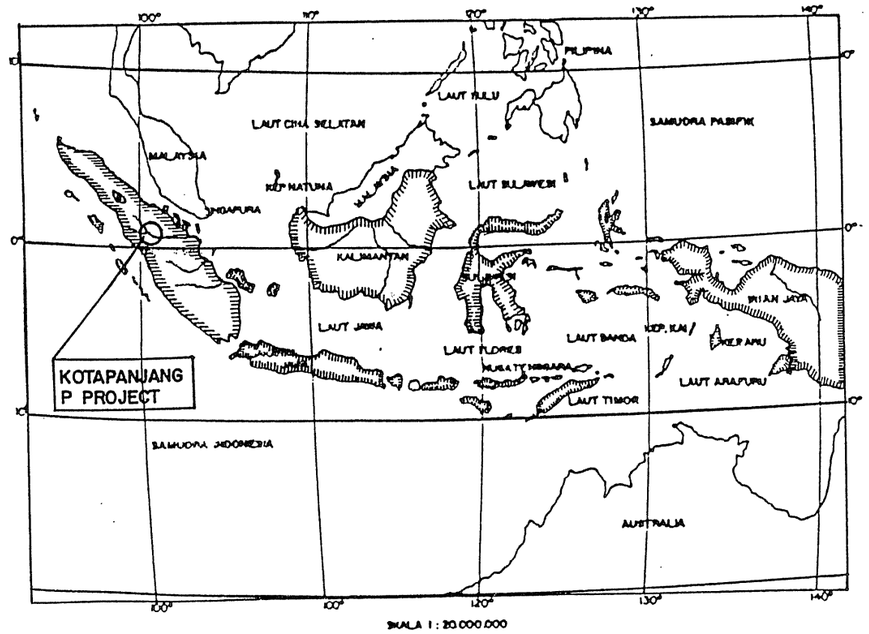
LOCATION MAP

REPUBLIC OF INDONESIA





LEGEND

- NATIONAL ROAD
- LOCAL ROAD
- NATIONAL BORDER
- PROPINCE BORDER
- PROPINCIAL CAPITAL

PROJECT MAIN DATA

RESERVOIR

Effective storage capacity	:	1040 x 10^6 m^3
Water surface area	:	124 km^2
High water level	:	EL. 85.0 m
Low water level	:	EL. 73.5 m
Catchment area	:	3337 km^2

DAM

Type	:	Concrete gravity
Height	:	58 m
Crest length	:	257.5 m
Dam volume	:	301,300 m^3

POWER STATION

Type	:	Ground
Firm discharge	:	165 m^3/sec
Peak discharge	:	348 m^3/sec
Firm output	:	54 MW
Peak output	:	114 MW
Installed capacity	:	114 MW (38 MW x 3 units)
Annual energy output	:	583 GWh
Penstock	:	3 lines x 5.0 m (D) x 86.9 m (L)
Power plant	:	35.6 m (W) x 43.2 m (H) x 80.3 m (L)

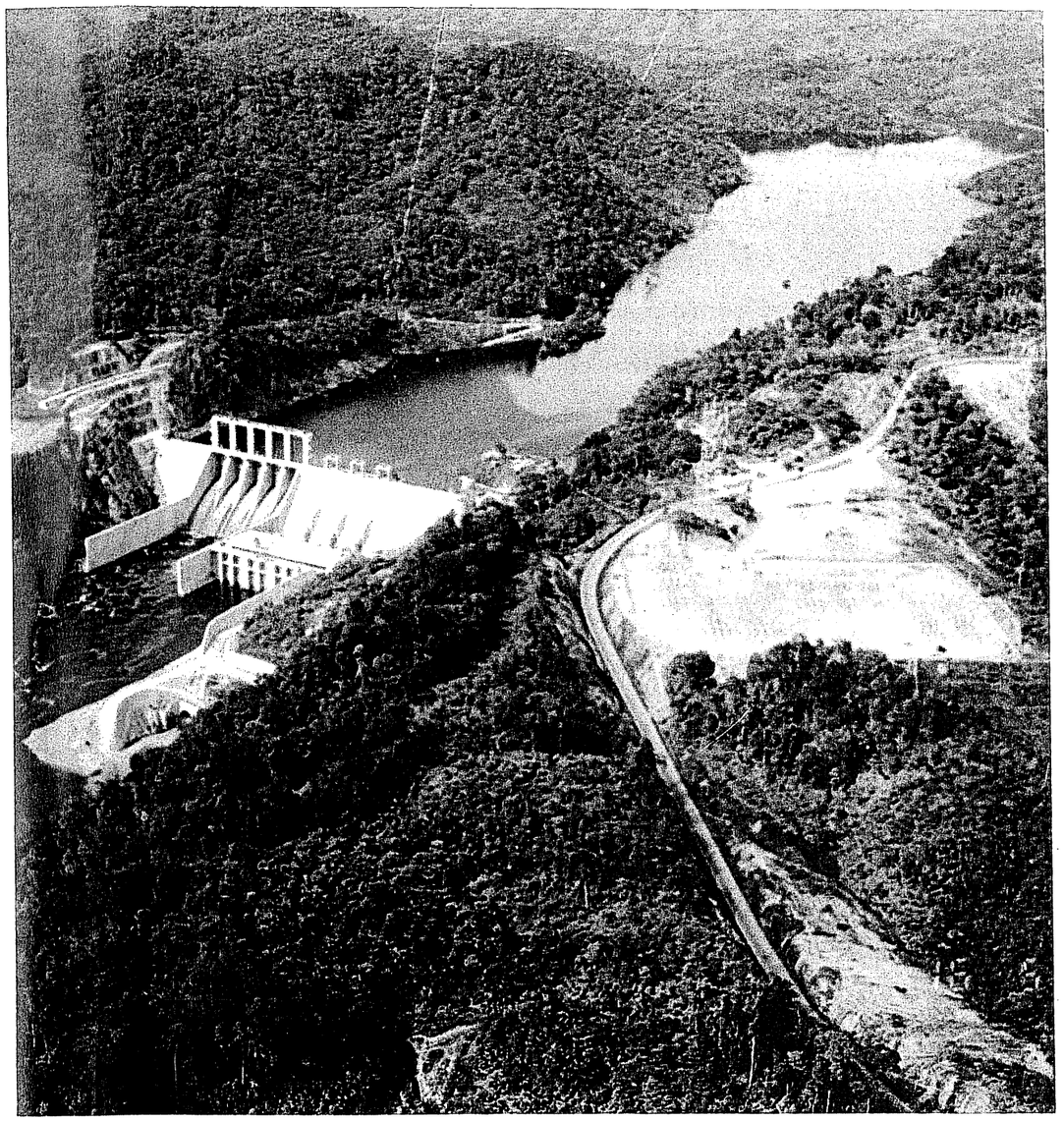
TRANSMISSION LINE

Line voltage & No. of circuits	:	150 kV, double-circuit
Type and No. of towers	:	Steel structure, 448
Type of conductor	:	ACSR 435 or 330 mm^2
Length of line	:	149.1 km

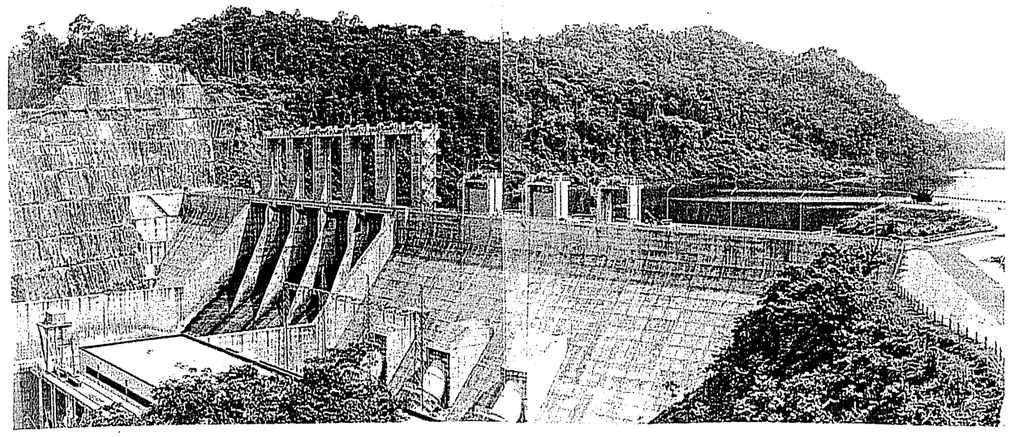
RELOCATION ROAD

Provincial road	:	6.0 m (W) x 22.0 km (L) gravel layer
National road	:	6.0 m (W) x 22.0 km (L) asphalt pavement
Bridges	:	Kampar Kanan 6.0 m (W) x 293.1 m (L)
		Gulamo 6.0 m (W) x 28.1 m (L)

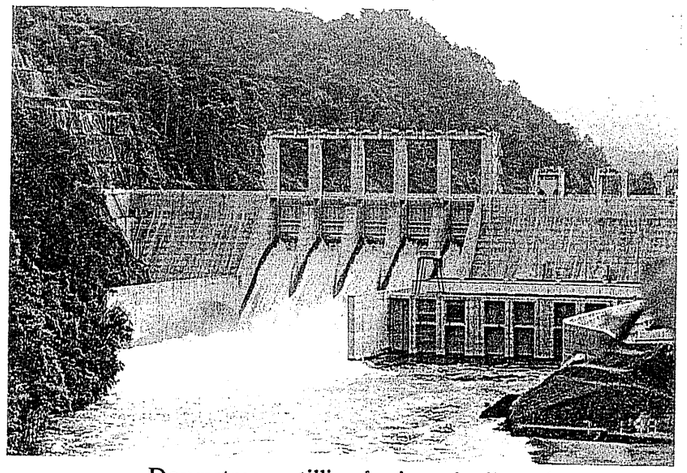
CONSTRUCTION PERIOD	:	1992 – 1999
CONSTRUCTION COST	:	32,320 million Yen



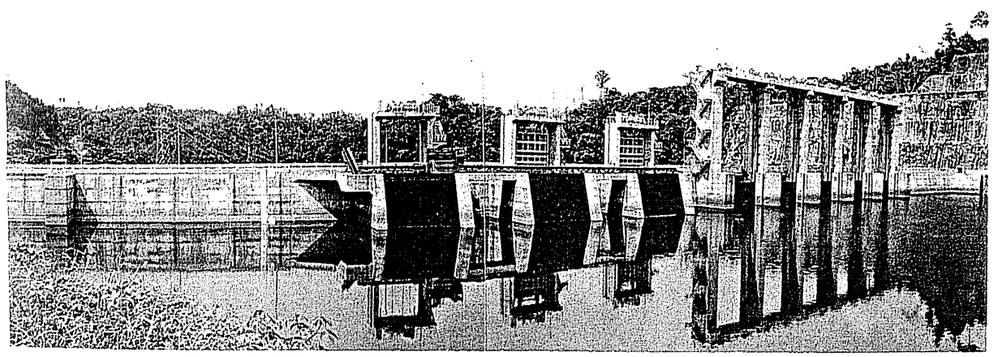
General View of the Project Site



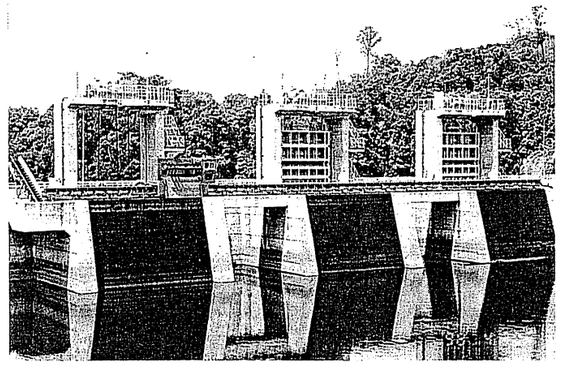
Dam and powerstation, looking from downstream left bank



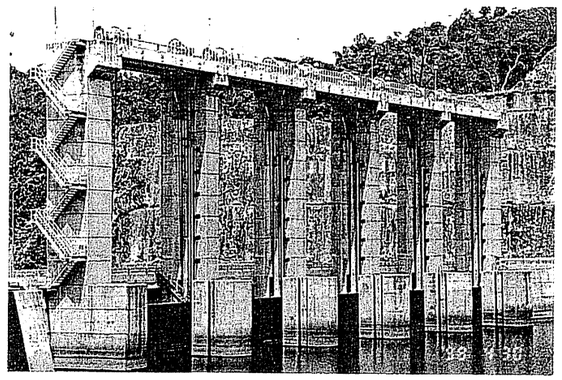
Downstream stilling basin and tailrace bay,
looking from downstream left bank



Upstream of dam, spillway and intake gate, looking from upstream left bank



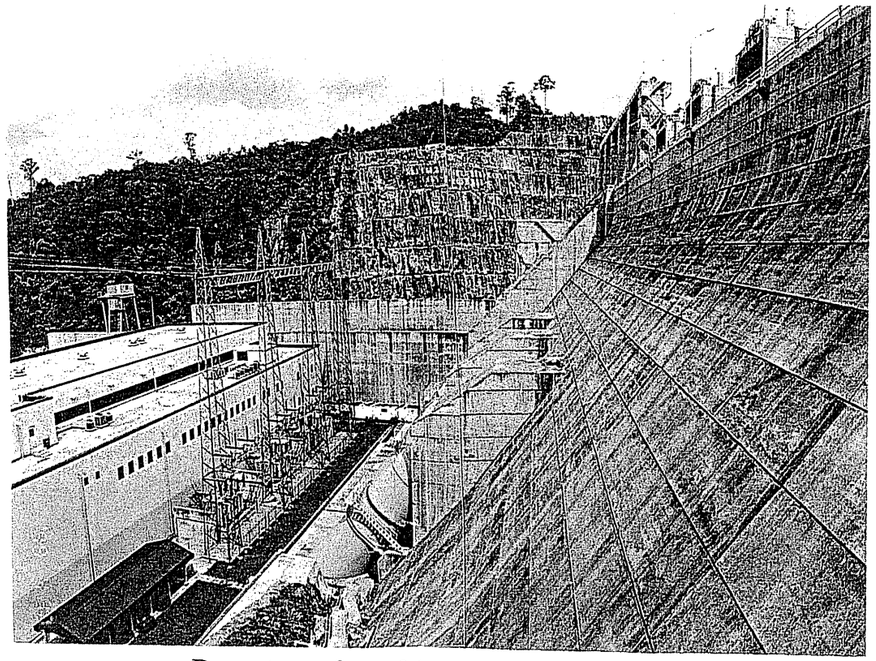
Intake structure, looking from upstream



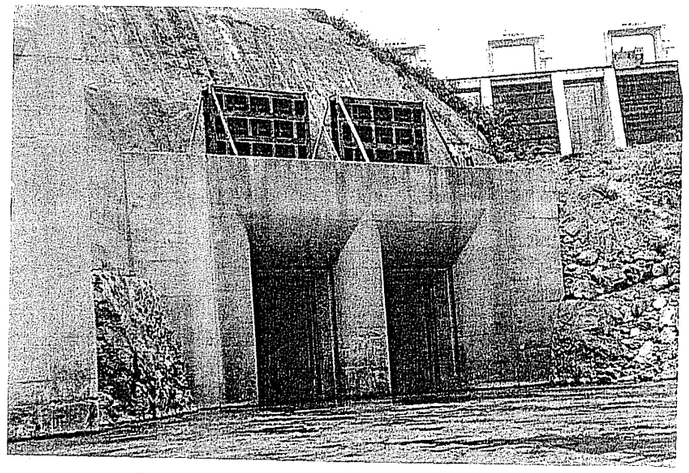
Spillway piers and hoist deck, looking from upstream



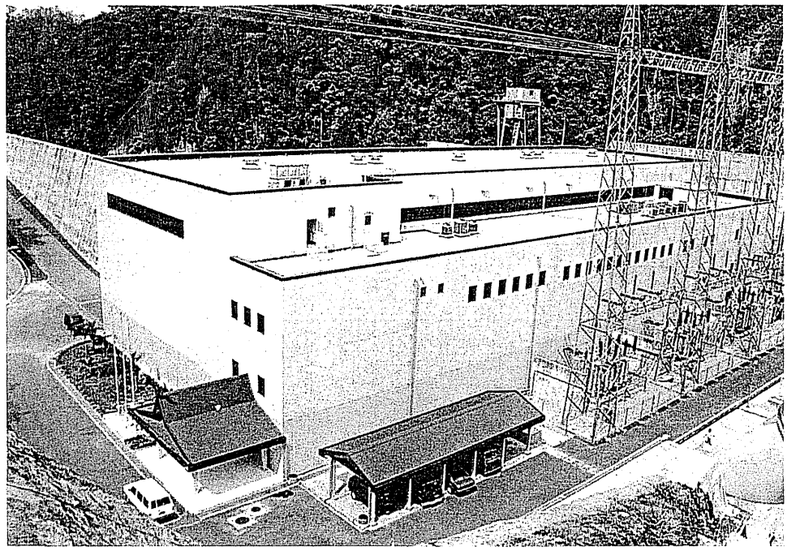
Gallery in dam



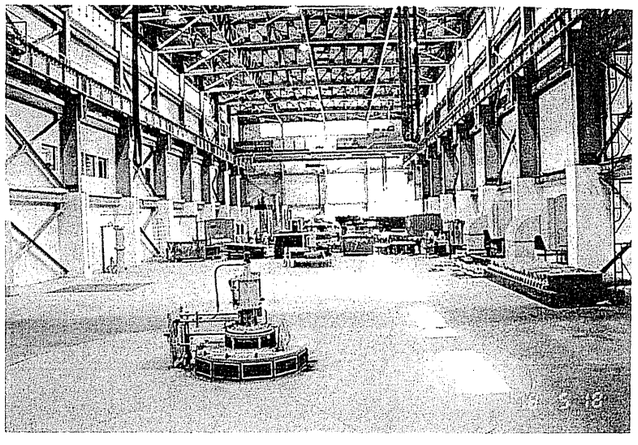
Downstream face of dam, penstock, powerhouse
and transformer, looking from left dam abutment



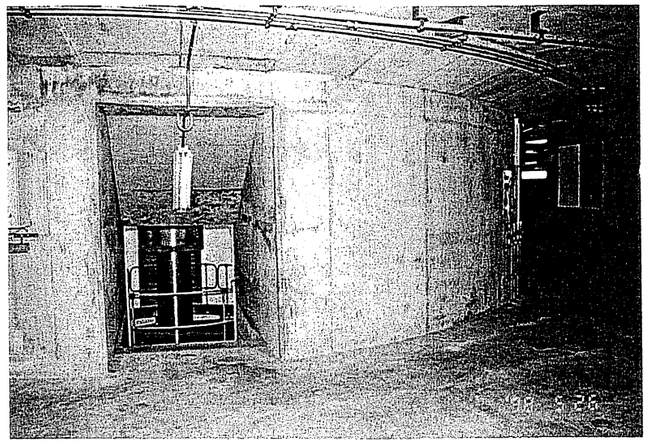
Diversion tunnel inlet and closure gates,
looking from upstream left bank



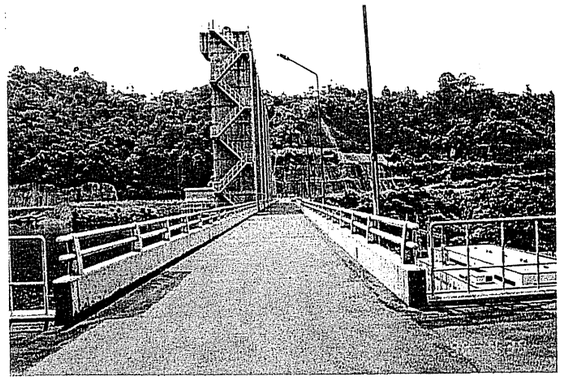
Powerhouse looking from left dam abutment



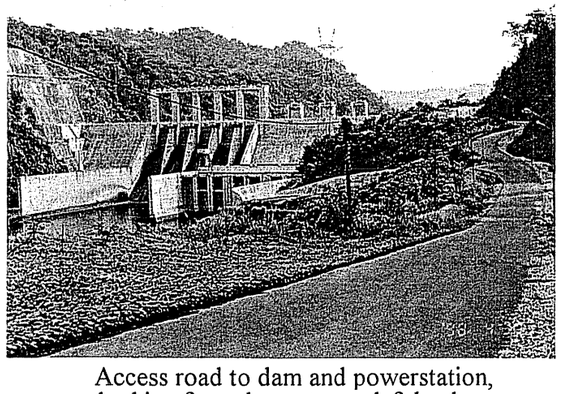
Powerhouse generator room



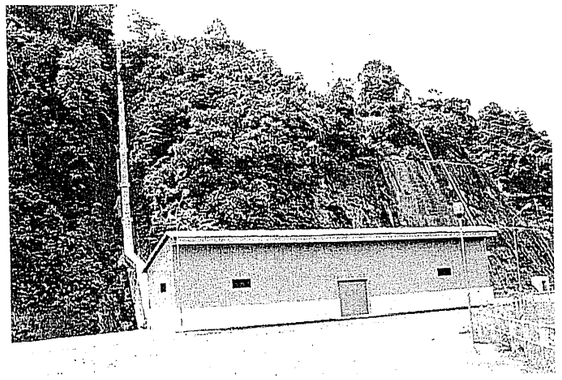
Around barrel of turbine room



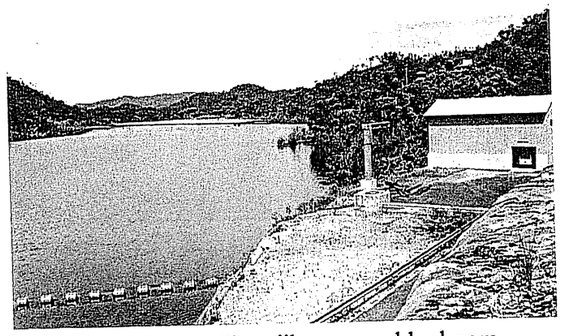
Dam crest road, looking from right bank



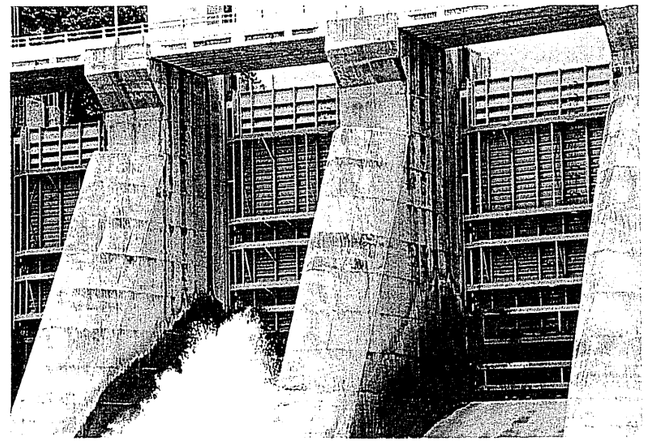
Access road to dam and powerstation,
looking from downstream left bank



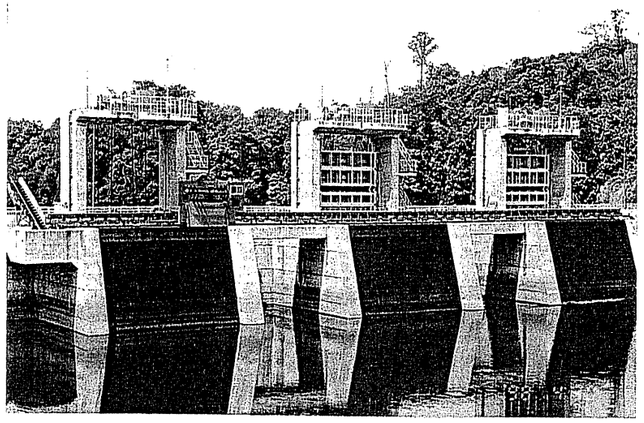
Water supply werehouse and cable duct
from switchyard to powerstation



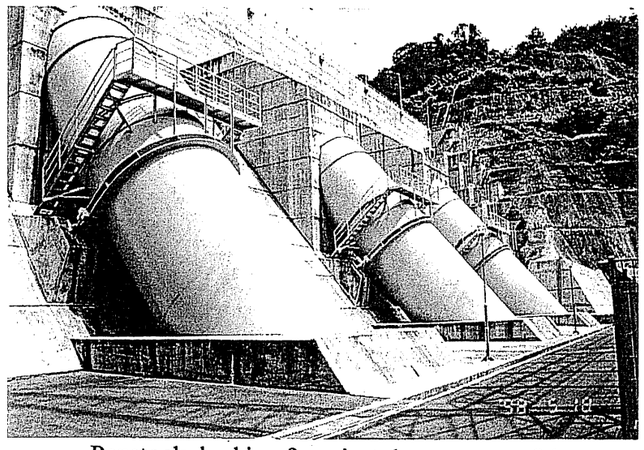
Boat handling, jibcrane and logboom,
looking from left bank



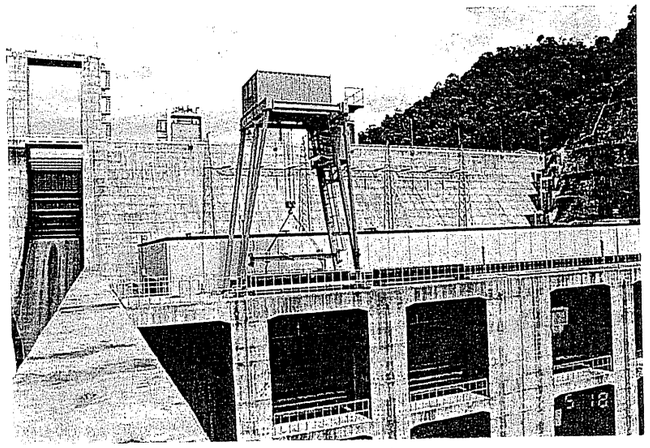
Spillway gate, looking from just downstream of dam



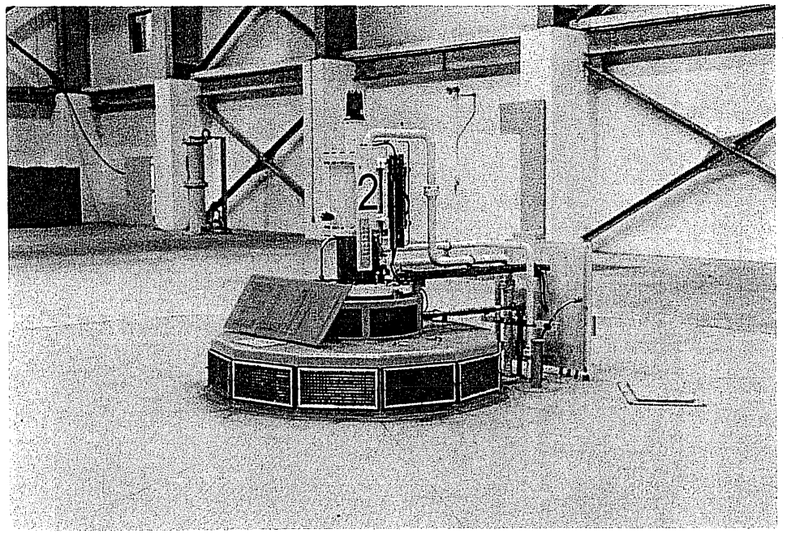
Intake gate, screen trash rake, looking from upstream of dam



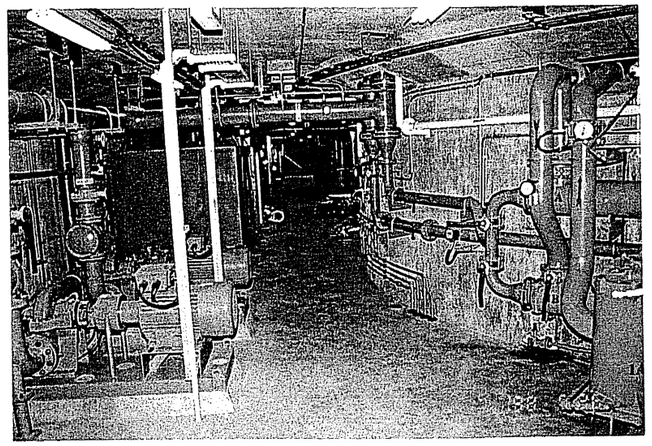
Penstock, looking from just downstream of dam



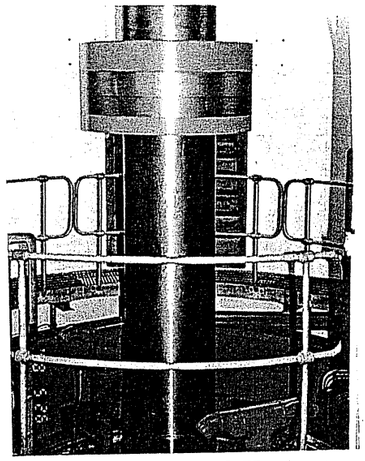
Tailrace stoplogs and gantry crane



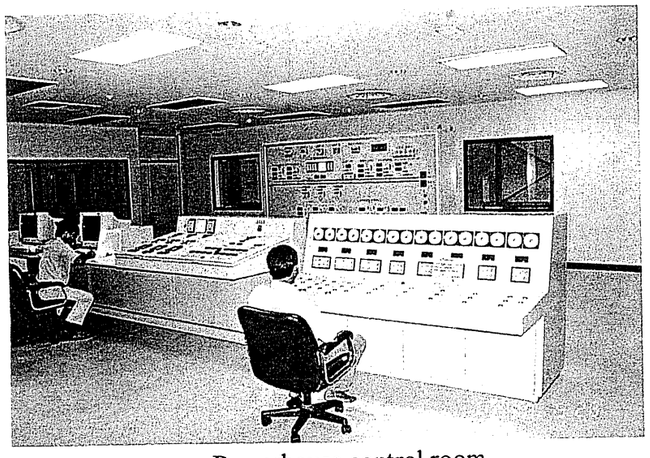
3-phase AC Generator



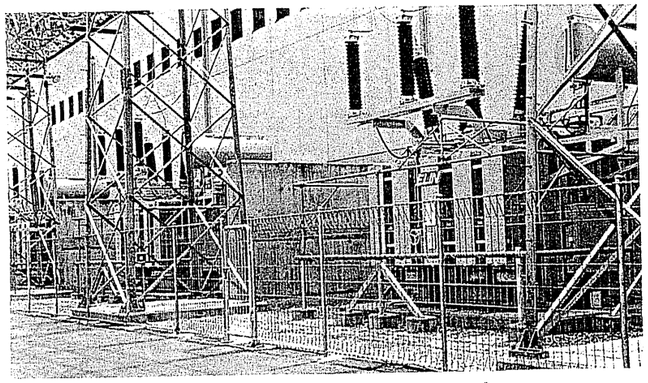
Powerhouse turbine room



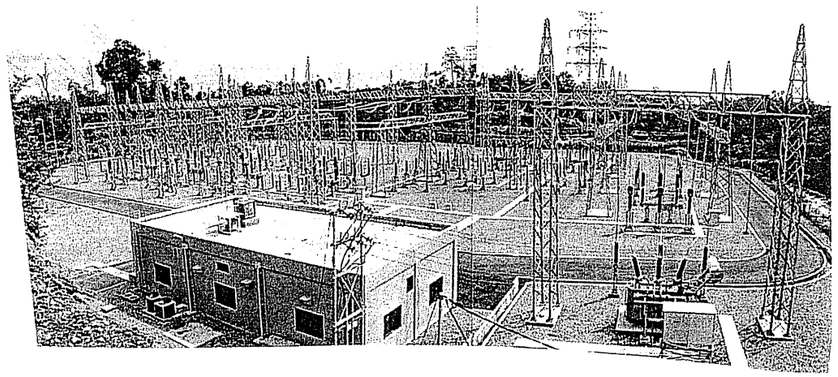
Turbine (Vertical shaft Kaplan type)



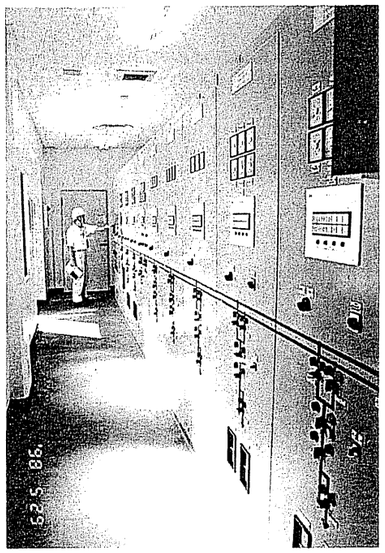
Powerhouse control room

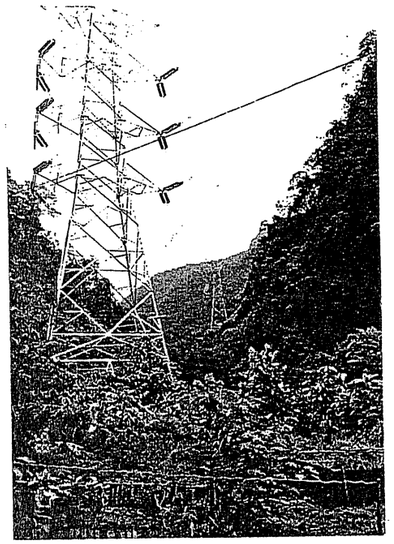


Transformer adjacent to powerhouse

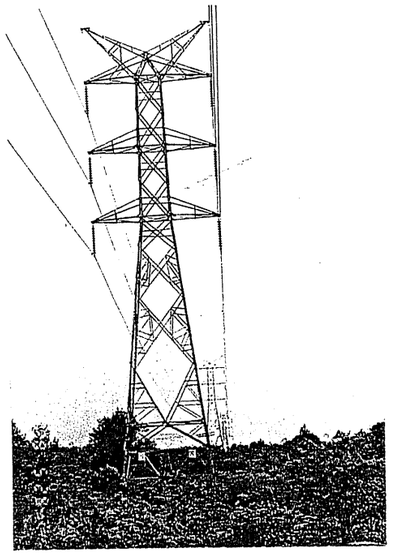


Switchyard and relayhouse

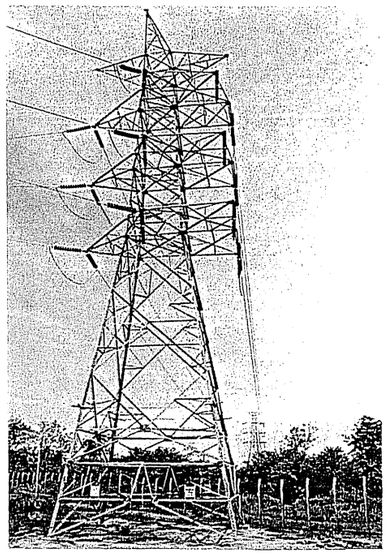




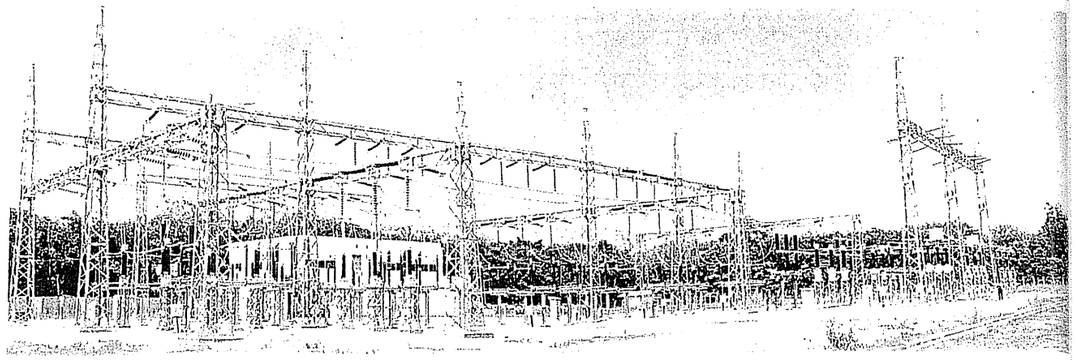
Transmission line (P/S - Payakumbuh S/S)

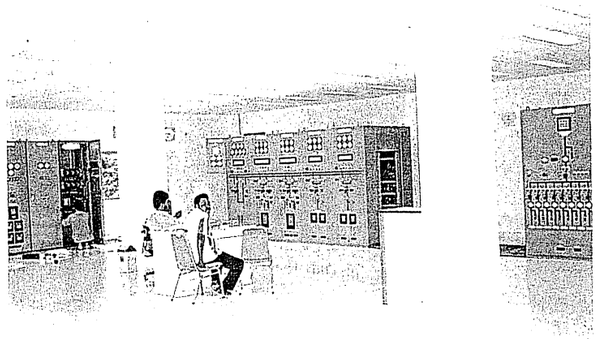


Transmission line (P/S - Pekanbaru S/S)

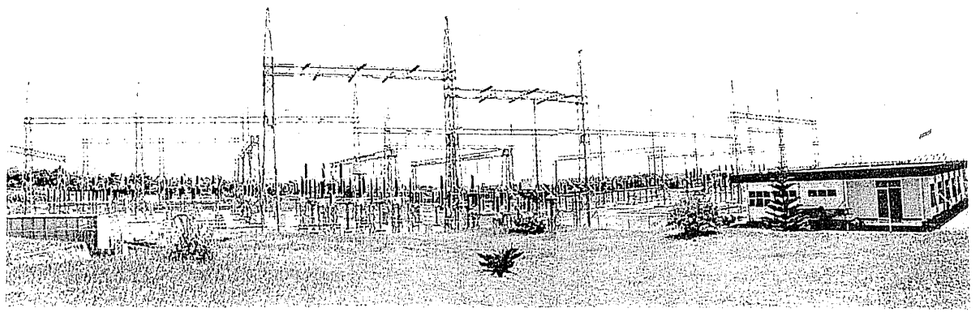


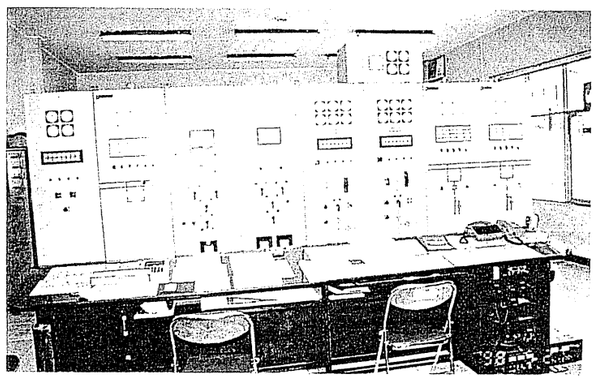
Transmission Tower in Pekanbaru S/S





Pekanbaru substation and relay room

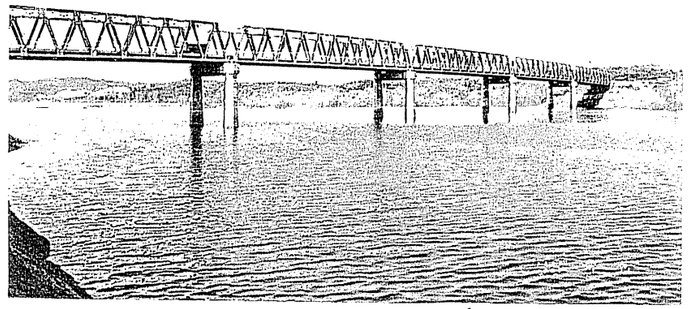




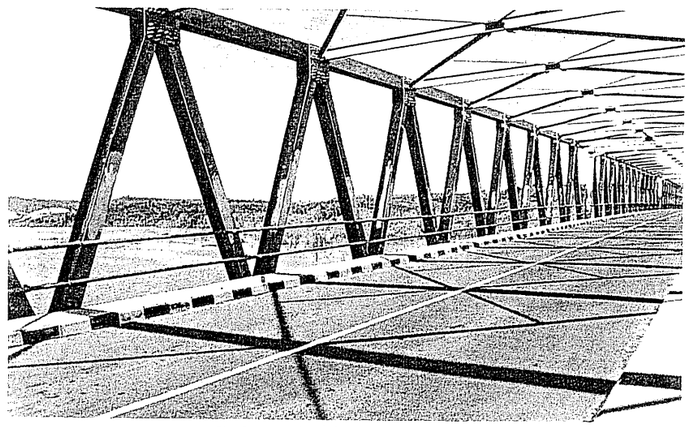
Payakumbuh substation and relay room



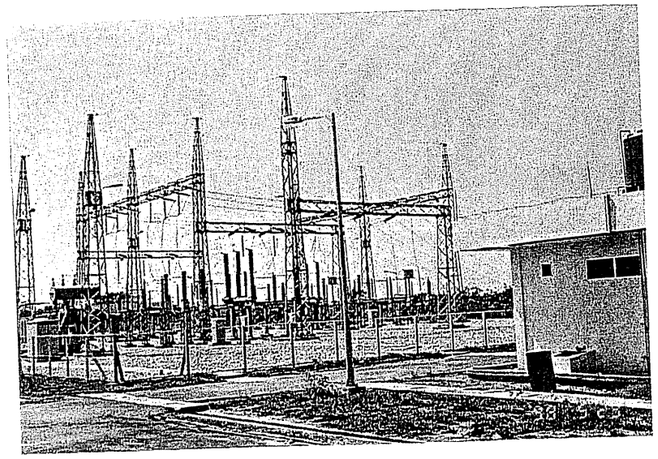
New National Road



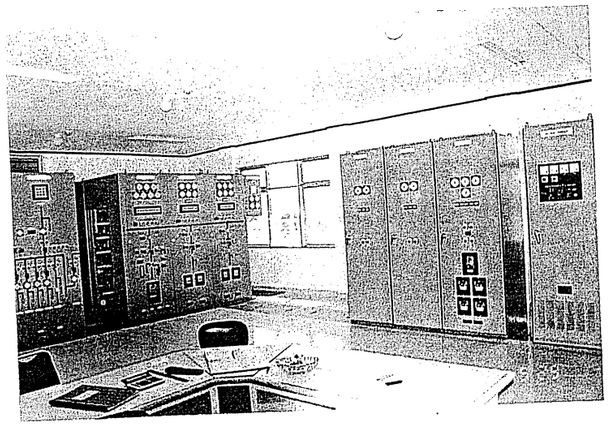
Gulamo bridge (New National Road)

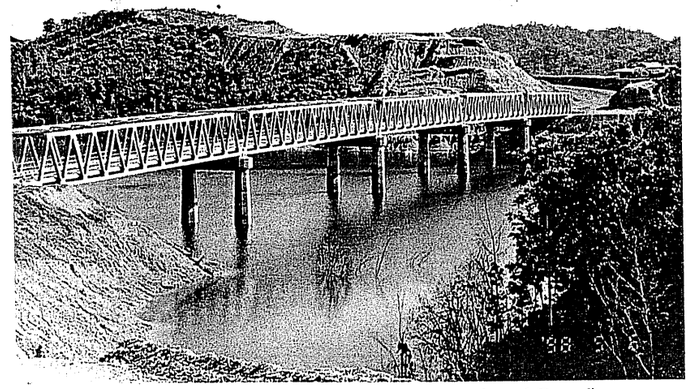


Gulamo bridge upper structure

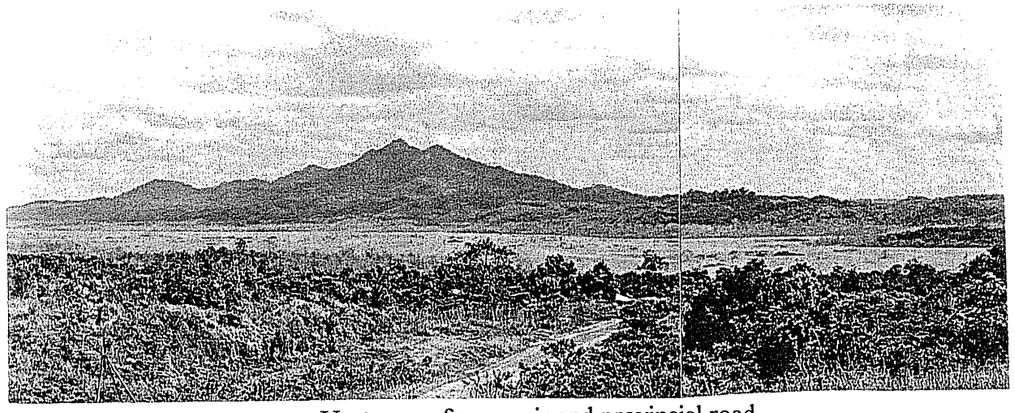


Bangkinang substation and relay room

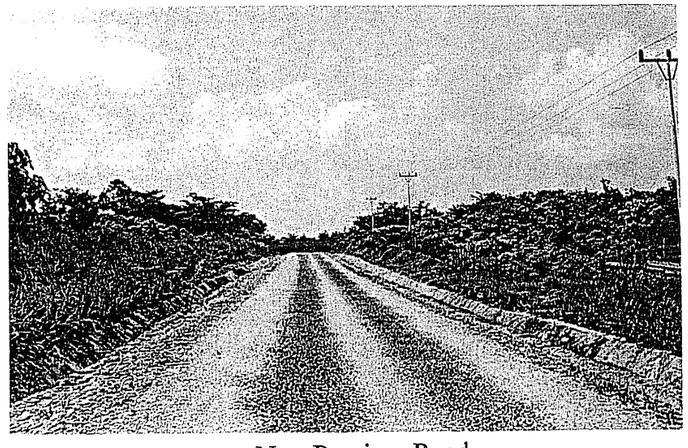




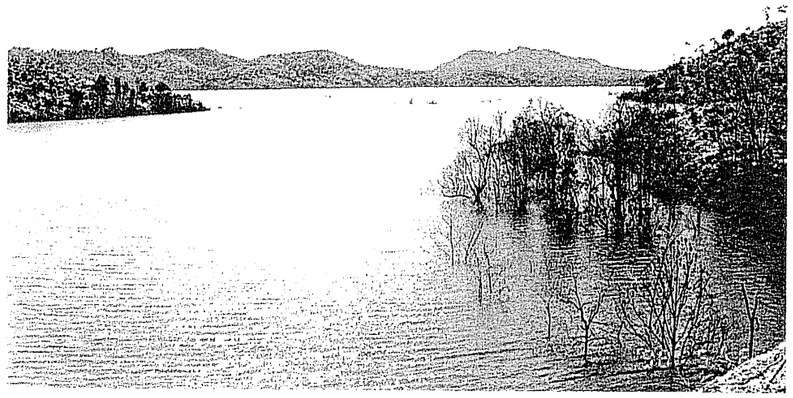
Kampar kanan bridge (New National Road)



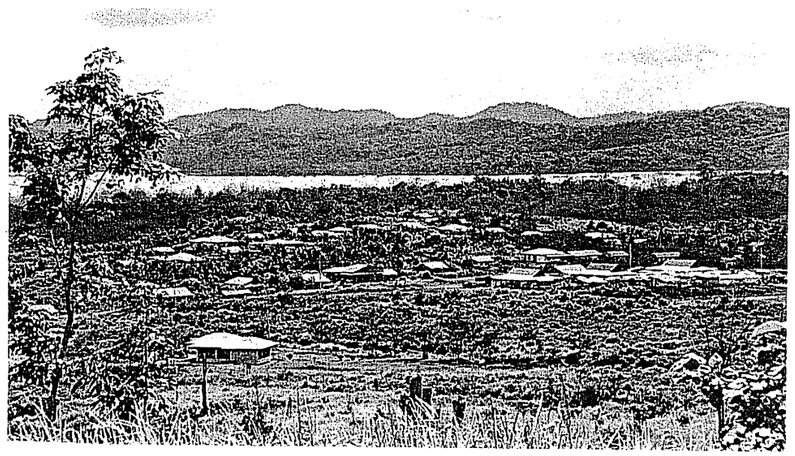
Upstream of reservoir and provincial road



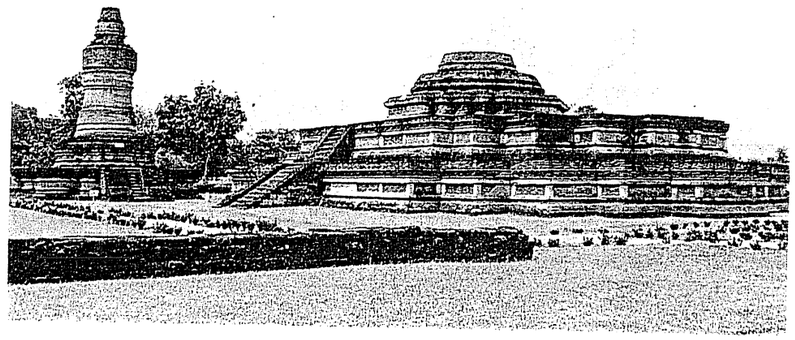
New Province Road



A view of reservoir, looking from National Road



A view of resettlement area



Muara Takus Buddhist remains

PROJECT COMPLETION REPORT
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PHOTO RECORD (A Separate Volume)

PREPAFE

The Kotapanjang Hydroelectric Power Project is one of the largest single project in the history of Indonesian hydropower development, consisting of a concrete gravity dam, a power station, and a transmission line. The National Road and Provincial Road relocation and people resettlement submerged by reservoir impounding were carried out on a very extensive scale. This project completion report is the fruit of many man-months of detailed study and analysis of all aspects of the planning, design, construction, and commissioning of this Project and its features.

This report is consisting of thirteen parts with the following titles. An Summary Report and Photo Record are separately prepared.

- Summary Report (A Separate Volume)
- Main Report
  - Part 1 Project Planning and Design of Civil Works
  - Part 2 Construction of Civil Works
  - Part 3 Architectural Works
  - Part 4 Telemetering and Flood Warning System
  - Part 5 Metal Works (Hydromechanical Works)
  - Part 6 Generating Equipment (Powerstation and Switchyard)
  - Part 7 Transmission Line
  - Part 8 Substation
  - Part 9 Road Relocation
  - Part 10 Project Cost
  - Part 11 Contract Administration
  - Part 12 Training and Transfer of Knowledge
  - Part 13 Environmental Aspects
- Photo Record (A Separate Volume)

Each Part contains several chapters. Part 1, which is general in content, comprises six chapters and treats the entire project with respect to historical background and project features, description of project planning and facilities, geology, hydrology, concrete and materials. The other parts are more specific in content and discuss the design aspects of the principle parts of the project.

The Parts 2 treats the construction of the civil works for the river diversion, dam, powerhouse, tailrace and switchyard including temporary facilities. Part 3 covers the building structure, finishes, mechanical and electrical facilities of powerhouse including control building and relay house control building as well as outdoor planting works and miscellaneous works around the powerhouse. Part 4 treats flood forecasting and warning and telemetering system which covers the design, manufacture, testing and erection at site.

Part 5 discusses the design, fabrication and erection for the gates, stoplogs and penstock. The design, manufacturing and delivery and construction of the generating equipment are discussed in Part 6. Transmission line and substation on equipment and material and erection works including test are respectively discussed in Part 7 and Part 8. Part 9 covers planning, design and construction of relocation of national and provincial roads including two large bridges.

The project cost for each contract lot is discussed in part 1, followed by part 11 describing its contractual issues and problems. Part 12 introduces the transfer of the knowledge and knowhow, and training of Indonesian personnel during the period of the Services I and II which was implemented in the manners of the on-the-job and off-the-job training. Part 13 treats the environmental issue and problems, especially on the resettlement program which had been conducted during the project.

This completion report was prepared in fulfillment of the requirements regarding engineering services as stipulated in Agreement No.060.PJ/070/1991/M concluded between Tokyo Electric Power Services Co., Ltd in association with PT. YODYA KARYA AND PT. TRIMITRA NUSA ENGINEERING on June 3, 1991.

GLOSSARY

METRIC SYSTEM

	Unit	Abbreviation or Symbol
LENGTH :	millimeter	mm
	centimeter	cm
	meter	m
	kilometer	km
AREA :	square millimeter	mm^2
	square centimeter	cm^2
	square meter	m^2
	square kilometer	km^2
	hectare	ha
VOLUME :	cubic centimeter	cm^3
	cubic meter	m^3
	liter	ℓ
	million cubic meters	MCM or preferably $1 \times 10^6 m^3$
TIME :	year	yr
	day	d
	hour	h
	minute	min
	second	s
MASS AND WEIGHT :	milligram	mg
	gram	g
	kilogram	kg
	metric ton	t
	liter per minute	ℓ/m
	liter per second	ℓ/s
POWER :	milliampere	mA
	ampere	A
	kiloampere	kA
	volt	V
	kilovolt	kV
	volt-ampere	VA
	kilovolt-ampere	kVA
	megavolt-ampere	MVA
	watt	W
	kilowatt	kW
	megawatt	MW
	reactive power (Var)	var
	kilovar	kVar
	megavar	MVar
	kilowatt-hour	kWh
	megawatt-hour	MWh
	gigawatt-hour	GWh
	frequency (hertz)	Hz
	kilohertz	kHz
	megahertz	Mhz

	ohm	Ω
	kiloohm	kΩ
	farad	F
	microfarad	μF
	picofarad	pF
	decibel	dB
	revolution per minute	rpm
OTHER MEASURES :	parts per million (mg/lit)	ppm
	scale for acidity	pH
	kilocalorie	Kcal
	Percent	%
	degree centigrade	^0C
	degree	^{0}
	minute	'
	second	"

DESIGN SYMBOLS

Loads and Stresses :	kilogram per square meter	= kg/m^2
	kilogram per square centimeter	= kg/cm^2
	kilogram per cubic meter	= kg/m^3
	ton per square meter	= t/m^2
Moments :	ton meter	= tm
	kilogram meter	= kgm
Acceleration Due to Gravity :	g	9.80 m/s^2 = 980 gal
Flowing Water :	liter per second	= ℓ/s
	cubic meter per second	= m^3/s = 1,000 ℓ/s
	meter per second	= m/s = 100 cm/s
Angle :	degree	= ^{0}
	minute	= '
	second	= "
Ration :	percent	= %
Elevations :	elevation above sea level	= EL.
	high water level	= HWL
	low water level	= LWL
	tail water level	= TWL
Currency :	U.S. Dollar	= US$
	U.S. cent	= c
	Japanese Yen	= ¥
	Indonesian Rupiah	= Rp.

ABBREVIATIONS OF VOCABULARY

Organizations

GOI	Government of the Republic of Indonesia
BAPPENAS	National Planning and Development Board
BAPPEDA	Provincial Planning and Development Board
Bupati	Chief of Kabupaten (Regency)
Camat	Chief of Kecamatan (District)
PU	Ministry of Public Works
P&K	Ministry of Education and Culture
PLN	PT. Perusahaan Umum Listrik Negara (State Electricity Cooperation)
LMK	Electric Power Research Center, PLN
Wilayah III	Business area of PLN (Region III) in the Province of West Sumatra, Riau, Jambi the district of Kerinci)
Sumatra Barat	West Sumatra
DPMA	Water Resource Research Institute
GS I	Geological Survey of Indonesia
BMG	Bureau of Meteorology and Geophysics
P3SA	Water Resource Development Project
JICA	Japan International Cooperation Agency
OECF (or the Fund)	The Overseas Economic Cooperation Found of Japan
TEPSCO (or the Engineer)	Tokyo Electric Power Services Co., Ltd in association with PT. Yodya Karya & PT. Trimitra Nusa Engineering
JSCE	Japan Society of Civil Engineers
ASCE	American Society of Civil Engineers
JIS	Japanese Industrial Standard
JASS	Japanese Architectural Standard Specifications
AIJ	Architectural Institute of Japan
JEMAS	Japan Erection Manufactures Association Standard
HASS	Heating, Air Conditioning and Sanitary Standard of Japan
ASTM	American Society for Testing and Materials
JANCOLD	Japan National Committee on Large Dams

JCI	Japanese Concrete Institute
JEC	Standard of the Japanese Electrotechnical Committee
ICE	International Electrotechnical Commission

Agreements and Contracts

Agreement	Agreement concluded between PLN and TEPSCO on June 3, 1991.
Service I	Engineering Services for Phase I for the construction supervision of the Civil Works and the design of the Transmission Line between Payakumbuh Substation and Kotapanjang Switchyard for the PROJECT.
Service II	Engineering services for Phase II for the construction supervision of the Metal, Generating Equipment, Relocation Road Works, and the Transmission Line between Payakumbuh Substation and Pekanbaru Substation through Kotapanjang Switchyard for the PROJECT.
The Project	Kotapanjang Hydroelectric Power Project and Associated Transmission Line Project.

Other Abbreviations Used in the Report

REPELITA	Five Years Development Plan
GRDP	Gross Regional Domestic Product
FOB	Free on Board
CIF	Cost, Insurance and Freight
F/S	Feasibility Study
D/D	Detailed Design
O & M	Operation and Maintenance
G/S	Gauging Station
P/S	Power station
S/S	Substation
S/Y	Switchyard
KTP	Kotapanjang
PKU	Pekanbaru
PYK	Payakumbuh
BKN	Bangkinang

PART 1

PROJECT PLANNING AND DESIGN OF CIVIL WORKS

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Chapter 1 INTRODUCTION

1.1 Background of the Project

Located in the central part of Sumatra, Riau Province is the largest oil producing province in Indonesia where about 150 million barrels of crude oil, equivalent to about 50% of total crude oil production throughout the country, is produced a year. Riau Province, which comprises a vast area of 94,562 km^2 where the big four rivers, namely, the Rokan River, Siak River, Kampar River and Indragiri River flow, has a great potential for the development of agriculture.

Up to the time of the Feasibility Study (1982-1984), in spite of such abundant natural resources, the economy of Riau Province had been relatively stagnant when compared with that of other provinces due to a delay in investment in essential infrastructure.

However, in order to improve the living standard of the people and promote balanced regional development, the Government of Indonesia has so far implemented three 5-year development plans. In Riau Province, the effect of these 5-year programs has gradually been taking shape, and the regional development including replenishment of social capital and industrialization is expected to materialize rapidly in the near future along with the progress of the transmigration program, etc.

The power generating facilities of PLN in Riau Province was small (120,000 kW in 1995), and the electrification rate was as low as 30% because of insufficient power distribution facilities. Electric power is supplied by PLN mainly to the urban areas and surrounding areas, and power demand in other areas is covered independently by private diesel power plants.

The Feasibility Study found the annual average growth rate of power demand of PLN in Riau Province excluding Kepulauan Riau was 10% during the ten year period from 1985 through 1995, and power demand was expected to grow at a rate as high as 30% a year from 1996 through 2006, and at an average rate of 20% a year from 1996 through 2016.

In order to meet such rapidly increasing power demands, PLN perceived the utilization of abundant hydro power sources identified on the Kampar Kanan River, Rokan River and Indragiri River. Development of these hydro-power sources will make it possible to greatly reduce domestic consumption of a large amount of oil, secure foreign currency reserve and ultimately contribute extensively to the national economy of Indonesia in the future.

The Kotapanjang Hydro-Power Project, located along the middle reaches of the Kampar Kanan River, has been selected as the first hydro-power development project in Riau Province in consideration of the above factors. This project is reasonably economical in that a maximum output of 114 MW can be obtained by constructing a medium scale dam with a height of 58 m and a total reservoir capacity of 1,545 million m^3, and the site is near and accessible from Pekanbaru, the capital of the province.

In consideration of the necessity and urgency of this project, the Government of Indonesia requested the Government of Japan in June 1981 to carry out a feasibility study for this project. The feasibility study started in January 1982 and was completed in March 1984. After that, the final design and preparation of the tender and contract documents commenced in February 1987 and were completed in August 1988. After due evaluation of the tenders and the necessary processes followed by the Indonesian Government, the construction of the Kotapanjang Project started in June 1991.

1.2 Development of the Project

In September and November 1979, a project finding was conducted regarding the entire basin of the Kampar River. At that time, a development plan of the Mahat River, a tributary of the Kampar River was considered by PLN. Following reconnaissance study, the Kotapanjang site was also considered. Then, a two-stage development plan for a staged development of the Kotapanjang site and Mahat site was worked out, and summarized in the Reconnaissance Report in March 1980.

In August 1980, a further detailed study was carried out with respect to the area along the Kampar River. Regarding the Kotapanjang site, a single stage large scale development plan was worked out and proposed for subsequent comparative study with the two stage development plan. This single stage development plan is roughly similar to the plan adopted in the feasibility study. The results of the study were summarized in the Pre-feasibility Report in October 1980.

It was found that approximately 110 MW could be generated at Kotapanjang and that the energy produced annually could well exceed 540 GWh. The report recommended that further investigations as required to implement the project be started as soon as possible.

In June 1981, the Government of Indonesia requested the Government of Japan to carry out the feasibility study for this project. The objectives of this study were to work out an optimum development plan for the Kotapanjang Hydro-Power Development Project being planned in the area along the Kampar Kanan River, by executing feasibility grade study including field investigations in order to prove the technical, economic and financial feasibility of the project.

The feasibility study was completed in March 1984, when the report on these studies was submitted to PLN. The site investigations included topography, geology, construction materials, adequacy of hydrologic data, sedimentation, sociologic and environmental aspects of the project, costs of land acquisitions and site access, and resettlement compensation. The feasibility design comprised design and drawings of all major features of the project, project arrangement and its justification, cost estimate including cash flow, construction schedule and output studies. In addition, further economic and financial studies were also conducted.

The detailed design and preparation of the tender and contract documents commenced in February 1987. The foreign exchange portion for this work was financed by OECF of Japan. During this stage, further detailed field work consisting of surveying, borings, test adits and pits, in-situ and laboratory testing, and hydraulic model tests were undertaken as required.

The tender documents for the Civil and Penstock Metal Works were completed in August 1988. The international tenders were invited and opened on November 1991. The OECF accepted to provide the necessary finances for part of the cost for the construction of the Civil and Metal Works. After the completion of the design stage, OECF also accepted to provide finance for supply of the major hydroelectric equipment, turbine and generators.

After due evaluation of the tenders and the necessary process followed by the Indonesian Government, the construction of the Kotapanjang Project started in June 1992 with the award of the contract to the Lot I Works Contractor. The work covered by this contract included the construction of the diversion tunnel, the dam, the spillway and power station. TEPSCO was engaged to perform construction supervision required for the project with the contract signing in June 1991, between PLN and TEPSCO.

The major events and dates during the construction of the project are described as below:

(i)	River diversion	October 1, 1993
(ii)	Installation of the gate and penstock completed	November 15, 1996
(iii)	Completion of the dam	January 15, 1997
(iv)	Opening of the national road relocation	February 28, 1997
(v)	Impounding of the dam commenced	March 12, 1997
(vi)	Installation of the transmission line completed	June 30, 1997
(vii)	Commissioning test	July 1, 1997
(viii)	Completion of the main civil works	November 15, 1997
(ix)	Commencement of commercial operation	
	unit 1	November 21, 1998
	unit 2	April 20, 1998
	unit 3	February 28, 1998

1.3 Description of the Project Features

(1) Site Conditions

The dam site is located about 85 km from Pekanbaru, the provincial capital and about 20 km from a small town of Bangkinang. The site is easily accessible from the National Road running along the Kampar River.

The topography around the dam site consists of a narrow valley and a vast quasi-table exists about 10 km upstream of the dam site. Thus, the dam site is in an area having excellent topography whereby a medium size dam can create a large reservoir. However, the river gradient around the dam site is a gentle 1:1,090, so a head can only be obtained from the dam.

The dam site mainly consists of dacitic turf. The left bank is a gentle slope of about 25^0. Class D rock is deeply distributed from the intermediate stage to the upper stage, and the occupancy depth of Class CM rock becomes shallower from 24 to 4m toward the river bed. The appearance depth of CL class rock is 14m at EL. 80m.

The right bank has a steep slope of 40^0, and its upper part comprises a precipice with height of about 50m where foundation bedrock is exposed. The appearance depth of Class CM rock is 4 to 9m, and that of Class CL is within 2m. Generally, the excavation line was rather deep on the left bank. The left and right banks were evaluated to be sufficient as foundation bedrock. The foundation on the left bank required some treatment.

The annual rainfall within the catchment area of the dam site is estimated to be approximately 2700 mm, making the area one of the most intensive in Indonesia. The yearly average runoff at the dam site over the past fifteen years was 184.4 m^3/sec, with an annual total inflow of 5815 million m^3. When these values are converted into monthly values, the maximum runoff is 225.59 m^3/sec in December, and the minimum runoff is 111.03 m^3/sec in August. The yearly average specific yield per 100 km^2 is 5.20 m^3/sec. Therefore, it is clear that river discharge is considerably high. The design flood discharge is estimated to be 8000 m^3/sec based on a 200-year probability.

(2) Power Generation and Transmission Line Plan

Regarding the development system of this site, a single stage development system and two stage development system were considered during the initial stage. The single stage development system will make it possible to realize a large scale reservoir, though a large amount of compensation will be required. In the case of the two stage development system, the two sites have to be of the regulating pondage type and the scale of development has to be made smaller than the single stage system. As a result of a comparative study regarding these two systems, the single stage development system proved to be the one most optimal.

With respect to the type of a dam to be built, a comparative study of a concrete gravity dam and a fill dam was carried out. In the case of a fill dam, countermeasures against flooding and flood discharge facilities are required due to the scale of flooding at the site. Therefore, a fill dam is more costly than a concrete dam. Topographically, the river width is small, further negating any advantage of a fill dam type. On the other hand, selection of a concrete gravity dam is much more advantageous in view of topography, and construction materials. As a result, the concrete gravity dam was adopted.

As the water level rises, the capacity of the reservoir tends to increase rapidly, with the net benefit (Benefit-Cost) also increasing. However, due to the Muara Takus Buddhist remains and Pangkalan Kotabaru (population : 8572) at the terminal of the reservoir, the high water level of the reservoir is limited to EL. 85m in order to avoid flooding of the remains and submergence of the town. As it is considered possible to obtain sufficient reservoir capacity even with water at this level, the high water level has been determined to be EL. 85m.

Regarding the scale of the power plant, comparative studies were carried out by assuming several different turbine discharges. As the scale becomes large, both Benefit/Cost (B/C) and Benefit-Cost (B-C) tend to increase. Even if the scale of development is made larger, the major part of construction cost, which consists of the dam and compensation costs, will remain unchanged, while the construction cost of the facilities from the intake through to the tailrace will only increase to a certain extent. Therefore, the kW value will increase.

Still, if the scale is made too large, it will be impossible to realize effective operation of the power plant. In consideration of the future power demand and supply plan in Riau Province, the optimum scale of development was determined to be 114 MW when the equivalent continuous time of peak load is approximately 12 hours.

The main transformer adjacent to the power station steps up the voltage of the generated power. A 150 kV transmission line conveyers the power to a switchyard, which is located an adjacent at EL 174 m, 500m north of the power station. The transmission line of 150 kV with 2 circuits has been constructed for a length of 64 km from the switchyard to a substation in Pekanbaru and for a length of 85 km from the switchyard to a substation in Payakumbuh.

(3) Submerged Property and Environmental Study

The area submerged due to construction of the reservoir was estimated to include 10 villages (4886 families), 12450 ha of land, approximately 45 km of national road and 22 km of provincial roads, etc.

Appropriate compensation was provided for submerged households. Also, compensatory consideration was made for sites of relocated households and resettlement area were chosen as close as possible to their former residences within transmigration and plantation projects. Nine prospective transmigration sites were selected after discussion with Provincial Planning and Development Board (BAPPEDA).

The relocated roads were selected along the routes as close as possible to the existing roads after a comparative study taking into account regional development, economic and other factors for the future. Study of the relocated roads was also done by Ministry of Public Works (PU), the routes for the relocated roads was finally decided by the Government of Indonesia.

There are Buddhist religious temple remains near Muara Takus at the terminal of the reservoir. These remains, which date back to the 11th and 12th centuries are historically important cultural assets. Therefore, the high water level of the reservoir was determined to be 85m, which is lower than the lowest elevation of the remains (86.25m). In this way, the remains can be preserved and be safe from submergence.

Also it is predicted that the highest water level of the reservoir for the flood of 200 year return period would be EL. 84.70 m by spillway gate operation at Kotapanjang dam. In consideration of back water arising in the flood time, the water level is predicted to rise to EL. 86.10m which is 0.15 m lower than the elevation of the Temple site.

(4) Design Outline of Main Features

(a) General

The Kotapanjang Hydroelectric Power Project comprises of a dam on the middle reaches of the Kampar Kanan River, a system of the Kampar Kanan River, with capacity to store 1,545 million water and to generate 114 MW of power by a head created by the dam.

The dam site is located approximately 10 km downstream of the confluence of the Kampar Kanan and the Mahat Rivers. The dam is 58 m high, with a crest length of 257m and a volume of approximately 301300 m^3 to suit the topography and geology.

The reservoir created by the dam has an active storage capacity of 1,040 million m^3 which regulates the annual inflow at the site. A maximum of 348 m^3/sec of water is conveyed through three penstocks of 5.0 meters diameters each about 87 m long, to a powerhouse located on the left bank immediately downstream of the dam. The Kotapanjang Power Plant generates a maximum of 114 MW of electricity at a rated head of 38.1 m and produces 583 x 10^6 kWh of generated energy annually.

(b) Reservoir

On the basis of the optimum plan, the dimensions of the reservoir were established. The reservoir capacity is 1545 million m^3 and the surface area being 124 km^2 at HWL 85.0 of the reservoir. The design discharge of the spillway was determined to allow the 200-year flood discharge of 8000 m^3/sec at HWL 85.0m.

Although this project was planned solely for power generation, it will provide the benefit to mitigate flood inflow into an average outflow by this reservoir. Further more, it will also be possible to cut down the peak of a flood by means of preliminary discharge at the time of the flood. An amount of soil sedimentation will occur in the reservoir, the 100-years soil sedimentation of 167 x 10^6 m^3 was determined, and the design soil sedimentation level was estimated to be EL. 64.0m.

Gross storage volume	: 1545 x 10^6 m^3
Active storage volume	: 1040 x 10^6 m^3
High water level (HWL)	: EL.85.0 m
Low water level (LWL)	: EL.73.5 m
Effective depth of reservoir	: 11.5 m
Surface area for HWL 85.0	: 124 km^2
Project catchment area	: 3337 km^2
Annual average inflow at dam site	: 184 m^3/sec

(c) Dam

The dam selected on the basis of the optimum plan was a concrete gravity dam, which was constructed along the dam axis-4 on the extreme downstream side. The axis and type of the dam were selected based upon the results of economic comparative study carried out by taking into account the topography, geology, materials, flood and other various conditions. The dam was constructed on the river bed at EL 29.5m. With the crest of the non-overflow level at EL. 87.5 m and a height of 58.0 m from the river bed, the freeboard of the dam from HWL 85.0 m will be 2.5 m.

The volume of concrete for the dam including the spillway was 340400m^3, with a base width extending to 55.1m. The gradient of the upstream surface and that of the downstream surface is 1:0.15 and 1:0.8, respectively. The traverse joints of the dam were provided at an spacing of 15m making 16 blocks in total.

In consideration that the maximum base width is 55.1m, no vertical joint is provided. One gallery is provided inside the lower part of the dam body near the dam axis, so the required dam monitoring measurements can be carried out.

The foundation rock of the dam was excavated down to the bedrock class CM on the river bed section and right bank, and down to the rock class CL on the upper part of the left bank. Grouting was conducted particularly carefully on the upper part of the left bank. Consolidation grouting was carried out with respect to the entire foundation of the dam, and curtain grouting was executed in one line.

Dam type		: Concrete gravity
Dam height		: 58.0 m
Freeboard		: 2.5 m
Dam crest length		: 257.5 m
Dam crest width		: 5.0 m
Non overflow dam crest		: EL.87.5 m
Overflow dam crest		: EL.67.5 m
Dam volume		: 301300 m^3
Dam base width		: 55.1 m
Dam surface slope	Upstream	: 1:0.15
	Downstream	: 1:0.80

(d) Spillway

The spillway is of a construction letting over-flow water from the central part of the dam flow down over the downstream surface of the dam, and the energy dissipator is a horizontal apron system dissipator of a hydraulic jump type provided with chute block and dentated sill. The final design of the spillway was determined by the results of a hydraulic model test carried out in the detailed design stage.

Five (5) sets of roller gates with a height of 18m and a width of 11m were provided on the overflow crest of the dam so as to permit a 200-years flood discharge of 8000 m^3/sec at HWL 85.0m. Five piers and guide walls were installed on both sides of the right and left banks, with the crest elevation of the guide walls being 61.5m. The discharge of the energy dissipator was designed to be 7200 m^3/sec. In order to dissipate the energy of a flood a chute block was provided on the starting point of the apron, and a dentated sill on the terminal of the apron.

Spillway type	: Overflow, chute and dentated sill type.
Design flood	: 8000 m^3/sec (200 - year probable flood)
Overflow depth	: 17.5 m
Overflow length	: 71.0 m
Type of gate	: Steel construction, fixed wheel gate
Size and sets	: 18.0m (H) x 11.0m (W) x 5 sets
Materials	: SS400B, SS400, SM490YA etc

(e) Intakes

Three (3) sets of intakes were installed on the upstream surface of the left bank of the dam body. These intakes are positioned at the center of the respective dam blocks. The amount of water intake per one intakes is 116 m^3/sec.

Since LWL of the reservoir is EL.73.5 m and the soil sedimentation level is EL. 64.0m, and the water depth between the LWL and the sedimentation level is only 9.5m, a guide wall was provided on the bottom of each intake in order to ensure smooth intake of flowing water. Three (3) sets of screens with a height of 20m and a width of 13m were installed in front of the intake.

An intake trash rake of traveling gantry type was installed at the dam crest level at EL.87.5m for removal of debris from the front face of the intake fixed trash rack. The trash rake 3.0m width for rake operation on clear width of 10m for the trash rack.

Intake type	: Pressure type
Elevation of intake bed	: EL 61.0 m
Type of gate	: Steel construction, fixed wheel gate
Size and sets	: 6m (H) x 6m (W) x 3 sets
Trashrack	: 23.5m (H) x 10.0m (W) x 3 sets
Materials	: SM400B, SS400, SM490RYA etc.

(f) Diversion Work

Since the river around the dam site was narrow, it was difficult to perform sluice type diversion or open channel type diversion. Therefore, two diversion tunnels were constructed. Since the river downstream of the dam was crooked toward the left side and easily accessible from the national road, the two tunnels were provided toward the left bank side. The design discharge of the tunnels is 1300 m^3/sec, which is equivalent to the 3.0-year flood discharge. The tunnel is of a horseshoe-shaped section with an inside diameter of 10.0 m and a total length of 478.8m for No. 1 tunnel and 423.0m for No. 2 tunnel.

The cofferdam sites were selected so as to permit smooth execution of the works for construction of the dam, powerhouse, and tailrace. The major dimensions of the cofferdams and the diversion tunnels are as listed as follows :

Primary	Upstream	: Rockfill 12.5 m (H) x 84.8 m (L)
	Downstream	: Rockfill 11.2 m (H) x 106.8 m (L)
Secondary	Upstream	: RCC 19.5 m (H) x 132.0 m (L)
	Downstream	: Rockfill 10.5 m (H) x 100.3 m (L)

Diversion tunnel	Unit	: 2 units
	Length	: No.1 = 478.8 m No.2 = 423.0 m
	Diameter	: 10.0 m
	Capacity	: 1300 m^3/sec for 2 units

(g) Penstocks

Three (3) sets of penstocks each with a discharge capacity of 116 m^3/sec were installed and connected from the intake to the water turbines house in the powerhouse. The penstock line starts at an intake portion of the upstream dam surface and terminates at the entrance of the turbine. The upstream portion of the penstocks was buried horizontally inside the dam body, and the downstream portion, being supported by a concrete anchor block along the downstream surface of the dam.

Penstock type	: Buried and opened type
Length	: 86.9 m
Units	: 3 lines
Diameter	: 5.00 m
Thickness	: 19 mm - 14.5 mm
Material	: SM400B, SS400, SM490YA etc.

(h) Powerhouse

The powerhouse was built at the downstream end of the dam on the left bank, with the main building being 80.3m in length, 35.6m in width and 43.2m in height. The foundation of the powerhouse was excavated down to level of EL.21.0m. Powerhouse structure comprises reinforce of concrete for basement and control building. Precast concrete paneled steel framed erection bay.

Geological conditions of the foundation were made clear by the drilling and seismic prospecting along the dam axis-4 which was adjacent to the power house. The results of the prospecting along the powerstation showed that the foundation rock was fresh, because it's seismic wave velocity was 3.2 km/sec.

Three (3) units of vertical shaft kaplan turbines (39400 kW x 3 units) were installed. The turbine discharge and effective head per turbine is 116 m^3/sec and 38.1m, respectively. Three (3) sets of 45,000 kVA generators were also installed. Overhead traveling cranes were installed in the main building.

Three (3) sets of 45,000 kVA main transformers were arranged on the upstream side adjacent the main control building. On the upstream western side of the main building, a three-storied control building of 80m in length, 11m in width and 14m in height was built. In this control building, central control room, computer and relay room, air conditioning machine room, communication room, auxiliary equipment room, office room, conference room, etc. were provided.

Powerhouse type	: Ground type
Powerhouse size	: 43.2m (H) x 35.6m (W) x 80.34m (L)
Type of turbine	: Vertical shaft kaplan type
Installed capacity	: 114 MW (38 MW x 3 units)
Rated discharge	: 116 m^3/units

Effective head	:	38.1 m
Type of generator	:	3-phase AC generator
Capacity, voltage and frequency	:	45000 kVA x 3 units, 11 kV, 50 Hz
Type of main transformer	:	3-phase oil immersed oil material air forced
Capacity and voltage	:	45000 kVA x 3 units, 11/150 kV

(i) Tailrace

One (1) trapezoid type tailrace was provided to collect water from the turbine drafts into one open channel and discharge it into the river at a rate of 348 m^3/sec (maximum). Two sets of tailrace gate comprising two blocks each and four sets of stoplog comprising five log each, 6.5m wide by 4.6m high, vertical lift slide type, were respectively installed and provided at the tailrace. The gates and the stoplogs are used to dewater the draft tube for inspection and maintenance purposes. A gantry crane with a lifting beam for raising, lowering and moving the stoplog, were provided on the tailrace.

Tailrace type	:	Open channel with trapezium section
Length	:	80.0 m
Width of inverts	:	50.5m - 65.2m
Type of gate	:	Steel slide gate
Size and sets	:	4.55m (H) x 6.0m (W) x 2 sets
Type of stoplogs	:	Steel slide stoplog
Size and sets	:	0.92m (H) x 6.0m (W) x 20 sets

(j) Switchyard

The switchyard site is located on the mountain ridge about 500m north of the powerhouse. A land space of about 200m x 100m was developed and the area of 146m x 109m was graded to accommodate main switchgear equipment and relay house. In this case, the space required for interconnection of the power plant to the power system in West Sumatra was be graded in advance. The elevation of the switchyard site is EL.174m, 86.5m above top of the dam crest EL.87.5. The access road to the switchyard connects with the national road. The major equipment to be installed in and around the switchyard is as listed below.

Switchyard area	:	146 m x 109 m
Relay house	:	10.5 m x 10 m
Local transformer	:	150 kV/20 kV, 3-phase
Circuit breaker	:	150 kV, 3-phase
Disconnecting switch	:	150 kV, 3-phase
Potential transformer	:	150 kV, 3-phase
Arrester	:	150 kV, 3-phase

(k) Transmission Line

Transmission lines was constructed from Kotapanjang (KTP) Switchyard to Pekanbaru (PKU) Substation and Kotapanjang Switchyard to Payakumbuh (PYK) Substation. Both lines are double circuit construction. But, one circuit of Kotapanjang to Pekanbaru was made via Bangkinang Substation. The transmission line was constructed along the national road as close as possible because of the good access to the tower site.

Transmission line between Kotapanjang Switchyard and Pekanbaru Substation was 69.3 km in original plan. However, Pekanbaru Substation was relocated and the line length was reduced to 64.4 km.

Nominal voltage	:	150 kV, double Circute
Line length	:	64.4 km (KTP-PKU), 84.7 km (KTP-PYK)
Conductor (ACSR)	:	435 mm^2 (KTP-PKU), 330 mm^2(KTP-PYK)
Ground wire	:	GSW 55 mm^2
Number of suspension tower	:	167 (KTP-PKU), 131 (KTP-PYK)
Number of tension tower	:	33 (KTP-PKU), 117 (KTP-PYK)
Insulator	:	254 mm
Unit/String	:	11

(l) Substation

A new trunk substation for distributing the generated output at the Kotapanjang Power station to the major power consumption area in Riau Province was constructed at a site in the outskirts of Pekanbaru. Although the space required for the substation under this project is 250 x 200m, an additional space was prepared for construction of a bay for an other incoming transmission line scheduled to be completed simultaneously with this project or in near the future.

A new substation near the town of Bangkinang located about 15 km from Kotapanjang Powerstation was constructed and an existing substation (Payakumbuh S/S) in West Sumatra was extended. The 150 kV power system in West Sumatra and Payakumbuh was connected by the completion of Kotapanjang Powerstation.

Pekanbaru S/S		
150 kV transmission line	:	2 bays
150 kV/20 kV power transformer	:	2 bays with 50 MVA x 2 units
150 kV bus couple	:	1 bay
20 kV express feeder	:	2 feeders
20 kV outgoing feeder	:	14 feeders
20 kV capacitive compensator	:	2 feeders (future)

Bangkinang S/S		
150 kV transmission line	:	2 bays
150 kV/20 kV power transformer	:	1 bay with 10MVA x 1 unit
20 kV outgoing feeder	:	4 feeders

Payakumbuh S/S (Extension work)
150 kV transmission line : 2 bays

(m) Relocation of Road

Approximately 45 km of existing National Road were submerged by Kotapanjang dam reservoir water between Pulau Gadang village in Riau Province and Tanjung Balit village in West Sumatera Province. The Relocation of National Road was necessary for the diversion of the existing National Road to new location above proposed high water level of reservoir, with more suitable geometric standard to meet the future traffic demand.

The Relocation of National Road which includes construction of two major bridges is a part of the Kotapanjang Hydroelectric Power Project. The National Road joins the capital of Riau Province Pekanbaru with Bukittinggi and capital of West Sumatera Padang.

The provincial road lies just southward of the Kotapanjang Dam reservoir and goes just westward from a junction point of the National Road. And it crosses the Kampar Kanan River at Muara Takus by ferry. After crossing the Kampar Kanan River, it goes up to Tanjung through Gunung Bungsu resettlement area.

National Road
Total length : 42.13 km (Asphalt pavement)
Road width : 6.0 m
Maximum grade : 10%
Major truss bridges : 293.1 m (Kampar Kanan)
288.1 m (Gulamo)

Provincial Road
Total length : 22.0 km (Gravel pavement)
Road width : 6.0 m
Maximum grade : 10%

(n) Access Road to Muara Takus Temple Site

The existing village road about 480 m long and 5.0 m wide branching from the provincial road provides access to the historic Muara Takus Temple Site. The road however, often flooded and traffic gets suspended in the rainy season. Based on the discussion with Ministry of Education and Culture, the road improvement was done for side ditch drain, raising the low sections of road masonry wall construction was made at the branch with the provincial road.

General Features of the Project (1)

Reservoir		
Gross storage volume		1545 x 10^6 m^3
Activity storage volume		1040 x 10^6 m^3
High water level (HWL)		EL.85.0 m
Low water level (LWL)		EL.73.5 m
Surface area at HWL 85.0		124 km^2
Catchment area		3337 km^2
Annual average inflow at dam site		184 m^3/sec
Dam		
Type		Concrete gravity
Dam height		58.0 m
Freeboard		2.5 m
Dam crest length		257.5 m
Dam crest width		5.0 m
Non over flow dam crest		EL. 87.5 m
Over flow dam crest		EL. 67.5 m
Dam volume		301300 m^3
Dam base width		55.1 m
Surface slope upstream and downstream		1:0.15 and 1:0.80
Spillway		
Type		Overflow, chute and dentated sill type
Design flood		8000 m^3/sec (200 year probable flood)
Overflow depth		17.5 m
Overflow width		71.0 m
Gate Type and size		Fixed wheel gate, 18m (H) x 11m (W) x 5 sets
Intake		
Type		Pressure type
Elevation of intake bed		EL. 61.0 m
Gate type and size		Fixed wheel gate, 6.0m (H) x 6.0m (W) x 3 sets
Trashrack		23.5 m (H) x10 m (W) x 3 sets
Diversion Tunnel		
Tunnel shape		Standard horse-shoe
Discharge capacity		1300 m^3/s (2 Units)
No. 1 Tunnel		10.0 m (Dia) x 478.8 m (L)
No. 2 Tunnel		10.0 m (Dia) x 423.0 m (L)
Primary	upstream	Rockfill 12.5 m (H) x 84.8 m (L)
	downstream	Rockfil 11.2 m (H) x 106.8 m (L)
Secondary	upstream	RCC 19.5 m (H) x 132.0 m (L)
	downstream	Rockfill 10.5 m (H) x 100.3 m (L)
Penstock		
Type		Buried and opened
Size and lines		86.9 m (L) x 5.0 m (D) x 3 lines
Thickness		19 mm - 14.5 mm
Material		SM400B, SS400, SM490YA etc.

General Features of the Project (2)

Powerhouse		
Type	Ground type, 43.2m (H) x 35.6m (W) x 80.3m (L)	
Type of Turbine	Vertical shaft kaplan type	
Installed capacity	114 MW (38 MW x 3 units)	
Annual generated energy	583 Gwh	
Rated discharge	116 m^3/units	
Effective head	38.1 m	
Type of Generator	3-phase AC generator	
Capacity, voltage and frequency	45000 kVA x 3 units, 11 kV, 50 Hz	
Type of Main Transformer	3-phase oil immersed oil material air forced	
Capacity and voltage	45000 kVA x 3 units, 11/150 kV	
Tailrace		
Type	Open channel with trapezium section	
Size	80.0m (L) x 50.5m (W)	
Gate	Steel slide gate, 4.55m(H) x 6.0m(W) x 2 sets	
Stoplogs	Steel slide stoplog 0.92m(H) x 6.0m(W) x 20 sets	
Switchyard		
Space	146m x 109m (EL 174.0m)	
Local transformer	150 kV/20 kV, 3-phase	
Circuit breaker	150 kV, 3-phase	
Disconnecting switch	150 kV, 3-phase	
Potential transformer	150 kV, 3-phase	
Arrester etc.	150 kV, 3-phase	
Transmission Line		
Nominal voltage	150 kV, double circute	
Line length	64.4km (KTP-PKU)	84.7 km (KTP-PYK)
Conductor	ACSR 435mm^2 (KTP-PKU)	ACSR 330mm^2 (KTP-PYK)
Ground wire	GSW 55 mm^2	
Number of suspension tower	167 (KTP-PKU)	131 (KTP-PYK)
Number of tension tower	33 (KTP-PKU)	117 (KTP-PYK)
Insulator	254 mm	
Unit/String	11	
Substation		
Pekanbaru S/S (new)		
150 kV transmission line	2 bays	
150 kV/20 kV transformer	2 bays with 50 MVA x 2 units etc	
Bangkinang S/S (new)		
150 kV transmission line	2 bays	
150 kV/20 kV transformer	1 bay with 10 MVA x 1 unit etc	
Payakumbuh S/S (extension)		
150 kV transmission line	2 bays	
Relocation of Road		
National Road		
Road length and width	Asphalt pavement	
Two major bridges	42.13 km x 6.0 m	
	293.1 m (Kampar Kanan), 288.1 m (Gulamo)	
Provincial Road		
Road length and width	Gravel pavement	
	22.0 km x 6.0 m	

1.4 Project Chronology

Chronology of important events related to the construction of project is tabulated below. The construction phase of the Kotapanjang Hydroelectric Power Project has been performed by 13 contracts and 4 minor contracts which are introduced in detail in Part 11 Contract Administration.

(1) Investigation, Design and Contract Award

January	1982	:	Started Feasibility Study on the Kotapanjang Hydroelectric Power Development Project (Kotapanjang F/S)
March	1984	:	Completed Kotapanjang F/S
February	1987	:	Started Detailed Design and Preparation of Tender and Contract Documents (Kotapanjang D/D)
August	1988	:	Completed Kotapanjang D/D
June	1991	:	Concluded the engineering services for the construction supervision of main civil works and metal, generating equipment, relocation road and bridge, the transmission line works
April	1992	:	Awarded main civil works contracts
July	1992	:	Awarded relocation of national road works contracts
August	1992	:	Awarded relocation of provincial road works contracts
January	1993	:	Awarded generating equipment (generator) works contracts
February	1993	:	Awarded metal works (hydromechanical works) contract
April	1993	:	Awarded transmission line material contracts
August	1993	:	Awarded generating equipment (turbine) works contracts
August	1993	:	Awarded generating equipment (switchyard) works contracts
September	1993	:	Awarded equipment for sub-station contract
September	1993	:	Awarded erection, civil architectural works for substations contracts
September	1993	:	Awarded erection of transmission lines works contracts
September	1996	:	Termination of original contractor because of his severely disrupted work schedule and extended commitment of resources
November	1996	:	Awarded new contracts of erection of transmission lines works
June	1997	:	Awarded telemetering and warning systems works contracts
October	1998	:	The engineering services for the construction supervision of the project completed

(2) Main Civil Works and Related Hydromechanical Works

October	1992	:	Started excavation for diversion works
April	1993	:	Started concreting diversion tunnels
May	1993	:	Started excavation for main dam
July	1993	:	Started excavation for powerhouse
September	1993	:	Completed concreting diversion tunnels
October	1993	:	River diverted and cofferdams closed
February	1994	:	Completed excavation for main dam and powerhouse

March	1994	:	Started concreting main dam and powerhouse
April	1994	:	Held commemorative stone ceremony for dam inviting former Vice President Try Sutrisno and other VIPs
August	1994	:	Started installing steel penstock pipes
October	1994	:	Partially completed dam concreting ready for inside steel penstock pipe installation
August	1995	:	Started installing spillway gates
February	1996	:	Started installing intake gates
June	1996	:	Completed concreting main dam
			Completed installing penstock steel pipes
September	1996	:	Completed installing spillway gates and intake gates
November	1996	:	Taking over certificate for metal works issued
March	1997	:	Commenced impounding of reservoir water
November	1997	:	Taking over certificate for civil works issued

(3) Main Civil Works and Related Elector-Mechanical Works

October	1992	:	Started excavation and embankment for switchyard
July	1993	:	Started excavation for powerhouse
February	1994	:	Completed excavation for powerhouse
March	1994	:	Started concreting powerhouse
June	1994	:	Completed excavation and embankment for switchyard
December	1994	:	Started installing turbine draft tube
January	1995	:	Completed first-stage concrete for powerhouse
February	1995	:	Started installing powerhouse steel structure
April	1995	:	Completed installing turbine draft tube
			Completed second-stage concrete for powerhouse
June	1995	:	Completed concreting switchyard foundations
November	1995	:	Set overhead travelling crane
			Started installing spiral case
January	1996	:	Started installing steel structures on switchyard
June	1996	:	Started installing generator
August	1996	:	Completed installing steel structures on switchyard
			Taking over certificate for switchyard issued
October	1996	:	Started installing main transformer
March	1997	:	Commenced impounding for reservoir water
April	1997	:	Started pre-commissioning test for generating equipment
August	1997	:	Completed control building works
			Started first commissioning test for Unit 1
			Taking over certificate for generator Unit 1 issued
September	1997	:	Taking over certificate for generator Unit 3 issued
October	1997	:	Taking over certificate for generator Unit 2 issued
January	1998	:	Switchyard energizing from Payakumbuh-2 transmission line

February	1998	:	Re-started commissioning test for Unit 1
			Completed generating equipment Unit 3 and started commercial operation
		:	Taking over certificate for turbine Unit 3 issued
April	1998	:	Completed generating equipment Unit 2 and started commercial operation
		:	Taking over certificate for turbine Unit 2 issued
October	1998	:	Re-started commissioning test for Unit 1
November	1998	:	Completed generating equipment Unit 1 and started commercial operation
		:	Taking over certificate for turbine Unit 1 issued

(4) Relocation of National Road and Provincial Road, and 150kV Trunk Transmission Line

February	1993	:	Started relocation of provincial road construction
February	1993	:	Started relocation of national road construction
March	1994	:	Started two long bridges sub-structure construction
April	1994	:	Started construction transmission tower foundation
May	1995	:	Started erection of transmission steel tower
July	1995	:	Started the bridges super-structure construction
September	1995	:	Started transmission stringing work
October	1995	:	Completed provincial relocation road construction
December	1996	:	Completed the bridged construction
February	1997	:	National relocation road was opened for traffic
March	1997	:	Commenced impounding of reservoir water
July	1997	:	Completed transmission tower foundation
August	1997	:	Completed transmission steel tower erection
November	1997	:	Completed transmission stringing work
November	1997	:	The whole work of transmission completed and taking over certificate for the works issued
January	1998	:	Line charging test made

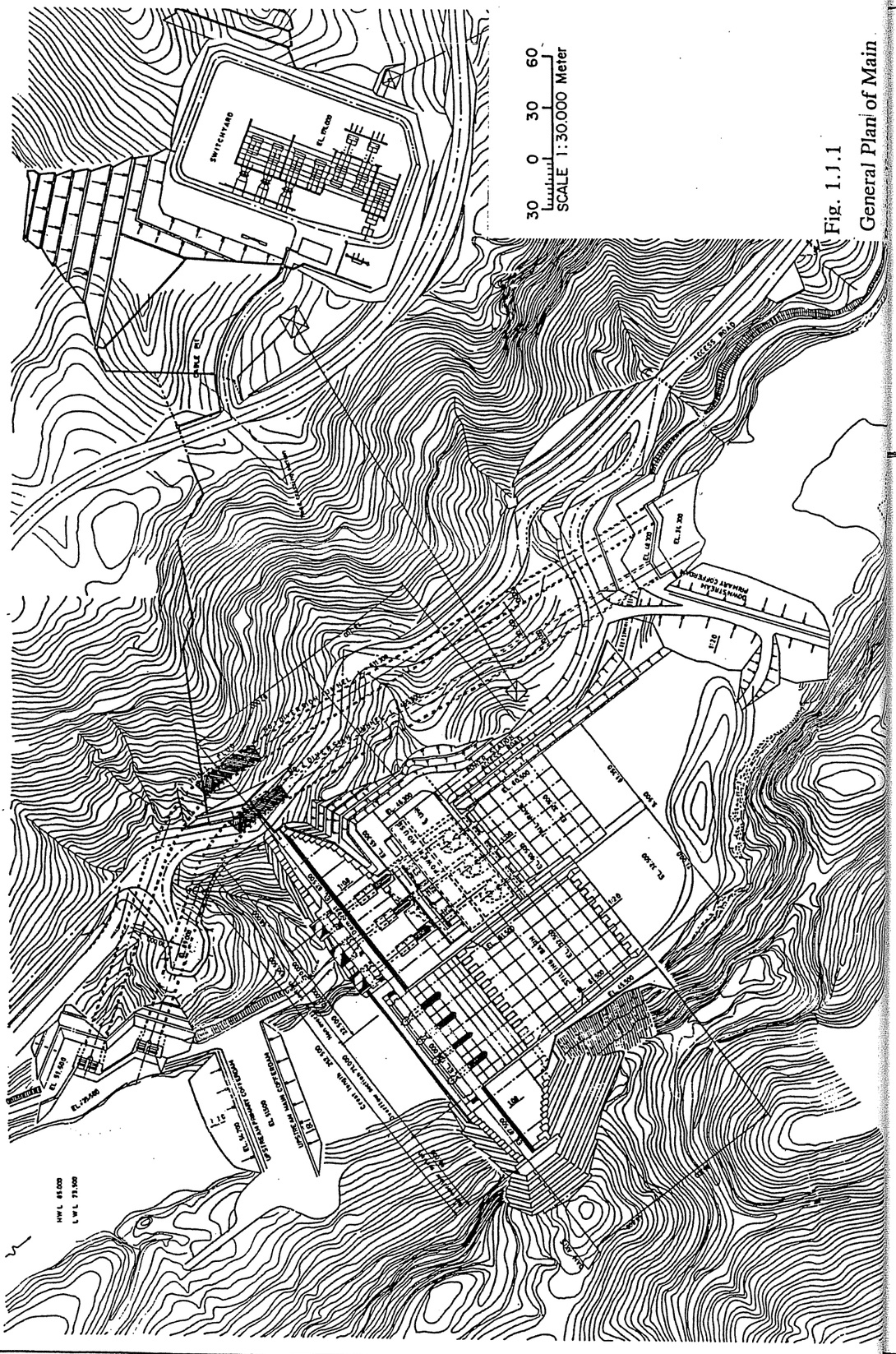


Fig. 1.1.1 General Plan of Main

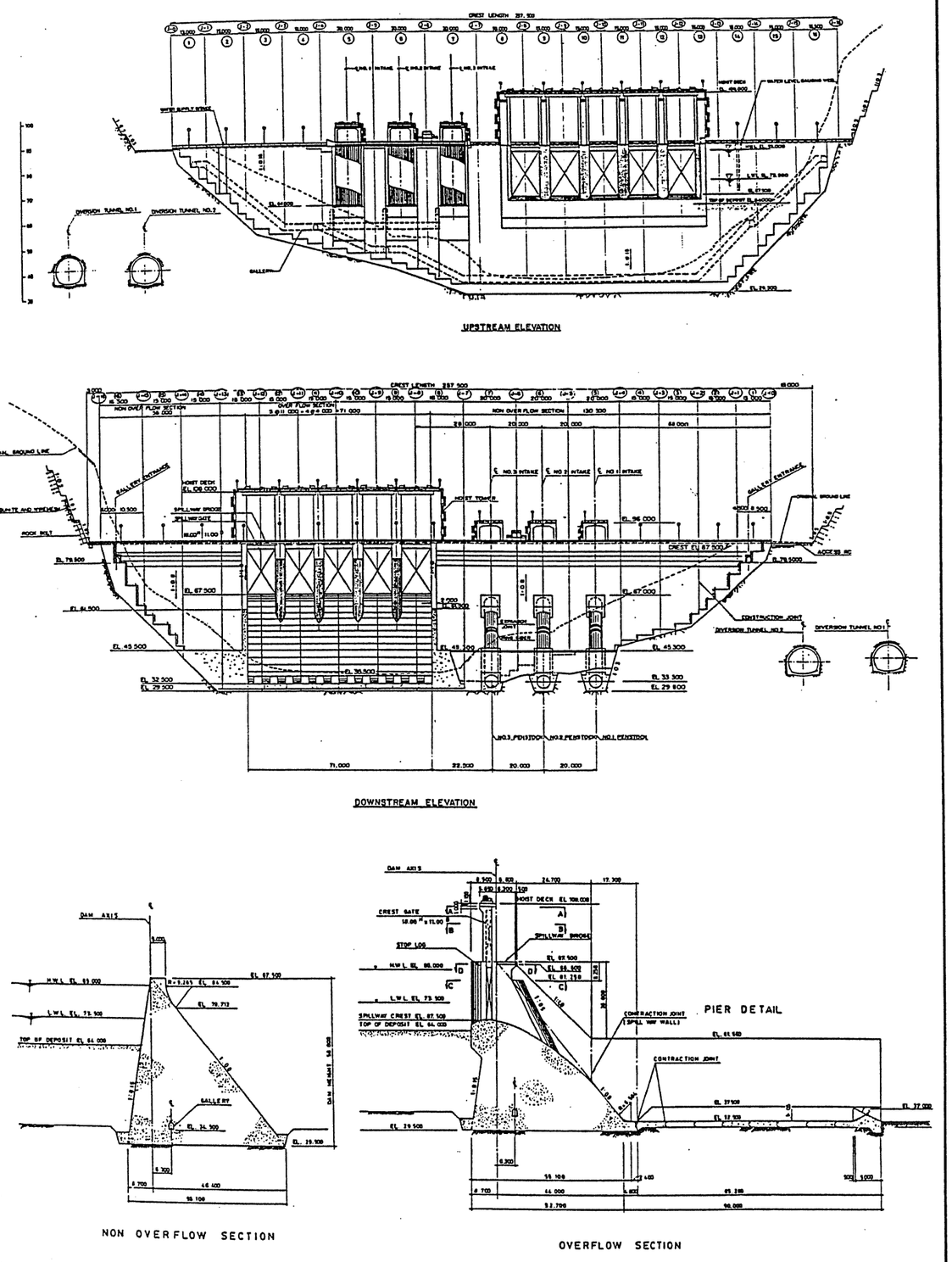
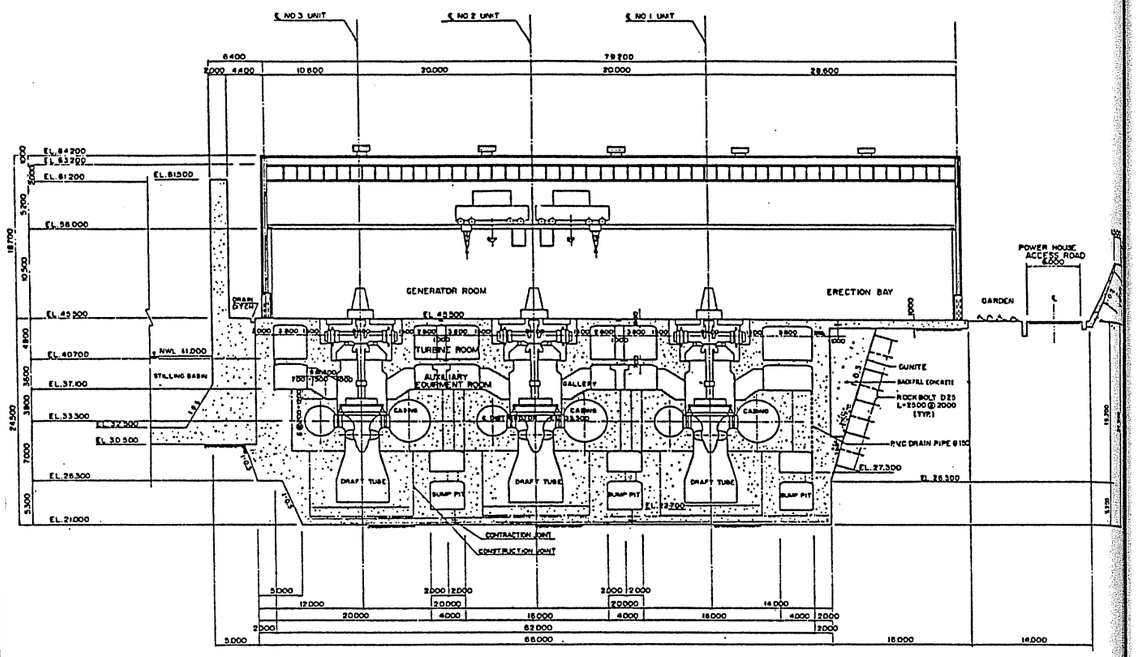


Fig. 1.1.2 Upstream and Downstream of Dam, and Section of Dam



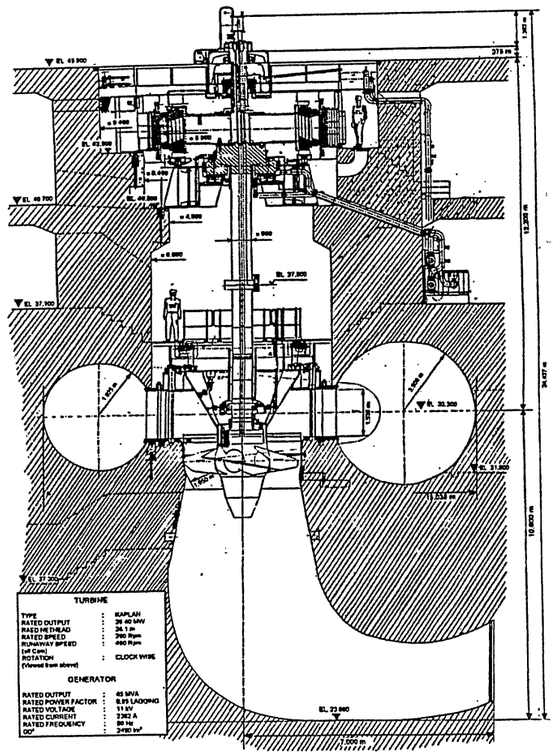


Fig. 1.1.3 Powerhouse and Assembly of Turbine and Generator

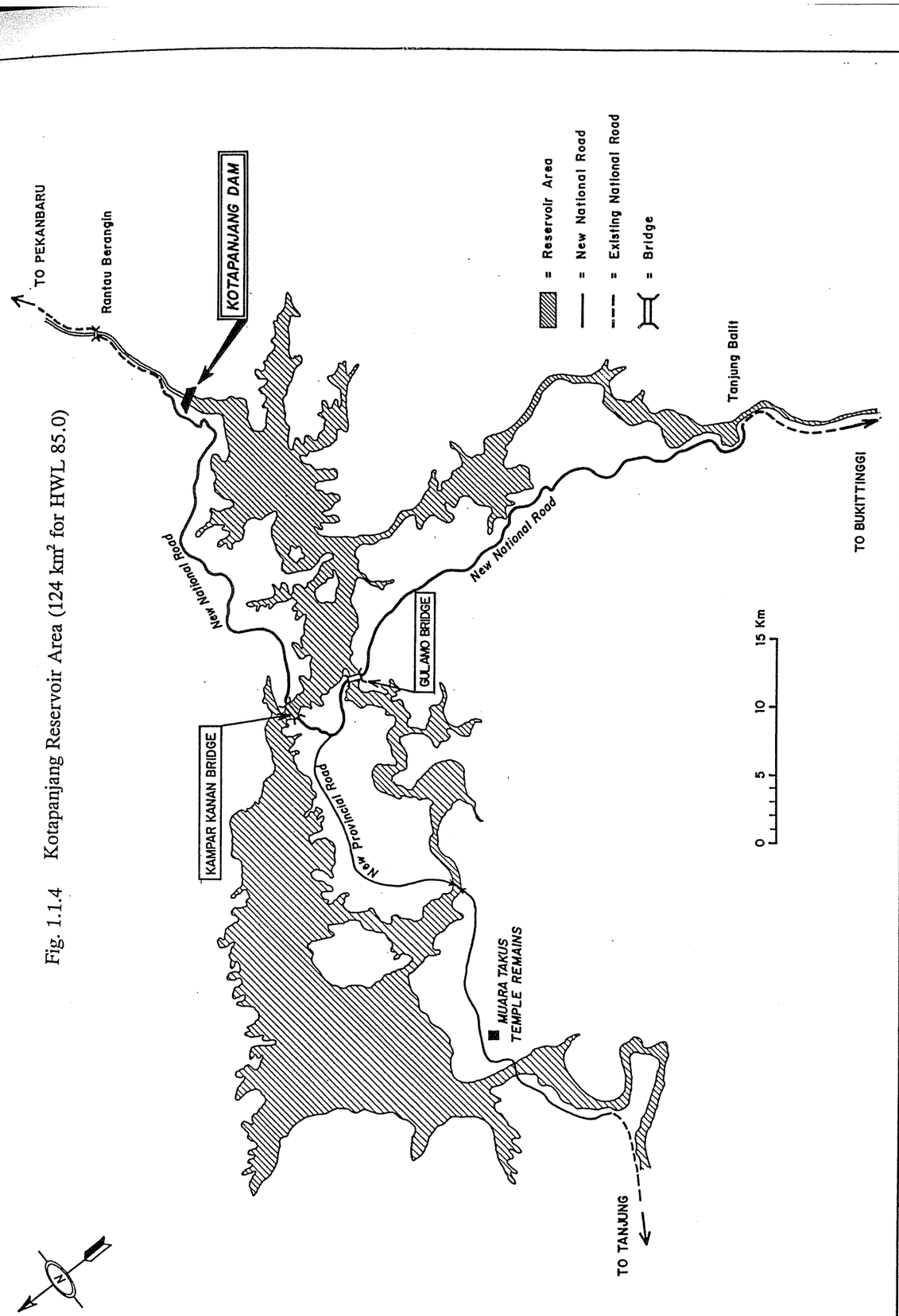
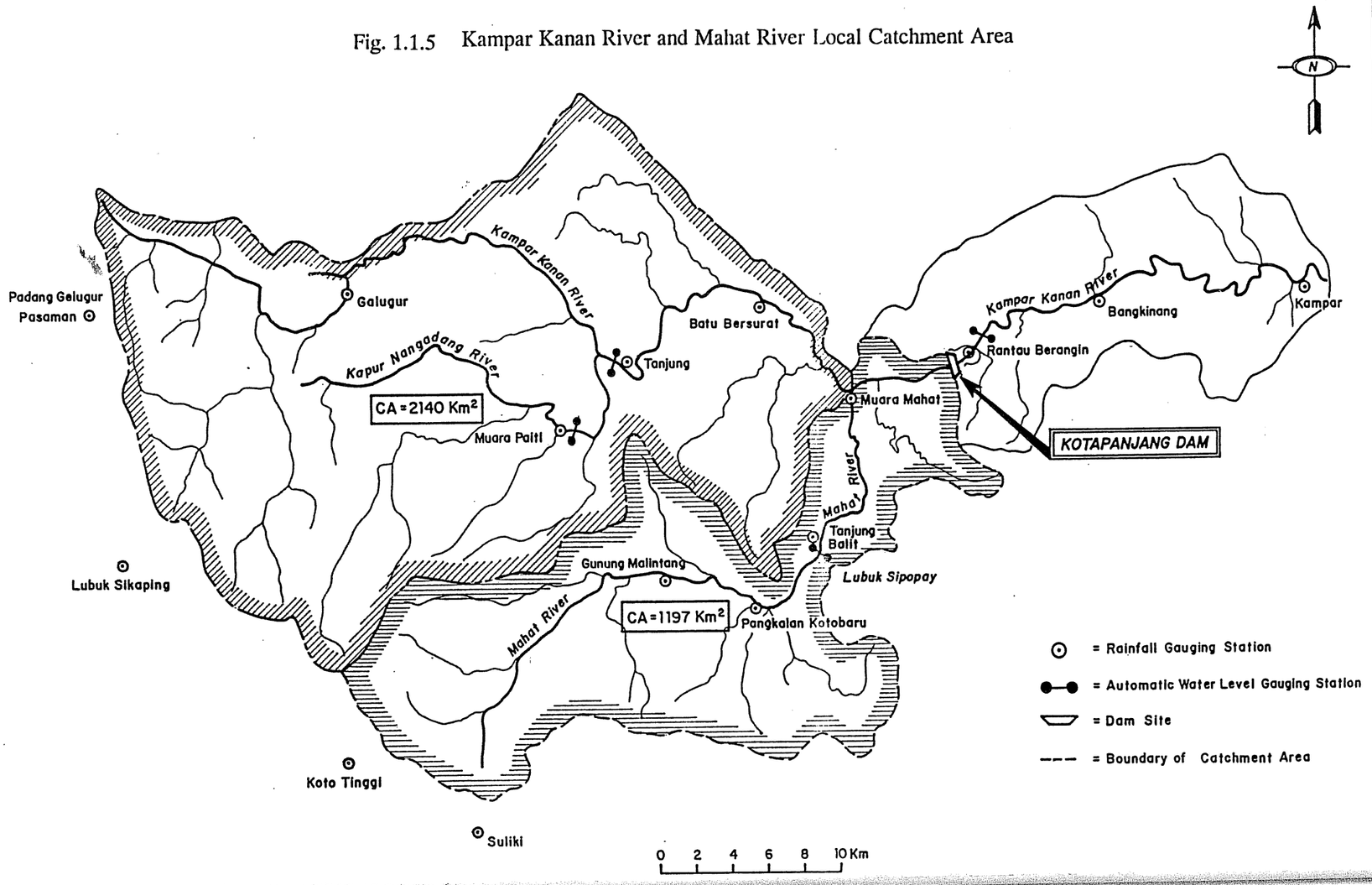


Fig. 1.1.4 Kotapanjang Reservoir Area (124 km² for HWL 85.0)

Fig. 1.1.5 Kampar Kanan River and Mahat River Local Catchment Area



Chapter 2 GEOLOGY

2.1 General Geology

2.1.1 Topography

The most prominent topographic feature of Sumatra is the Barisan Range, which extends along the western coast for a distance of 1,650 km and which is about 100 km wide.

The Kampar Kanan River flows toward the east from its upper reaches at the Bukit Dusun Tinggi, Bukit Lumut, Gunung Hijau and Bukit Tandang Kombang on the Barisan Range. After joining with the tributaries such as (in order) Pakayu River, Lolo River, Kapur River, Gulamo River and Mahat River from upstream to downstream, it runs slowly on the quasi-palin plateau. After joining with Mahat River at Muara Mahat, the river reaches the Kotapanjang Dam Site. Near Rantau Berangin about 4 km downstream of the dam the river runs on the flat alluvial plain and joins with the Kampar Kiri River near Langgam about 90 km east of the Dam site. The Kampar River finally flows into the Malaca Strait after running a meandering course.

The Kampar Kanan River basin can be divided into five (5) physiographic zones, each of which is closely related to basement techtonism, and which trend approximately NW-SE.

Barisan Geanticline (Refer to the dashed line I and I' in Fig. 1.2.1) : This zone consists of high relief mountains generally ranging in attitude from 500 to 1,000 meters and is the major geological feature of Western Sumatra.

Eastern Barisan Foothills (Refer to the dashed line II, II' and III in Fig. 1.2.1) : This zone lying on the east of the Geanticline extending to Rantau Berangin near Kotapanjang dam site is approximately 45 km wide and consists of low relief hills generally ranging in altitude from 150 to 500 meters. It is essentially a series of NW-SE Horst and Graben in which rivers are incised. The horst have young incised water channels, while the graben frequently show meander belts.

Mountain Front (Refer to the dashed line IV in Fig. 1.2.1) : The eastern margin of the foothills is known as the mountain front. It is about a 20 km wide zone of relatively low but still distinct relief between 30 and 150 meters above sea level. This zone exposes Tertiary sediment, sometimes capped with terrace composed of Pleistocene deposits.

Alluvial Plain : This zone lying to the east of the first three regions is an extremely low relief, densely forested area of mainly alluvium and swamp. Modern river courses are often bordered by well developed flood plains with numerous abandoned channels and oxbow lakes.

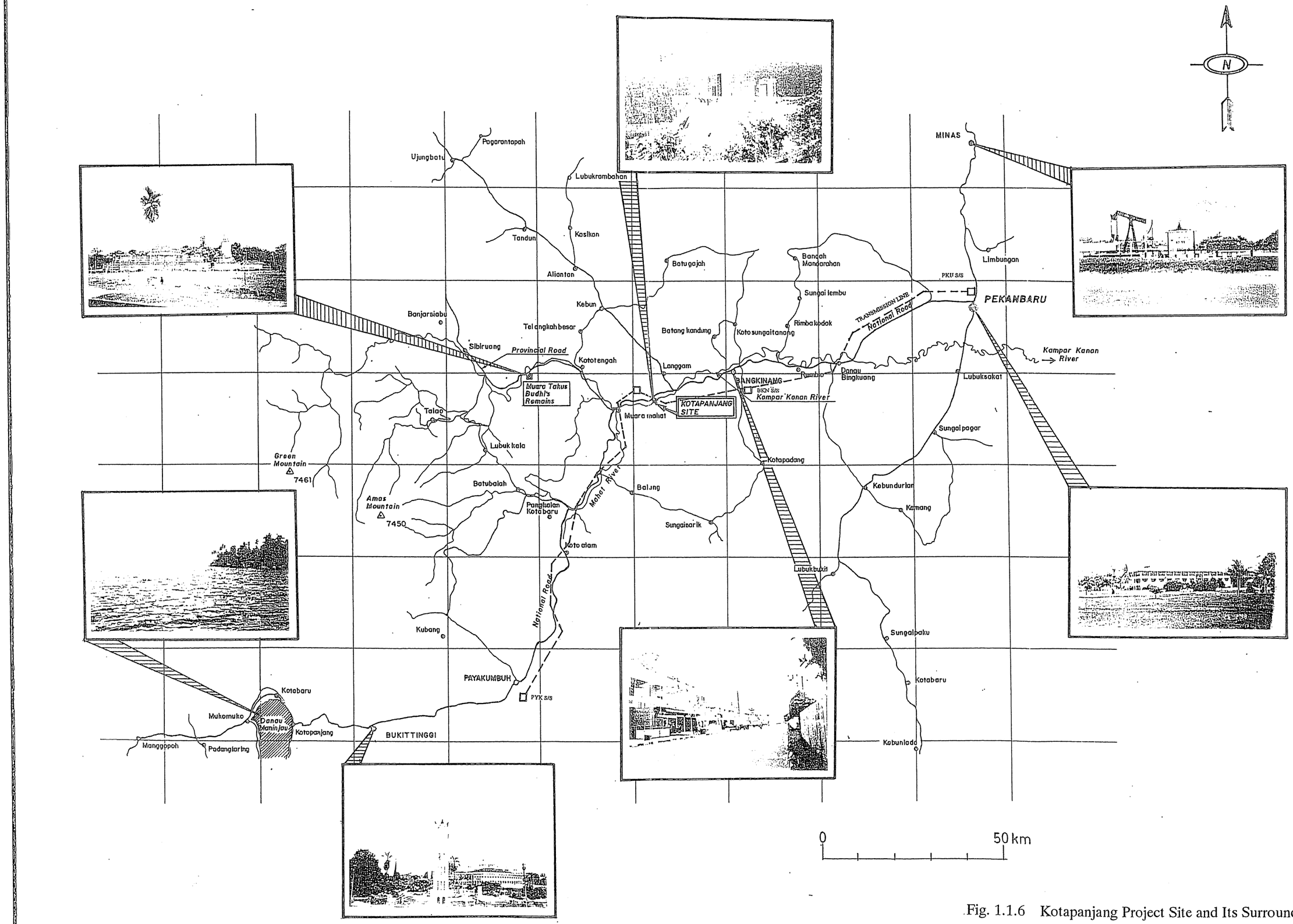


Fig. 1.1.6 Kotapanjang Project Site and Its Surrounding Area

Minas Hills (Refer to the dashed line V in Fig. 1.2.1) : At intervals , broad areas of positive relief protrude from the alluvial plain. They are collectively termed the Minas Hills. The Best example is underlain by the Minas oil field, which reaches an elevation of 100 meters above sea level. The strongly incised drainage pattern indicates that the structure is an elongate dome.

The Kotapanjang Dam Site is located in the last range among the Eastern Barisan Foot Hills, about 10 km downstream of Muara Mahat at the confluence of the Kampar Kanan River and Mahar River. In other words, the dam site is about 85 km from Pekanbaru, the capital city of Riau Province and about 20 km from Bangkinang. The dam site is easily accessible through the Bukittinggi - Pekanbaru National road running on the left bank.

Regarding the topography around the dam site, the river is a valley form over a distance of about 1 km upstream and downstream of the dam site. The dam site was selected at the position where the width of the valley is smallest. The river at the dam site is about 80m in width and the elevation of its basin is about 35 m with its depth between drought discharge and wet discharge varying from about 2m to 5m.

The river gradient of the reservoir area is gentle, being about 1:1,070 at the Kampar Kanan river, 1:1,500 at the Mahat river and 1:1,090 from the dam site to Rantau Berangin. Moreover, a large space surrounded by the mountain is extended upstream of the dam site. Therefore, this site obtain a large capacity reservoir by constructing a medium size dam.

The main existing data available before the start of the feasibility study were the topographic maps of 1:250,000, 1:100,000 and 1:50,000 in scale. In the feasibility stage aerial maps of 1:10,000 were prepared for the reservoir area and survey maps of 1:1,000 and 1:500 were also prepared for the area around the dam site and sand and gravel material sites.

In the detailed design stage, topographic survey mainly in the dam and powerhouse sites including appurtenant civil structures site was carried and the survey maps of 1:500 and 1:200 were prepared for the detailed design. In the construction stage, further detail survey was carried out for some additional work as occasion demands.

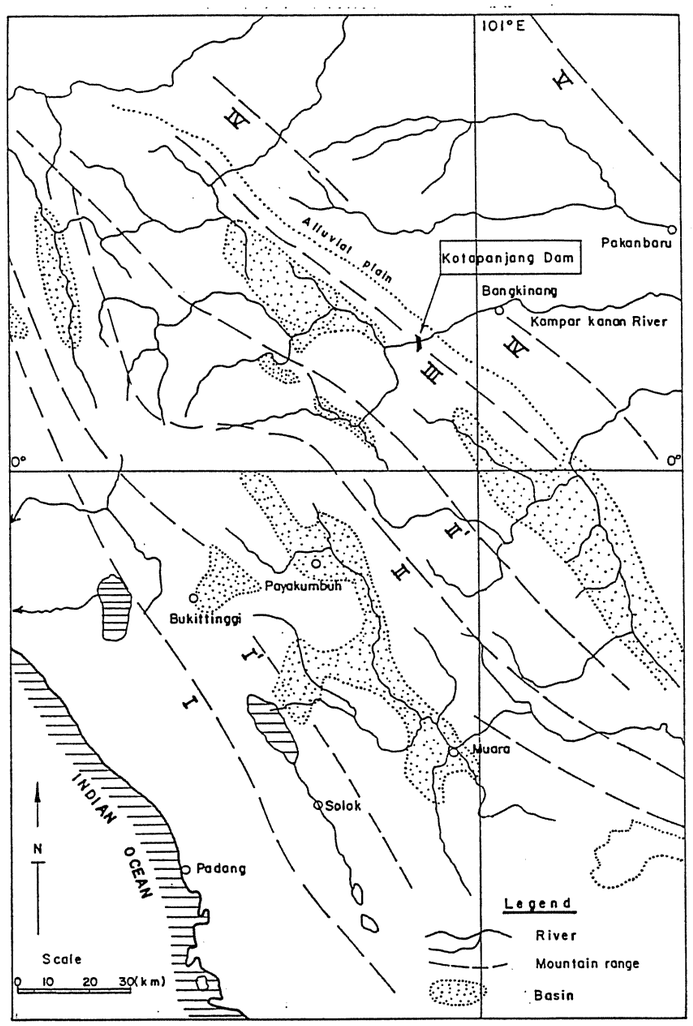
Table 1.2.1 Topographic Survey Carried Out in Feasibility Study Stage

Survey items	Locations of survey	Quantity	Scale	No. of maps
Aerial photogrammetry	Reservoir area	535 km^2	1:10000	34 sheets
Topographic survey	Dam site	77 ha.	1:1000	4 sheets
	Kuok sand and gravel site	6.25 ha.	1:1000	1 sheet
Cross sectioning	Dam site	10 lines	1:500	5 sheets
Levelling	Muara Mahat - Pangkalan Kotabaru	60 km	-	1 sheet
	Rantau Berangin - Triangulation T.630	55 km	-	1 sheet

Table 1.2.2. Topography Survey Details Carried Out in Engineering Design Stage

Survey items	Locations of survey	Quantity	Scale	No.of maps
Cross section	Dam body, power house and tailrace	39 Lines	1 : 500	12 sheets
Cross section	Power house excavation line	16 Lines	1 : 200	3 sheets
Cross section	Power house and tailrace excavation line	22 lines	1 : 200	2 sheets
Cross section	Power house control building	24 lines	1 : 50	3 sheets
Cross section	Power house, tailrace and river bed	12 lines	1 : 500	4 sheets
Topographic survey	Propincial road	22.125 km	H = 1 : 1000 V = 1 : 200	33 sheets
Topographic survey	National road I	3.150 km	H = 1 : 1000 V = 1 : 200	4 sheets
Topographic survey	National road II	42.175 km	H = 1 : 1000 V = 1 : 200	62 sheets
Topographic survey	Access road to dam	878.8 m	1 : 500	2 sheets
Topographic survey	Access road to power house	395 m	1 : 500	1 sheets
Topographic survey	Access road to boat handling	160 m	1 : 500	1 sheets
Topographic survey	Access road to Switchyard and surrounds	550 m	1 : 500	2 sheets

Fig. 1.2.1 Simplified Physiographic Map



2.1.2 Geological History and Stratification

The Barisan Range came into existence in the cretaceous age of the Mesozoic Era. At the beginning of the Tertiary Period, the Central Sumatra Basin was an extensive landmass with numerous orogeny subsequently continental clastic sedimentation began filling the trough. Following the continental deposition, a regional marine transgression began in the Late Oligocene or Early Miocene period.

Neogen strata record a transgressive - regressive cycle of sedimentation, consisting of an older transgressive phase of fluvial-deltaic near shore to shallow marine sediments. Deeper marine shales were recorded in some areas when transgression reached its maximum.

The last major period of uplift and block faulting occurred in the Plio-Pleistocene period. The basin and range topography of the foothills region was established at this time. Debris from uplifted areas is represented by Pleistocene continental deposits which overlay the Tertiary sediment with slight unconformity. Through Late Pleistocene, in Kampar Kanan River there were deposited Terrace or older alluvium along the river, while the land was rising. Further east, subsidence resulted in a large area of recent alluvium, often in the form of deep swamps.

The geology of the Kampar Kanan River basin area is stratigraphically shown in the following table.

Fig. 1.2.3 Stratigraphy in the Project Area

Geological Age			Sediments and Metasediments		Volcanics and Intrusives	
CENOZOIC		Holocene	Young Alluvium		Maninjau Pumice Tuff	
	Quartermary	Pleistocene	Oldel Alluvium			
			Minas Fmn.	Kerumutan Fmn.		
	Tertiary	Pliocene	Kampar Group	Telisa Fmn.	Koto Alam Volcanic Fmn.	Undifferentiated Minor Volcanics
		Miocene		Sihapas Fmn.		
		Oligocene & Eocene		Pematang Fmn.		
MESOZOIC		Jurasic & Cretaceous				
		Triassic	Peusangan Group	Tuhur Fmn.		
PALEO-ZOIC		Permian			Granit	
		Carboiferous	Tapanuli Group	Bohorok Fmn / Kuantan Fmn		

According to the data prepared by the Geological Research and Development Centre in 1982 and the result of the field reconnaissance carried out in the Detailed Design Stage, the stratification of each formation are described below, from the oldest up to the youngest respectively.

(a) Pre-Tertiary succession

Kuantan Formation : This formation consists of phyllites and muscovite schist, thin limestone and sandstone. The Kuantan Formation is found in the west part of the area, between Tanjung Pauh and Pulau Gadang, and Gulamo River. The thickness of this formation is around 3,000 meters.

Bohorok Formation : This formation is composed of Dacitic Tuff in some places conglomeratic. The Bohorok Formation is found from Rantau Berangin Bridge Site to 10 km upstream at Pulau Gadang village and is distributed in a northwest to southeast direction. The thickness of this formation is around 3,000 meters.

Tuhur Formation : This formation is composed of gray to dark grey slate, mainly shale and intercalations of brown chert, black silicified shale and thin layers of metamorphosed greywacke. The Tuhur Formation is found in the middle of the area near Lipat Kain Town located about 150 south-east of the dam site.

(b) Tertiary succession

Pematang Formation : The Pematang Formation consists of carboniferous shale, fine-grained sandstone and silts tone. The pre-Tertiary basement trough or grabens on low-lying areas, were filled by these sediments. This formation is found near the Rantau Berangin Bridge and at Tanjung Balit about 30 km south of the dam site, and is underlain by pre-Tertiary rocks in unconformable contact. The thickness of this formation is about 200 m and thickening towards Barisan .

Sihapas Formation : The Sihapas Formation consists mainly of medium to coarse-grained sandstone with minor shale intercalation, fining upward sequence with conglomeratic at the base. This formation is underlain by the Pematang Formation in unconformable contact. Also this formation will be found near Rantau Berangin, upstream Batu Anjing Bridge and Tanjung Alai. The Thickness of this formation is 100 to 300 meters.

Telisa Formation : This formation consists mostly of dark grey calcareous mudstone, thin limestone and minor glauconitic sandstone. In some places, this formation is composed mostly of shale with minor silts tone intercalation. The shale is rich in foraminifera. This formation is distributed around Muara Takus remains located about 40 km east of dam site. The thickness of this formation is 100 m to 300 m and underlain by the Sihapas Formation in conformable contact and probably interfinger.

Petani Formation : The Petani Formation is composed of greenish grey shale with sandstone interbeds, carbonaceous mudstone and in some places lignite or coal bed. This formation is well exposed along the national road between Bangkinang and Air Tiris about 30 km east of dam site. The thickness of this formation is 300 m to 700 m. It is underlain by the Telisa Formation in unconformable contact.

(c) Quaternary succession

Minas Formation : This formation is composed of unconsolidated to semi-consolidated mud, sand and gravel. This formation is found in more than 15% of the study area. The thickness of the Minas Formation is about 100 m. This formation is underlain by the Tertiary Formation in unconformable contact.

Terrace / Older Alluvium : After Minas Formation, through Late Pleistocene the terrace / older alluvium was deposited along the Kampar Kanan River. This sediment consists of mud, sand and gravel.

Alluvium : The Alluvium consist of muds, sand and gravel, and also swamps sediment of organic materials. This sediment is distributed along the river, especially in the meandering area.

2.2 Field Investigation

The field geological investigation carried out in the feasibility study stage and in the engineering design stage are summarized in Tables 1.2.4 and 1.2.5.

The surface reconnaissance in the feasibility study was conducted in as much detail as possible in the reservoir area and river basin in addition to the dam site, quarry sites, and river deposits sites. At the Kotapanjang dam site, field investigations were conducted with respect to the dam axis-1, 3 and 4 which had been selected from among the dam axis 1 to 4 as a result of a comparative study in accordance with the results of field reconnaissance in the preliminary investigation stage.

The geological investigation in the detailed design stage was conducted for eight months from May 1987 to January 1988. This investigation consisted mainly of surface geological survey, drilling, seismic prospecting, editing, test pitting, trenching and auger boring. In-situ testing, laboratory testing, sounding, and geophysical logging were carried out to obtain the engineering properties of the bedrock and soil.

The geological investigation was performed at the following sites :

- Dam and Power Station, Diversion Tunnels
- Switchyard, Transmission Line Route and Substation
- Pulau Gadang Gravel Pit, Kuok Gravel Pit and Quarry Site
- Relocation Road and Bridges
- Reservoir Area

Moreover, the aerial survey with the aid of a helicopter was performed to observe the landform, especially any landslide of the entire reservoir area and relocation road route. In the following Table 1.2.5 is summarized the items and quantities covered by the filed work and laboratory testing.

Table 1.2.4 Summary of Geological Investigation Carried out in Feasibility Study Stage

Survey Items	Survey Sites	Contents of Survey
Drilling	Dam site	27 holes and 1,200m in total length
	Quarry sites River deposits	14 holes and 525m in total length 2 holes and 20m in total length
Seismic prospecting	Dam site Quarry sites	6 lines and 2,805m in total length 9 lines and 2,805m in total length
Test adits	Dam site	4 adits and 260m in total length
Permeability test	Dam site	20 holes
Lugeon test	Dam site	5 holes
Grouting test	Dam site	2 locations, 12 holes 420m in total length
Block shearing test	Dam site	4 locations
Test pits	River deposits	7 holes and 19m in total length
Laboratory test of sampled materials	Dam site, quarry sites and river deposits sites	Rock : 81 samples from 51 locations and test of 6 items Sand and gravel : 300 kg samples from 6 locations and test of 5 items Microscopic observation of rock : 5 thin test pieces from 5 locations.

Table 1.2.5 (1) Summary of Geological Investigation Carried Out in Engineering Design Stage

Work Items	Location	Quantity	Total
Drilling	Dam Site	860 m (19 holes)	
	Quarry Site	290.5 m (7 holes)	
	Pulau Gadang Gravel Pit	40 m (6 holes)	
	Kuok Gravel Pit	90 m (9 holes)	
	Switchyard	30 m (3 holes)	
	Transmission Line	537 m (41 holes)	2250.5m (104 holes)
	Substation	120 m (4 holes)	
	Gulamo Bridge Site	125 m (7 holes)	
	Kampar Bridge Site	123 m (6 holes)	
	Muara Takus Bridges Site	35 m (2 holes)	
Lugeon Test	Dam Site	97 stages (14 holes)	97 stages (14 holes)
Standard Penetration Test	Dam Site	15 times (6 holes)	
	Switchyard	30 times (3 holes)	
	Transmission Line	539 times (41 holes)	
	Substation	120 times (4 holes)	834 times (69 holes)
	Gulamo Bridge Site	35 times (7 holes)	
	Kampar Bridge Site	90 times (6 holes)	
	Muara Takus Bridges Site	5 times (2 holes)	
Seismic Prospecting	Dam Site	1800 m (8 lines)	
	Quarry Site	1625 m (7 lines)	3425 m (15 lines)
Test Adit Excavation	Dam Site	210 m (5 adits)	
	Quarry Site	66 m (1 adit)	276 m (6 adits)
Plate Load Test	Dam Site	6 units	6 units
Rebound Test	Dam Site	38 units	
	Quarry Site	66 units	94 units
Point Load Test	Dam Site	38 units	
	Quarry Site	30 units	68 units
Trench Excavation	Pulau Gadang Gravel Pit	35 cubic m (1 trench)	
	Kuok Gravel Pit	820 cubic m (7 trenches)	855 cubic m (8 trenches)
Test Pit Excavation	Pulau Gadang Gravel Pit	4 m (2 test pits)	
	Kuok Gravel Pit	19.2 m (8 test pits)	34.2 m (15 test pits)
	Relocation Road	6 m (3 test pits)	
	Gulamo Bridge Site	5 m (2 test pits)	
Cone Penetration Test	Relocation Road	80.2 m (20 point)	80.2 m (point)
Auger Boring	Relocation Road	49.4 m (20 holes)	49.4 m (20 holes)
Dilatometer Test	Transmission Line	4 units	
	Gulamo Bridge Site	4 units	11 units
	Kampar Bridge Site	3 units	
Electric Logging Test	Switchyard	3 times	
	Substation	3 times	6 times

Table 1.2.5 (2) Summary of Geological Investigation Carried Out in Engineering Design Stage

Work Items		Location	Quantity	Total
Laboratory Test (Rock)	Specific Gravity and Absorption	Dam Site Quarry Site Bridge Site	20 units 30 units 16 units	66 units
	Unconfined Compressive Strength	Dam Site Quarry Site Bridge Site	20 units 30 units 16 units	66 units
	Tensile Strength	Dam Site Quarry Site	20 units 30 units	50 units
	Sonic Velocity	Dam Site Quarry Site Bridge Site	20 units 30 units 16 units	66 units
	Density	Bridge Site	16 units	16 units
Laboratory Test (Soil)	Specific Gravity and Absorption	Relocation Road	6 units	6 units
	Moisture Content	- ditto -	6 units	6 units
	Moisture Unit Weight	- ditto -	6 units	6 units
	Atterberg Limit	- ditto -	6 units	6 units
	California Bearing Ration	- ditto -	3 units	3 units
	Proctor Modified	- ditto -	3 units	3 units
	Unconfined Compressive Strength	- ditto -	3 units	3 units
	Triaxial	- ditto -	3 units	3 units
	Consolidation	- ditto -	3 units	3 units

To identify the quality of rock, the Tanaka's classification standard which is commonly used for hydropower projects in Japan is applied and classified into several grades from the physical and mechanical viewpoint, by analyzing the physical changes of rocks based on the weathering, alteration and fissure distribution.

Table 1.2.6 Rock Classification Standard by Tanaka

Classification	Characteristic
A	Rock-forming minerals are fresh and not weathered or altered. Joints and cracks are very closely adhered with no weathering along their planes. A clear sound is emitted when hammered.
B	Rock-forming minerals are weathered slightly or partially altered, the rock being hard. Joints and cracks are closely adhered. A clear sound is emitted when hammered.
CH	Rock-forming minerals are weathered but the rock is fairly hard. The bond between rock blocks is slightly reduced and each block is apt to be exfoliate along joints and cracks by strong hammering. Joints and cracks contain clay and other material which may be colored by limonite. A slightly dull sound is emitted when hammered.
CM	Rock-forming minerals are weathered and the rock is soft. Exfoliation of the rock occurs along joints and cracks by normal hammering. Joints and cracks sometimes contain clay and other material. A somewhat dull sound is emitted when hammered.
CL	Rock-forming minerals are weathered and the rock is soft. Exfoliation of the rock occurs along joints and cracks by light hammering. Joint and cracks contain clay. A dull sound is emitted when hammered.
D	Rock-forming minerals are weathered and the rock is very soft. There is virtually no bond between rock blocks, and collapse occurs at the slightest hammering. Joints and cracks contain clay. A very dull sound is emitted when hammered.

2.3 Geological Features of the Project Site

2.3.1 Dam Site Area

Based on the results of the investigation, the geological conditions in the dam site area were concluded as follows :

(i) The whole area of Dam Site is composed of Dacitic Tuff of Carboniferous to Early Permian age. The formation at this site consists mainly of fairly hard dacitic tuff, and is dotted with pumice without definite signs of stratification. Moreover, the rock is cut through by frequent joints, but the joints are filled with quartz and hardened iron-oxide veins. In some places, this rock is overlain by river deposit, terrace deposit, and talus deposit of Quaternary age.

The river deposits are observed mainly on the right bank at the dam site at elevations below the water level, and consist of sand and gravel with a maximum diameter 12 to 50 cm, with the maximum thickness being estimated to be about 5 m.

The terrace deposits which are observed around the central part of the right bank at the dam site are composed of unconsolidated fine sand and silt. The terrace deposits are also observed on the right side of the left bank, and at around 50 m downstream from the dam axis.

Though the talus materials and top soil are observed everywhere on the ground surface, these strata are very thin around the slope of the left bank and adjacent to the ridges of both banks. The strata have a thickness of 4 to 5 m covering the terrace deposit on the right bank toward the mountain side.

The foundation rock consists mainly of massive, hard, brittle, and well-jointed Dacitic Tuff. The Joints had been firmly filled by iron-oxide and/or quartz veins due to hydrothermal alteration. When fresh, the Dacitic Tuff is, in general, strong and stiff, and of good quality.

The rock-mass discontinuity recorded are (i) no fault encountered (ii) fracture zones were discovered at the dam abutments and surrounding areas (iii) joint and seam distribute irregularly (iv) dipping of discontinuities could be classified into gentle, steep and vertical. Information about the distribution and number of discontinuities was put into a stereographic projection and is shown Fig. 1.2.10 to 1.2.14.

(ii) The Quality of Dam Foundation Rock can be shown as follows :

Grade	Left Bank	River Bed	Right Bank
D	0 to 2 m	0 m River bed deposit composed of sand and gravel.	0 to 2 m Talus deposits was found in a thickness of approx. 4 m.
CL	2 to 7 m	0 to 4 m	0 to 4 m
CM	7 to 22 m	0 to 4 m	6 to 13 m
CH	25 to 45 m	15 to 37 m	10 to 40 m

(iii) The Permeability of Dam Foundation Rock tends to vary. Permeability of 2 lugeon or less was detected at about 50 m depth in the left bank, and more than 35 m depth in the river bed, and more than 20 m depth in the right bank.

(iv) The result of in-situ and laboratory rock tests of the dam site area have the following typical, physical and mechanical properties. The rock condition with these physical properties was considered to be strong enough to construct the concrete gravity dam, power station, and large size diversion tunnels.

Natural Density	2.53 g/cm^3
Saturated Density	2.95 g/cm^3
Dry Density	2.51 g/cm^3
Natural Water Content	0.78 %
Absorption/Saturated Water Content	3.13 %
Degree of saturation	26.25 %
Apparent Specific Gravity	2.51
True Specific Gravity	2.73
Void Ratio	0.0785
Longitudinal Wave Velocity	4952.24 m/sec
Shear Wave Velocity	2820.44 m/sec
Tensile Strength	52.90 kg/cm^2
Uniaxial Compressive Strength	430.615 kg/cm^2
Static Modulus of Elasticity	5.58x10^4 kg/cm^2
Poisson Ratio	0.24
Young Modulus of Elasticity	2.10x10^5 kg/cm^2
Shear Strength	
Apparent friction angle	38^0
Apparent Cohesion	18.4 kg/cm^2

The Area of Dam Site comprises mainly Dam Foundation, Power Station, and Diversion Tunnels. The following investigations were carried out at the Dam Foundation before construction stage. The plan and profile of the dam site powerhouse and diversion tunnel are shown in Fig. 1.2.2 to 1.2.8.

- Drilling, Standard Penetration Test, Lugeon Test
- Seismic Prospecting and Test Additing
- Plate Loading Test, Point Load Test, Rebound Test, Rock Mechanical Laboratory Test.

Test adits and drillings carried out at the dam site, powerhouse and diversion tunnels are as follows.

Location	Test Adits		Drilling	
	Number	Length (m)	Number	Length (m)
Right bank	2	110	5	265
River bed	-	-	-	-
Left bank	3	100	5	595
Powerhouse	-	-	6	-
Diversion Tunnels	-	-	3	-
Total	5	210	19	860

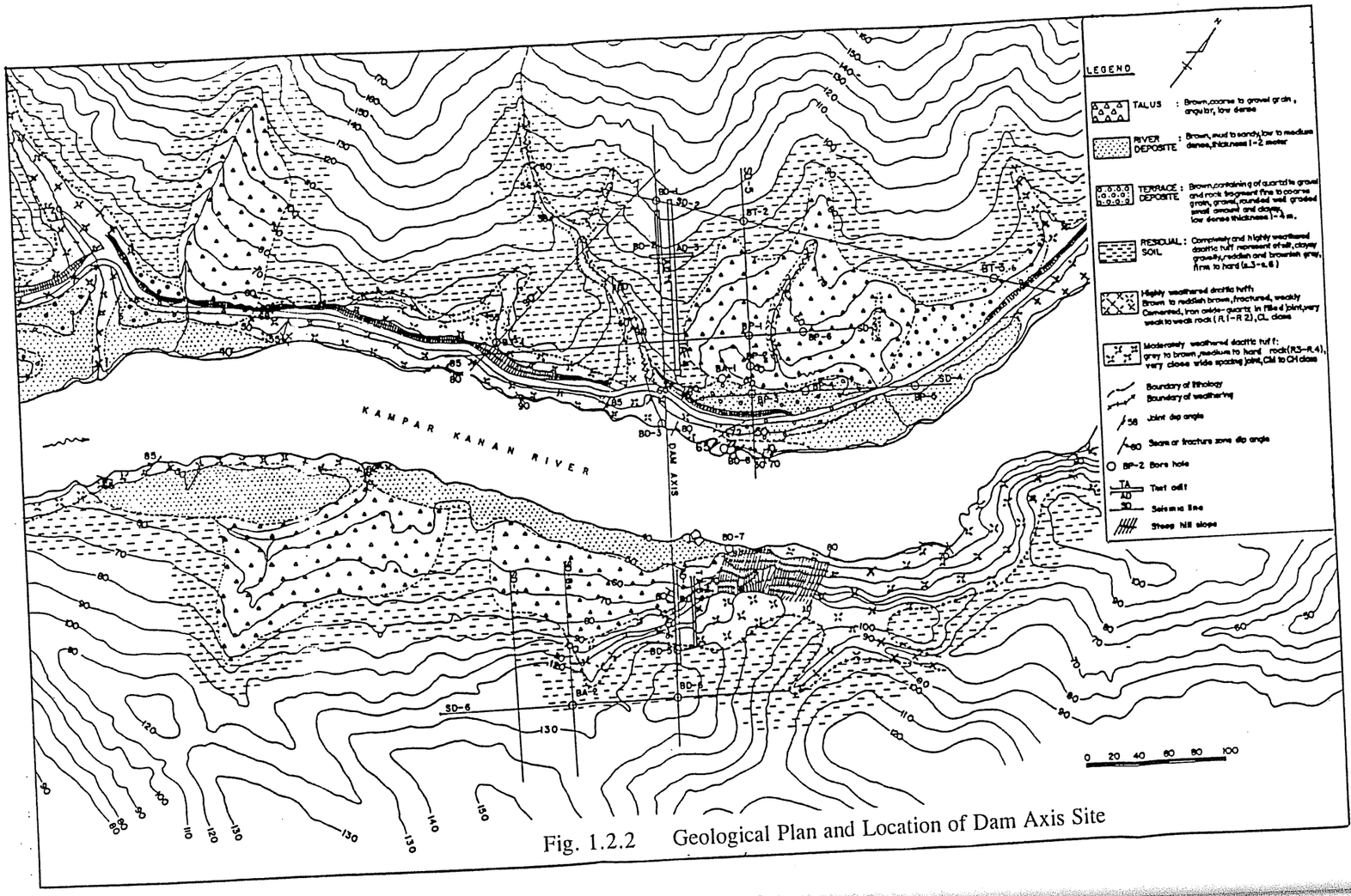


Fig. 1.2.2 Geological Plan and Location of Dam Axis Site

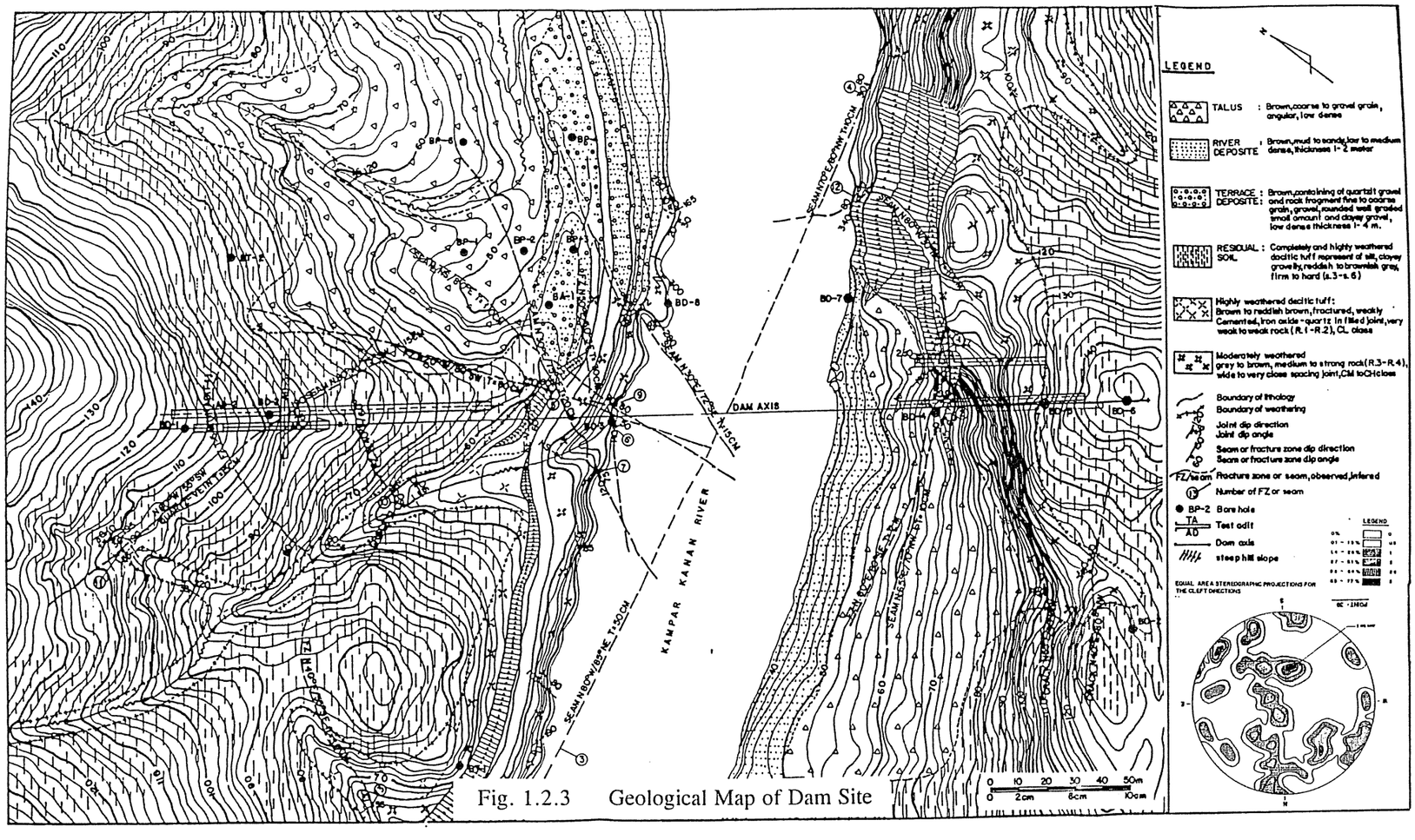


Fig. 1.2.3 Geological Map of Dam Site

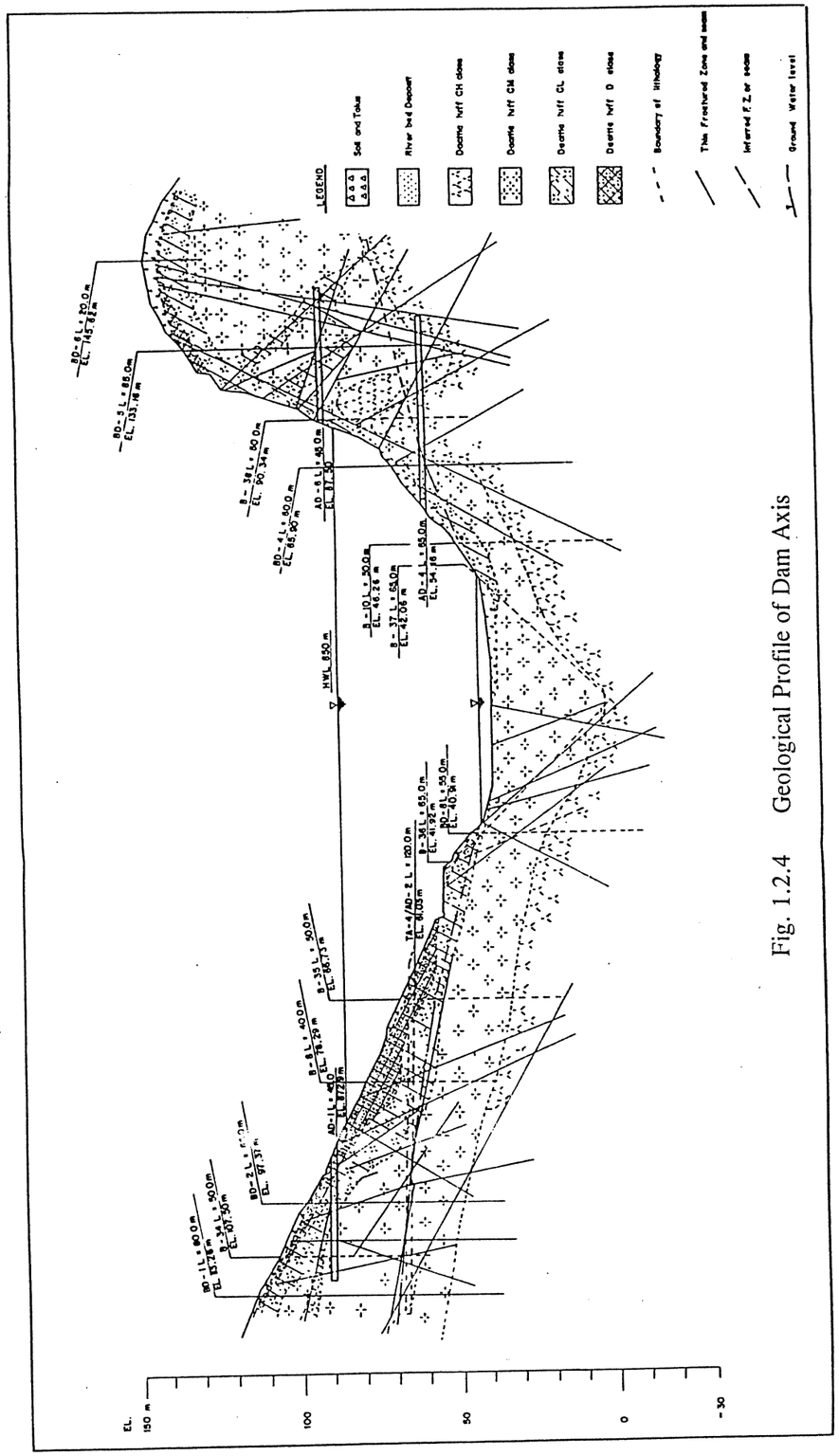


Fig. 1.2.4 Geological Profile of Dam Axis

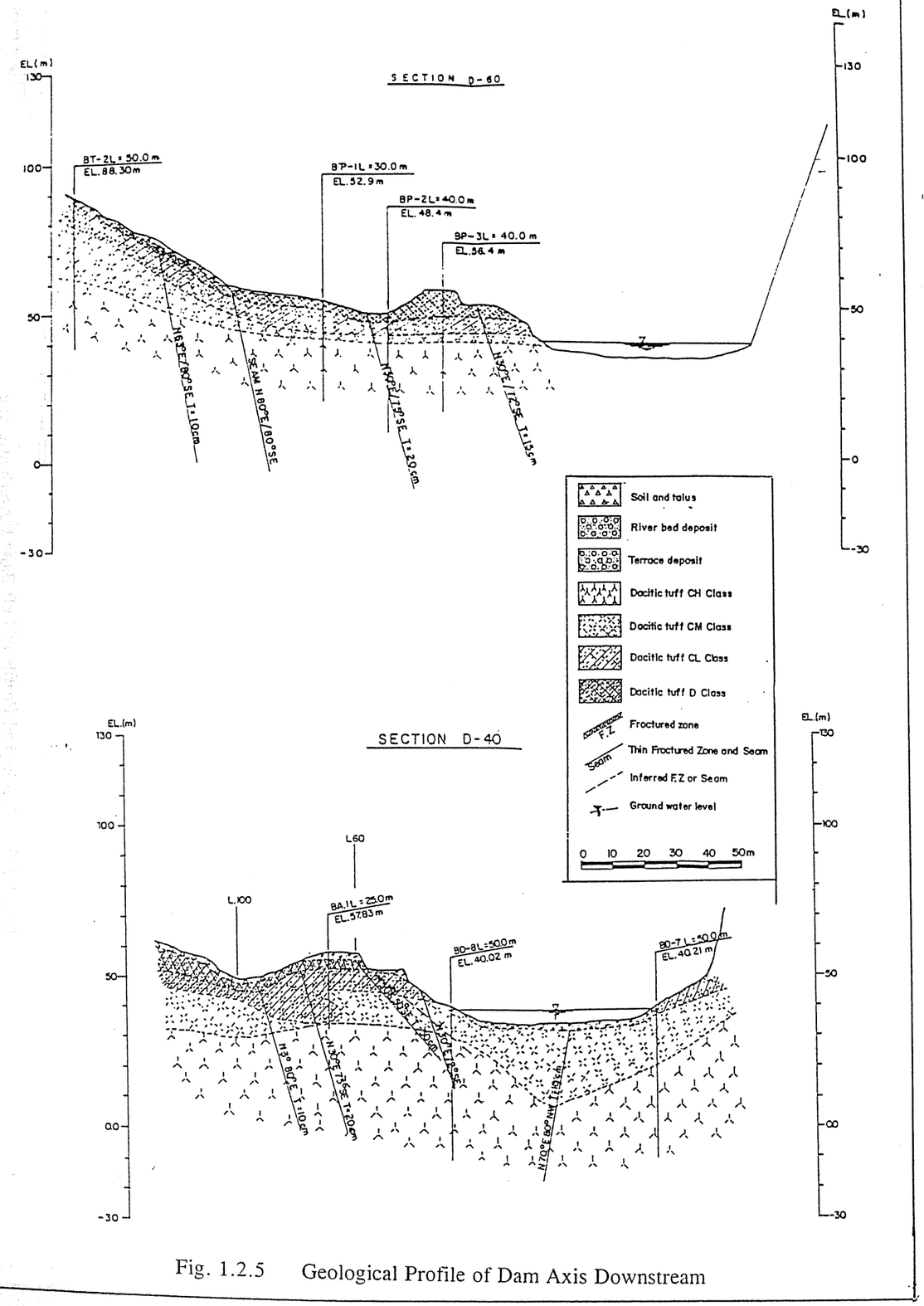


Fig. 1.2.5 Geological Profile of Dam Axis Downstream

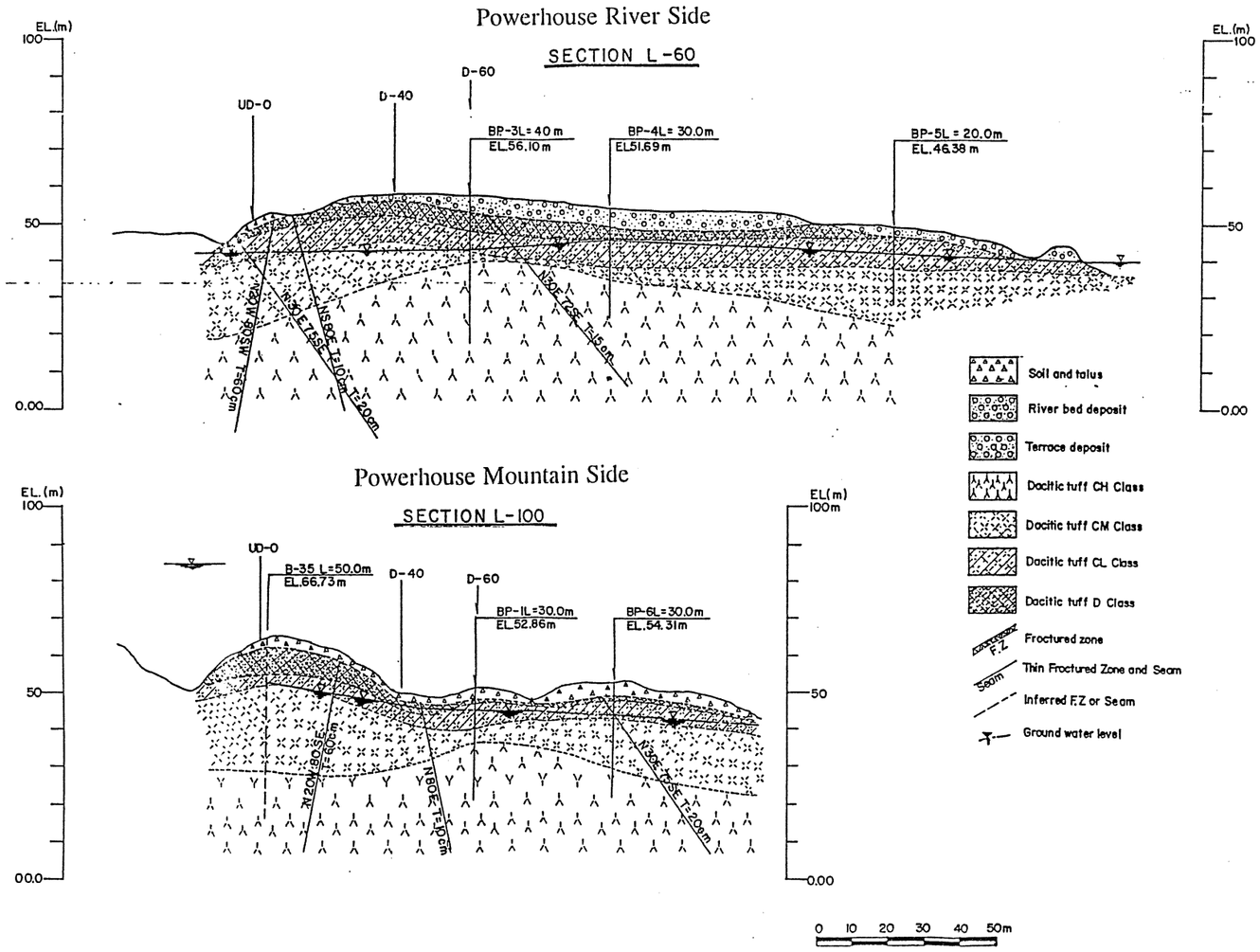


Fig. 1.2.6 Geological Profile of Left Bank Powerhouse Site

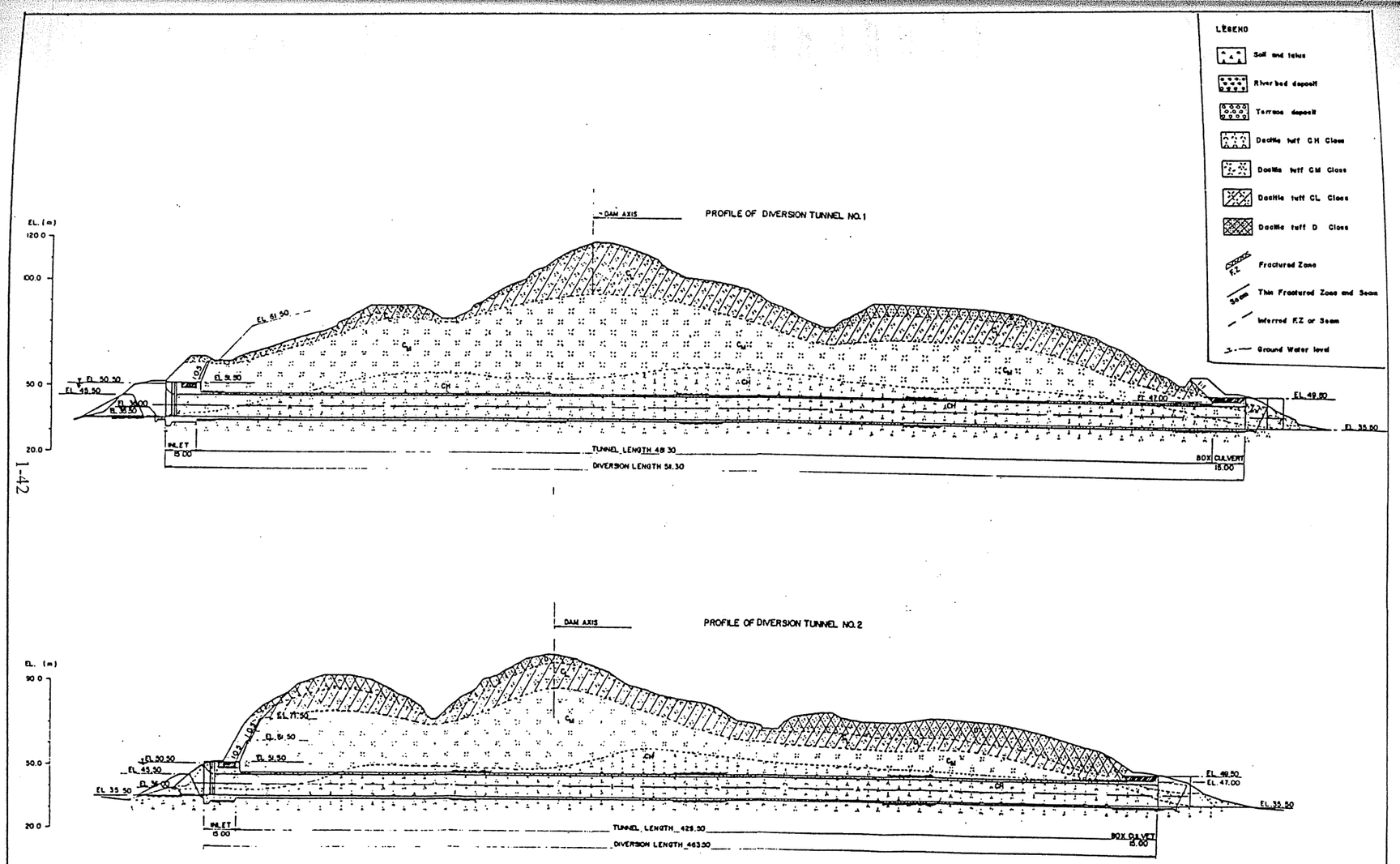


Fig. 1.2.7 Profile of Diversion Tunnel No. 1 and No. 2

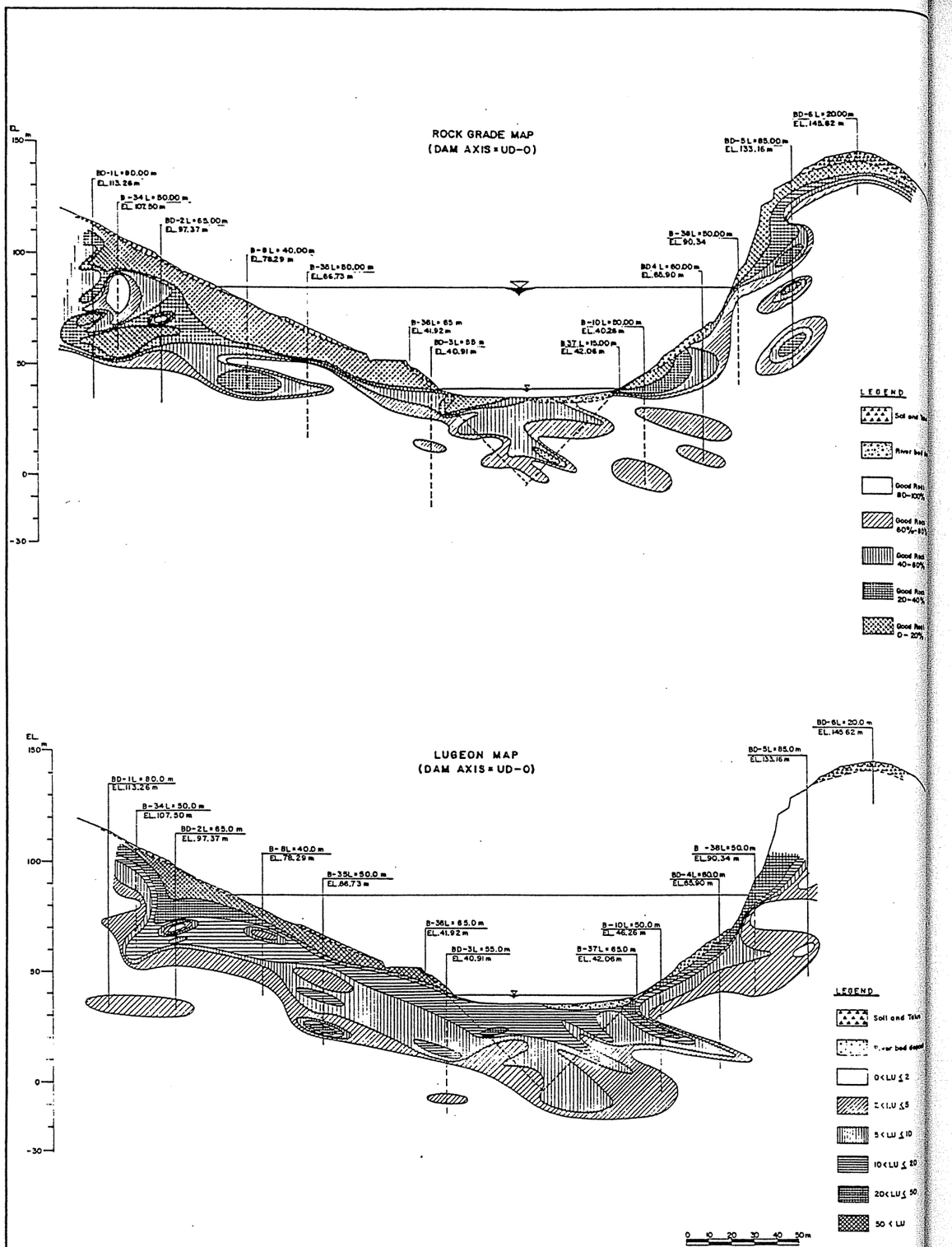


Fig. 1.2.8 Rock Grade and Lugeon Maps of Dam Axis Site

2.3.2 Gravel Pits

One quarry site and two gravel pits were selected mainly for the material of dam concrete. The quarry site is located about 10 km upstream of the dam, whereas the two gravel pits, Pulau Gadang Gravel Pit which is located about 3 km upstream of the dam site and Kuok Gravel Pit which is located about 8 km downstream of the dam.

In order to obtain data about the subsurface geological condition, the following investigations were made at the site.

- Drilling;	at the Quarry Site and Gravel Pits
- Test Pitting;	at the Gravel Pits
- Test Trenching;	at the Gravel Pits
- Test Aditting;	at the Quarry Site
- Seismic Prospecting;	at the Quarry Site
- Point Load Test;	at the Quarry Site
- Rebound Test;	at the Quarry Site
- Rock Mechanical Laboratory Test;	material from Quarry site.

The geological conditions of the above sites are as below:

(i) Quarry site

The main rock type at the Quarry site is granite, which has exposures of fresh rock in a small area. The rock has been greatly altered by surface weathering down to about 35 m in depth at the ridge of the mountain body, and strong solid granite exists underneath the weathered rock. The result of in-situ testing which consists of rebound test and point load test, and the result of laboratory tests can be found in the detailed design report.

(ii) Gravel Pits

The gravel pits are located at Pulau Gadang and Kuok. The concrete aggregate is produced from fluviatile deposits which are composed of sand and gravel. The result of investigation such as drilling, test pitting, and test trenching as well as concrete aggregate tests were reported in the detailed design report. The probable reserve of sand and gravel was estimated about 190,000 m^3 at Pulau Gadang Gravel Pit, whereas the sand and gravel bed covers approximately 350,000 m^2 and is 5 to 6 m thick at Kuok Gravel Pit.

Since material collection at the quarry site had proved to be uneconomical compared to collection at the gravel pits before the construction stage, the contractor planned to use the quarry site only in case the material quantity proved in sufficient at the gravel pits. During construction stage, it was found that the material quantity was enough at the gravel pits and therefore the quarry site was not used.

2.3.3 Switchyard, Transmission Line Route and Substation

The transmission line from the Kotapanjang Switchyard to Pekanbaru substation will be constructed over an extension of some 64 km, of which about 28 km will be situated in the mountain and hill area, and the remaining 36 km in the plain and swamp area.

In the mountain and hill area between the dam site and Rantau Berangin bridge located 4 km downstream, the bedrock is weathered dacitic tuff and the surface is covered by clay of maximum thickness about 3 m. The dacitic tuff is in the form of clay containing fragmental rocks. The plain area from Rantau Berangin to Air Tiris 30 km downstream is composed mainly of silty clay and gravelly sand. The bearing capacity of the gravelly sand layer is generally recorded sufficient for tower foundations of standard spread types.

Along the Kampar Kanan river towards the Pekanbaru substation, the swamp area is covered by peat and clay layers, and the well compacted gravelly sand layer is as much as 10 m below the ground surface at some locations. The tower foundations need suitable support by piling or substitution of soil beneath the embedded tower foundations.

The switchyard site is located on the crest of the mountain ridge at 150 - 180 m at EL above the dam and powerhouse site and slope of ridge about 30° to 50°. This area is underlain by Dacitic Tuff which can be divided into completely weathered, to highly weathered, and highly weathered. The field investigations carried out at the switchyard site are :

- Drilling
- Standard Penetration Test
- Dilatometer Test (Lateral Loading Test)
- Electrical Resistivity Logging

Location of Pekanbaru S/S was moved approximately 5km from original location because of very week soil at original location. Soil investigation was carried out at 3 locations inside substation area and one location at tower No. 194. For the soil investigation, boring, standard penetration test (SPT) and sampling of undisturbed soil and one set of laboratory test have been carried out. Total length of boring was 120m. As the result of the investigation, it was found that the area was covered by silty sand layer up to about 20m depth and the soil of GL-5m to -10m was weak. N value was less than 10. Ground water level was GL-13m.

2.3.4 Relocation of Road and Bridges and Reservoir Area

The relocated road is situated in a wide belt of Paleozoic carboniferous and Tertiary sedimentary rocks forming the southwest barisan geanticline. Along the mountain and hilly area, the relocated road route is generally covered by silt, sand, or clay layers or the bedrock of CL - class or higher with fractures. In the level area, the thickness of residual soil and completely weathered rock of mudstone and sandstone reaches up to 5 m.

The sites for the bridges are covered by sand and gravel of thickness 5 - 9 m at proposed Kampar Bridge site, and 7 - 12 m at the proposed Gulamo Bridge site, with bedrock of sandstone, shale or phyllite of Kuantan Formation.

In general, landslides that pose the most serious threat to the safe operation of a reservoir are large-volume, high-velocity types such as rock slides, rock falls, debris flows and debris avalanches.

The investigation of the reservoir area was conducted by the aid of helicopter and air-photograph interpretation in order to observe any mass movements phenomena such as landslides.

The reservoir rim consists of low relief hills generally ranging in altitude from 50 to 300 m. The area is occupied by the Kuantan and Bohorok Formations of Paleozoic age and the Pematang, Sihapas, and Telisa formations of Tertiary age. The Paleozoic Formations consist mainly of Shale and Sandstone with steep dip 50° to 80°, whereas the tertiary formation consist mostly of poorly stratified sandstone with gentle dip of 5° to 10°.

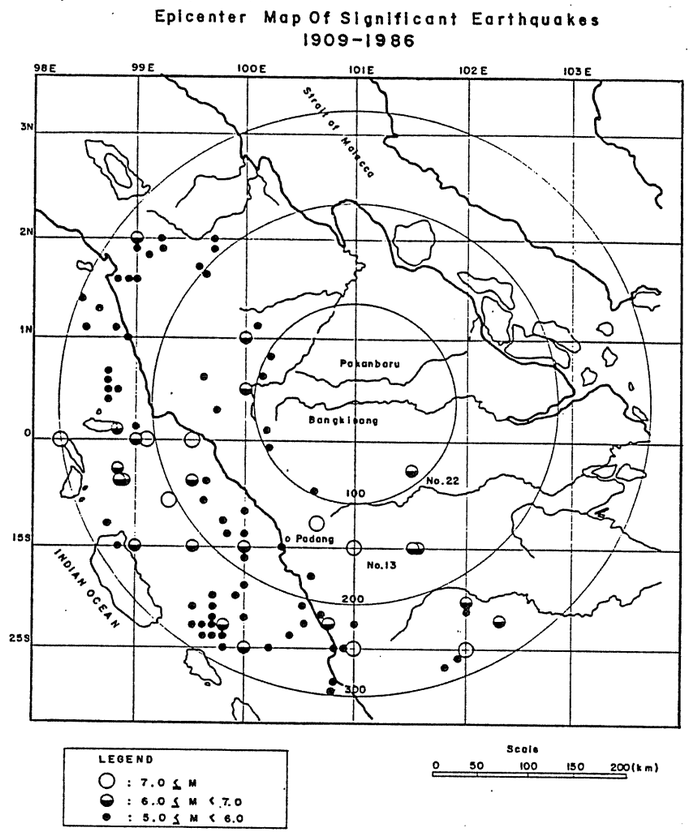
Based on the result of this investigation, no existing and dormant landslide was discovered with exception of one existing landslide situated about 4 km north of Tanjung Balit, but it will not affect the planned reservoir.

2.3.5 Seismicity and Seismic Force

The dam site is situated in longitude 100.878 E and latitude 0.289 N. In other words, the dam site is located in rather seismically active "Eurasian Seismogenic Zone". Based on the available data obtained from the Meteorological and Geophysical Institute, Jakarta for 78 years (1909 to 1986) the epicenters of seismic disturbances were mostly under the sea.

The design seismic intensity was determined based on the statistical data calculated from the seismic observation records at various areas in central Sumatra. The maximum acceleration in the past record which could be applied to the project site was determined by comparative study of various formula and Okamoto's Formulae was selected among other formulae.

To determine the design seismic coefficient, the probable acceleration was calculated for the return periods of 200 and 500 years. From the result of calculations, the design seismic coefficient for the structures of the Kotapanjang project was 0.08 for a 200 years and for a 500 years return period respectively.



The following table shows the Technical Standard Pertaining to Hydroelectric Power Facilities based on the intensive and weak earthquake zones for dam types. (Ordinance Governing the Technical Standards Pertaining to Hydroelectric Power Facilities of the Ministry of International Trade and Industry (MITI)). In considering this standard and the result at value by Okamotos formula the design seismic coefficient of 0.10 was determined for use on this project.

Type of Dam	Concrete Gravity Dam and Concrete hollow gravity dam	Arch Dam	Fill Dam	
			In case nearly homogeneous materials are used for levee	In other cases
Intensive earthquake zone	0.12	0.24	0.15	0.12
Weak earthquake zone	0.10	0.20	0.12	0.10

2.3.6 Permeability of the Dam Foundation

Before the construction stage, permeability tests (Lugeon) of the rock-mass were carried out by the packer method at 5 m intervals in boreholes, or other intervals depending on the rock condition. A Cycle system of pressure application was used during the tests, where pressures used were as follows :

P (kg/cm^2)	0.5-1-2-3-4-5-7-10-7-5-3-1.5	For depth less than 15 meters
	1-2-3-5-7-10-7-5-3-1-1.5	For depth more than 15 meters

Permeability values obtained during the investigation of the dam abutments are shown in the table below. This table indicates that the value of rock mass permeability tends to be small at the lower part of the foundation and becomes large in the upper part. This is caused by (i) distribution of rock-mass discontinuities are more developed in the upper part than the lower part and (ii) open discontinuities and weathering are found in the upper part.

Table 1.2.7 Result of Lugeon Test of Dam Foundation Rock (l/min/m)

Depth (m)	LEFT BANK				RIGHT BANK		
	BD.1 (113.26)	BD.2 (97.367)	BD.3 (40.91)	BD.8 (40.02)	BD.4 (65.90)	BD.5 (133.16)	BD.7 (40.21)
0-5			18.2	8.2			0.4
5-10							
10-15		31	1.8	17	5.7		0.09
15-20		31.3	9.8	12.3	2.5		0.45
20-25		13	97	8.5	2		0.3
25-30		1.3	15.4	8.2	1.7		0
30-35		29	10.4	9.4	0.3		
35-40	1.6	3	0.8	5.5	0.32		0.9
40-45	7	4.2	0.4	1.6	0.13	0.6	0.3
45-50	0.3	1.5	2.5	2.7	1.5	5.8	
50-55	0.3	13.6	1.2		7.6	0.5	
55-60	0.6	1.2			1	1.1	
60-65	1.1	2.1				2	
65-70	0.9					2.7	
70-75	1.2					0.8	
75-80	2.8					4	
80-85						1	

During the construction stage permeability test, were carried out in the foundation for the foundation treatment operations, that is consolidation grouting and curtain grouting. The consolidation grouting was carried out for the purpose of reducing the permeability, improving the bearing capacity of dam foundation rock, and enhancing the efficiency of the grout curtain. Meanwhile, the objectives of the curtain grouting were to restrain outflow from a reservoir by control of seepage through foundation rock, and to prevent the foundation from piping by controlling seepage, especially for concrete dams, and reduce uplift pressure.

The grouting design and the work during construction are described in detail in Chapter 4 Design of Main Structure and Part 2 Civil Works - Construction.

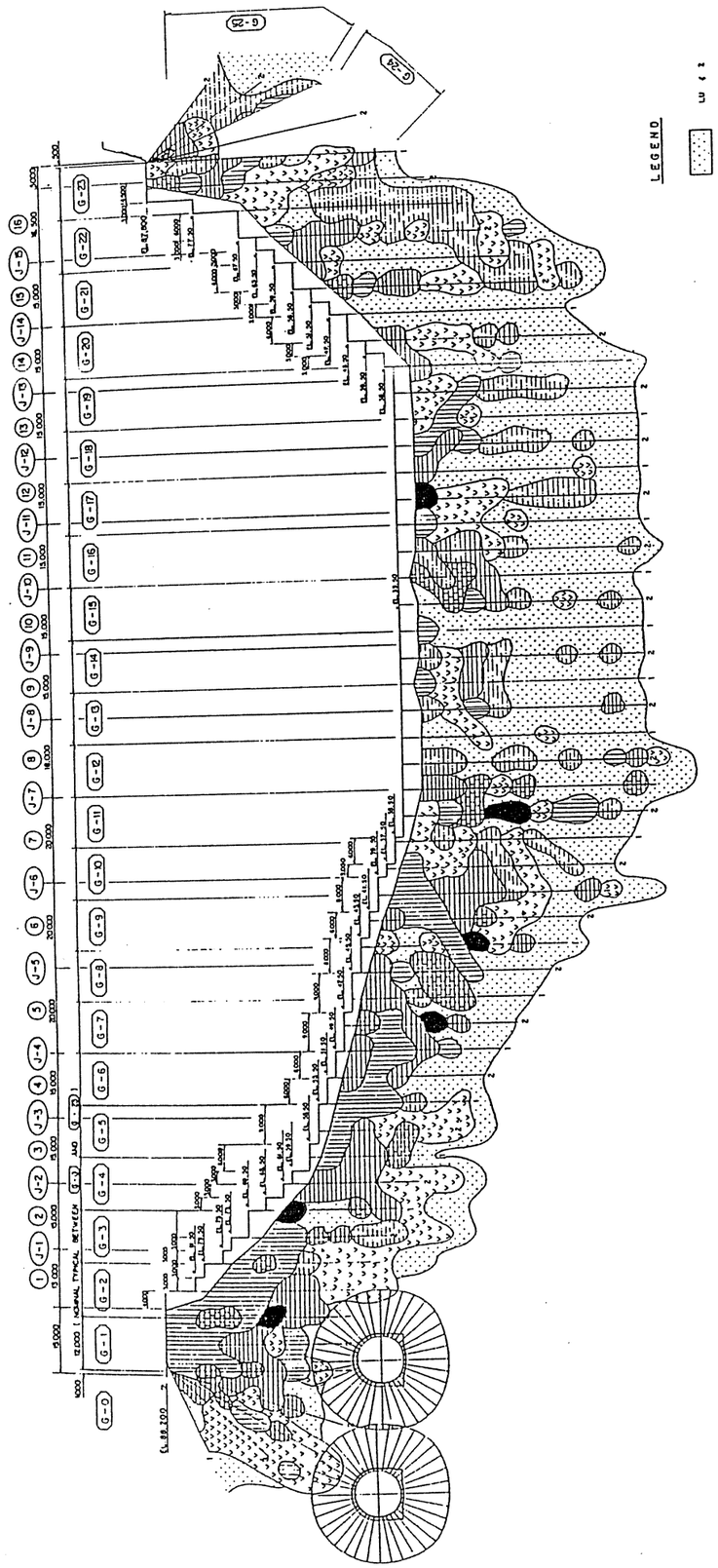


Fig. 1.2.9 Lugeon Map of Dam Foundation Rock

2.4 Geological Structure of Dam Site

The geological structure of the dam site has been exposed in the excavations of the foundation for dam, powerhouse and tailrace, stilling basin, and diversion tunnels. Geological maps of each foundation were prepared during construction through rock foundation examination prior to concrete placing.

2.4.1 Dam Foundation

The dam foundation consists of the left bank abutment, the river bed, and the right bank abutment. The geological structure of each of these sections is as follows :

(1) Left Bank Abutment

The geological structure of the left bank abutment are as shown in Fig. 1.2.10. (J0-J13). Several discontinuities of rock-mass distribute in form of joints, clay-filled joints or seams, quartz veins, fracture zones, and brecciated zones.

Based on the indication mentioned above, the left bank area has already been altered by a hydrothermal process in the past. Moreover, a brecciated zone indicates that this area was crossed by a fault which is categorized as an inactive fault and is not a main fault at present. The thickness of this brecciated zone is approximately 2 m and it is oriented north-south and steeply dipping towards the east. There are two orientations of the rock-mass discontinuities, in general they are north-south and west-east orientation.

According to equal area stereographic projections for the cleft direction, the discontinuity of the rock-mass is generally classified into 1 (one) set of joints. This joint set is a group of 5.5 percent of rock-mass discontinuities which is steeply dipping towards the south and distributed in wide spacing of continuous type.

The rock classification of the rock-mass, in general, belongs to CM class, CH class of rock is however encountered in some locations. One location of ground water seepage coming out of the foundations was found at the lowest part near the dam axis.

Geological features are summarized as follows :

Weathering	Fresh to slightly weathered
Solidity	Hard to very hard
Discontinuity spacing	1 - 2 m
Tightness	Close-open (mostly filled by foreign material)
Rock classification	CM (mostly) - CH (localized)

(2) River Bed

The geological structure is as shown in Fig. 1.2.11 (J6-J13). The discontinuities of rock-mass are similar to the left bank, however there are fewer fracture zones and one new brecciated zone was encountered with an orientation of north-east to south-west, steeply dipping towards the south-east. The thickness of this brecciated zone is approx. 2 m in the upstream, increasing to more than 8 m downstream of the dam axis. Another brecciated zone occurred in the upstream left side corner of the new one has the developed from the brecciated zone in the lower left bank.

According to equal stereographic projections for the cleft directions, 1 (one) joint set resulted. This joint set is a group of about 4 percent of rock-mass discontinuities with a north-east to south-west orientation and vertical dipping. Ground water seepages from the foundation were found, five seepages in the upstream foundation and five seepages in the downstream, of which two are near the dam axis.

Geological features are summarized as follows :

Weathering	Fresh to slightly weathered
Solidity	Hard to very hard
Discontinuity spacing	1 -3 m
Tightness	Close to open (filling material)
Rock classification	CM to CH (mostly)

(3) Right Bank Abutment

The discontinuities of rock-mass are generally in the form of joints (closed discontinuity), as shown in Fig. 1.2.12 (J13-J16). Equal stereographic projections for the cleft directions reveal 3 (three) joint sets. The joint sets develop fairly steep to very steep towards the north, south-west, and north-east directions of 3 percent of rock-mass discontinuities. There are common indications of ground water seepage coming out of up from the joints, and they are located at the upstream and downstream.

Geological features are summarized as follow :

Weathering	Fresh to slightly weathered
Solidity	Hard to very hard
Discontinuity spacing	more than 5 m
Tightness	Closed (in general)
Rock classification	CH (mostly) - CM (localized)

2.4.2 Powerhouse and Tailrace

The geological condition of the powerhouse and tailrace is shown in Fig. 1.2.13. The rock-mass discontinuities have been altered by hydrothermal process. Fracture zone with slicken side at the slope side are situated at the left of the control building area. This indication is related to the lower left bank abutment where brecciated zones occur.

Equal area stereographic projections for the cleft directions shows 1 (one) joint set which dips steeply towards the north-west and north-east to south-west direction. The joint set is a group of 9 percent of rock-mass discontinuities with a wide spacing. Ground water seepage is only found at the tailrace chute in three locations.

Geology features are summarized as follows:

Weathering	Fresh to slightly weathered
Solidity	Hard to very hard
Discontinuity spacing	more than 5 m
Tightness	Closed
Rock classification	CH (mostly) - CM (localized)

2.4.3 Stilling Basin

Geological structure of the stilling basin is relatively similar to the river bed of dam abutments as shown in Fig. 1.2.14. One brecciated zone passes through from upstream to downstream and is the developed brecciated zone of the dam river bed foundations. The orientation of the brecciated zone is north-east to south-west and it is steeply dipping towards the south-east. The thickness is approx. 13 m at the upstream and becomes small towards the downstream. Fracture zones are present with clay-seams concentrated to the centre around 20 - 50 cm thick. Quartz veins are abundant at the downstream portion.

According to the equal area of stereographic projections for the cleft directions, 2 (two) joint sets result from around 7 percent of rock-mass discontinuities. These joints sets are a group of rock-mass discontinuities which have north-west to south-east and north-east to south-west orientations and are steeply dipping towards the south-west and south-east. The 2 (two) ground water seepage paths follow the brecciated zone.

Geology features are summarized below :

Weathering	slightly to moderately weathered
Solidity	hard to moderately hard
Discontinuity spacing	more than 5 m
Tightness	closed - open (predominantly)
Rock classification	CM (mostly) - CH (localized)

2.4.4 Diversion Tunnels

The 2 (two) diversion tunnels, which have lengths of 478.8 m and 423.0 m, are located in generally CH class rock. According to the geomechanic classification shown in Table 1.2.8, the main geological classification is good rock with rating of 80-61, Class II. There are 8 (eight) locations of ground water percolation coming down from the roof and right wall. Over break has occurred at STA 370 in diversion tunnel no. 1. Geological features are summarized as below :

Weathering	Fresh to slightly weathered
Solidity	Hard to very hard
Discontinuity spacing	1 - 2 m
Tightness	tending to closed
Rock classification	CH (mostly) - CM (localized)

Table 1.2.8 CSIR Geomechanic Classification of Jointed Rock Mass (Bieniawski 1984)

1. RATING FOR STRENGTH INTACT ROCK MATERIAL

PARAMETER		RANGES OF VALUES						
Strength of intact rock material	Point load strength index	10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range uniaxial compressive test is preferred		
	Uniaxial compressive strength	250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	<1 MPa
Rating		15	12	7	4	2	1	0

2. RATING FOR R.Q.D

Drill core spacing	90% - 100%	75% - 90%	50% - 75%	25% - 50%	25%
Rating	20	17	13	8	3

3. RATING FOR SPACING OF DISCONTINUITIES

Spacing of joint	> 2 m	0.6 - 2 m	0.2 - 0.6 m	60 - 200 m	< 60mm
Rating	20	15	10	8	5

4. RATING FOR CONDITION OF DISCONTINUITIES

Condition of joint	Very rough surface Not continuous No separation	Slightly rough surface Separation < 1 mm	Slightly rough surface Separation < 1 mm	Slickensided surface Gouge > 5 mm thick Joints open 1-5 mm	Soft gouge > 5 mm thick Joints open > 5 mm
Rating	30	25	20	10	0

5. RATING FOR GROUNDWATER

Ground water	Inflow per 10 m tunnel length	None	< 10 ltr / min	10 - 25 ltr/min	25 - 125 ltr/min	> 125 ltr/min
	Ratio: Joint water pressure / Major principal stress	or 0	0.0 - 0.1	or 0.1 - 0.2	or 0.2 - 0.5	or > 0.5
	General condition	or Completely dry	Moist. (interstitial water)	or Wet	or Water under moderat pressure	or Severe water problems
Rating		15	10	7	4	0

6. RATING FOR ADJUSTMENT TO DISCONTINUITY ORIENTATION

Strike and dip orientation of joint		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Rating	Tunnels	0	-2	-5	-10	12
	Foundation	0	-2	-7	-15	-25
	Slope	0	-5	-25	50	-60

THE EFFECT STRIKE AND DIP ORIENTATION IN TUNNELING

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0° - 20° Irrespective of strike
Drive with dip		Drive against dip				
Dip 45° - 90°	Dip 20° - 90°	Dip 45° - 90°	Dip 20° - 45°	Dip 45° - 90°	Dip 20° - 45°	
Very favourabl	Favourable	Fair	Unfavourable	Very unfavourable	Fair	Unfavourable

7. TOTAL RATING 1 2 3 4 5 6

8. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 21
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
MEANING ROCK MASS CLASSES					
Class No.	I	II	III	IV	V
Average stand-up time	10 Years for 15m span	6 Month for 8m span	1 Week for 5m span		
Cohesion of the rock mass	> 400 KPa	300 - 400 KPa	200 - 300 KPa	100 - 200 KPa	< 100 KPa
Friction angle of the rock mass	> 45°	35° - 45°	25° - 35°	15° - 25°	< 15°

2.4.5 Geological Structure of Curtain Grouting Line

Geological features, in general, along curtain grouting line were interpreted from observations of core taken from primary holes, as mentioned below :

(i) Fracture zones and brecciated zones were observed at several locations, especially in the lower left bank and river bed area.
(ii) Open discontinuities of rock-mass distribute in shallow depth (around 25 m from surface) and tend to close at greater depth.
(iii) CM-class rock was observed at shallow depth, while CH-class rock was at greater depth.
(iv) Some iron-staining was observed in the rock-mass discontinuities at shallow depth.
(v) Fine cracks due to blasting effect were encountered in the upper part near the surface.

2.4.6 Modification of Excavation Line

The modification of the design excavation line of dam site made in several locations due to the geological condition. The modified locations are situated at the following areas:

(i) <u>Powerhouse Area (EL. +45.00 to +35.60)</u> : Vertical slope (original design) was changed to the a steep slope of 1 : 0.625. The volume of removed rock material was about 320 cubic meters.

(ii) <u>Dam Body in Left Bank (EL. +46.362 to +35.600 and EL. +41.000 to +35.600)</u> : The dam body that is located in this area should be founded on competent rock. The excavation had to continue deeper to reach suitable rock for the dam body. Approximately 2800 cubic meters of rock material was removed below the design foundation line.

(iii) <u>Dam Body in river bed (EL. +29.50 to +28.00) and Stilling Basin (EL. +30.50 to +28.00)</u> : Due to the fact that a river bed gravel deposit and completely weathered rock were still found at the elevations of 29.50 and 30.50 (according to the design requirement), these materials had to be removed.
Suitable rock condition was reached by elevation 28.00 m.

(iv) <u>Control Building Foundation (EL. +35.60)</u> : The foundation of control building no. 3 is designed to be EL. +35.00, but CL to D class rock was still encountered in the upper part. In order to reach the suitable rock condition, the bed rock had to be removed an additional half meter below the design foundation line.

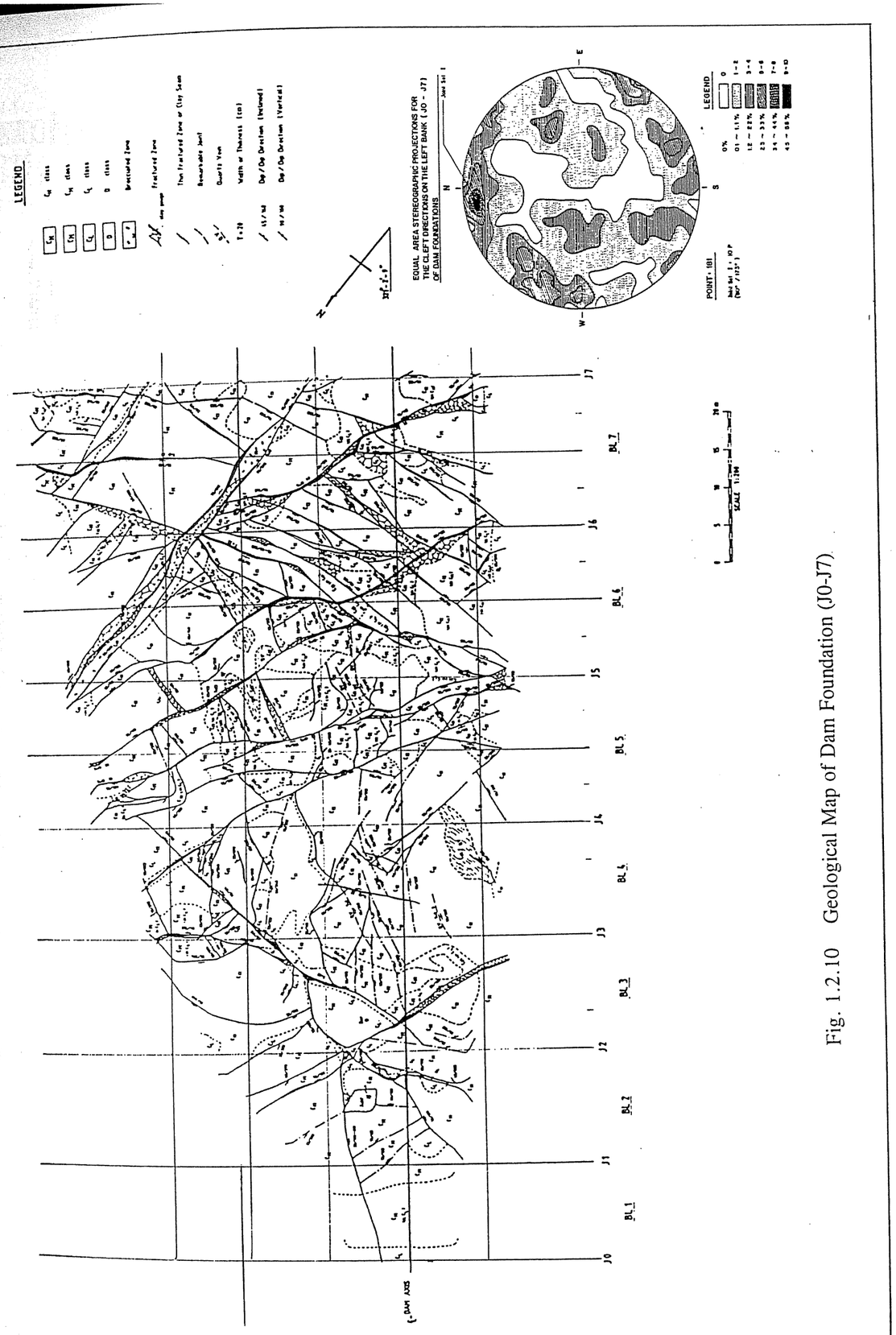
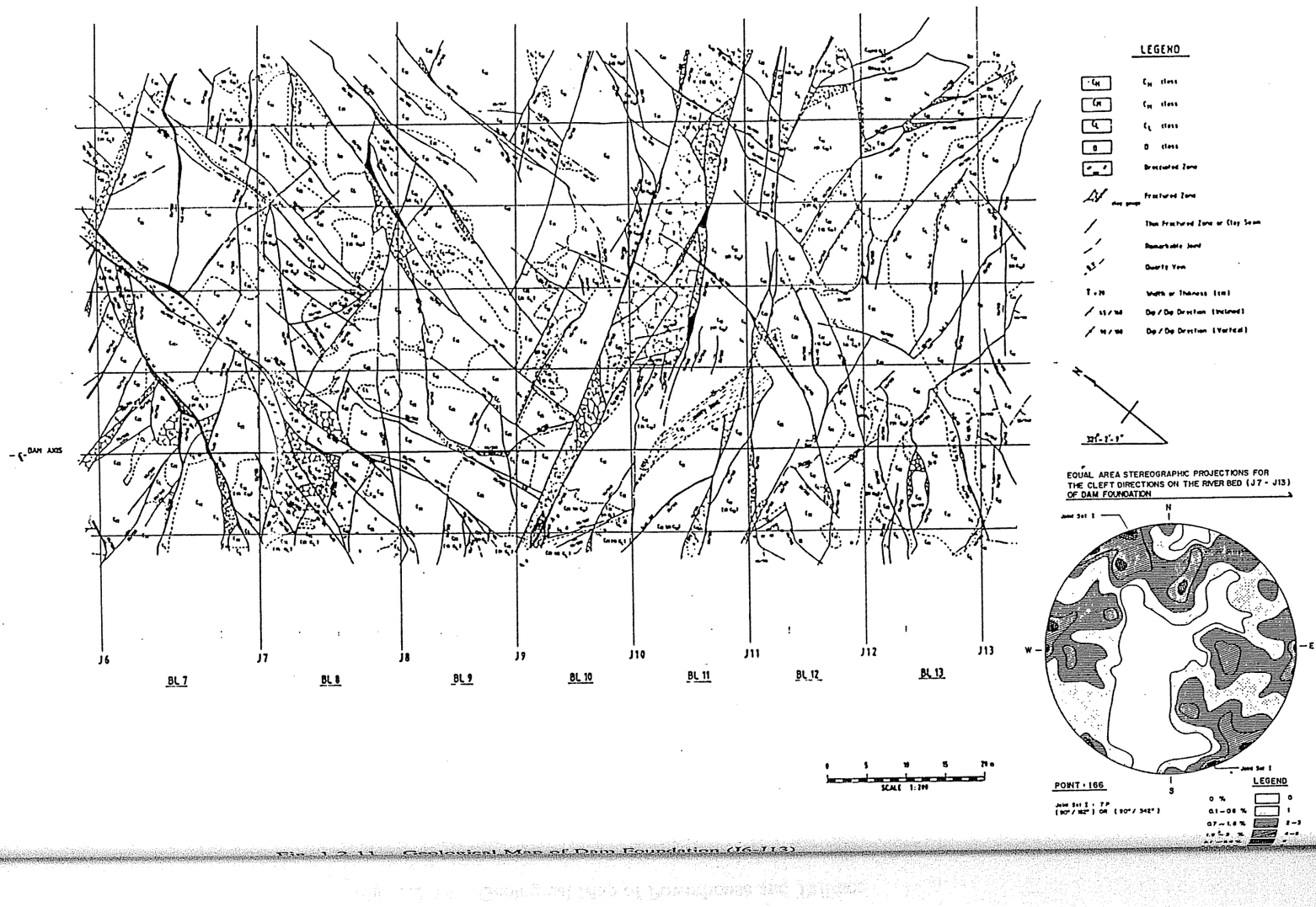


Fig. 1.2.10 Geological Map of Dam Foundation (J0-J7).



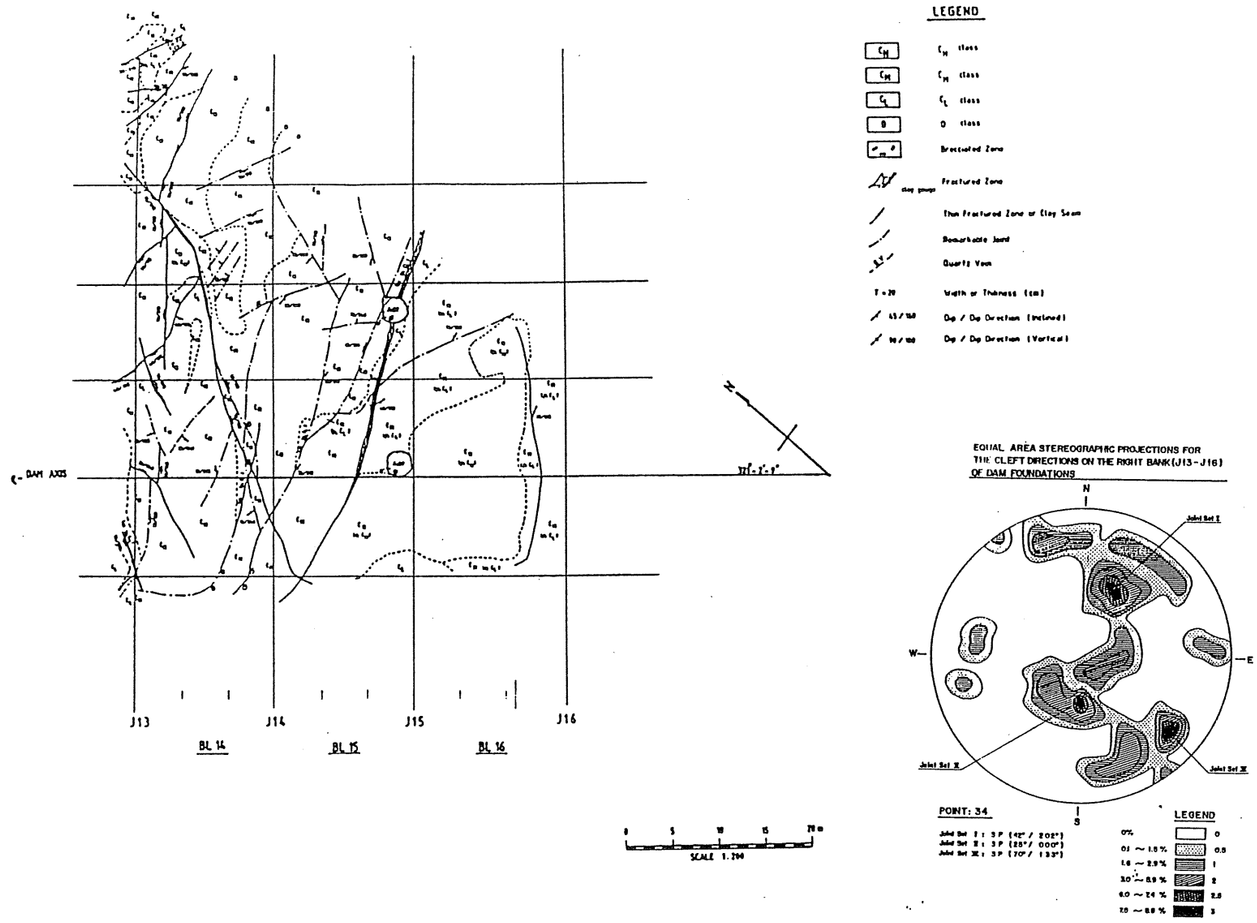


Fig. 1.2.12 Geological Map of Dam Foundation (13-J16)

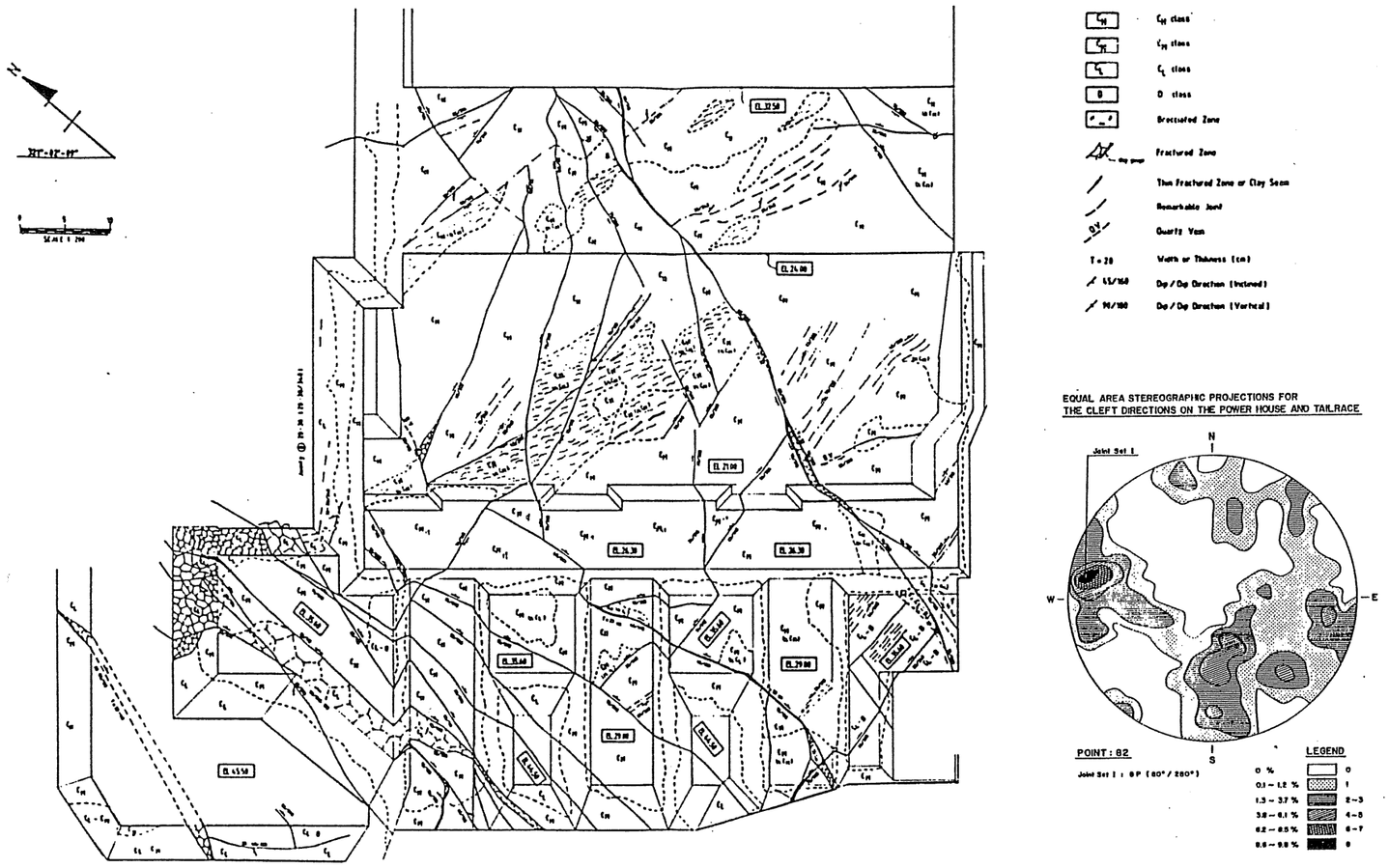


Fig. 1.2.13 Geological Map of Powerhouse and Tailrace

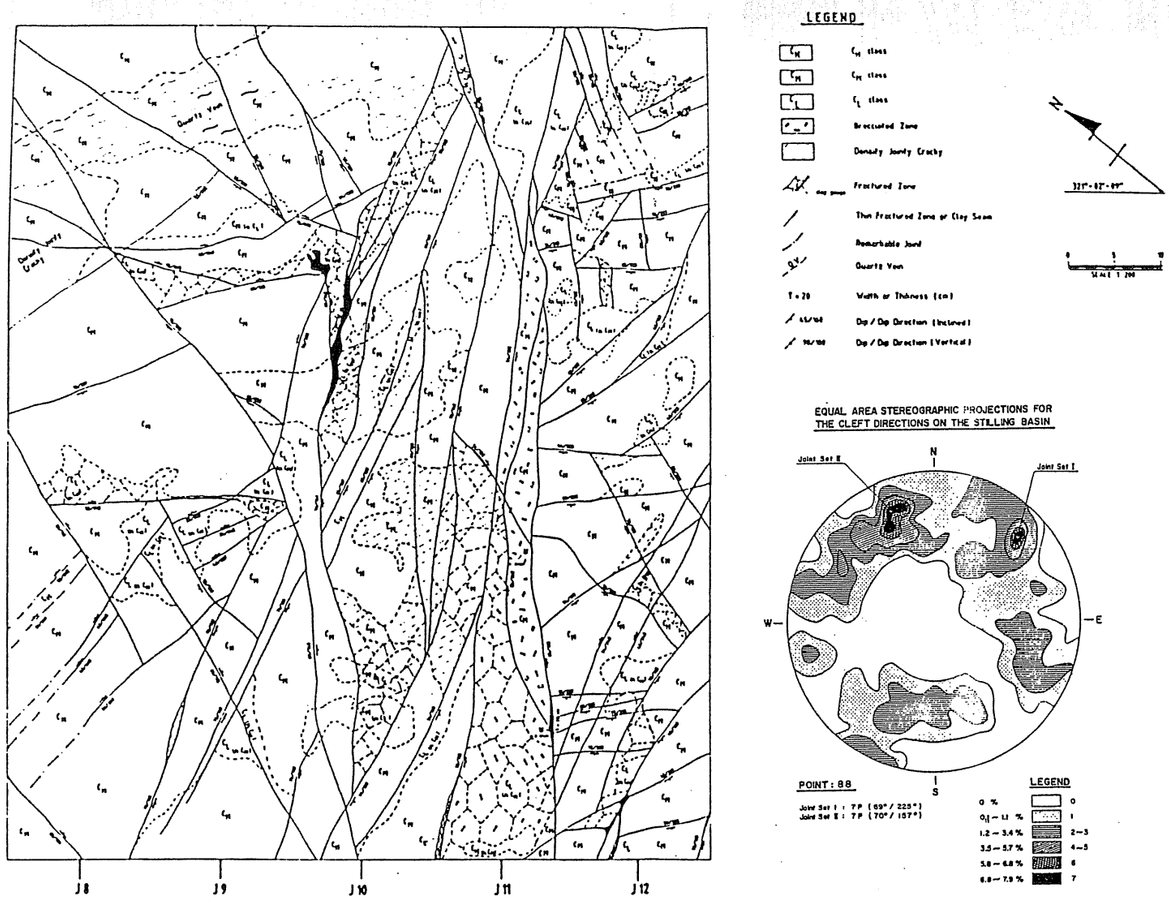
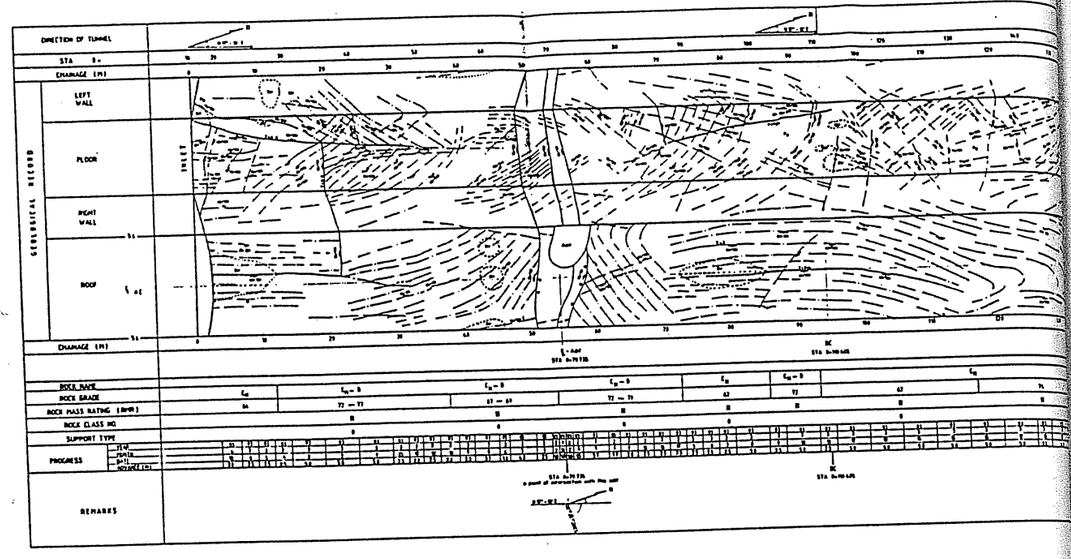


Fig. 1.2.14 Geological Map of Stilling Basin

Fig. 1.2.15 (1) Geological Map of Diversion Tunnel No. 1

From 0 m (Tunnel Inlet) to 135 m



From 135 m to 275 m

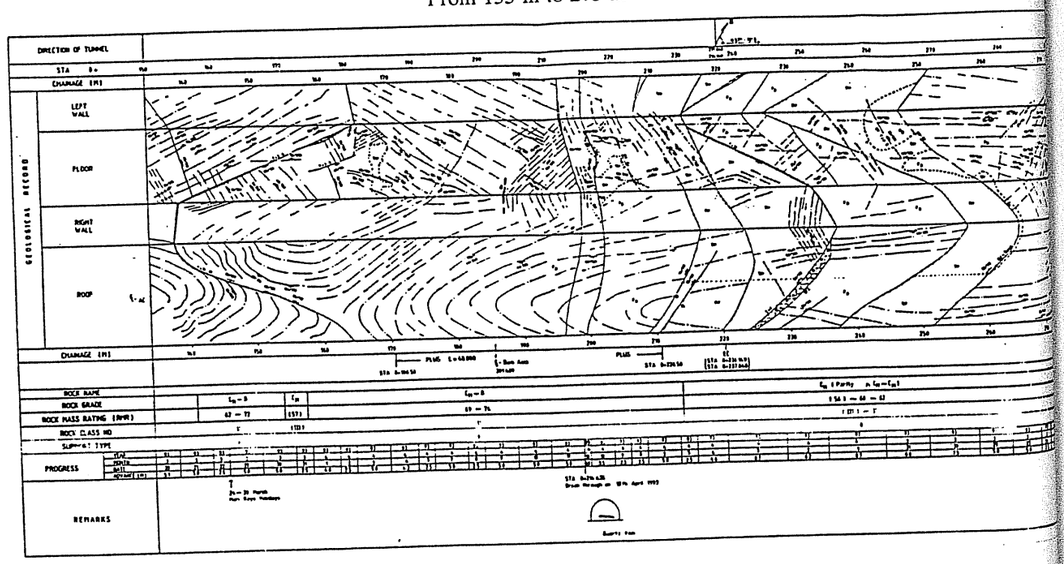
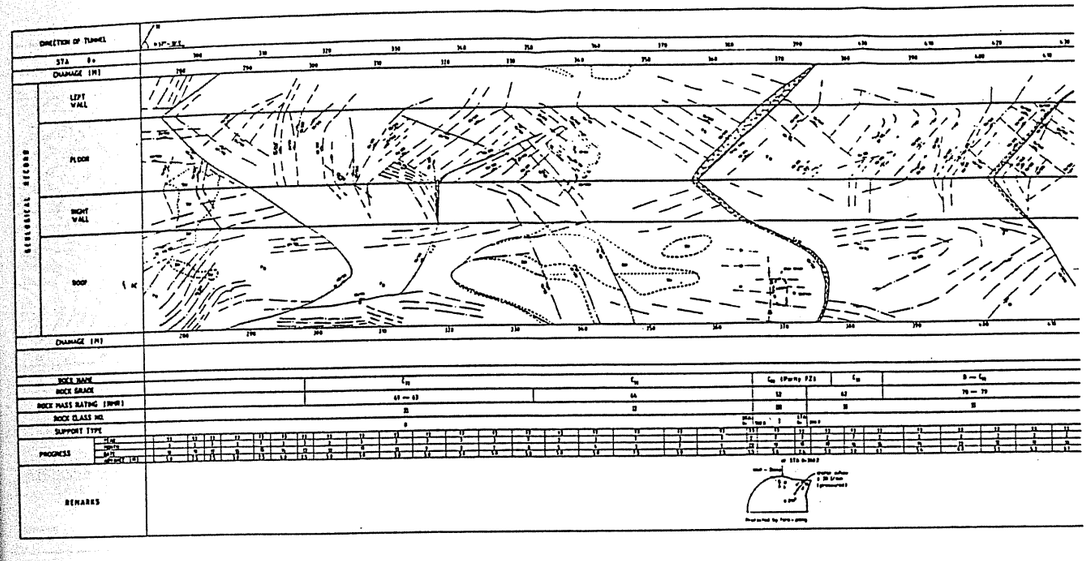


Fig. 1.2.15 (2) Geological Map of Diversion Tunnel No. 1

From 275 m to 415 m



From 415 m to 480 m (Tunnel Outlet)

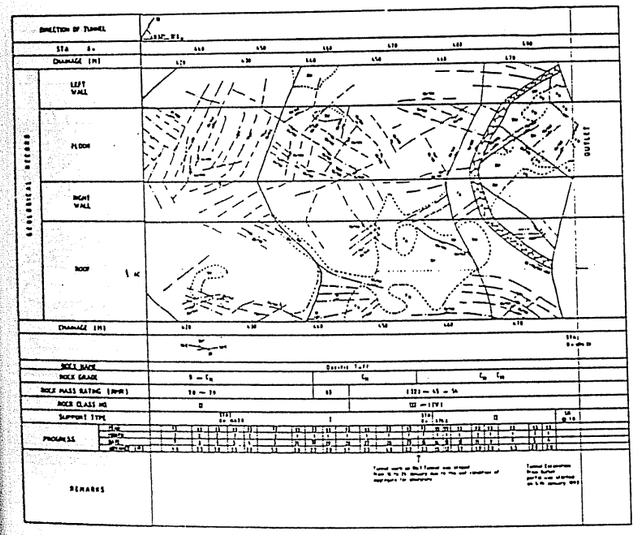
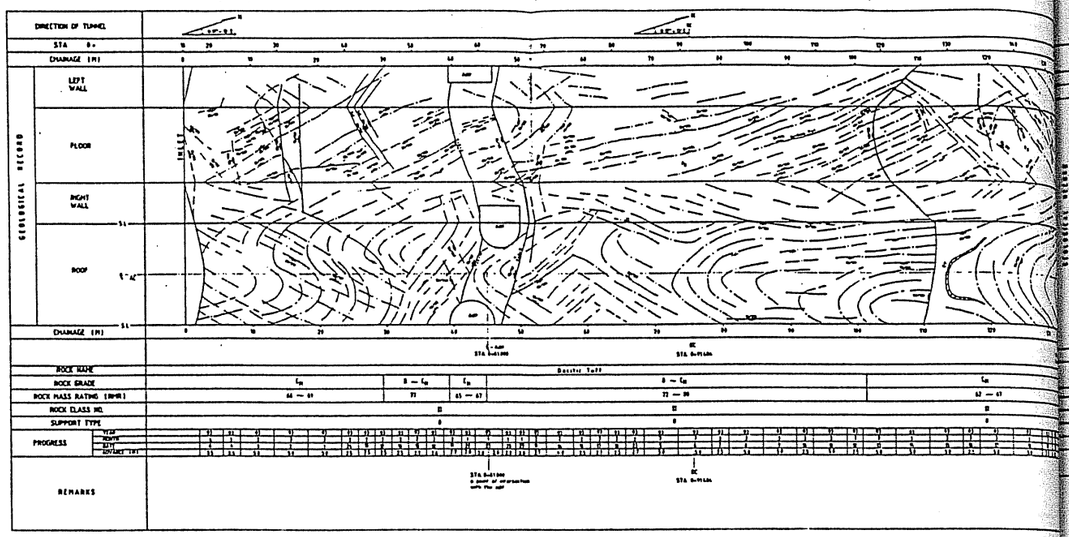


Fig. 1.2.16 (1) Geological Map of Diversion Tunnel No. 2

From 0 m (Tunnel Inlet) to 135 m



From 135 m to 275 m

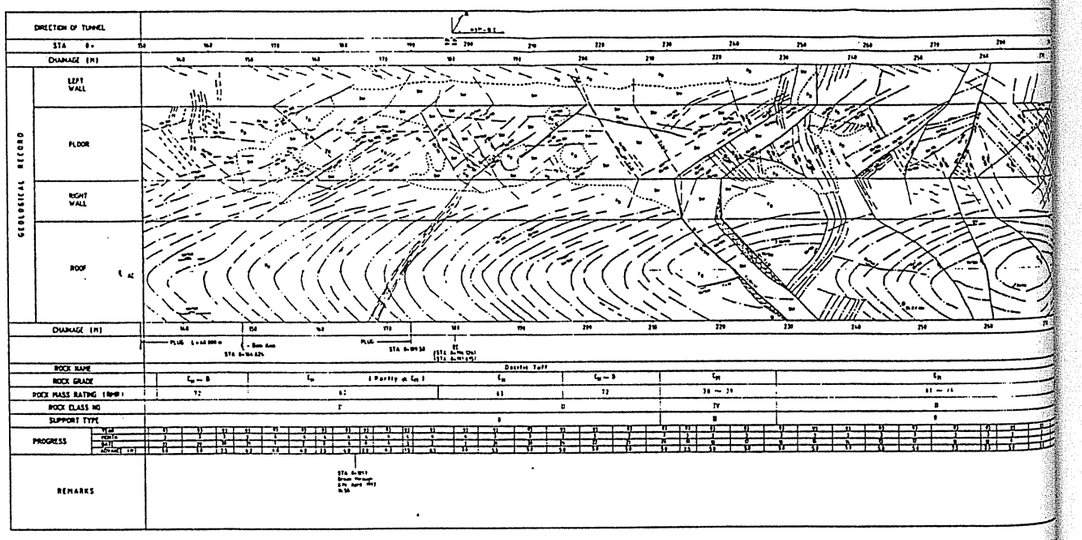
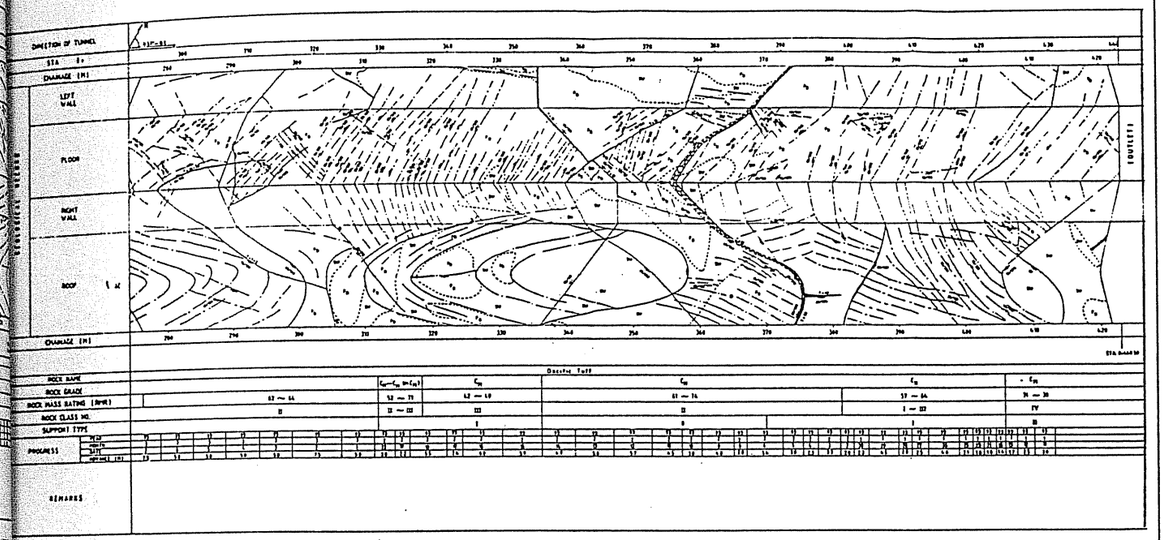


Fig. 1.2.16 (2) Geological Map of Diversion Tunnel No. 2

From 275 m to 425 m (Tunnel Outlet)



Chapter 3 HYDROLOGY

3.1 General

The first comprehensive hydrological study on the Kotapanjang catchment area was conducted by the Feasibility Study Team under control/financing by Japan International Cooperation Agency (JICA) from 1982 to 1984. The result of the study, based on the data up to 1981, was compiled in "The Feasibility Study Report on the Kotapanjang Hydroelectric Power Development Project" submitted to Perusahaan Umum Listrik Negara (PLN) in March 1984.

The feasibility report indicated that the annual average discharge at the dam site was estimated at 173.5 m^3/s (1971 - 1981), and the design flood for 200-year return period was 9000 m^3/s. The design sedimentation yield was estimated at 500 $m^3/km^2/year$. The feasibility study team also installed meteor-hydrological stations in the catchment area to obtain more detail data for further study.

In February 1987, the detailed design stage was conducted from February 1987 to August 1988. The main objective of the hydrological study for this stage was to supply meteorological data, to estimate the inflow data for the power generation plan, to revise the flood discharge for hydraulic structure design and to confirm the water quality for concrete mixture, hydro power installation and water supply for the camp facility during the constructions stage.

The main result of hydrological study in Engineering Design Stage was as follows;

- Rainfall
  Annual average rainfall in the catchment area : 2666 mm (1971-1986)

- Riverflow
  Annual average discharge at dam site : 184.4 m^3/s (1971-1986)
  Design flood discharge for spillway : 8000 m^3/s (200-year)
  Design flood discharge for river diversion : 1300 m^3/s (3 year)

- Sedimentation yield
  Annual sedimentation yield : 500 $m^3/km^2/year$
  Sediment level in the Reservoir : EL.64.0 (for 100 years)

- Water quality
  The water quality of the Kampar Kanan River at the dam site can be used for the concrete mixture and is not corrosive to the Hydroelectric Power Plant. The water supply for camp service is available at the proposed base camp site for the construction stage.

3.2 Climate of Project Area

(a) General

The project is located in the tropical zone where monsoon winds, heavy rainfall and high humidity with little variation in temperature dominate the climate. By these monsoon winds, there are distinct seasons, the wet season with the northeast monsoon from November to May and the dry season with the southwest monsoon from June to October.

During the wet season, the moist northeast monsoon causes heavy rainfalls on east slope of the Barisan mountains where the project area is located. On the other hand, rainfall decreases during the dray season with the southwest monsoon, since the project area is located in the lee side of this monsoon wind by the Barisan mountains.

(b) Rainfall

The annual average rainfall for the past sixteen years over the reservoir catchment area was estimated at 2666 mm. At Kotabaru on the Mahat River about 30 km southwest of the dam site, the average monthly rainfall reached 338 mm/month in December and decreases to 116 mm/month in August. The monthly rainfall fluctuation is large.

During the wet season, a heavy rainfall which often continues three to five days causes floods in the downstream area. Based on the collected data the maximum observed rainfalls were about 540 mm/5 day and 320 mm/day at Kotabaru. The maximum hourly rainfall was 78.5 mm/hr recorded in 1985, at Rantau Berangin about 4 km downstream of the dam site.

(c) Temperature

The annual average temperature in the project area is 27^0C. The monthly temperature varies very little throughout in the year. At Pasar Kampar 50 km downstream from the dam site, the monthly temperature fluctuates 27^0C in January to 28.4^0C in May. The daily temperature varies from about 35.9^0C to 20.8^0C in the project area.

(d) Relative Humidity

There is not a large variation in the average value of relative humidity throughout the year. At the Pasar Kampar relative humidity fluctuates from 79.9% in August to 86.2% in May. The annual average humidity is 84.0% approximately.

(e) Evaporation

Evaporation was measures by "Class A Pan" at several gauging stations near the project area. At Pasar Kampar, the annual average evaporation is 4.3 mm/day or 1564.7 mm/year. The monthly evaporation varies from 3.8 mm/day in January to 4.9 mm/day in March.

(f) Wind Velocity

The annual average wind velocity at Pasar Kampar is 0.382 m/s varying from 0.417 m/s in August to 0.298 m/s in December. The maximum monthly wind velocity is 0.78 m/s in November.

3.3 Field Investigation

The items of the site survey carried out under the feasibility study and the detailed design stage are as shown in the following tables. Among these survey items, the rain gauge stations and flow gauging station and the observation and measurement devices were installed and a hydrological survey was carried out.

Table 1.3.1 (1) Hydrological Survey Carried Out in Feasibility Study Stage

Survey Item	Survey Site	Contents of Survey
Field reconnaissance	Kampar Kanan River	By obtaining data and information pertaining to the Kampar Kanan River on vehicles, boats and on-foot, the survey policy was studied.
Installation of rain gauge stations	Kampar Kanan River	Three observation stations and one station were installed respectively along the Kampar Kanan River and Mahat River.
Installation of flow gauging station	About 4.5 km downstream of the dam site	One flow gauging station was installed in Rantau Berangin.
Discharge observation	Three locations	Rantau Berangin flow gauging station : establishment of discharge curve formula Immediately downstream of the dam site : establishment of discharge curve formula Rantau Berangin flow gauging station : check of discharge curve formula Lubuk Sipopay flow gauging station : check of discharge curve formula
Survey of the amount of suspended sediment	Rantau Berangin flow gauging station	Sampling of suspended sediment specimens and analysis thereof at the Bangkinang Office.
Water quality analysis	Rantau Berangin flow gauging station	Sampling of water specimens and analysis.

Table 1.3.1 (2) Hydrological Survey Carried Out in Detailed Design Stage

Survey Items	Survey Site	Contents of Survey
Meteorological Observation devices - Long-term Automatic Rain Gauge - Long term Manual Rain Gauge - Long term Mechanical Windvane Recorder - Long term Automatic Standard Psychrometer - Long term Manual Sun-shine Recorder - Long term Manual Floating Thermometer - Weather Hut - Plastic Water Tank	At Dam Site Camp Desa Merangin about 600 downstream of the dam axis.	Manual : 2 times daily Automatic : Weekly check
Water level Observation Device - Long term Manual Staff Gauge	At Desa Merangin on the left bank side of Kampar Kanan River about 200 m downstream of dam	2 times daily by manual devices
Flow-velocity Measurement Device - Current Meter (CM-1 B Type)	At Desa Merangin on the left bank side of Kampar Kanan River about 200 m downstream of dam	16 times for normal flow condition
Water Sampling Device - Simple Water Sample (US DH 48 Type)	Rantau Berangin about 4 km downstream of dam	5 times from September to December 1987

3.4 Flow Analysis

3.4.1 Low-flow Analysis

(1) Procedure Low-flow Analysis

The main objective of the low-flow analysis was to confirm water availability at the dam site. To complete this objective, a study of the low-flow analysis was carried out to estimate the stream-flow at the dam site.

The Low-flow analysis consists of i) Rainfall analysis, ii) Stream-flow analysis at Dam site and Danau Bingkuang and iii) Correlation analysis for stream-flow at Danau Bingkuang and Rantau Berangin.

Due to the lack of both rainfall data and stream flow data around the project area, correlation analysis and supplemental works was made to derive the river basin character.

(2) Rainfall Analysis

Up to the Detailed Design Stage, rainfall data from 24 rain gauging stations in Riau Province and West Sumatra Province were collected for the year up to 1986, as shown in Table 1.3.2 five (5) of these stations were installed during the Feasibility Study by the JICA Team and PLN. Since then, more rainfall data have been collected by PLN-LMK, in particular the hourly rainfall data.

During the Detailed Design Stage, 12 rain gauging stations, inside and outside the Kotapanjang reservoir catchment area were selected for estimation of the annual average rainfall, 3 additional rain gauging stations were also selected from the Danau Bingkuang catchment area located about 50 km downstream of the dam, as shown in Figure 1.3.1.

All these stations were considered from geographical and climatological point of views for these 2 catchment areas.

(a) Correlation among Stations

Rainfall data for the period of 1971 - 1986 were selected for estimation of the annual rainfall. In the case of missing records of some stations, rainfall correlation analysis was made by using a simple statistical method in order to derive the complete monthly mean rainfall of each station.

(b) The Annual Average Rainfall

The annual average rainfall from 1971 to 1986 was estimated for the dam site and Danau Bingkuang gauging station catchment. From the summary of the monthly mean annual rainfall, the annual average rainfall over 2 catchment areas were estimated by using the Thiessen polygon in order to derive rainfall depth.

The results of the rainfall depth at dam site is given as (R') and the rainfall depth at the Danau Bingkuang (R_{DB}) is obtained from a ratio of the catchment area by the following equation :

$$R_{DB} = (R_{DS} \times A_{DS} + R' \times A') / (A_{DS} + A')$$

Where ; R_{DB} : rainfall depth at Danau Bingkuang in mm
R_{DS} : rainfall depth at the Dam Site in mm
R' : rainfall depth at the residual catchment in mm
A_{DS} : catchment area of dam site in km^2
A' : residual catchment area in km^2

The result indicates that the annual average rainfall in Kotapanjang catchment is only slightly higher than that of the catchment at Danau Bingkuang gauging station.

Annual Average Rainfall

Location	Catchment	Annual Average Rainfall
Dam Site	3,337 km2	2,665.6 mm
Danau Bingkuang	4,035 km2	2,622.2 mm

(3) Stream-flow Analysis

The water level records and discharge measurements were collected for the year up to 1986, from 2 gauging stations on the Kampar Kanan River. The Danau Bingkuang gauging station, located about 50 km downstream of the dam site, was selected for the stream-flow analysis, since it has the longest record on the Kampar Kanan since 1977. The Rantau Berangin gauging station was selected for correlation analysis from stream-flow because it is located only 4 km downstream of the dam site, and has data since 1982.

(a) Run-off Studies

The daily discharge at the Dam Site for the years 1971-1986 were estimately by 2 methods, i) 1971-1976, by Tank Model and ii) 1977-1986, by the Danau Bingkuang stream-flow data.

The Tank Model was calibrated by using the observed hydro graphs at the Danau Bingkuang and the daily rainfall over the catchment area. The stream-flow data at the Danau Bingkuang have been studied by P3SA-PU, and the relation between discharge and water level is as follows :

$$Q = 26.306\ H2 + 131.985\ H + 12.164$$

where ; Q : discharge in m^3/s
H : water level in m

By using this equation, the daily mean discharge at Danau Bingkuang have been calculated. The annual average discharge from 1977 to 1986 at Danau Bingkuang (CA.4035 km^2) is 231.1 m^3/s consisting of the discharge from the dam site catchment area (CA.3,337 km^2) and the residual catchment area (CA. 698 km^2) between the dam site and Danau Bingkuang.

In order to derive the dam site discharge from the Danau Bingkuang data, it is necessary to estimate run-off from the residual catchment. The annual average rainfall of the dam site is estimated at 2666 mm, while it is 2522 mm in the residual area.

Concerning evaporation, the difference in topographic features of both catchment areas might affect the amount of evaporation which is essential factor for the water balance in the catchment areas. The dam site catchment is situated in a mountainous region, on the other hand the residual area is located in an alluvial plain where evaporation is higher.

Low rainfall and higher evaporation in the residual catchment indicates lower discharge from this area. However, for a conservative estimation for the dam site discharge from 1977 to 1986, the specific discharge from the dam site and the residual catchment was assumed to be 5.7 $m^3/s/100\ km^2$ recorded at Danau Bingkuang gauging station. Therefore, the discharge at the dam site was derived from the Danau Bingkuang data using the ratio of the dam site catchment to the Danau Bingkuang catchment as follows.

$$Q_{DS} = A_{DS} / A_{DB} \times Q_{DB}$$

where; Q_{DS} : discharge at the Dam site (m^3/s)
A_{DS} : catchment area of the dam site (3337 km^2)
Q_{DB} : discharge at the Danau Bingkuang (m^3/s)
A_{DB} : catchment area of the Danau Bingkuang (4035 km^2)

The annual average discharge at the dam site was estimated at 184.4 m^3/s.

(b) Double Mass Curve

In order to check consistency of the annual rainfall and the annual run-off at the dam site, a double mass curve analysis was carried out. Accumulated annual run-off and rainfall was plotted and the figure indicates that the annual run-off is closely related with the rainfall and run-off coefficient is 0.65. There is no substantial change in this rainfall relationship in the Kotapanjang catchment from 1971 to 1986.

(4) Correlation Analysis of Stream-flow

In order to check the daily discharge at the Dam site, correlation analysis was made for the Rantau Berangin and the Danau Bingkuang gauging stations.

(a) Discharge Data

The daily discharge compressing 625 data is available at both stations since 1982. Discharge data from both stations were converted to the Dam site by the ratio of both catchment areas, as follows :

$$Q_{DS}(X) = Q_{DB} \times A_{DS} / A_{DB},\ Q_{DS}(Y) = Q_{RB} \times A_{DS} / A_{RB}$$

where ; $Q_{DS}(X)$: Dam site discharge converted from the Danau Bingkuang (m^3/s)
$Q_{DS}(Y)$: Dam site discharge converted from the Rantau Berangin (m^3/s)
Q_{DB} : Danau Bingkuang discharge (m^3/s)
Q_{RB} : Rantau Berangin discharge (m^3/s)
Q_{DS} : Dam site catchment area (3337 km^2)
A_{DB} : Danau Bingkuang catchment area (4035 km^2)
A_{RB} : Rantau Berangin catchment area (3357 km^2)

(b) Lag Time between Stations

Daily discharge hydro graphs at both stations indicates about one day time lag therefore, one day lag time was given for the correlation analysis.

(c) Correlation and Regression Line

The correlation and regression line were obtained as follows,

$r_{XY} = 0.900$
$Q_{DS}(Y) = 61.506 + 0.6904 \times Q_{DS}(X)$ in m^3/s

where; r_{XY} : correlation coefficient
$Q_{DS}(X)$: dam site discharge from Danau Bingkuang
$Q_{DS}(Y)$: dam site discharge from Rantau Berangin

By this regression line, the annual average dam site discharge from 1977 to 1986 was estimated to be 193.3 m^3/s while the annual average dam site discharge from the Danau Bingkuang was estimated to be 191.1 m^3/s by the ratio of the catchment area. Therefore it is concluded that the discharge data derived from the Danau Bingkuang by the ratio of the catchment areas is applicable to estimation of the discharge data.

Fig. 1.3.1 Location Map of Rainfall Gauging Stations (Scale 1:500,000)

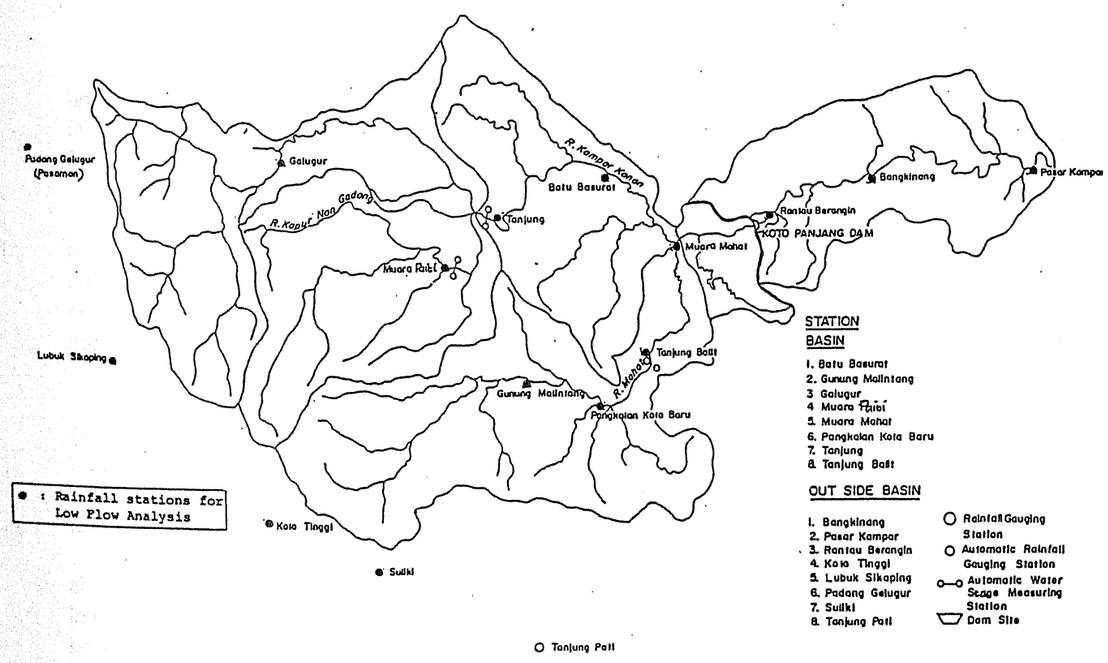
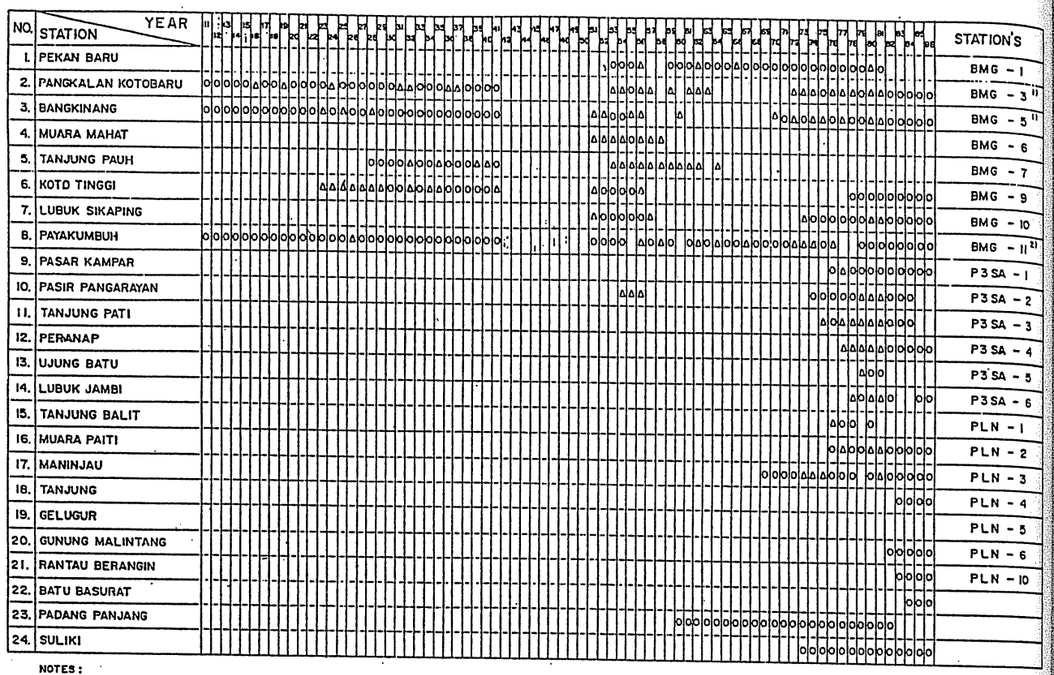


Table 1.3.2 Collected Daily Rainfall Data



NOTES :
Δ INCLUDED SUSPENDED DATA

3.4.2 Flood Analysis

(1) Procedure of Flood Analysis

Flood analysis was carried out to derive the design flood discharge for the spillway and to prepare the hydro graph for the reservoir operation study. Floods more than 1000 m^3/s scale are observed every year at the Danau Bingkuang gauging station located 50 km downstream from the Kotapanjang dam site.

The Rantau Berangin gauging station, located 4 km downstream from the dam site, has been operated since December 1983. This means the observed data of the Rantau Berangin station is only available 26 month up to 1987. Therefore the statistical method based on the stream flow data of both stations is not applicable to derive the design flood.

However, a number of automatic rainfall stations and automatic water level gauging stations have been installed and operated in the project area since the Feasibility Study. At present, about 5 years hourly rainfall data are available while the water level stations are also operated at Rantau Berangin, Lubuk Berangin, Lubuk Sipopay and Tanjung.

Under these hydrological monitoring conditions the storage function model, which is a commonly used model in hydrology analysis with hourly rainfall data and water level records, was employed to carry out the flood analysis. The procedure of the flood analysis is as follows ;

(i) Data collection for daily and hourly rainfall, water level records and river flow measurements.
(ii) Analysis of rainfall duration and intensity.
(iii) Statistic analysis of 1-day and 5-day rainfall for each 200- year and 1000-year return periods.
(iv) Selection of observed floods and storms
(v) Calibration of storage function model
(vi) Extension of observed rainfall to 200-year and 1000-year storm
(vii) Result of calculation.

(2) Data Collection

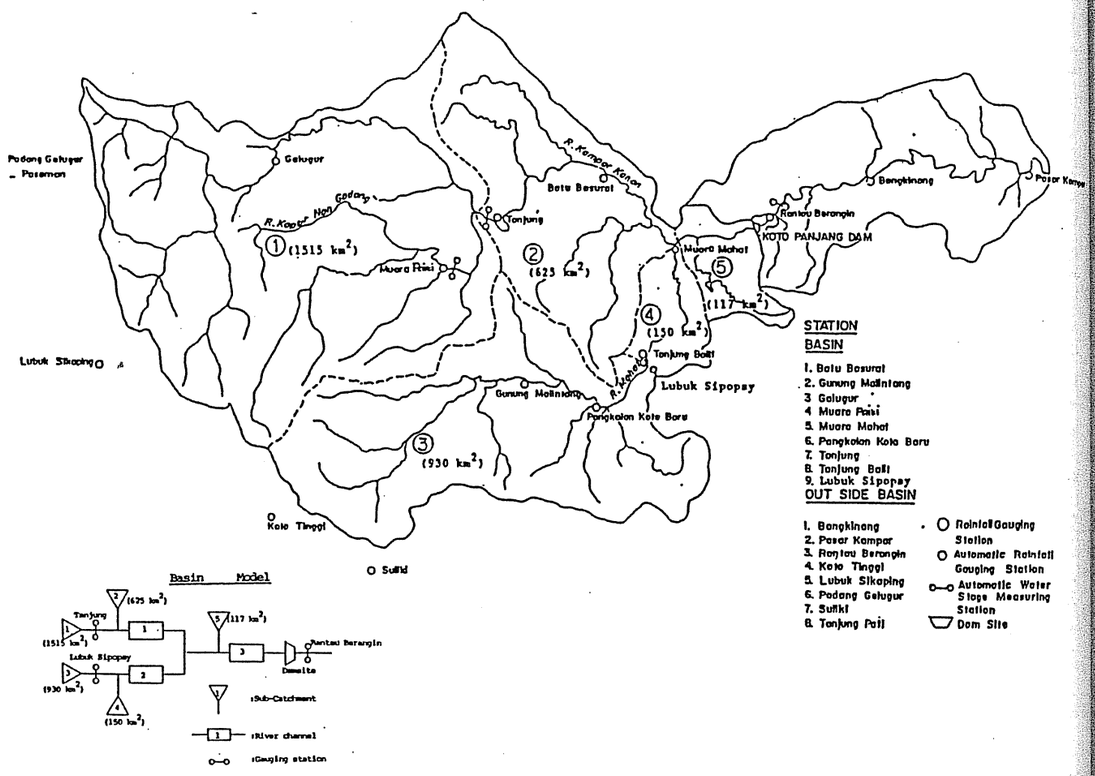
(a) Daily rainfall data were collected from the 24 gauging stations in and around the catchment area. These daily data were used for statistical analysis of 200-year and 1000-year rainfalls. Hourly rainfall data from 1982 to 1986 were also obtained from 12 stations among the above mentioned stations. These hourly rainfall data were used for rainfall duration and intensity analysis and input data for calibration of the storage function model.

(b) Water level records were obtained from four gauging stations on the Kampar Kanan River and the Mahat River as shown below in the Table. The main tributary of the Kampar Kanan and the catchment areas of these stations are shown in Figure 1.3.2.

Water Level the Gauging Stations

Station	Catchment Area (km^2)	Distance from dam site	Location
Danau Bingkuang	4,035	50 km downstream	the Kampar Kanan near Pasar Kampar
Rantau Berangin	3,357	4 km downstream	the Kampar Kanan
Muara Paiti	-	55 km upstream	Gadang, tributary of the Kampar Kanan
Lubuk Sipopay	930	30 km upstream	the Mahat River, tributary of the Kampar Kanan
Tanjung	1,515	50 km upstream	the Kampar Kanan

Fig. 1.3.2 Basin Model and Gauging Stations (Scale 1:500,000)



(3) Rating Curve

A number of river flow measurements were carried out at the water level gauging stations, as shown in the following Table. Rating curves at Rantau Berangin, Lubuk Sipopay, Tanjung and Muara Paiti stations were revised on the basis of the latest available data. After checking the water level records, the Muara Paiti data was excluded for further analysis because of its poor quality of records. The results are shown as follows :

Rating Curve

Station	Rating Curve
Danau Bingkuang	$Q = 26.306\ H^2 + 131.985\ H + 12.164$
Rantau Berangin	$Q = 63.872\ H^2 + 70.629\ H + 19.525$
Lubuk Sipopay	$Q = 52.736\ H^2 - 16.249\ H + 1.252$
Tanjung	$Q = 23.453\ H^2 + 33,205\ H + 11.735$
Muara Paiti	$Q = 45.844\ H^2 - 11,388\ H + 0.701$

(4) Rainfall Duration and Intensity

(a) Rainfall Duration

In order to determine the storm duration in the catchment, 1 to 12 days accumulated rainfall was examined for each gauging station. The mass curve analysis was made for the storms (i) Start of Storm (1-day rainfall exceeds 10 mm), (ii) End of Storm (1-day rainfall less than 3.0 mm) and (iii) Amount of rain (total amount of storm exceeds about 60 mm)

From the rainfall duration curves, the characteristics of storm durations in the Kotapanjang catchment are summarized as follows ;

- Duration of heavy storm lasts more than 3 days at Gunung Melintang, Galugur, Tanjung and Muara Paiti.

- Within 5 days, most of the total amount of rainfall were observed at Gunung Melintang, Galugur, Tanjung and Muara Paiti.

Based on the above mentioned meteorological conditions in the catchment, the storm duration was adopted for 5-days.

(b) Rainfall Intensity

The maximum 1-hour rainfall was examined for 7 hourly rainfall gauging stations. The maximum rainfall was 78.5 mm/hr observed at Rantau Berangin in April 6, 1985. It was assumed that the rainfall intensity would be approximately 60% to 70% of the daily rainfall,

Observed Maximum Rainfall Intensity

Station	Date	A. Maximum Hourly Rainfall in mm	B. Daily Rainfall in mm	A/B in %
Galugur	Dec. 12, 1982	74.4	112.0	66
Gunung Melintang	Apr. 24, 1984	70.0	89.1	79
Muara Paiti	Nov. 20, 1984	23.8	55.7	43
Tanjung	Nov. 20, 1984	60.2	67.6	89
Muara Paiti	Mar. 9, 1985	60.0	77.5	77
Tanjung Balit	Apr. 9, 1985	56.0	77.5	72
Rantau Berangin	Apr. 6, 1985	78.5	119.0	66

(5) Statistic Analysis of Daily Rainfall

Based on the latest available rainfall data in the project area, the probable rainfall for both 200-year and 1000-year return periods was estimated by applying the Hazen, Thomas, Iwai and Gumbel methods for the duration of the 5-days which was confirmed at section "5.4(2) Rainfall Duration and Intensity" above. In order to check the probable rainfall, the 1-day rainfall was also used.

The latest available data was extracted from Kota Tinggi, Lubuk Sikaping, Tanjung Pauh and Bangkinang, since these station are located in and near the catchment area of the Kotapanjang dam and have a long observation period as shown below.

Available Years of Data

Station	Available Years of Data
Bangkinang	55 years
Kota Baru	48
Kota Tinggi	23
Lubuk Sikaping	37
Tanjung Pauh	21

The Hazen, Thomas, Iwai and Gumbel methods were applied to these data. From the result of these methods, the average value of each method was adopted as the probable rainfall, as shown below.

Result of Statistical Analysis

Station	200-year Rainfall		1000-year Rainfall	
	1-day (mm)	5-day (mm)	1-day (mm)	5-day (mm)
Kota Baru	340.9	663.0	405.9	815.7
Kota Tinggi	152.2	408.6	178.5	514.4
Lubuk Sikaping	250.4	376.4	293.2	431.2
Tanjung Pauh	201.5	362.3	227.1	418.1
Bangkinang	225.0	412.5	257.9	490.0

(6) Selection of Observed Storms and Floods

Since 1982, the storms in the catchment area have been recorded by hourly rainfall gauging stations, and the floods on the Kampar Kanan and the Mahat rivers have been observed at the same time at Tanjung, Lubuk Sipopay, Rantau Berangin and Danau Bingkuang.

The following 10 floods were extracted from these stations data be cause these high floods and heavy rainfall data are suitable for calibration of the Storage function model. Hydrographies and rainfall duration curves were made for these floods and storms.

Selected Floods for Calibration (m^3/s)

Flood No.	Date	Rantau Berangin	Tanjung	Lubuk Sipopay	Danau Bingkuang
3.	Dec. 01, 1982	-	361.8	-	-
4.	Dec. 11, 1982	-	493.7	-	-
5.	Dec. 18, 1983	942.4	1184.9	-	1232.6
6.	Feb. 05, 1984	1396.2	419.2	-	1317.9
7.	Apr. 10, 1984	1028.2	611.9	-	1172.2
8.	Nov. 01, 1984	1062.5	742.8	808.7	981.6
9.	Nov. 28, 1984	1153.3	-	391.9	624.7
10.	Jan. 28, 1985	1047.7	-	1934.1	1321.8
11.	Mar. 12, 1985	1401.9	-	1187.8	1286.6
12.	May 13, 1985	873.7	-	521.9	880.3

Selected Storms for Calibration (mm/hr)

Flood No.	Date	Galugur	Gunung Melintang	Rantau Berangin	Tanjung	Tanjung Balit	Tanjung Pati	Muara Paiti
3.	Dec. 01, 1982	143.8	85.7	-	71.0	-	-	-
4.	Dec. 11, 1982	82.1	88.0	-	94.6	-	-	-
5.	Dec. 18, 1983	40.0	197.2	132.8	108.2	0.0	-	-
6.	Feb. 05, 1984	114.5	74.0	70.2	34.4	-	52.0	-
7.	Apr. 10, 1984	171.3	151.3	70.1	174.0	-	97.5	-
8.	Nov. 01, 1984	142.1	171.2	-	154.1	-	-	209.1
9.	Nov. 08, 1984	49.3	140.3	-	103.4	-	34.3	90.1
10.	Jan. 28, 1985	68.3	122.2	91.8	98.8	147.8	-	136.9
11.	Mar. 12, 1985	64.3	79.8	131.9	182.7	155.9	-	100.9
12.	May 13, 1985	113.8	334.5	-	124.5	122.2	-	136.3

(7) Storage Function Model

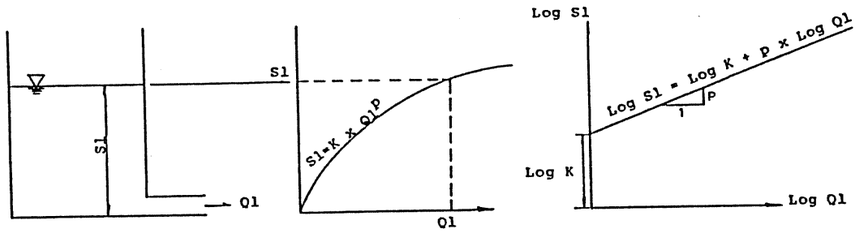
The storage function model is one of the deterministic models which simulate a series of rainfall inputs to the resulting stream-flow hydrograph. The model was proposed by Dr. T. Kimura and has been widely applied for flood routing in Japan.

The basic assumption of the model is that storage (S1) of a basin or a river channel is an exponential function of the discharge (Q1) as follows.

$S1 = K.Q1^P$ where K, P : constant

This equation expresses the storage run-off relation as shown below.

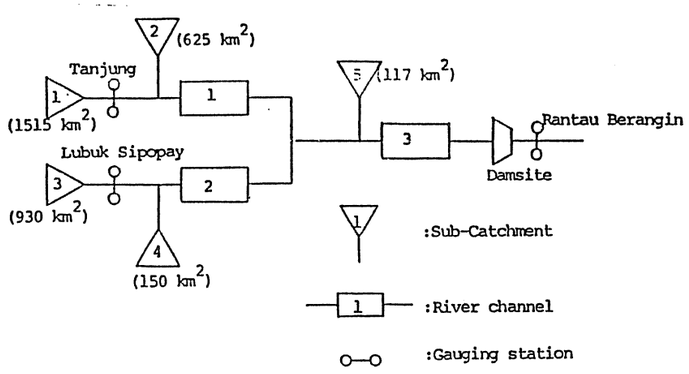
Storage Function Model



This equation is combined with the following continuous equation for a basin or river channel.

For the basin 1/3.6 x f x R x A - Q1 = d (S1) / dt,
For the river channel I - Q1 = d (S1) / dt
where; f : Run-off coefficient
R > Rsa (Saturation raintall) F=1.0
R : Rainfall in the basin (mm/hr)
A : Catchment Area (km^2)
T1 : Lag Time (hr)
I : Inflow to the channel from the basin
Q1(t) : Q1(t) = Q (t+T1)
S1 : Apparent storage in the basin

The catchment was divided into a number of sub-areas. The storage function model simulates each separately and combines them to obtain the catchment response. Based on the location of the gauging stations, the Kotapanjang catchment area was divided into the five sub-areas and three river channels as shown in the following model.



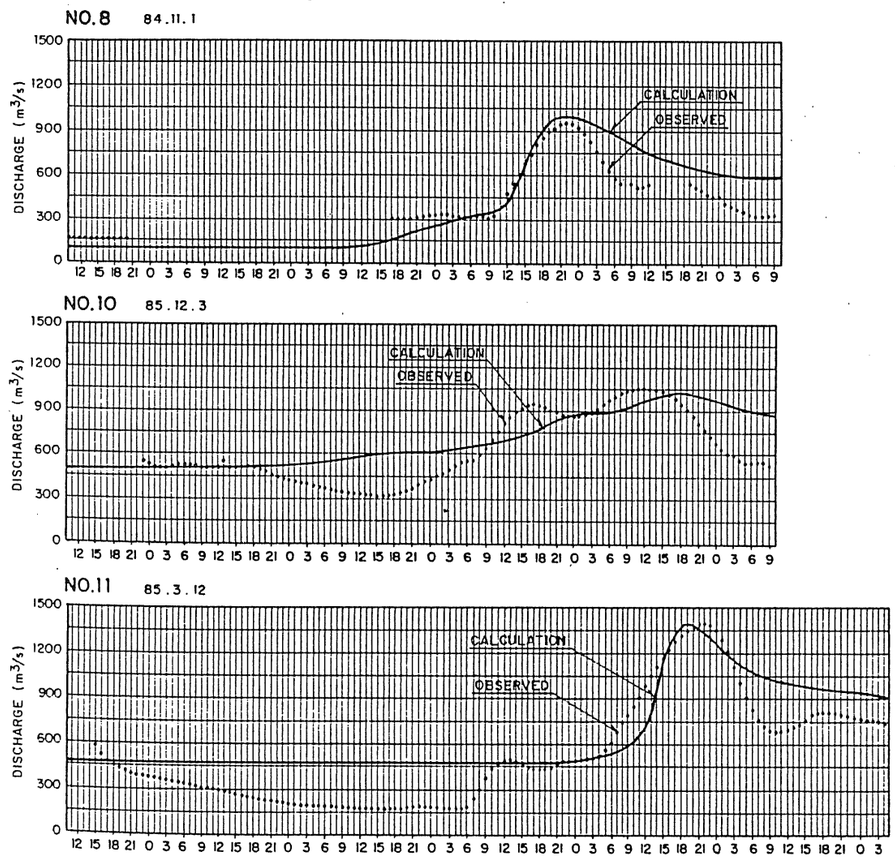
(8) Calibration of Storage Function Model

The storage function model contains several parameters which are to be calibrated with observed hydrographies and rainfall data. These parameters are initially estimated by the empirical equations based on the analysis of the rivers in Japan as follows. After a number of trial calculations, the peak discharge and a ratio of run-off to the observed run-off is as shown below.

No.	Observed	Simulation	Observation/Simulation
8	947 m^3	1.000 m^3	1.075
10	1,048	1,042	1.083
11	1,402	1,401	1.076

Although the hydro graph by the calibrated model is slightly shifted from the observed hydro graph, the peak discharge and total run-off of the model simulates the observed floods as shown in Fig. 1.3.3.

Fig. 1.3.3 Peak Discharge and Total Run-off of the Model Simulation



(9) Input data for the Storage Function Model

In order to derive the input data for the calibrated storage function model, the observed rainfall data was extended to the probable 200-year and 1000-year rainfall by using the extension ratio.

Firstly, both observed rainfall and probable rainfall at each gauging station were converted to the sub-catchments of the basin model by using the Thiessen method as shown in the three tables below "The result of probable rainfall, the Recorded 5 day storm and the recorded 1 day storm".

Secondly, at each sub-catchment, a ratio of observed rainfall to probable rainfall was calculated as follows.

$$R = Rp/Ro$$

Where ; Rp : Probable rainfall for 1-day and 5-day
Ro : Observed rainfall for 1-day and 5-day

Result of Probable Rainfall

(mm)

	CA-1 (1,515 km²)	CA-2 (625 km²)	CA-3 (930 km²)	CA-4 (150 km²)	CA-5 (117 km²)
200-year 5-day	399.0	398.8	535.4	438.3	362.3
200-year 1-day	209.2	218.2	246.4	236.7	201.5
1000-year 1-day	478.2	465.8	664.7	518.7	418.1
1000-year 5-day	244.6	248.5	291.9	272.3	227.1

Recorded 5-day Storm

(mm/5days)

Storm No.	Catchment 1 (1,515 km²)	Catchment 2 (625 km²)	Catchment 3 (930 km²)	Catchment 4 (150 km²)	Catchment 5 (117 km²)	Basin Average
7	170.2	167.1	150.2	118.6	70.1	158.2
8	164.2	156.2	174.8	169.5	161.7	165.8
9	55.2	103.2	123.1	127.4	113.3	88.4
10	92.5	107.8	127.9	147.8	97.9	107.9
11	77.1	164.7	93.3	150.5	129.0	103.1
12	105.2	115.0	216.5	79.9	79.9	136.0

Recorded 1-day Storm

(mm/day)

Storm No.	Catchment 1 (1,515 km²)	Catchment 2 (625 km²)	Catchment 3 (930 km²)	Catchment 4 (150 km²)	Catchment 5 (117 km²)	Basin Average
7	62.5	76.8	94.7	62.7	51.5	73.8
8	71.1	56.8	79.2	73.9	54.6	70.2
9	26.7	100.3	110.0	116.0	106.8	70.5
10	20.7	43.2	37.6	42.2	35.3	31.1
11	43.5	137.0	46.7	64.6	52.7	63.2
12	35.9	66.1	106.4	49.0	49.0	62.3

For each storm, the ratio of the sub-catchment where the maximum rainfall occurred was adopted, as shown in the table below to extend the observed rainfall to input rainfall data for the calibrated storage function model.

Extension Ratio

Storm No.	200-year		1000-year	
	1-day	5-day	1-day	5-day
7	2.602	2.334	3.082	2.788
8	2.942	2.241	3.440	2.580
9	1.887	3.198	2.126	3.690
10	5.051	2.965	5.752	3.509
11	1.593	2.421	1.841	2.828
12	2.361	2.473	2.743	3.070

(10) Result of Calculation

In order to select the input data for the calibrated model, the following hydrological conditions were considered;

(i) Heavy rainfall should occur in the catchment No.1, No.2 and No.3, since the recorded storm indicates heavy rainfall in these areas.

(ii) Rainfall intensity should exceed at least 78.5 mm/hr which was the maximum observed intensity in the catchment area.

(iii) Rainfall extension ratio of the 5-day storm should not exceed that of 1-day rainfall since the return period for each day of 5-day storm should be equally or less than that of 1-day rainfall.

Under these hydrological conditions, the No. 7 storm (Apr.10, 1984), No. 8 storm (Nov. 01, 1984) and No. 10 storm (Jan. 28, 1985) were adopted for the input data.

From the results of calculation, the inflow design flood for the spillway is estimated to be 8,000 m^3/s with a 120 hour volume of $601x10^6$ m^3 or 180 mm run-off for 200-year return period for storm No. 7 which was applied to this project. 1140 m^3/s for 1000 year return period for given storm No. 7 are also shown in the table below.

Results of 200-year Flood

Storm No.	Average Rainfall (mm)	Rainfall Intensity (mm/hr)	Peak Discharge (m^3/s)
7	370.9	150.7	8,000
8	371.6	94.5	5,000
10	320.0	82.3	3,400

Results of 1,000-year Flood

Storm No.	Average Rainfall (mm)	Rainfall Intensity (mm/hr)	Peak Discharge (m^3/s)
7	441.1	179.7	11,400
8	428.7	109.3	6,600
10	378.6	97.2	4,200

3.5 Design Flood and Sedimentation

3.5.1 Design Flood Discharge

(1) Spillway Design Flood Discharge

The result of the flood analysis indicates that the peak discharge of the 200-year and 1000-year return periods is 8,000 m^3/s and 11,400 m^3/s respectively. Since the Kotapanjang dam is a concrete gravity dam, the 200-year return period was adopted for the inflow flood. Regarding the retention effect of the design flood in the reservoir, flood routing was carried out to examine the possibility to reduce the spillway design flood of 8,000 m^3/s with a surcharge capacity of 10.8 x $10m^3$ between the reservoir level of EL. 84.000 and EL. 85.000.

The result of this calculation shows that the peak discharge from the spillway can be reduced to 6,400 m^3/s or 80% of the peak inflow discharge. On the other hand of the flood routing, a comparative study was made for cost reduction in the spillway and gates cost and benefit decrease in power generation by lowered operating level. Output of power generation will decrease by 11.2 GWh and 2000 KW due to the lower operating level to install the surcharge capacity. The annual cost and benefit loss will be as follows.

Annual cost reduction by spillway and dam : 153,700 US$/year
Annual benefit decrease by power generation : - 618,140 US$ /year
Annual cost - Annual benefit : - 544,340 US$/year

From this result it is concluded that installation of surcharge capacity is not feasible from an economical point of view. Therefore, the design flood discharge for the spillway was adopted to be is 8,000 m^3/s for 200-year return period.

(2) Diversion Design Flood Discharge

The design of the river diversion works is estimated from the flood frequency analysis for the dam site (3337 km^2) with the discharge derived from Danau Bingkuang gauging stations (4035 km^2). The annual maximum discharge for ten years from 1977 to 1986 was converted to the Dam site by the ratio of both catchment areas as follows.

Maximum Recorded Flood

Date	Water Level (m) D. Bingkuang	Discharge (m^3/s) D. Bingkuang	Discharge (m^3/s) Dam site
Dec. 08, 1977	4.29	1298	1073
Dec. 15, 1978	6.90	2175	1798
Feb. 10, 1979	5.73	1632	1349
Nov. 25, 1980	5.02	1337	1106
Dec. 17, 1981	4.72	1221	1009
Dec. 14, 1982	4.86	1274	1054
Dec. 22, 1983	4.70	1213	1003
Feb. 10, 1984	4.97	1317	1089
Feb. 02, 1985	4.98	1321	1093
Jan. 10. 1986	6.06	1778	1470

In order to derive the probable discharge the Iwai method was applied to these annual maximum data. The results for the 2 to 20 year probable flood are as follows :

Probable Flood

Return Period (year)	Peak Discharge (m^3/s)
20	1624
10	1514
7	1452
5	1390
3	1285
2	1183

3.5.2 Design Sedimentation

(1) General

Estimation of the design sedimentation was based on the observed sampling data at the dam site and other reference data from Indonesia and other countries. Sediment measuring was carried out during the feasibility study for the period from 1982 to 1983.

(2) Sediment rating curve

Suspended sediment and river discharge data were recorded 12 times at the Rantau Berangin gauging station about 4 km downstream from the dam site. Based on these data, the sediment rating curve in Fig. 1.3.4 was constructed using the least square method. The envelope curve was adopted for the rating curve as shown below, since available data is very limited.

$$Qs = a \times q^b = 2.818 \times 10^{-5} \times Q^{3.07}$$

where qs : suspended sediment discharge at dam site (m^3/day)
Q : River discharge (m^3/s)
a,b : constant

(3) Annual suspended load

The annual suspended load was calculated using the sediment rating curve with the daily discharge data at the dam site. The discharge data in 1986 including several floods during the year were used for a more conservative estimation of the suspended load. The annual suspended sediment was derived at 1,362,536 m^3/year or 408 m^3/km^2/year at the dam site.

This total sediment load will be trapped in the reservoir as shown in Fig.1.3.5. The trap efficiency was assumed to be 95 % corresponding to the ratio of the reservoir capacity, 1545 x 10^6 m^3, to the annual inflow, 5815 x 10^6 m^3. Therefore, the design sediment for the reservoir is derived as follows :

$$Qs = 490 \times 0.95 \fallingdotseq 500 \text{ m}^3/\text{km}^2/\text{year}$$

where Qs : specific sediment per year (m^3/km^2/year)

(4) Design Sedimentation

The design sedimentation of other projects in Indonesia was checked. Generally, the projects in Java indicate a higher value of sedimentation. On the other hand, the Riam Kiwa project in Kalimantan adopted a relatively low value of sedimentation as there is no volcanic activity and deep forest covering the catchment.

The design sedimentation of the Kotapanjang project was also compared with other major rivers in the world. Based on this figure, the design sedimentation of Kotapanjang is located at the higher sedimentation group compared with Japanese rivers, while it is in the low sediment group compared with the Nepal rivers where water erosion and glaciation prevails in the Himalayas.

Thus, the design sedimentation of 500 m^3/km^2/year is appropriate for the Kotapanjang project based on the above consideration. From the reservoir storage and area curves, the sedimentation level will be EL. 64 m for the calculated volume of 167 x 10^6 m^3 for the 100-year flood.

Fig. 1.3.4 Sediment Rating Curve

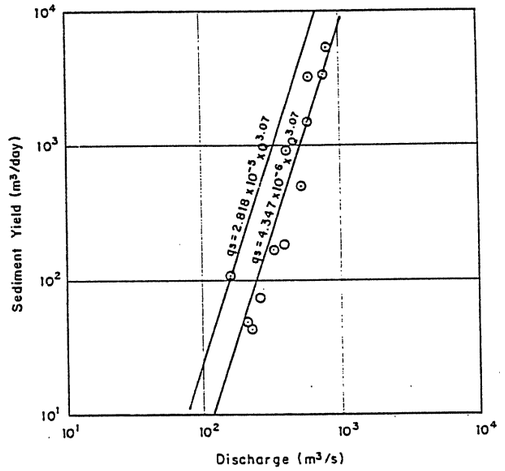
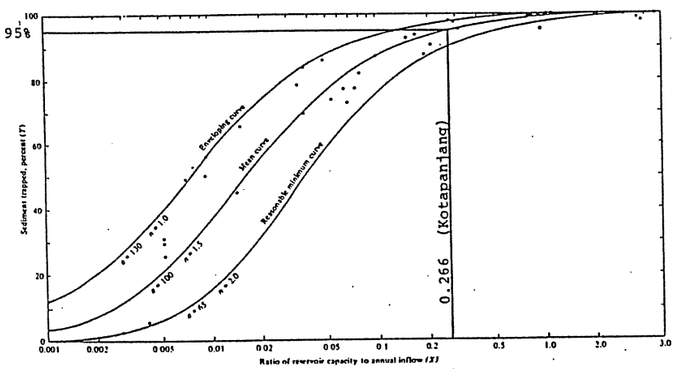
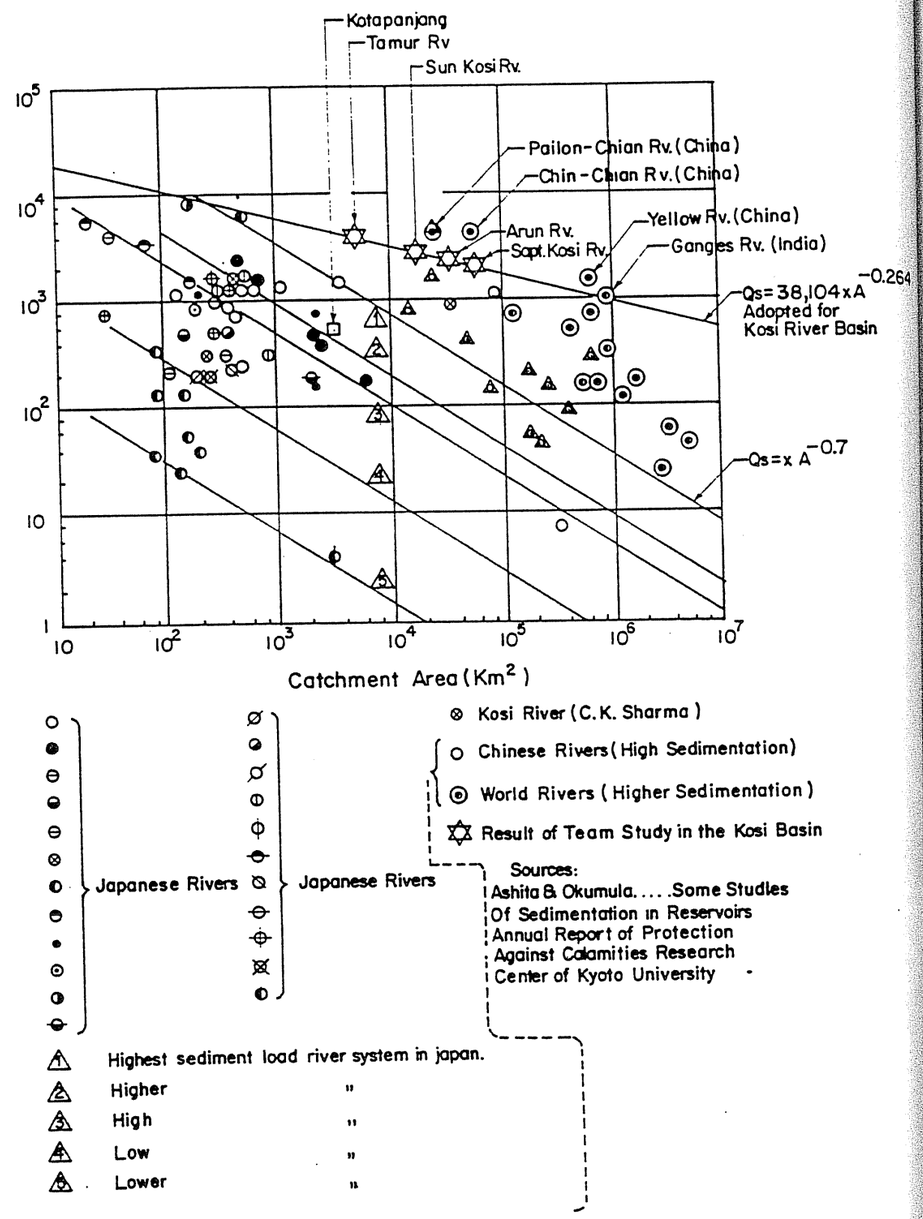


Fig. 1.3.5 Sediment Load Trap Efficiency



Note:This figure is adopted from pp.332 of "Hydrology for Engineers" by Ray K.Linsley,Jr.

Fig. 1.3.6 Relation Between Specific Field and Catchment Area of Major Rivers in the World



Chapter 4 POWER GENERATION PLAN

4.1 Demand and Supply of Electric Power

(1) Electric Power Condition

At the time of the feasibility study the energy consumption in both West Sumatra and Riau Provinces in the eleven years since 1975 indicated an annual average growth rate of 20.4 % in both Provinces, 22.3% in West Sumatra and 16.2 % in Riau. The installed capacity of power source facilities as of April 1986 were as shown below. Diesel power plants comprised about 40 % of the total installed capacity.

Province	Diesel	Gas Turbine	Hydro	Total
West Sumatra	38.8 MW	42.7 mw	78.5 MW	160.0 MW
Riau	31.5 MW	-	-	31.5 MW
Total	70.3 MW	42.7 MW	78.5 MW	191.5 MW

With respect to the transmission line system, a 163 km long, 150 kV transmission line was in operation from Maninjau Hydro Power Station to Solok, passing through Padang city. A further 42 km long section was under construction to Padang Luar in West Sumatra. In Riau, however, there was no transmission line at that time and the only power was supplied by diesel power plants. These plants were located near the center of each city/town and power was only distributed locally. This highlighted the shortage of the power supply and the lack of general distribution. In addition the system had rather low reliability.

Several industrial plants and enterprises generated their own captive power and the capacity of these were as indicated below.
It was expected that these industries/enterprises with captive power would be prepared to switch over to the PLN supply once it became available.

Province	Capacity	Remarks
West Sumatra	32,216 KVA	Include Padang Cement
Riau	103,212 KVA	Include PERTAMINA & exclude CALTEX
Total	135,428 KVA	

(2) Power Demand

A detailed power demand forecast was made in the Feasibility Study for the Electric Power System Development Project in Central Sumatra in 1985 - 1986 (hereinafter called Central Sumatra F/S). This study also took into account the results of the power demand by the Feasibility Study of the Kotapanjang Hydroelectric Power Project in 1982 - 1984 (hereinafter called Kotapanjang F/S).

After the record of solid energy used in 1985/1986 in Central Sumatra was checked and studied, it was found that the actual energy consumed was 7.5% lower than forecast, mainly due to a reduction in demand in Padang and Pekanbaru cities. The lower consumption appears to have been caused by economic recession and/or a shortage of reliable power sources.

The demand forecast derived from the Central Sumatra F/S, is shown in Table 1.4.1. The annual power demand forecast was as follows :

Annual Power Demand	Including PERTAMINA	Excluding PERTAMINA
Annual Power Demand Forecast 1995	1193 GWh	1097 GWh
Annual Average Growth Rates 1985 - 1995		
National	14.4%	13.5%
West Sumatra	12.3%	12.3%
Riau	19.2%	16.4%
Maximum Power in 150 KV System :	232 MW	202 MW

(3) Power Facility Extension Plan

To meet the growth in power demand, PLN is making concerted efforts to utilize non-oil resources, particularly hydro power and coal, for power generation. In addition, it is concurrently carrying out the expansion of the transmission line system in order to supply economical and reliable power to areas requiring such power.

With respect to the transmission line system, a detailed study was carried out in the Central Sumatra F/S, in which the optimum system to be installed by 1995 was determined. In the transmission line system study, the importance of early interconnection of the power systems between West Sumatra and Riau was established, assuming that the Ombilin Coal-fired Power Plant (50 MW x 2 Units) was to be commissioned in the early 1990's.

However by 1995 further new power sources, equivalent to approximately 100MW, were expected, even if all planned diesel plants together with the Ombilin Power Plant were commissioned on schedule. Therefore, the Kotapanjang Hydro Power Plan can meet this electric power demand. The peak load forecast and the proposed power plant plan in Riau Province carried out in the F/S are shown in Fig. 1.4.1.

To promote the interconnection of the power system between West Sumatra and Riau, a transmission line between Ombilin and Pekanbaru was planned. This line was divided into two (2) sections. The first section was 64 km long between Kotapanjang and Pekanbaru. A second line nearly 140 km long between Ombilin and Kotapanjang, was originally proposed. The second line was changed from Kotapanjang to Payakumbuh (nearly 85 km long) and included in the scope of the Kotapanjang Project.

Both these sections of transmission line, together with the Pekanbaru Substation and other associated substations, should be commissioned by March 1994, so that an economical and stable power can be transmitted to Pekanbaru.

From the latest data in 1988, the installed capacity in Riau Province at that time was only 6 MW which was provided by Diesel Power Plants, however, the demand forecast by Fifth Five Year Plan (1988 - 1993 REPELITA V) showed 150 MW. This was mainly based on the future industries such as industrial sector, forestry industrial sector etc..In consideration of the above situation, the Kotapanjang Hydroelectric Power Project was judged to be very viable.

Fig. 1.4.1 Power Demand Program

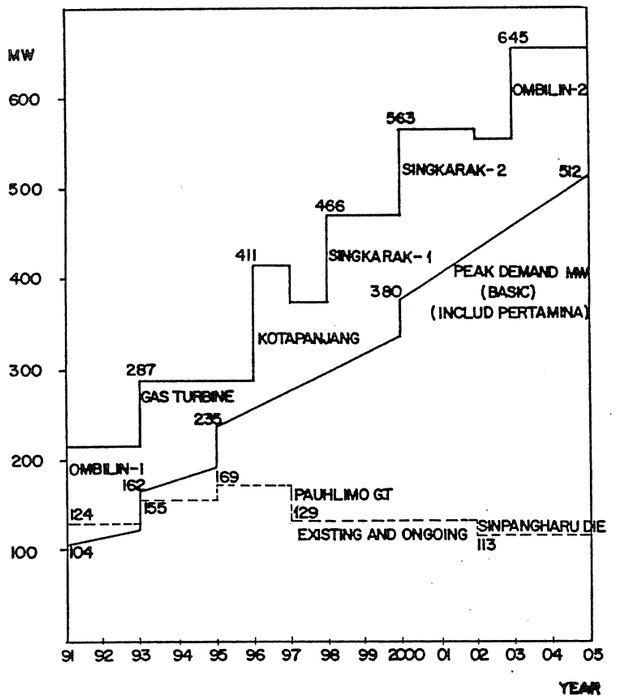


Table 1.4.1 Power Demand Forecast

1.1 In case the Power Demand of PERTAMINA is Included

(Unit : Gwh, MW, %)

Province	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995	'85-'90 Annual growth rate %)	'90-'95	'85-'95
West Sumatera Province	231.1	275.4	330.5	378.0	429.0	476.9	519.8	566.3	618.0	674.6	736.5	15.6	9.1	12.3
Riau Province	78.6	91.6	105.3	120.1	135.8	152.9	173.4	197.1	286.3	331.5	456.4	14.2	24.4	19.2
Total of Wilayah III	309.7	367.0	435.8	498.1	564.8	629.8	693.2	763.4	904.3	1006	1193	15.3	13.6	14.4
Maximum System Demand	54	63	72	79	88	96	103	111	164	178	232	12.2	19.3	15.7

1.2 In case the Power Demand of PERTAMINA is not Included

(Unit : Gwh, MW, %)

Province	1985	1986	1987	1988	1989	1990	1991	1992	1993	1994	1995	'85-'90 Annual growth rate %)	'90-'95	'85-'95
West Sumatera Province	231.1	275.4	330.5	378.0	429.0	476.9	519.8	566.3	618.0	674.6	736.5	15.6	9.1	12.3
Riau Province	78.6	91.6	105.3	120.1	135.8	152.9	173.4	197.1	286.3	331.5	360.4	14.2	18.7	16.4
Total of Wilayah III	309.7	367.0	435.8	498.1	564.8	629.8	693.2	763.4	904.3	1006	1097	15.3	11.7	13.5
Maximum System Demand	54	63	72	79	88	96	103	111	164	178	202	12.2	16.0	14.1

Notes 1. The demand in the respective isolated islands in both provinces was excluded.
2. The demand of Padang Cement is included in the demand of West Sumatera Province.
3. The maximum system demand refers to a maximum value of total demand incorporated into the Central Sumatera Power System.

4.2 Power Development Plan

(1) Alternative Development Plans

In formulation of the Kotapanjang Development Project in F/S, two alternative plans, i.e. a single stage development plan and a two stage development plan were studied.

The single stage development plan was to build a dam of 58m height (HWL 85m) at Kotapanjang site approximately 10 km downstream of the confluence of the Kampar Kanan River and its tributary, the Mahat River, and to construct a power station of 114 MW with a reservoir of active storage capacity of 1.040 million m^3. This plan permits effective utilization of the rivers by means of a large-scale power development, and flood mitigation of the downstream area. On the other hand, 2,644 houses, 8,989 ha of cultivated land and 25.3 km national road and 27.2 km provincial road would be submerged.

The two stage development plan was to build a regulating pondage by two steps as a measure to reduce compensation costs for land and houses to be submerged. Specifically, a downstream dam of 30.5m height (HWL 58m), a regulating pondage with an active storage capacity of 20 million m^3 and a power generation plant of 42 MW at the aforementioned Kotapanjang dam site, and further upstream a 38 meter high dam (HWL 85m) was proposed to create a regulating pondage of active storage capacity of 20 million m3 and building the Mahat power plant of 23 MW at Tanjung Pauh on the Mahat River. According to this plan, about 390 houses, 1,860 ha agricultural land and 16 km national road would be submerged.

Locations and profiles of the two alternative plans are as shown in Fig. 1.4.2. The comparison study shown in table 1.4.2 indicated that the single stage development plan was better.

Table 1.4.2 Comparison of Development Plan

Project Dimension	Single stage development	Two stage development		
	Kotapanjang	Kotapanjang	Mahat	Total
Catchment area (km^2)	3,337	3,337	1,075	
Annual average inflow (m^3/sec)	173.5	173.5	55.9	
High water level (m)	85	58	85	
Low water level (m)	73.5	54	81	
Active storage capacity ($10^6 m^3$)	1,040	20	20	
Power generation type	Reservoir	Pondage	Pondage	
Dam height (m)	58	30.5	38	
Type of dam	Gravity	Gravity	Gravity	
Max. output (kW)	111,000	42,000	23,000	65,000
Max. discharge (m^3/sec)	348	348	110	
Effective head (m)	38.1	14.2	24.5	
Annual generated energy (GWh)	495	175	96	271
Facility utilization rate (%)	50	44	49	
Construction cost (10^3 US$)	190,194	109,700	56,300	166,000
Construction cost per kW (US$)	1,713	2,612	2,448	2,554
Construction cost per kWh (US$)	0.38	0.63	0.59	0.61
B/C	1.47	0.92	0.98	0.94
B-C (10^3US$)	12,551	- 1,252	- 144	- 1,396

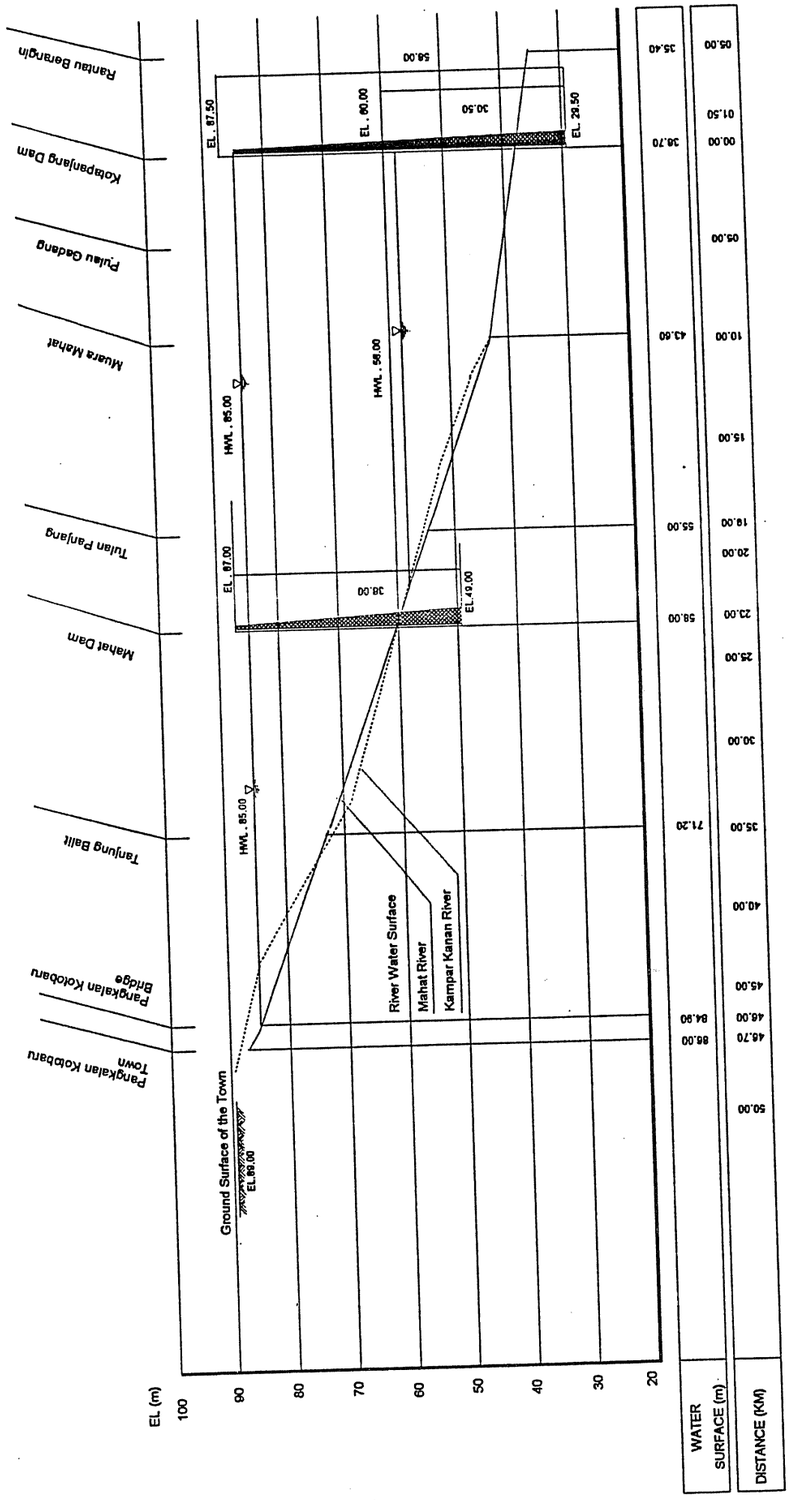


Fig. 1.4.2 Profile for Single Stage and Two Stage Development Plan

(2) Dam Site and Type

Upon selection of the dam site, comparative studies were carried out on dam axes-1, 2, 3 and 4, using existing topographic maps of 1:25.000 scale. As axis-2 was found to be relatively close to axis-3, it was excluded from further study. Field investigations including drilling investigation were also applied to these axes shown on Fig. 1.4.3.

Concrete gravity type dams on the dam axes-1, 3 and 4 were studied to select the optimum site for a concrete dam. For a fill type dam, on the other hand, dam axis-3 was selected as most suitable topographically and the type of center core was studied. In this case, a comparative study for the location of spillway was done at both bank of left and right bank.

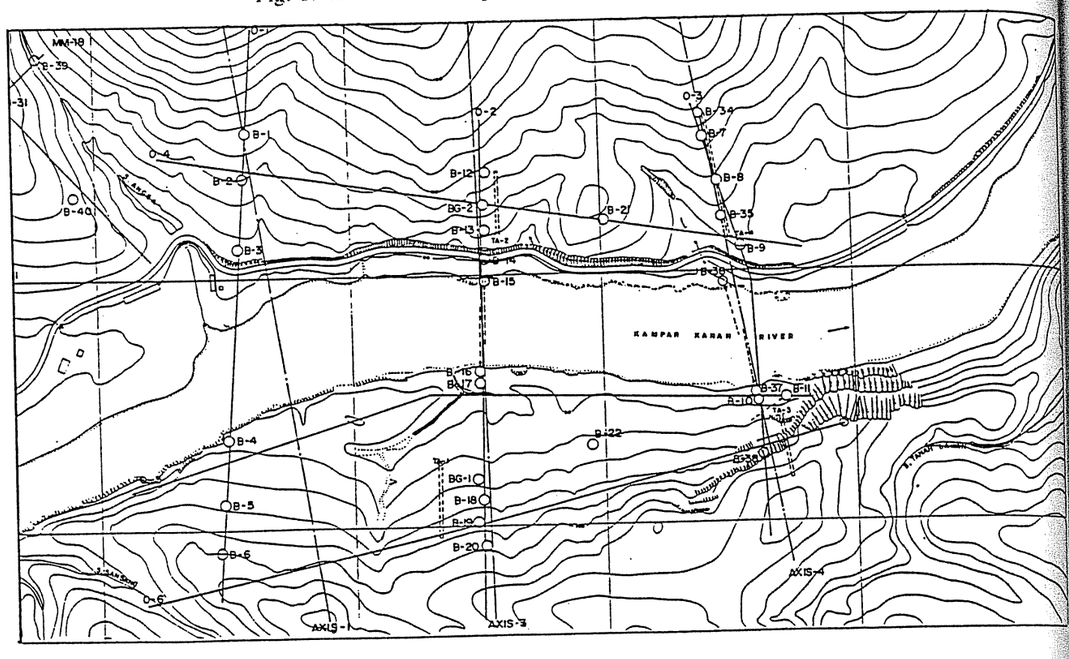
Such studies were carried out using the following procedure. The optimum layout was drawn for each case and the total construction costs were estimated and compared for main dam structure, spillway, diversion work and powerhouse excluding common work costs.

As a result, the construction of a concrete gravity dam on the dam axis-4 as shown in Table 1.4.3 was found to be the optimum one because of the high cost of countermeasures necessary against large flood discharge during and after construction at the other axis.

Table 1.4.3 Selection of Dam Sites and Dam Types

Dam type and Dam Axis / Item	Concrete gravity dam			Fill dam	
	Axis-1	Axis-3	Axis-4	Axis-3 (Spillway at right bank)	Axis-3 (Spillway left bank)
Dam					
- Dam body	22,791	16,626	16,331	10,421	6,974
- Spillway	6,804	7,009	6,756	16,519	19,617
- Diversion work	6,047	6,434	5,149	10,911	12,221
- Concrete plug work	1,617	1,617	1,617	3,234	3,234
- Others	3,728	3,170	2,913	4,111	4,328
Total	40,987	34,856	32,766	45,196	46,374
Power house	9,689	11,455	9,936	13,174	13540
Metal works					
- Gate	9,072	9,072	9,072	10,498	9,272
- Penstock	1,362	1,362	1,362	1,872	1,940
Total	10,434	10,434	10,434	12,370	11,212
Grand Total	61,110	56,745	53,136	70,740	71,126

Fig. 1.4.3 Location Map of the Kotapanjang Dam Axis



(3) Dam Scale

An optimization study on the scale of the proposed dam, annual generated energy and construction costs was calculated for there cases as shown in Table 1.4.4. The study took into account the height of the dam, operating hours, effective water storage capacity and effective depth, and then each case was evaluated on the basis of a cost-benefit analysis. According to this analysis, the case of HWL 85m was the optimum. Considering a part of Pangkalan Kotabaru, upstream the Mahat River, is located at an elevation of 88.2 - 91.5m (population 8,572) and that the Buddhist temple remains (Muara Takus) lie at an elevation of 86.25m the proposed high water level (HWL) was set at 85m.

Table 1.4.4 Comparison of Dam Scales

HWL		76 m	85m	100 m
Max. output	(kW)	90,000	111,000	160,000
Max. discharge	(m^3/sec)	348	348	348
Effective head	(m)	30.7	38.1	54.4
Annual generated energy	(kWh)	393 x 10^6	495 x 10^6	697 x 10^6
Construction cost	(10^3 US$)	155,447	190,194	268,796
Construction cost per kW	(US$)	1,727	1,713	1,680
Construction cost per kWh	(US$)	0.40	0.38	0.39
Benefit - Cost (B/C)		1.43	1.47	1.47
Benefit - Cost (B-C)	(10^3 US$)	9,534	12,551	17,923

4.3 Installed Capacity and Unit Size

(1) Installed Capacity

Considering the power demand and the system requirement and available stream flow of the Kampar Kanan River, the power generation plan was reviewed based on the Kotapanjang F/S carried out 1982 - 1984.

The study indicated that the maximum output 167 MW with a plant factor of 30% was more economical than that of 111 MW on the construction cost and benefit of power generation as shown in Table 1.4.5 and Fig. 1.4.4.

It was recommended in the study, however, that the installed capacity of 111 MW with a plant factor of approximately 40% based on the firm discharge would be the most feasible because a load factor in Pekanbaru area was estimated at about 50% in the future and an excessive scale of power generation will not meet high efficiency of plant utilization.

Moreover the Feasibility Study on Electric Power System in Central Sumatra carried out in 1985 - 1986 recommended that a new power source with 100 MW would be required by 1995. According to the daily duration curve in 1995, the load factor will be approximately 81% which indicates the requirement of a base load type power station.

Assuming that the Kotapanjang Project will not be commissioned before 1995 at the earliest the Kotapanjang power station should have capacity of at least 100 MW together with base load capability. To meet these requirements, the maximum installed capacity of 114 MW was selected from the following considerations.

- The minimum requirements of the Kotapanjang power station is at least 100 MW.
- The base load operation is possible approximately 22 MW during the dry season as shown in Fig.1.4.5.
- For peak operation, approximately seven hours peak operation with 81.5 MW is possible during the dry season as shown in Fig.1.4.6.

Central Sumatra F/S also recommended that the Riau and West Sumatra power system be interconnected by 1995, since the Ombilin thermal power station would be commissioned in the early 1990's. The interconnection of these two system make up the base load.

(2) Unit Size

After the interconnection of the Riau and West Sumatra power system, the maximum peak demand will increase to 202 - 232 MW in 1995. In this system, the largest single unit will be 50 MW in the Ombilin thermal power station. Therefore, the maximum single unit of the system is limited to approximately 50 MW to avoid unacceptable frequency drops on the loss of a unit.

For the Kotapanjang power station, 38 MW x 3 Units and 58 MW x 2 Units were compared with regard to construction cost, generated energy and system stability. The 2 Unit has slight advantages on cost and generated energy. However, for frequency fluctuation on a unit shutdown, the 3 units is more favored than that of the 2 Units. Therefore, 38 MW x 3 Units were finally determined for the Kotapanjang Powerstation taking into consideration the total system operation for Central Sumatra.

Table 1.4.5 Comparison of the Installation Capacity

Max. Output (MW)	Max. discharge (m^3/sec)	Annual generated energy (10^6 kWh)	Annual benefit B (10^3 US$)	Total Construction cost (10^3 US$)	Annual cost C (10^3 US$)	B/C	B-C (10^3 US$)
66.8	208	423.3	30,583	171,596	24,195	1.264	6,388
74.2	231	459.1	33,361	174,481	24,602	1.356	8,759
83.5	260	495.2	36,363	178,128	25,116	1.448	11,247
95.4	297	495.2	37,663	182,565	25,742	1.463	11,921
111	348	495.2	39,368	190,194	26,817	1.468	12,551
134	416	495.2	41,882	197,655	27,869	1.503	14,013
167	520	495.2	45,489	209,140	29,489	1.543	16,000

Note : B = 109.3 (US$/kW) x (Max. output) + (US$/kWh) x (Annual generated energy)
C = (Total construction cost) x 0.141
These data are based on the feasibility study carried out in 1982 - 1984.

Fig. 1.4.4 B/C and B-C Cures of Power Generation Plan

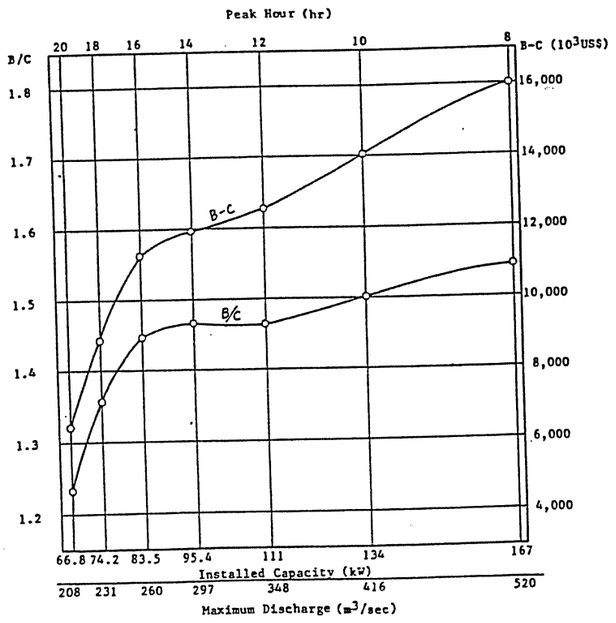


Fig. 1.4.5 Forecast Load Curve of 150 kV System

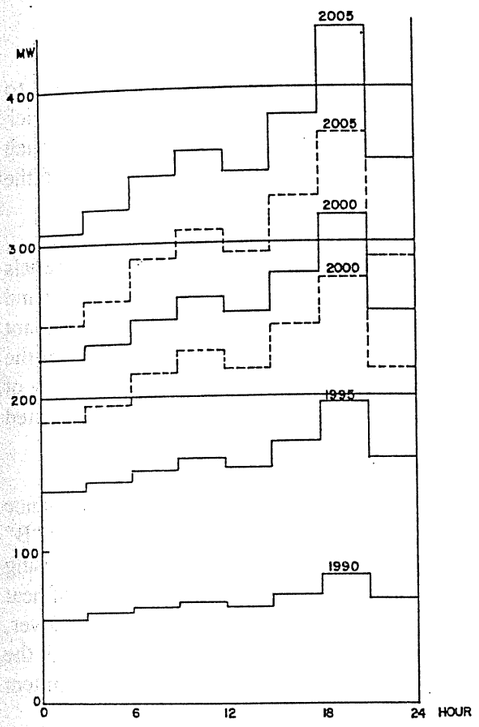
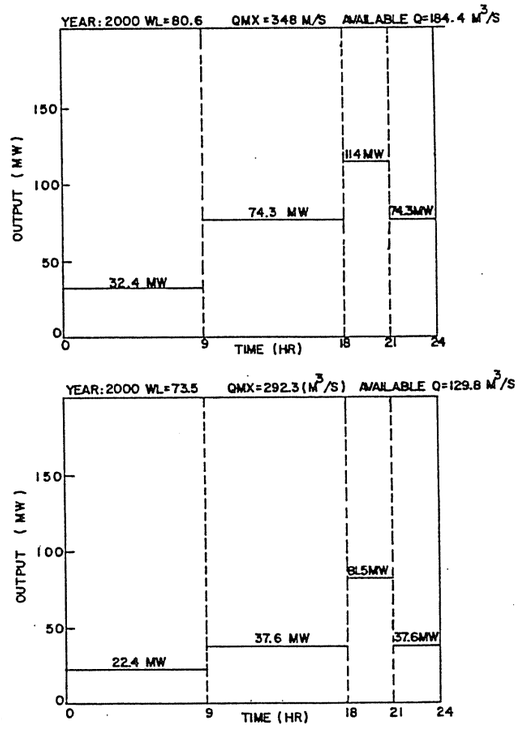


Fig. 1.4.6 Mode of Daily Operation



Chapter 5 CONCRETE AND MATERIALS

5.1 General

The concrete to be used in the dam, powerhouse and other structures was designed to satisfy the required properties such as design strength, durability and water tightness, which were determined for each structure. The concrete was also designed to be of such consistency that it could be worked into the corners and angles of the form and around the reinforcement without segregation of the constituent materials.

Moreover, the quality of the concrete depends on the quality of its constituent materials and their interaction. Therefore, production of good concrete required proper selection and processing of aggregate, and mix proportioning using cement and other additives relevant to the desired properties together with continuous control for the assuring quality of the concrete and its constituent materials. During production of the concrete, the suitability of the mix proportions including water content was checked and where necessary adjusted using the quality control system which was established at the site.

The concrete to be used in Lot-1 civil works was divided into two classes in accordance with the kinds of the structures, namely, class "A" concrete for the dam and class "B" concrete for the powerhouse and other structures. The former was produced by using moderate heat cement for minimizing heat of hydration of cement and reduction of heat generated, while the later was produced by using ordinary portland cement. Moreover, both classes of concrete were divided into several types of concrete in accordance with the required properties, design strength, maximum size of coarse aggregate, slump, air content and so on.

During the design stage, various material tests and concrete tests were carried out by the Engineer in order to decide the source of materials and develop the specification for the concrete works of the project. The results of these tests are summarized as follows.

Cement

Among the cements produced in Indonesia, ordinary portland cement (type I) and moderate heat cement (type II) manufactured by PT. Semen Padang, West Sumatera were selected to test. From the results of the physical, chemical and hydration heat tests, both types (type I, type II) complied with the ASTM standard (ASTM C.150-85) and JIS standard (JIS R.5210) as shown below.

Table 1.5.1 Result of Cement Test

Test Item	Test Result			ASTM C.150-85	
	TYPE I (1)	TYPE I (2)	TYPE II	TYPE I	TYPE II
[Physical Property]					
Specific Gravity	3.18	3.21	3.29	-	-
Fineness specific surface					
- Air permeability (m^2/kg)	317	307	310	min 280	min 280
Auto clave expansion (%)	0.05	0.12	0.01	max 0.8	max 0.8
Compressive strength (kg/cm^2)					
- 3 days	188	182	155	min 124	min 103
- 7 days	258	256	216	min 193	min 172
- 28 days	372	377	321	min 276	min 276
Time of setting					
- Vicat test (minutes)					
Initial set	120	115	135	min 45	min 45
Final set	300	285	335	max 375	max 375
Heat of Hydration (cal/gr)					
- 7 days	-	-	59.14		max 70
- 28 days	-	-	68.81		
[Chemical Property]					
Insoluble residue (%)	0.14	0.11	0.07	max 0.75	max 0.75
SiO2 (%)	21.08	20.78	22.45	-	min 20
AL2O3 (%)	5.54	5.95	3.72	-	max 6.0
Fe2O3 (%)	3.36	3.33	5.20	-	max 6.0
CaO (%)	65.34	65.62	64.90	-	-
MgO (%)	0.97	0.93	1.05	max 6.0	max 6.0
SO3 (%)	2.19	1.92	1.11	max 3.0	max 3.0
K2O (%)	0.17	0.30	0.11	-	-
Na2O (%)	0.24	0.26	0.09	-	-
Alkalies (Na2O+0.658 K2O) (%)	0.35	0.46	0.16	max 0.60	max 0.60
S (%)	0.01	0.01	0.01	-	-
LSF	96.17	97.40	91.89	-	-
SIM	2.29	2.24	2.52	-	-
ALM	1.51	1.77	0.72	-	-
Loss on Ignition (%)	0.99	0.76	0.65	max 3.0	max 3.0
Mineral Compounds (%)					
- C3S	53.24	55.43	55.81	-	-
- C2S	20.36	17.85	22.35	-	-
- C3A	8.50	10.14	1.07	-	max 8
- C4F	11.13	10.12	15.81	-	-
- C3S + C3A	-	-	56.87	-	max 58

Note : Type I (1) : Ordinary Portland Cement taken at Packing Plant (PT. Semen Padang).
Type I (2) : Ordinary Portland Cement taken at the local market.
Type II : Moderate heat Cement taken at Packing Plant (PT. Semen Padang).

Aggregate

The sand and gravel samples were excavated from the gravel pits at Kuok and Pulau Gadang located at Kampar Kanan river side, about 10 km downstream and 4 km upstream of the dam site respectively. The samples were graded into 3 ranges of grain size, 5 mm under, 5-50 mm and 50 mm over, transported into laboratory. The sand was used for fine aggregate (0-5) mm and the gravel was used for the coarse aggregate in the grain size of (5-80) mm.

The physical tests were conducted in twelve parts, which were sieve analysis, specific gravity, unit weight, washing loss, absorption, abrasion, soft particles, soundness, clay lumps, organic impurity, particles less than 1.95 specific gravity aggregate and alkali-aggregate reaction.

The results of the tests are shown below. The samples (fine and coarse aggregate from Kuok and Pulau Gadang gravel pit) complied with the ASTM standard (ASTM C 33-85) and JSCE standard specification on the condition that the fine aggregate used for concrete was adjusted to meet the standard particle size distribution. Afterwards, adjusting the graduation of fine aggregate from Kuok and Pulau Gadang gravel pit was confirmed not to be necessary following examination of the result of sieve analysis executed at trial mixing.

Therefore, sand and gravel materials extracted from both gravel pits were confirmed to be suitable for producing concrete for the works.

Table 1.5.2 Result of Aggregate Test

	Test Result					Standard	
	S	G1	G2	G3	G4	Fine	Coarse
Specific gravity	2.62	2.58	2.61	2.62	2.70	min 2.5	min 2.5
Absorption (%)	0.93	0.84	0.71	0.54	0.15	max 3	max 3
Unit weight (t/m3)	1.69	1.72	1.70	1.72	1.85	1.50~1.85	1.55~2.00
Solid volume (%)	65.1	67.5	65.9	65.9	68.6	-	min 55
Organic impurity	lighter	lighter				lighter	lighter
Soundness (%)	4.4	-	1.22	2.98	4.31	max 10	max 12
Abrasion loss (%)	-	23.4	30.3	20.2	-	-	max 40
Clay lump (%)	0.35	0.20	0.28	0.03	0.09	max 1.0	max 0.25
Washing loss (%)	2.7	0.55	0.26	0.09	0.10	max 3.0	max 1.0
Particles less than 1.95 Specific gravity Aggregate (%)	0.64	0.17	0.07	0.04	-	max 0.5	max 1.0
Soft particles (%)	-	0.24	1.05	2.33	-	-	max 5.0
Alkali-aggregate Reaction (%)	-	0.014	0.013	0.012	0.016	-	max 0.05

Note : S : Fine aggregate
G1 : Course aggregate grading (20-5 mm)
G2 : Course aggregate grading (40-20 mm)
G3 : Course aggregate grading (80-40 mm)

The concrete test was divided into three (3) stages, preliminary test, mixing test and final mixing test . In the preliminary test, concrete mixing trials were conducted by changing the unit water and sand-aggregate ratio (s/a) until the appropriate unit water and (s/a) were found to meet the required workability, plasticity, slump, and air content. In the mixing test, several water - cement ratios (w/c) were used with a fixed value of unit water and sand - aggregate ratio (s/a) in order to obtain the relationship between the compressive strength and cement - water ratio (c/w). In the final mixing test, it was verified that the compressive strength of the specified mix exceeded the target strength of the concrete.

Concrete

A total of nine (9) types of concrete shown in the table below were tested to obtain the mix proportions using the cement and aggregate tested above, and the city water and the chemical admixtures complied with the ASTM standard (ASTM C.494).

Five (5) types of concrete were specified for inner and outer concrete of the dam and apron of the stilling basin with different the maximum size of aggregate, target strength and slump. The remaining four (4) types of concrete were specified for the powerhouse and other structures that had different the target strength and slump.

The results of the tests show below that the concrete mixes fulfilled the target strength, slump and air content specified for all types of concrete. Moreover maximum unit water, minimum cement content and maximum water-cement ratio were specified for the dam inner and outer concrete taking into the consideration the durability and water tightness requirements specified in the in JSCE standard specification.

Table 1.5.3 Result of Concrete Test

Type	Gmax	Slum	Air	W/C	s/a	Unit weight per 1m³ (kg)							Application
	(mm)	(cm)	(%)	(%)	(%)	W	C	S	G1	G2	G3	G4	
A1	150	3±1	3±1	57	26	95	167	567	322	322	403	564	Dam inner
A2	150	3±1	3±1	41	26	95	232	553	314	314	393	550	Dam outer
B1	80	3±1	3±1	65	30	105	162	644	516	479	479	-	Dam inner
B2	80	3±1	3±1	47	30	105	223	616	504	468	468	-	Dam outer
C	80	8±2	4.5±1	59	34	114	193	694	475	441	441	-	Stilling basin
D1	40	12±2	4.5±1	52	37.5	142	273	704	649	531	-	-	Powerhouse
D2	40	12±2	4.5±1	62	37.5	142	229	717	661	541	-	-	Retaining wall
D3	40	12±2	4.5±1	73	37.5	142	195	728	671	549	-	-	Tunnel
D4	40	8±2	4.5±1	60	37.5	137	228	722	666	545	-	-	Pier etc.

Note : W : Water G1 : Coarse Aggregate (5-20 mm) Water Retarding Agent: Cx0.3%
C : Cement G2 : Coarse Aggregate (20-40 mm) Air Entraining Agent : Cx0.15%
S : Fine aggregate G3 : Coarse Aggregate (40-80 mm)
G4 : Coarse Aggregate (80-150 mm)

The maximum size of coarse aggregate for the dam body was selected at 80 mm following the result of temperature stress analysis and economical study found that the production of maximum size of 150 mm would require the quarry and crushing plant.

The location of both gravel pits and the flow chart for marking the cement, aggregate and concrete tests are shown in Fig. 1.5.1 and Fig. 1.5.2 respectively.

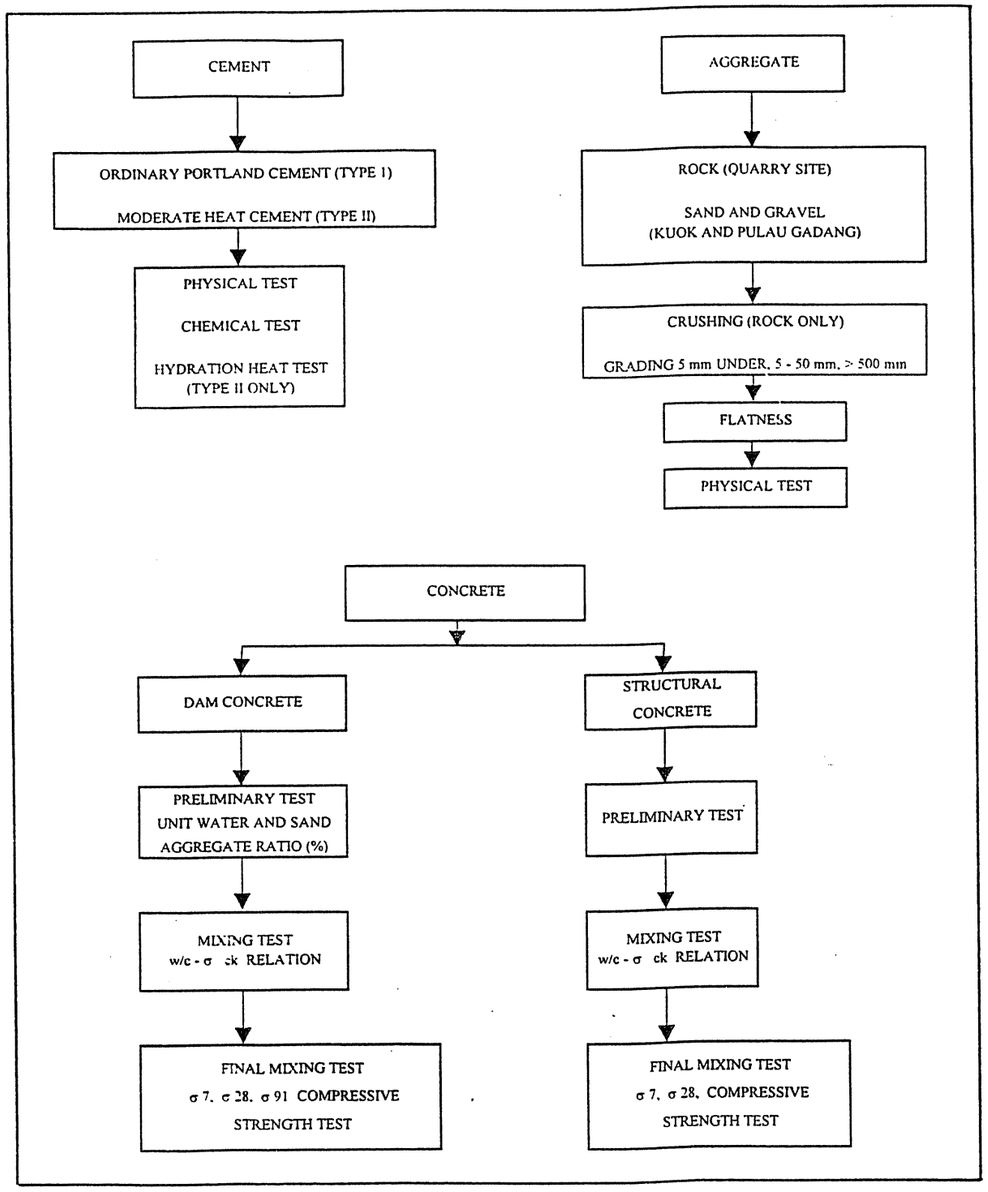


Fig. 1.5.2 Flow chart for Aggregate and Concrete Test

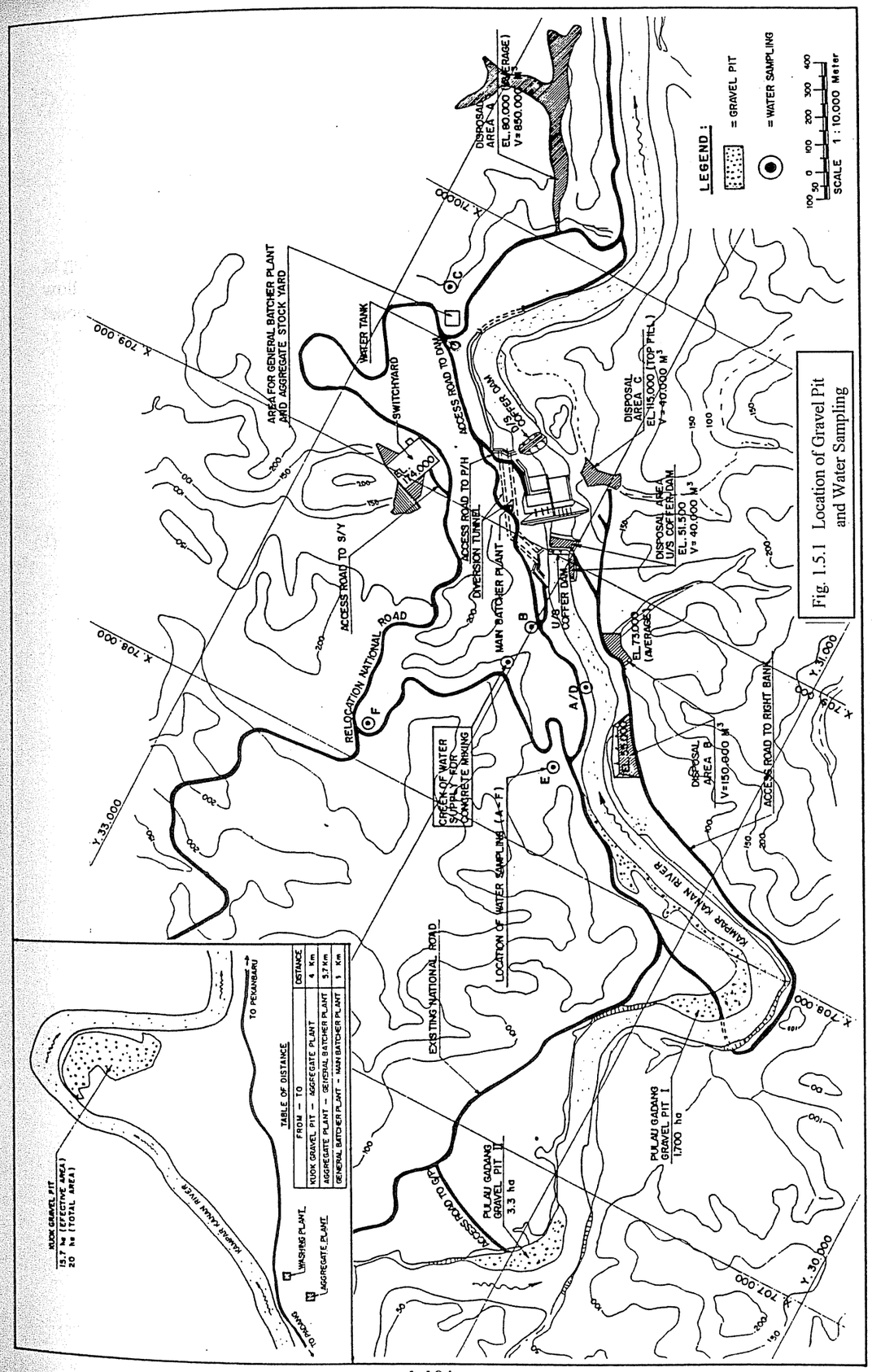


Fig. 1.5.1 Location of Gravel Pit and Water Sampling

5.2 Specification for Concrete and its Materials

5.2.1 Specification for Materials

(1) Cement

It was specified that ordinary portland cement and moderate heat cement conforming to ASTM Specification C.150 or JIS R.5210 should be used. The two types of cement follow the respective standard chemical requirement and physical requirement including optional requirement prescribed below.

Table 1.5.4 Standard and Optional Chemical Requirements

Cement Type	I and IA	II and IIA
Silicon dioxide (SiO2)	-	20.0 min
Aluminium oxide (AL2O3)	-	6.0 max
Ferric oxide (Fe2O3)	-	6.0 max
Magnesium oxide (MgO)	6.0 max	6.0 max
Sulfur trioxide (SO3)	-	30 max
When (C3A) C is 8% or less	3.0 max	-
When (C3A) C is more than 8%	3.5 max	-
Loss on ignition	3.0 max	3.0 max
Insoluble residue	0.75 max	0.75 max
Sum of tricalcium-silicate and tricalcium-aluminate	-	58 max
Alkalies (Na2O+0.658K20)	0.60 max	0.60 max

Table .1.5.5 Standard and Optional Physical Requirements

Cement Type	I	II
Fineness, specific surface (m^2/kg) (alternative methods) :		
Turbidimeter test	160 min	160 min
Air permeability test	280 min	280 min
Autoclave expansion	0.80 max	0.80 max
Compressive strength, psi (MPa)		
3 days	1800 (12.4) min	1500 (10.3) min
7 days	1800 (19.3) min	2500 (17.2) min
Time of setting (alternative methods)		
Gillmore test :		
Initial set, min, not less than	60 min	60 min
Final set, min, not more than	600 max	600 max
Vicat test :		
Time of setting, not less than	45 min	45 min
Time of setting. not more than	375 max	375 max
False set, final penetration (%)	50 min	50 min
Heat of hydration :	-	
7 days cal/g (kJ/kg)	-	70 (290) max

(2) Water

It was specified that the water for mixing and curing of concrete, mortar, plaster, and grout must be clean and free from deleterious substances including salt, oil, alkali or organic matter and conform to ACI. 318-77 section 3.4 or JASS 5T - 301.
The standard for water quality is tabulated below :

Table 1.5.6 Standard of Water Quality

Test Item	Unit	Standardized Value
pH	-	4.6 - 8.5
Suspension Substance	mg/l	< 2000
Solid Substance	mg/l	< 2000
Organic Substance	mg/l	< 2000
Sulfate	mg/l	< 10.000
Chloride	mg/l	< 10.000
Oil	%	< 2 %

(3) Aggregate

It was specified that fine and coarse aggregate for concrete must be obtained from gravel pits as shown on the Drawings, Pulau Gadang and Kuok gravel pits, and the Contractor was required to carry out all the processing operations necessary to obtain the required aggregate.

It was also specified that the aggregate must possess the suitable physical characteristics. In particular, the aggregate for concrete may be obtained by the Contractor from any source not specified, the Contractor must subject to the Engineer's approval submitting such samples and results of the tests to confirm that the aggregate comply with the Specification.

The other hand, it was determined that the aggregate from both gravel pits specified must possess the selected physical characteristics shown in the Table 1.5.7 because all of their physical characteristics were already confirmed to comply with the Specification by the results of the tests conducted during design stage and among the control items, important objects of control were gradation of coarse aggregate, and fineness modulus and surface moisture content of fine aggregate which greatly affect the various properties of concrete. In addition, unit weight test, specific gravity, absorption and surface moisture test must be carried out.

Table 1.5.7 Specification of Aggregate and Testing Items

Item	Standard requirement		Requirement from both gravel pits	
	Fine	Coarse	Fine	Coarse
Grading	shown below	shown below	o	o
Fineness modulus	2.3-3.1	-	o	o
Clay lumps	1 5 max	0.25 % max	-	-
Washing loss	3 % max	1 % max	o	o
Particles less than 1.95 specific gravity aggregate	0.5 % max	1.0 % max	-	-
Organic impurity	lighter	lighter	o	o
Soundness	10 % max	12 % max	-	-
Soft particles	-	5 % max	-	o
Abrasion loss	-	40 % max	-	o
Specific gravity	2.5 min	2.5 % min	o	o
Absorption	3 % max	3 % max	o	o
Unit weight	1.50-1.85 t/m^3	1.55-2.00 t/m^3	o	o
Surface moisture	-	-	o	-

Note : 0 : Testing item

Table 1.5.8 The Standard Grading of Fine Aggregate

Nominal Size to Sieve (mm)	10	5	2.5	1.2	0.60	0.30	0.15
Mass Passing (%)	100	90 to 100	80 to 100	50 to 90	25 to 65	10 to 35	2 to 15

Table 1.5.9 The Standard Grading of Coarse Aggregate

Mass of Passing (%)	Nominal Size of Sieve (mm)										
	100	80	60	50	40	25	20	15	10	5	2.5
Crushed stone G1 (05-20)mm	-	-	-	-	-	100	90 to 100	-	20 to 55	0 to 10	0 to 5
Crushed stone G2 (20-40)mm	-	-	-	100	90 to 100	20 to 55	0 to 15	-	0 to 5	-	-
Crushed stone G3 (40-80)mm	100	90 to 100	45 to 70	-	0 to 15	0 to 15	-	-	-	-	-

(4) Admixture

(a) Air Entraining Agents

It was specified that an air entraining agent had to conform to ASTM C.260 or JIS A.6204. The amount of air entraining agent used in each concrete mix had to be such as well effect, in concrete as discharged from the mixer, entrainment of air in the percentage as shown in the following tabulation :

Table 1.5.10 Total Air of Concrete

Maximum size of course aggregate (mm)	Total Air, Percent by Volume of Concrete
20	4.5 ± 1
40	4.5 ± 1
80	3.0 ± 1

(b) Water-reducing, set-retarding agent (WRA)

It was specified that a water-reducing agent (WRA) had to conform to ASTM C.494 or JIS 6204. WRA, set retarding agent (ASTM C.494, type D, or JIS A.6204) had to be used if the concrete was placed when the temperature of the concrete would exceed 30°C at the placement sites, as determined by the Engineer.

5.2.2 Specification for Concrete

(1) Mix proportion

The specifications for concrete were determined by type in accordance with the design strength (σck) at 28-day age, the maximum size of aggregate, types of cement and slump as shown in Table 1.5.11. The Contractor was responsible for the design of the concrete mix proportions to produce the required grade of concrete with minimum workability consistent with the position in which it was placed and with the method of placing and compaction used. If these conditions were not met during execution of the works, the mix proportion had to be redesigned.

Table 1.5.11 Design Strength (σ_{ck}) of Concrete

Concrete Type	Design Strength (σ_{ck}) (at 28 days) kg/cm^2	Max. Size of Aggregate	Slump
A1	330	80	3 ± 1
A2	220	80	3 ± 1
A3	280	40	8 ± 2
A4	280	40	12 ± 2
B1	280	40	8 ± 2
B2	260	40	12 ± 2
B3	260	40	8 ± 2
B4	240	40	12 ± 2
B5	220	40	8 ± 2
B6	210	20	16 ± 2
B7	280	20	10 ± 2
B8	120	20	8 ± 2

(2) Batching and Mixing

The minimum time of mixing and admissible error in the batching are shown in the tables below:

Table : 1.5.12 Minimum-Time of Mixing

Capacity of Mixer (m^3)	Minimum time of Mixing (Minutes)
3 to 2	2.5
2 to 1.5	2
1.5 or less	1.5

Table :1.5.13 Admissible Error in Batching

Material	Unit	Error
Cement	% by weight	1
Water	% by volume or weight	1
Admixture	% by volume or weight	3
Aggregate	% by Weight	3

(3) Sampling and Testing of Concrete

All concrete samples were based on the ASTM specification C-172.

- The test specimens for concrete strength are of cylindrical shape, 15 cm in diameter and a height of 30 cm.

- For testing purposes, the Contractor provided two sets of three cylinders taken from each class of fresh concrete placed in the works for each 300 m^3 of concrete but not less than six cylinders for each class for each day (or shift) of concreting.

- The test cylinders were made and cured in accordance with the ASTM specification C.31.

- The samples shall be tested for compressive strength in accordance with ASTM specification C. 39 or JIS A. 1108, and the test result shall be the average of the strengths of the three cylinders.

(4) Concreting in adverse weather conditions.

It was specified that the temperature of the fresh concrete shall not exceed 30 ^{0}C when placed in the forms. Effective measures such as precooling of aggregate and mixing water and placing at night had to be undertaken to meet the requirements.

5.3 Concrete Mix Design

Prior to starting the main construction works, trial tests were carried out during March 1994 up to March 1996 to determine the specified mix proportions for each concrete type by the Contractor. Specified mix proportion thus decided on were examined and adjusted utilizing actual batching plant to be used.

5.3.1 Type of Concrete

Before starting the main structural construction works, the mix proportions for the twelve (12) types of concrete, A1-A4 and B1-B8 specified corresponding to design strength, maximum size of aggregate and slump, were determined through the trial testing to satisfy the all requirements for each type of concrete, especially the target strength that was determined in accordance with the coefficient of variation assumed to be 15% temporarily.

As the construction works progressed, some of the original mix proportions mentioned above were modified based on the actual coefficient of variation or actual relation between the compressive strength and cement - water ratio through the quality control process.

On the other hand, several types of concrete were designed newly because of change of placing method, modification of design strength and so on.

In conclusion, the total numbers of concrete types designed in the works reached 33 types and their design condition and reason for modification or supplement were shown in the table below.

Types of Concrete in Civil Works

Type	σ ck (kg/cm²)	G max (mm)	Slump (cm)	Air (%)	Reason for modification	Main Application
A-1	330	80	3±1	3±1	Modify σt due to change of Cn	Dam (Outer)
A1-R1	330	80	3±1	3±1		ditto
A2	220	80	3±1	3±1	Modify C for placing on Foundation rock	Dam (Inner)
A2-R1	220	80	3±1	3±1		Dam (on Foundation rock)
A3	280	40	8±2	4.5±1	Modify s/a for placing around Penstock by pumping	Dam (Gallery, Penstock) Stilling basin, Powerhouse, Tailrace.
A3-P	280	40	8±2	4.5±1	Modify σt due to change of Cn	Dam (Penstock)
A3-R1	280	40	8±2	4.5±1	Modify slump due to reduce cement content	Dam, Penstock, Powerhouse (Draft tube)
A3-R2	280	40	4±2	4.5±1		Dam, Spillway pier
A4	280	40	8±2	4.5±1	For placing secondary concrete	Stilling basin, Tailrace Switchyard.
A7	280	20	10±2	4.5±1		Spillway, Intake.
B1	280	40	8±2	4.5±1		Diversion tunnel, Powerhouse.
B1-R1	280	40	8±2	4.5±1	Modify σt due to change of Cn	Stilling basin, Powerhouse, Tailrace, penstock.
B1-R2	280	40	12±2	4.5±1	For placing to training wall in stilling basin by pumping	Stilling basin
B1-R3	280	40	10±2	4.5±1	For placing to training wall in stilling basin by pumping	Stilling basin
B1-R4	280	40	8±2	4.5±1	Modify w/c due to change of σ28-c/w	Stilling basin, Powerhouse, Switchyard
B2	260	40	12±2	4.5±1		Diversion tunnel.
B2-R1	260	40	12±2	4.5±1	Modify σt due to change of Cn and σ28-c/w	Diversion tunnel, Inlet, Outlet.
B2-R2	260	40	12±2	4.5±1	Modify σt due to change of Cn and σ28-c/w	Powerhouse, Tailrace, Test adit
B2-R3	260	40	12±2	4.5±1	Modify w/c due to change of σ28-c/w	
B3	260	40	8±2	4.5±1		Diversion tunnel, Tailrace
B4	240	40	12±2	4.5±1	Modify σt due to change of Cn and σ28-c/w	Diversion tunnel, Inlet, Outlet
B4-R1	240	40	12±2	4.5±1	Modify σt due to change of Cn and σ28-c/w	Inlet, Outlet
B4-R2	240	40	12±2	4.5±1		Powerhouse, Tailrace, Switchyard.
B5	220	40	8±2	4.5±1		Powerhouse, Switchyard, Test adit
B6	210	20	16±2	4.5±1		Powerhouse, Switchyard (Relay house)
B7	280	20	10±2	4.5±1		Powerhouse
B7-R1	280	20	10±2	4.5±1	Modify σt due to change of Cn and σ28-c/w	Powerhouse, Tailrace, Precast panel, Dam (hoist deck)
B8	120	20	8±2	4.5±1		Gantry Crane
B8-R1	120	20	8±2	4.5±1	Modify w/c based on trial mixing text by batching plant.	Switchyard (Relay house), Cable trench
B9-R3	85	20	8±2	-	For back filling	Powerhouse (backfill)
B11	85	20	10±2	-	For back filling	Powerhouse (backfill), Tailrace (backfill)
PC-R1	500	20	18-24	-	For prestress concrete bridge	Spillway (Bridge), Hoist deck)

5.3.2 Design of Mix Proportion

(1) Procedure of Mix Design by trial Methods

Mix proportions of concrete must be selected to make the most economical use of available materials that produce concrete of the desired properties. The most satisfactory combination of fine and coarse aggregate is that which enable the desired workability and strength with minimum cement consumption.

The trial mix procedure was carried out according to the following steps.

(i) Test of material used
(ii) Selection of maximum solids content of coarse aggregate
(iii) Selection of optimum unit water content from relation ship between unit water content and slump.
(iv) Selection of optimum sand percentage (s/a) form relation ship between sand percentage and slump.
(v) Selection of water cement ratio (w/c) from relationship between compressive strength and water - cement ratio.
(vi) Mix proportion was selected by mixing trial batches with the optimum unit water content, sand percentage and water - cement ratio adjusting to obtain the required slump, air content and target strength and regulating fine and coarse aggregate ratio on judgements of the workability of concrete.

The trial mix procedure of dam concrete is shown in Fig. 1.5.3 and power station concrete is shown in Fig. 1.5.4.

The layout of concrete laboratory is shown in Fig 1.5.5 and major pieces of the equipment are listed inTable 1.5.14.

(2) Material Used and Specifications Requirement

(a) Cement

The ordinary Portland cement type I and the moderate heat Portland cement type II were purchased from PT. Semen Padang Factory. Type I cement was used for type B concrete for pouring in Powerhouse and other structures work and type II cement was used for type A concrete for pouring Dam Work.

The cement was sampled and tested for strength and physical properties according to ASTM C. 150 or JIS R. 5210. The test result or cement certificate is shown in Table 1.5.15.

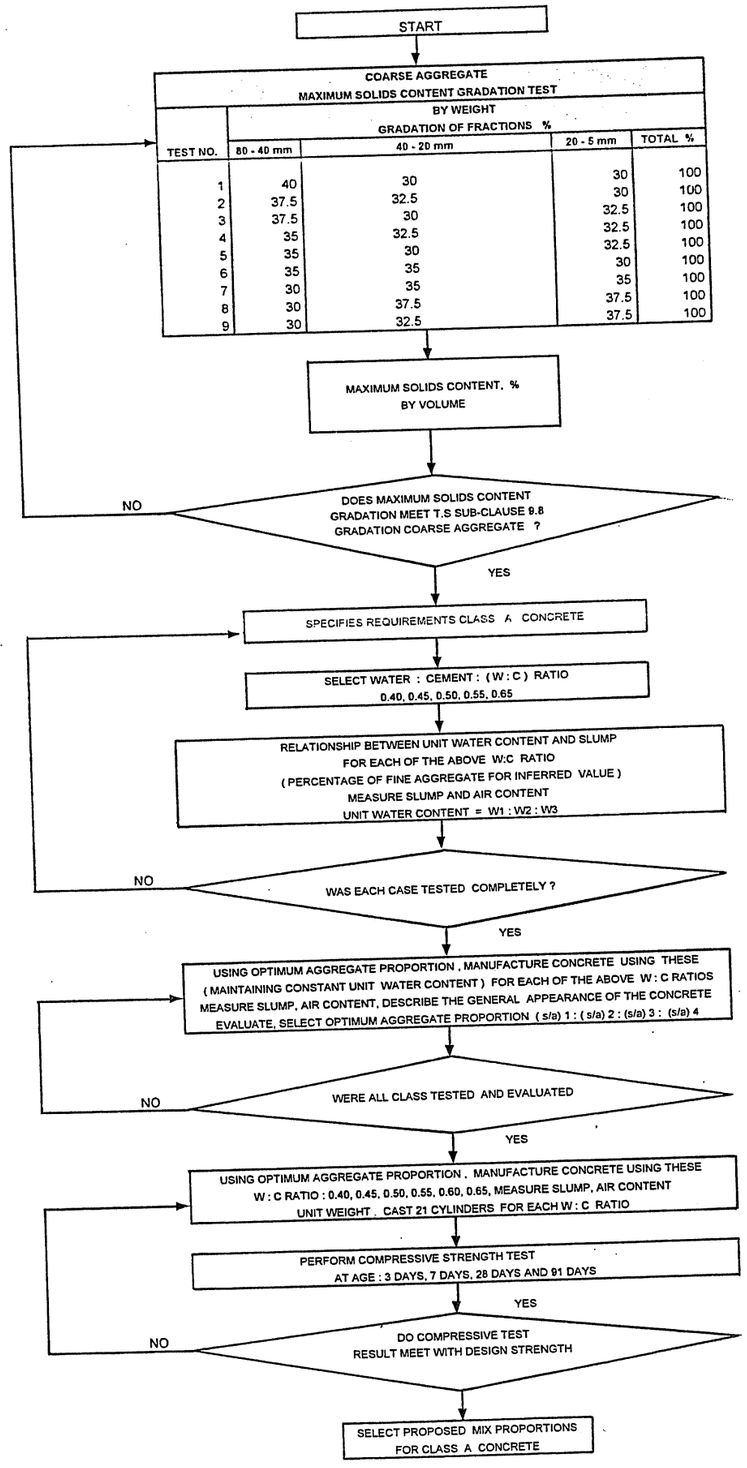


Fig. 1.5.3 Flow-Chart of Trial Mix Procedure for Dam Concrete

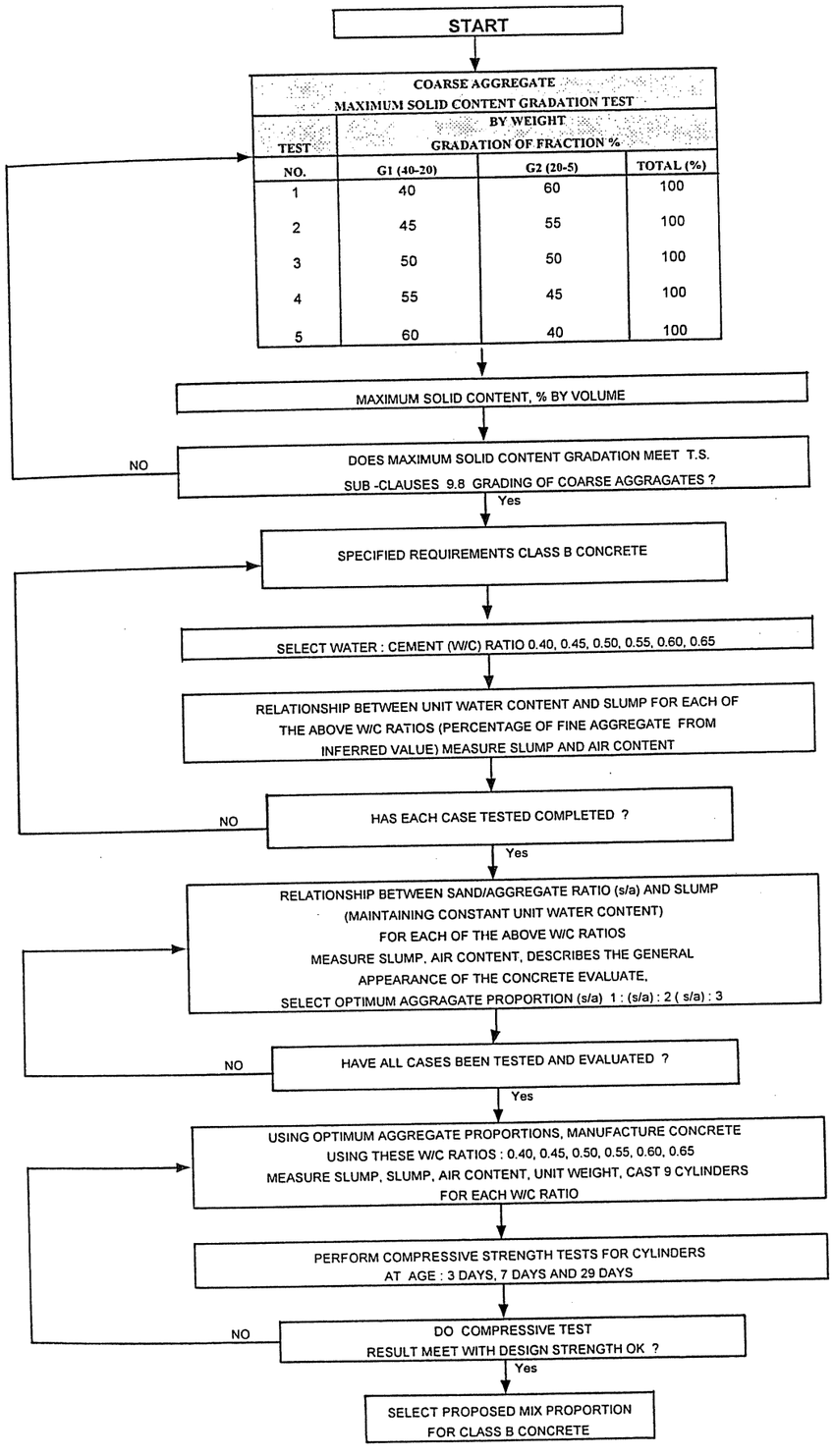
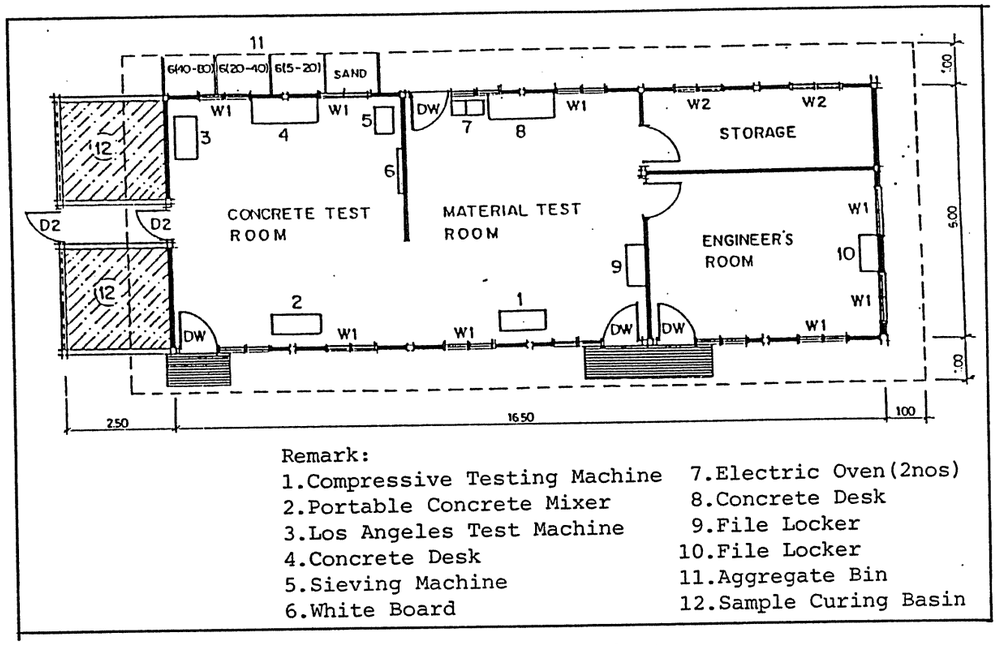


Fig. 1.5.4 Flow-Chart of Trial Mix Procedure for General Concrete



Remark:

1.Compressive Testing Machine	7.Electric Oven(2nos)
2.Portable Concrete Mixer	8.Concrete Desk
3.Los Angeles Test Machine	9.File Locker
4.Concrete Desk	10.File Locker
5.Sieving Machine	11.Aggregate Bin
6.White Board	12.Sample Curing Basin

Fig. 1.5.5 Layout of Concrete Laboratory

Table 1.5.14 (1) Major Pieces of Equipment

No.	Description	Code	Quantity
I	CEMENT TEST		
1	Vicat Apparatus With Needle	CE - 120	1 SET
2	Mixing Bowl	GE - 363	1 SET
3	Spoon	GE - 810	1 SET
4	Spatula	GE - 820	3 PCS
5	Le Chatelier Flask	CE - 111	5 PCS
II	MORTAR TEST		
1	Mortar Flow Apparatus	CE - 160	1 SET
2	Beam Mould 40 x 40 x 160 mm	CE - 242	3 SET
3	Cylinder Mould (ϕ5 x 10)	EL.39 - 1810/01	3 SET
4	Proctor Penetrometer ASTM With Needle	EL.34 - 1810	
5	Mechaelis Bending Tester	EL.39 - 1810/01	1 SET
III	AGGREGATE TEST		
A	FINE AGGREGATE		
1	Standard Sieve Analysis (10 s/d 0.15)	MBT	2 SET
2	Sand Absorption Cone	AG - 811	1 SET
3	Chapman Flask. 500 cc	CE - 412	5 SET
4	Organic Impurities Test Set	AG - 910	1 SET
B	COARSE AGGREGATE		
1	Standard Sieve Analysis 100, 80, 50, 40, 30, 20, 15, 10 mm	MBT	1 SET
2	Los Angeles Test Machine	AG - 700	1 SET
3	Sieve # 0.074 mm	AG - 242 A	1 PCS
	# 0.088 mm	AG - 241 A	1 PCS
4	Wooden Frame Hand Sieve, (2/8", 5/8", 3/4", 1", 1 1/4", 1 1/2", 2", 2 1/2", 3", 4")	MBT	1 SET
IV	CONCRETE TEST		
1	Slump Test Set	CO - 370	1 SET
2	Washington Type Air Meter Cap. 7 Liters	MIC : 138.1.01	1 SET
3	Washington Type Air Meter Cap. 15 Liters	MIC : 138.1.15	1 SET
4	Internal Vibration	MIC : 138.0.01	1 SET
5	Cylinder Mould (ϕ 15 x 30)	CO - 301	40 PCS
6	Straight Edge	GE.890	3 PCS
7	Specimen Griper	BI - 407	1 PCS
8	Cement Knife Stainless	MBT	3 PCS
9	Concrete Pan	GE. 303	2 PCS
10	Cement Can Cap	MBT	3 PCS

Table 1.5.14 (2) Major Pieces of Equipment

NO	DESCRIPTION	CODE	QUANTITY
11	Rubber Mallet	GE - 900	10 PCS
12	Cylinder Tongs	BI - 407 A	3 PCS
13	Cylinder Carries	CO - 470 A	3 PCS
14	Vertical Capping Set	CO - 470 A	1 SET
15	K. Slump Tester	EL. 34 - 0580	1 SET
16	Concrete Test Hummer.	CO - 550	1 PCS
IV	SOIL TEST AN EMBANKMENT CONTROL		
1	Hand Auger Set	SO - 110	1 SET
2	Picnometer With Stopper	GE - 412	6 PCS
3	Hydrometer Jar, 1000 CC	SO - 337	6 PCS
4	Hydrometer 0.095. 1.050	SO - 332	2 PCS
5	Mechanical String	SO - 333	1 SET
6	Soil Analysis Sieve Set 50.8 - 0.074 mm (2", 1 1/2", 3/4", 3/8", No. 4, 10, 20, 40, 60, 140, 200, PAN)		1 SET
7	Liquid Limit Test Set	SO - 310	1 SET
8	Plastic Limit Test Set	SO - 315	1 SET
9	Standard Compaction Hold 04"	SO - 351	1 SET
10	Standard Compaction Hold 6"	SO - 352	1 SET
11	Standard Compaction Hammer, Weigth 2.5 kg	SO - 353	1 SET
12	Standard Compaction Hammer, Weight 4.5 Kg	SO - 354	1 SET
13	Sand Density Cone Set	SO - 400	1 SET
14	Field Density Plate	SO - 403	2 SET
15	Field CBR Set Job Set Test	SO - 260	1 SET
16	Infra Red Moisture Meter Kett Japan	FD - 1A	1 SET
17	Evaporating Dish	GE - 500	24 PCS
		GE - 501	6 PCS
		GE - 502	6 PCS

Table 1.5.14 (3) Major Pieces of Equipment

No.	Description	Code	Quantity
VI	GENERAL		
1	Thermometer 100 C	GE - 601	6 PCS
2	Thermometer 200 C	GE - 603	3 PCS
3	Stop Watch 60 Second	GE - 700	2 PCS
4	Shovel	GE - 815	3 PCS
5	Large Spoon	GE - 806	2 PCS
6	Graduated Cylinder 100 CC	GE - 402	3 PCS
7	Graduated Cylinder 25 CC	GE - 400	2 PCS
8	Graduated Cylinder 10 CC	MBT	2 PCS
9	Enameled Iron Ware 1000 CC	GE - 424	3 PCS
10	Enameled Iron Ware 500 CC	GE - 422	3 PCS
11	Pipette 25 CC	GE - 533	2 PCS
12	Pipette 50 CC	GE - 534	2 PCS
13	Enameled Tray	GE - 320	5 PCS
14	Enameled Tray	GE - 320	5 PCS
15	Steel Ruler 300 mm	GE - 895	1 PCS
16	Steel Ruler 450 mm	GE - 895 A	1 PCS
17	Hand Level	GE - 74	2 PCS
18	Vernier Calipers	GE - 713	1 PCS
19	Pressure Gauge 100 ton	MBT	1 PCS
20	Dial Gauge 30 mm	GE - 137	5 PCS
21	Magnetic Stand Dial Holder	GE - 140	4 PCS
22	Brushes for Fine	GE - 921	10 PCS
23	Brushes for Concrete	GE - 920	10 PCS
24	Laboratory Timer	EL. 81.0518	1 PCS
25	Electrical Oven "MEMMERT"	GE - 171	2 PCS
26	Hot Plate 600 Watt	Sanyo	1 SET
27	Heavy Duty Solution	GE - 157	1 SET
28	Triple Beam Balance Cap 2.610 gr	GE - 152	1 SET
29	Sample Extruder	SO - 355	1 SET
30	Desiccator	GE - 452	1 PCS
31	Mortar	GE - 511	1 SET

Table 1.5.15 Results of Cement Test

Test Item	Test Result		ASTM standard	
	Type II	Type I	Type II	Type I
- Chemical Composition (%)				
Silicon dioxide	21.68	-	min 20.0	-
Aluminum oxide	4.16	-	max 6.0	-
Ferric oxide	4.36	-	max 6.0	-
Magnesium oxide	0.57	0.79	max 6.0	max 6.0
Sulfur trioxide	2.09	2.37	max 3.0	max 3.0
Loss on ignition	1.26	1.27	max 3.0	max 3.0
Insoluble residue	0.26	0.30	max 0.75	max 0.75
Tricalcium aluminate	0.36		max 8.0	
Alkalies (Na2O + 0.658 K2O)	0.20	0.22	max 0.60	max 0.60
- Physical Properties				
Fineness, specific surface air Permeability test (m^2/kg).	314	318	min 280	min 280
Auto calve expansion compressive strength (kg/cm^2).	0.14	0.10	max 0.80	max 0.80
3 days	170	176	min 103	min 124
7 days	253	247	min 172	min 193
Time of setting, vicat test :				
Initial set (minutes)	130	115	min 45	min 45
Final set (minutes)	245	210	max 375	max 375
Meat of Hydration 7 day (cal/gr).	63.50	-	max 70	
False set, final penetration (%).	86.60	62.60	min 50	min 50

(b) Water

The water for mixing and curing of concrete was taken from Kampar River. The test for water quality for the concrete mix was done by the laboratory of Department of Public Work of Riau Province. The sample for water test was taken at 6 locations in the Kampar river as shown in Fig. 1.5.1 and the result is shown in Table 1.5.16.

Table 1.5.16 Test Result of Water

Test Item	Unit	Technical Requirement	Test Result					
			Kampar Kanan (A)	River near Dam Access (B)	Silanuk River (C)	Kampar Kanan after rain (D)	River near Powder Magazine (E)	River near Relocation Road (F)
pH	-	4.6 - 8.5	7.1	7.1	6.8	7.0	5.7	7.0
Suspension Substance	mg/l	< 2,000	19	44	21	88	43	95
Organic Substance	mg/l	< 2,000	3,84	11.65	3.84	1.9	1.17	9.54
Sulfate	mg/l	< 10,000	35.78	77.18	23.6	34.97	37.4	21.98
Chloride	mg/l	< 10,000	2.50	1.5	2.5	4.03	2.01	1.65

(c) Aggregates

The original gravel was supplied from the Kampar Kanan River, and taken from two resources, i.e. Pulau Gadang, approximately 4 km upstream of the dam site, and Kuok, approximately 6 km downstream of the dam site. The gravel was excavated by excavator and transported by dump trucks and it was stockpiled at the aggregate and crushing plant yard, which was located at Rantau Berangin, approximately 4 km downstream of the dam site.

Flow charts of the aggregate production plants for the dam concrete and general concrete purposes, are shown in Fig 1.5.6 and 1.5.7.

Aggregate produced from Rantau Berangin plant was classified into 4 ranges: Fine aggregate (0-5 mm) and coarse aggregate, G1 (5-20 mm), G2 (20-40 mm), G3 (40-80 mm). The result of the physical property test for aggregate is shown below.

	Fine	Coarse Aggregate			Standard Requirement	
	< 5	5-20	20-40	40-80	Fine	Coarse
Specific Gravity	2.60 2.64	2.64 2.61	2.61 2.62	2.63	2.5 min	2.5 min
Unit Weight (t/m^3)	1.63 1.67	1.69 1.73	1.66 1.74	1.08		
Absorption (%)	0.87 0.48	1.00 1.22	0.99 0.94	0.93	3 max	3 max
Washing Loss (%)	2.06 2.36	0.92 0.80	0.77 0.49	0.75	3 max	1 max
Organic Impurity	Lighter	-	-	-	Lighter	Lighter
Abrasion (%)	- 	23.60 25.85	22.70 25.85	16.30 	-	40 max
Softrock (%)	-	-	-	4.40	-	-
Fines Modulus	2.76 3.06	6.82 6.60	7.96 8.01	8.91 8.91	2.3-3.1	5 max

Note, Upper value : Aggregate for dam concrete
Lower value : Aggregate for general concrete

(d) Admixture

Air entraining admixture : Pozzolith 303 A
Water reducing admixture : Pozzolith 300 R

(3) Target Strength of Concrete

Target strength (σ_t) which was used for selecting mix proportion of concrete, was obtained as the design increased through multiplication by over design factor (site coefficient).

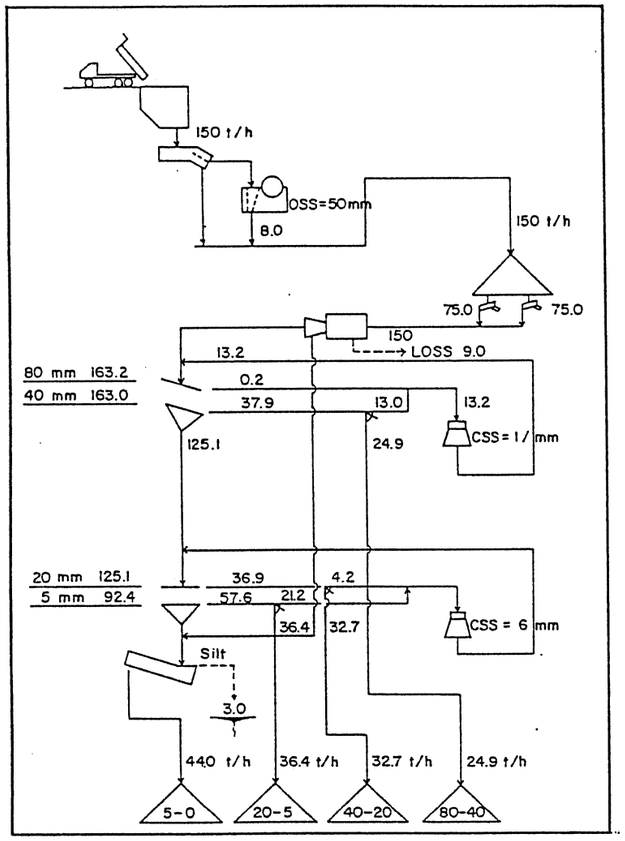


Fig. 1.5.6 Flow-Chart of Aggregate Production for Dam Concrete

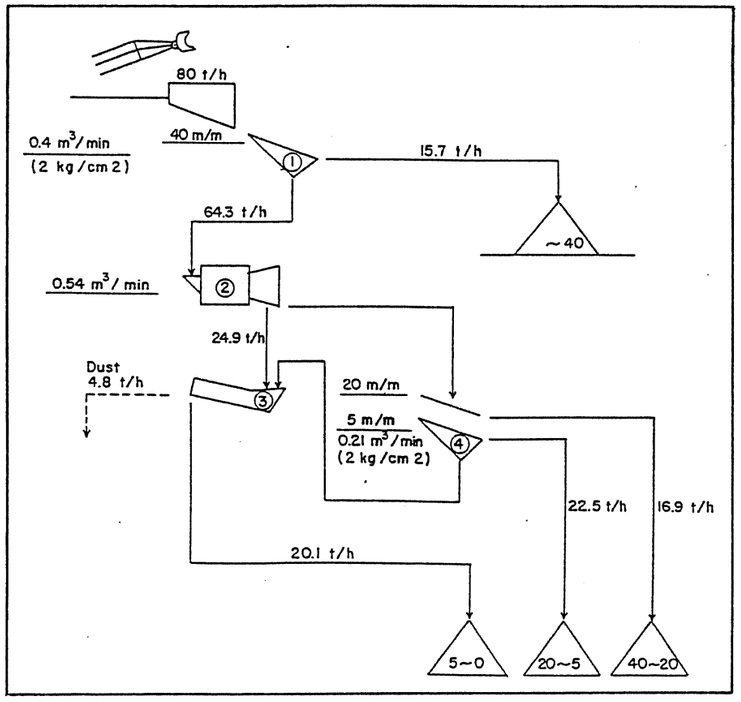


Fig. 1.5.7 Flow-Chart of Aggregate Production for General Concrete

The target strength partly depends upon the result of the compression tests satisfying the following condition :

(i) the probability for test results to have values lower than 80% of the design strength must be less than 5%.

(ii) the probability for test results to have values lower than the design strength must be less than 25%.

Based on the above, the target strength (σt) was formulated as shown below ;

$$\sigma t = \alpha \, \sigma ck$$

where, α : Site coefficient, which was expressed as either $\alpha 1$or $\alpha 2$ whichever the larger outcome from the following formulas

$$\alpha 1 = 0.80/(1-k1.Cn), \; \alpha 2 = 1.0/ (1-k2.Cn)$$

k1 : Probable deviation for a probability of 5% (=1.645)
k2 : Probable deviation for a probability of 25% (=0.675)
Cn : Coefficient of variation

Site coefficient α must be determined based on the Contractor's experience, equipment, variation of material and quality control technique. Before producing concrete, coefficient of variation (Cn) generally is set in the range 10-15%.

(4) Concrete Mix Design

(a) Maximum unit weight of coarse aggregates

Tests for unit weight and solid volume percentage of coarse aggregates was carried out to establish the optimum solid volume percentage of coarse aggregate fraction G1, G2 and G3 for 80 mm and 40 mm in coarse maximum size. The results of the test are shown below, the maximum solid volumes for maximum size of 80 mm and 40 mm were yielded by the solid volume percentage of coarse aggregate fraction 30% (G1) :35% (G2):35% (G3) and 45% (G1) : 55% (G2) respectively and both mix proportions were not segregated readily at concrete placing.

Table 1.5.17 Solid Volume Percentage of Coarse Aggregate

G max : 80 mm

Mix Proportion (%)	5-20	30.0	30.0	32.5	32.5	35.0	30.0	35.0	32.5	37.5	33.3
	20-40	30.0	32.5	30.0	32.5	30.0	35.0	35.0	37.5	32.5	33.3
	40-80	40.0	37.5	37.5	35.0	35.0	35.0	30.0	30.0	30.0	33.3
Solid Vol. (%)		64.4	64.2	64.1	63.6	64.1	64.4	63.0	63.0	63.4	63.1

G max : 40 mm

Mix Proportion (%)	5-20	40	45	50	55	60
	20-40	60	55	50	45	40
Solid Vol.	(%)	67.6	68.1	67.6	67.6	67.5

(b) Relationship between compressive strength and cement-water ratio

The table below shows relationships between compressive strength at 28 days age (σ_{28}) and cement-water ratio (c/w) for each type of concrete obtained through tests of concrete compressive strength for trial mixing batches with optimum unit water content and sand percentage (s/a) found by their tests. And modified relationships during construction period through the quality control control process are also shown in the table below.

Table 1.5.18 Relationship between σ_{28} - c/w

Type	Relationship	Type	Modified Relationship
A1, A2	- 112.3 + 217.6 c/w	B1 - R4	- 110.1 + 199.5 c/w
A3	- 39.4 + 133.3 c/w	B2 - R1	49.9 + 118.3 c/w
A4	- 56.5 + 175.5 c/w	B1 - R2, B2 - R2, B4 - R2	12.3 + 129.5 c/w
B1, B3, B5	- 130.1 + 199.5 c/w	B2 - R3	23.3 + 129.5 c/w
B2, B4	87.8 + 96.6 c/w	B4 - R1	87.8 + 118.3 c/w
B6	- 317.8 + 336.4 c/w	B7 - R1	- 264.0 + 285.9 c/w
B7	- 207.9 + 285.9 c/w		
B8	78.5 + 65.8 c/w	PC	355.2 + 64.2 c/w

(c) Target strength

The coefficient of variation (Cn) of the general batching plant showed 10% during trial tests. At the design mix stage, the coefficient of variation of 15% was selected to provide a margin. Thus, the site coefficient (α) calculated to be 1.112.

The coefficient of variation for mix design was changed to 10% based on the actual results where the coefficient showed less than 10% at both batching plants from results of Quality Control.

The coefficient of variation and the target strength adopted for each concrete type are shown in Table 1.5.19.

(d) Mix proportion

To establish the mix proportion of each concrete type, the water - cement ratio (w/c) is, in general, determined based on the required compressive strength (target strength).Moreover, w/c of outer concrete of the dam (Type A1) was also satisfied with the w/c of less than 55% required to be durable and water tight.

The mix proportions of each type of concrete actually used are shown in the Table. 1.5.19.

Table 1.5.19(1) Mix Proportion of Type A Concrete

Type	σ ck (kg/cm²)	Cn (%)	σt (kg/cm²)	G max (mm)	Slump (cm)	Air (%)	W/C (%)	s/a (%)	WTR (kg)	CMT (Kg)	Aggregate (kg)				Admixture (Gr)	
											Sand	G5-20	G20-40	G40-80	WRA	AE* 10
A1	330	15	367	80	3±1	3±1	45	26	101	223	539	468	539	544	789	405
A1-R1	330	10	354	80	3±1	3±1	47	26	101	216	547	465	545	545	648	390
A2	220	15	245	80	3±1	3±1	61	29	101	166	615	459	529	533	588	302
A2-R1	220	15	245	80	3±1	3±1	55	28	101	184	588	460	536	536	551	441
A3	280	15	311	40	8±2	4.5±1	49	36	137	280	688	549	673	-	1008	424
A3-P	280	15	311	40	8±2	4.5±1	45	35	136	302	669	553	678	-	906	540
A3-R1	280	10	300	40	8±2	4.5±1	50	36	135	270	694	553	678	-	956	486
A3-R2	280	10	300	40	4±1	4.5±1	50	36	125	250	714	565	693	-	900	450
A4	280	15	311	40	12±2	4.5±1	48	36	142	297	679	541	664	-	1069	450
A7	280	10	300	20	10±2	4.5±1	51	43	157	308	790	1044	-	-	1109	550

Note : WRA = Water-reducing, set-retarding agent : Pozzolith 300 R
AE = Air Entraining agent : Pozzolith 303 A

Table 1.5.19(2) Mix Proportion of Type B Concrete

Type	σ ck (kg/cm²)	Cn (%)	σt (kg/cm²)	G max (mm)	Slump (cm)	Air (%)	W/C (%)	s/a (%)	WTR (kg)	CMT (kg)	Aggregate (kg)				Admixture (Gr)		
											Sand	G5-20	G20-40	G40-80	WRA	AE* 10	RH 716
B1	280	15	311	40	8±2	4.5±1	45	35	135	298	668	552	677	-	894	536	-
B1-R1	280	10	300	40	8±2	4.5±1	47	35	135	290	671	554	680	-	870	609	-
B1-R2	280	10	300	40	12±2	4.5±1	45	35	140	311	663	547	672	-	1120	653	-
B1-R3	280	10	300	40	10±2	4.5±1	47	34	136	289	647	563	690	-	867	730	-
B1-R4	280	10	300	40	8±2	4.5±1	49	37	137	280	710	538	660	-	840	590	-
B2	260	17	295	40	12±2	4.5±1	47	37.5	153	325	709	526	643	-	1275	-	-
B2-R1	260	12	284	40	12±2	4.5±1	51	37.5	153	300	686	517	631	-	1176	-	-
B2-R2	260	10	279	40	12±2	4.5±1	49	36	141	290	684	541	664	-	870	609	-
B2-R3	260	10	279	40	12±2	4.5±1	51	37	143	280	699	533	654	-	840	504	-
B3	260	15	289	40	8±2	4.5±1	48	36	136	286	690	546	670	-	858	601	-
B4	240	20	279	40	12±2	4.5±1	51	37.5	153	300	717	532	651	-	1177	-	-
B4-R1	240	12	264	40	12±2	4.5±1	55	37.5	153	278	693	522	638	-	1090	-	-
B4-R2	240	10	257	40	12±2	4.5±1	53	37	142	268	709	537	659	-	804	482	-
B5	220	15	245	40	8±2	4.5±1	53	37	137	258	717	543	666	-	774	464	-
B6	210	15	234	20	16±2	4.5±1	61	45	179	279	825	997	-	-	837	335	-
B7	280	15	311	20	10±2	4.5±1	55	44	158	287	817	1028	-	-	861	344	-
B7-R1	280	10	300	20	10±2	4.5±1	51	43	157	308	789	1042	-	-	942	370	-
B8	120	15	133	20	8±2	4.5±1	85	40	153	180	805	1199	-	-	540	216	-
B8-R1	120	15	133	20	8±2	4.5±1	60	45	155	260	849	1026	-	-	780	312	-
B9-R3	85	-	-	20	8±2	-	102	48	170	166	964	1087	-	-	415	-	-
B11	85	-	-	20	10±2	-	103	55	185	180	1025	836	-	-	540	450	-
PC-R1	500	15	556	20	18-24	-	35	43	170	490	741	979	-	-	-	-	5880

Note : WRA = Water -reducing, set-retarding agent : Sika VZ
AE = Air-Entraining agent : Sika AER
RH-716 = Water-reducing, set-retarding agent : Rheobuild 716

5.4 Concrete Quality Control

The test item, method and sampling location of quality control selected for the project concrete works are listed in Table 1.5.20.

(1) Cement and Admixture

Certificate reports of cement and admixture indicated that all of their testing values met specificated quality requirement as shown in Table 1.5.21 (1) and (2).

(2) Aggregate

Table 1.5.22 (1) to (3) contain the test results on a monthly basis during March 1994-August 1997. The control charts of fineness modulus and surface moisture content of fine aggregate are shown in Fig. 1.5.8 and Fig. 1.5.9 respectively.

Fineness Modulus of Fine Aggregate

Fineness modulus of fine aggregate (F.M) was 2.69 in average and coefficient of variation was 3.2 %. This average value (2.69) met the specification requirement (2.3-3.1) and the coefficient of variation of 3.2 % was small. The difference between each sample and the average of the 10 consecutive test samples was controlled within 0.20 and then the control for fineness modulus of fine aggregate was carried out good.

Surface Moisture Content of Fine Aggregate

The moisture content of fine aggregate for the dam concrete and the general concrete were 7.6% and 4.9% respectively in average and the coefficient of variation for both concrete was 1.5%.

Other Quality Items

As for other physical properties, satisfactory results of quality control tests were obtained. In conclusion, fine and coarse aggregate exhibited good and stable quality.

(3) Concrete

The key check items of quality control were slump, air content and compressive strength for the typical concrete type which were placed in the powerhouse and other structures. Furthermore, for concrete placed in the dam, temperature of concrete at placing was important check item for the temperature control for dam concrete. The concrete temperature at placing was 27.0^{0}C in average and the coefficient of variation was 1.0% as shown in Table 1.5.22 and Fig. 1.5.10. This value met the specification requirement.

The results of the statistical analysis in term of average value, standard deviation and coefficient of variation are summarized in Table 1.5.23. The control charts of major type of concrete are shown in Fig. 1.5.11.

In conclusion, the concrete of all types was controlled within the allowable ranges of strength, temperature, workability and so on.

Table 1.5.20 Test Item and Method of Concrete Quality Control

	Test Item	Frequency	Method	Remarks
Material	1. Cement	Once a month or once each 1000 tons.	ASTM C.150-85	Through the certificate submitted by the Manufacturer
	2. Admixture - Water Reducing - Air-Entraining	Once for every shipment	ASTM C.260 ASTM C.494	Through the certificate submitted by the Manufacturer
	3. Coarse Aggregate - Sieve Analysis - Specific Gravity - Water Absorption - Unit Weight - Abrasion - Soft Rock	 once a week once a month once a month once a week once a month once a month	 JIS-A.1102 JIS-A.1110 JIS-A.1110 JIS-A.1104 JIS-A.1121 JIS-A.1126	
	4. Fine Aggregate - Sieve Analysis - Specific Gravity - Water Absorption - Unit Weight - Washing Loss - Organic Impurities - Surface Moisture Content	 twice a week once a month once a month once week once week once a month Once two hours and whenever slump was not met requirement.	 JIS-A.1102 JIS-A.1109 JIS-A.1109 JIS-A.1104 JIS-A.1103 JIS-A.1105 JIS-A.1111	
Concrete	5.Fresh Concrete - Slump - Air content - Temperature	Once 300 m^3 per each mix or once two hours	JIS-A.1101	
	6. Hardened Concrete - Compressive Strength	Once 300 m^3 per each mix or once two hours	JIS-A.1108	For each test sample - 9 test specimen for dam concrete - 6 test specimen for other concrete
Batching Plant	7. Checking Scale - Cement - Water - Admixture - Coarse Aggregate - Fine Aggregate	Once a month		The scales are checked at 25 kg intervals by actual static test weight

Table 1.5.21 (1) Result of Quality Control of Ordinary Portland Cement (1)

TEST ITEM	UNIT	SPECIFICATON REQUIREMENT	1993				1994				1995			
			N	x	Sn	Cn	N	x	Sn	Cn	N	x	Sn	Cn
CHEMICAL COMPOSITION														
Alkalies (Na2O + 0.658 K2O)	%	max 6.0 (Optional Requirement)	19	0.20	0.02	0.10	22	0.18	0.02	0.11	21	0.20	0.01	0.05
Insoluble Residue	%	max 0.75	19	0.32	0.05	0.16	22	0.36	0.06	0.17	21	0.38	0.04	0.11
Loss on Ignition	%	max 3.0	19	1.63	0.29	0.18	22	1.65	0.28	0.17	21	1.71	0.40	0.23
Magnesium Oxide (MgO)	%	max 5.0	19	0.79	0.03	0.04	22	0.80	0.01	0.01	21	0.82	0.04	0.05
Sulfur Trioxide (SO3)	%	max 3.0 when C3a < 8	19	2.44	0.29	0.12	22	2.47	0.19	0.08	21	2.44	0.25	0.10
		max 3.5 when C3a > 8	-	-	-	-	-	-	-	-	-	-	-	-
PHYSICAL PROPERTIES														
Fineness, Specific Surface - Air Permeability Test	m2/Kg	min 280	19	309.00	6.00	0.02	22	310.00	6.00	0.02	21	312.00	5.00	0.02
- Autoclave Expansion	%	max 0.80	19	0.11	0.03	0.27	22	0.09	0.02	0.22	21	0.10	0.01	0.10
Compressive Strength														
- 3 days	Kg/Cm2	min 124	19	175.00	10.00	0.06	22	160.00	6.00	0.04	21	158.00	7.00	0.04
- 7 days	Kg/Cm2	min 193	19	247.00	13.00	0.05	22	228.00	7.00	0.03	21	222.00	5.00	0.02
Time of Setting (Vicat Test)														
- Initial Setting Time	Minute	min 45	19	118.00	8.00	0.07	22	103.00	13.00	0.13	21	121.00	31.00	0.26
- Final Setting Time	Minute	min 375	19	217.00	17.00	0.08	22	202.00	22.00	0.11	21	237.00	21.00	0.09
False set, Final Penetration	%	min 50 (Optional Requirement)	19	69.12	6.78	0.10	22	76.86	4.39	0.06	21	80.16	4.93	0.06
Density			4	3.13	0.02	0.01	22	3.15	0.02	0.01	21	3.12	0.02	0.01

Note : N = number of samples , x = Arithmetical mean value , Sn = Standard deviation , Cn = Coefficient of variation (%)

Table 1.5.21 (1) Result of Quality Control of Ordinary Portland Cement (2)

TEST ITEM	UNIT	SPECIFICATON REQUIREMENT	1996				1997				TOTAL			
			N	x	Sn	Cn	N	x	Sn	Cn	N	x	Sn	Cn
CHEMICAL COMPOSITION														
Alkalies (Na2O + 0.658 K2O)	%	max 6.0 (Optional Requirement)	10	0.22	0.01	0.05	6	0.26	0.01	0.04	78	0.21	0.02	0.10
Insoluble Residue	%	max 0.75	10	0.36	0.10	0.28	6	0.26	0.04	0.15	78	0.35	0.23	0.66
Loss on Ignition	%	max 3.0	10	1.99	0.39	0.20	6	1.69	0.21	0.12	78	1.71	0.35	0.20
Magnesium Oxide (MgO)	%	max 5.0	10	0.86	0.04	0.05	6	0.86	0.05	0.06	78	0.81	0.10	0.12
Sulfur Trioxide (SO3)	%	max 3.0 when C3a < 8	10	1.65	0.19	0.12	6	1.46	0.06	0.04	78	2.27	0.42	0.19
		max 3.5 when C3a > 8	-	-	-	-	-	-	-	-	-	-	-	-
PHYSICAL PROPERTIES														
Fineness, Specific Surface - Air Permeability Test	m2/Kg	min 280	10	315.00	5.00	0.02	6	317.00	3.00	0.01	78	312.00	6.00	0.02
- Autoclave Expansion	%	max 0.80	10	0.11	0.01	0.09	6	0.14	0.02	0.14	78	0.10	0.02	0.20
Compressive Strength														
- 3 days	Kg/Cm2	min 124	10	157.00	6.00	0.04	6	171.00	7.00	0.04	78	164.00	10.00	0.06
- 7 days	Kg/Cm2	min 193	10	226.00	8.00	0.04	6	253.00	9.00	0.04	78	233.00	14.00	0.06
Time of Setting (Vicat Test)														
- Initial Setting Time	Minute	min 45	10	118.00	6.00	0.05	6	107.00	9.00	0.08	78	113.00	20.00	0.18
- Final Setting Time	Minute	min 375	10	256.00	6.00	0.02	6	235.00	15.00	0.06	78	225.00	26.00	0.12
False set, Final Penetration	%	min 50 (Optional Requirement)	10	80.95	6.97	0.09	6	78.03	8.07	0.10	78	76.48	7.35	0.10
Density			9	3.10	0.03	0.01	0	-	-	-	56	3.13	0.03	0.01

Note : N = number of samples , x = Arithmetical mean value , Sn = Standard deviation , Cn = Coefficient of variation (%)

Table 1.5.21 (2) Result of Quality Control of Moderate Heat Cement

TEST ITEM	UNIT	SPECIFICATON REQUIREMENT	1994				1995				TOTAL			
			N	x	Sn	Cn	N	x	Sn	Cn	N	x	Sn	Cn
CHEMICAL COMPOSITION														
Aluminium Oxide (AL 2O3)	%	max 6.0	43	4.27	0.14	0.03	26	4.33	0.31	0.07	69	4.23	0.56	0.13
Alkalies (Na2O + 0.658 K2O)	%	max 6.0 (Optional Requirement)	43	0.20	0.00	0.00	26	0.20	0.00	0.00	69	0.20	0.00	0.00
Ferric Oxide (Fe 2O3)	%	max 6.0	43	4.42	0.13	0.03	26	4.53	0.18	0.04	69	4.46	0.16	0.04
Insoluble Residue	%	max 0.75	43	0.22	0.04	0.18	26	0.22	0.08	0.36	69	0.22	0.06	0.27
Loss on Ignition	%	max 3.0	43	1.18	0.15	0.13	26	1.12	0.08	0.07	69	1.16	0.13	0.11
Magnesium Oxide (MgO)	%	max 5.0	43	0.65	0.05	0.08	26	0.69	0.17	0.25	69	0.66	0.12	0.18
Sulfur Trioxide (SO3)	%	max 3.0	43	2.03	0.10	0.05	26	2.01	0.07	0.03	69	2.02	0.09	0.04
Silicon Dioxide (Si O2)	%	min 20.0	43	22.11	0.14	0.01	26	21.86	0.27	0.01	69	22.01	0.23	0.01
Tetracalcium Aluminate Plus The Tricalcium Silicate	%	max 58 (Optional Requirement)	-	-	-	-	-	-	-	-				
Tricalcium Aluminate (C3A)	%	max 8.0	43	3.81	0.46	0.12	26	3.76	0.94	0.25	69	3.79	0.68	0.18
PHYSICAL PROPERTIES														
Fineness, Specific Surface : Air Permeability Test	m2/Kg	min 280	43	312.00	5.00	0.02	26	311.00	5.00	0.02	69	312.00	5.00	0.02
Autoclave Expansion	%	max 0.80	43	0.05	0.03	0.60	26	0.01	0.01	1.00	69	0.03	0.03	1.00
Compressive Strength														
- 3 days	PSi/MPa	min 10.30	43	16.49	0.73	0.04	26	16.00	0.61	0.04	69	16.31	0.73	0.04
- 7 days	PSi/MPa	min 17.20	43	22.49	0.81	0.04	26	22.12	2.22	0.10	69	22.35	1.52	0.07
Time of Setting (Vicat Test)														
- Initial Setting Time	minute	min 45	43	129.00	10.00	0.08	26	137.00	10.00	0.07	69	132.00	11.00	0.08
- Final Setting Time	minute	max 375	43	245.00	19.00	0.08	26	285.00	31.00	0.11	69	260.00	31.00	0.12
Heat Of Hydration, 7days	Cal/gr	max 70	43	65.71	2.00	0.03	26	65.89	3.47	0.05	69	65.78	2.65	0.04
False set , Final Penetration	%	min 50 (Optional Requirement)	43	77.38	6.85	0.09	26	77.05	8.82	0.11	69	77.26	7.66	0.10

Note : N = number of samples , x = Arithmetical mean value , Sn = Standard deviation , Cn = Coefficient of variation (%)

Table 1.5.22 (1) Result of Aggregate Test and Concrete Temperature

Test Item		Specification Requirement	1993								1994									
			May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Fitness Modulus (5-0 mm)											-	8	8	8	7	8	7	6	8	7	8
Sample nos.			8	9	9	4	9	9	8	8	-	2.67	2.71	2.67	2.73	2.68	2.68	2.68	2.69	2.72	2.73
Average		2.3 - 3.1	2.95	2.95	2.86	2.80	2.78	2.70	2.70	2.78	-	0.11	0.02	0.08	0.04	0.05	0.03	0.03	0.08	0.07	0.12
Standard dev.			0.22	0.14	0.06	0.03	0.03	0.16	0.18	0.05	-	4.30	0.90	3.00	1.30	1.80	1.00	1.20	3.00	2.70	4.50
Variation Coef.			7.40	4.70	2.10	1.20	1.10	5.90	6.70	1.80	-	2.94	2.74	2.75	2.78	2.73	2.71	2.72	2.88	2.80	2.95
Maximum			3.18	3.10	2.98	2.83	2.83	3.00	3.07	2.87	-	2.58	2.67	2.61	2.67	2.60	2.65	2.63	2.62	2.58	2.58
Minimum			2.66	2.76	2.80	2.73	2.74	2.45	2.55	2.74	-	2.63	2.63	2.63	2.62	2.63	2.63	2.63	2.85	2.65	2.63
Specific Gravity	(5 - 0 mm)	min 2.5	2.60	2.60	2.63	2.61	2.62	2.62	2.63	2.63	-	2.63	2.62	2.62	2.61	2.62	2.62	2.63	2.59	2.62	2.58
	(20 - 5 mm)	min 2.5	2.61	2.62	2.62	2.62	2.61	2.61	2.60	2.60	-	2.63	2.63	2.62	2.62	2.63	2.63	2.63	2.61	2.61	2.61
	(40-20 mm)	min 2.5	2.61	2.61	2.62	2.63	2.62	2.62	2.63	2.61	-	2.62	2.63	2.63	2.63	2.60	2.62	2.63	2.62	2.62	2.59
	(80-40 mm)	min 2.5	-	-	-	-	-	-	-	-	-	1.97	1.52	2.01	0.55	0.84	1.03	0.88	0.85	1.35	0.74
Absorption	(5 - 0 mm)	max 3.0	0.86	0.87	2.56	2.07	0.94	0.65	0.90	0.91	-	0.95	0.72	1.63	0.95	0.74	0.81	0.73	1.19	1.77	1.33
	(20 - 5 mm)	max 3.0	1.03	1.68	1.11	1.02	1.00	1.20	1.13	0.98	-	1.02	0.75	0.85	0.78	0.89	0.86	1.08	1.07	1.63	0.93
	(40-20 mm)	max 3.0	1.24	1.35	0.79	0.84	0.83	0.94	1.02	1.03	-	0.99	0.75	0.67	0.66	0.82	0.76	1.20	0.82	0.90	0.74
	(80-40 mm)	max 3.0	-	-	-	-	-	-	-	-	-	1.68	1.70	1.70	1.71	1.70	1.71	1.71	1.72	1.72	1.69
Unit Weight	(5 - 0 mm)	1.5 - 1.85	1.68	1.69	1.67	1.67	1.68	1.66	1.70	1.71	-	1.68	1.67	1.68	1.68	1.68	1.67	1.68	1.68	1.67	1.65
(Average)	(20 - 5 mm)	1.5 - 1.85	1.71	1.70	1.72	1.74	1.73	1.71	1.73	1.69	-	1.68	1.68	1.68	1.67	1.68	1.66	1.68	1.67	1.66	1.64
	(40-20 mm)	1.5 - 1.85	1.69	1.68	1.72	1.74	1.74	1.74	1.68	1.69	-	1.69	1.70	1.70	1.70	1.70	1.69	1.70	1.68	1.69	1.69
	(80-40 mm)	1.5 - 1.85	-	-	-	-	-	-	-	-	-	1.81	1.62	1.40	1.08	1.07	1.66	1.14	1.05	0.99	0.90
Washing Loss	(5 - 0 mm)	max 5.0	2.41	2.06	2.44	2.85	2.23	2.62	3.07	2.70	-	0.75	0.72	0.70	0.67	0.70	0.73	0.81	0.73	0.99	0.55
(Maximum)	(20 - 5 mm)	max 1.0	1.01	1.13	0.98	1.13	0.84	0.93	0.70	0.84	-	0.50	0.54	0.40	0.51	0.75	0.77	0.83	0.80	0.47	0.53
	(40-20 mm)	max 1.0	0.95	1.19	0.99	1.18	1.08	1.19	0.91	0.86	-	0.47	0.41	0.51	0.54	0.77	0.72	0.70	0.69	0.54	0.40
	(80-40 mm)	max 1.0	-	-	-	-	-	-	-	-	-	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok
Organic Impurities		Lighter	ok	ok	ok	ok	ok	ok	ok	ok	ok	11.58	23.54	24.02	23.10	23.80	24.28	0.00	25.00	27.46	23.70
Abrasion	(20 - 5 mm)	max 40.0	24.58	23.38	22.51	24.58	25.85	24.96	23.50	23.51	-	11.58	23.54	24.02	23.10	23.80	24.28	0.00	25.00	27.46	23.70
	(40-20 mm)	max 40.0	24.58	23.38	22.51	24.58	25.85	24.96	23.50	23.51	-	21.35	14.25	16.75	14.06	13.52	13.96	0.00	13.38	13.08	14.20
	(80-40 mm)	max 40.0	-	-	-	-	-	-	-	-	-	2.40	3.11	4.27	2.93	2.90	3.08	2.13	1.99	0.48	2.05
Soft Rock	(20 - 5 mm)	max 5.0	-	9.28	3.61	1.96	2.29	2.68	3.46	3.49	-	1.50	2.28	4.27	0.00	0.00	2.98	22.6	1.99	0.48	2.05
	(40-20 mm)	max 5.0	-	11.72	5.00	4.49	4.58	4.14	4.96	4.97	-	0.70	0.00	0.00	0.00	0.00	0.00	13.78	0.00	0.62	0.00
	(80-40 mm)	max 5.0	-	-	-	-	-	-	-	-	-										
Moisture Content	(%)										8	45	58	91	116	136	130	106	192	154	152
(GENERAL)	Sample nos.		-	-	-	-	-	-	-	-	3.2	4.7	5.6	4.8	6.3	6.5	6.6	6.4	6.7	7.7	7.7
	Average		-	-	-	-	-	-	-	-	1.0	0.8	0.9	1.4	1.6	1.8	2.2	2.0	1.6	1.9	1.9
	Standard dev.		-	-	-	-	-	-	-	-	-	94	130	187	217	260	238	277	233	254	199
(DAM)	Sample nos.		-	-	-	-	-	-	-	-	-	5.3	8.0	8.0	9.1	8.0	8.6	8.1	7.4	8.6	8.7
	Average		-	-	-	-	-	-	-	-	-	1.8	1.5	1.2	1.8	1.8	2.0	1.4	1.1	1.5	1.7
	Standard dev.		-	-	-	-	-	-	-	-	-										
Concrete Temperature	(°C)										30.1	28.5	28.9	30.0	29.1	29.2	29.2	27.3	28.3	28.1	28.4
GENERAL	Average		-	-	-	-	-	-	-	-	1.5	1.7	1.2	2.7	1.2	1.4	1.5	1.2	1.4	1.5	1.6
	Standard dev.		-	-	-	-	-	-	-	-	-	28.8	28.6	29.4	28.3	28.6	28.5	26.7	26.0	26.1	26.3
DAM	Average		-	-	-	-	-	-	-	-	-	1.1	1.5	0.8	1.3	1.1	1.0	0.9	1.0	1.0	1.3
	Standard dev.		-	-	-	-	-	-	-	-	-										

Table 1.5.22 (2) Result of Aggregate Test and Concrete Temperature

Test Item		Specification Requirement	1995 Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	1996 Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.
Fitness Modulus (5-0 mm)																					
Sample nos.			6	8	6	8	8	8	7	9	7	8	8	7	6	6	8	8	10	8	8
Average		2.3 - 3.1	2.75	2.68	2.65	2.71	2.67	2.67	2.67	2.66	2.74	2.76	2.69	2.73	2.69	2.63	2.61	2.62	2.60	2.65	2.66
Standard dev.			0.11	0.14	0.10	0.12	0.11	0.07	0.09	0.12	0.08	0.13	0.10	0.19	0.08	0.08	0.06	0.09	0.07	0.05	0.13
Variation Coef.			4.20	5.10	3.90	4.40	4.30	2.70	3.20	4.70	3.00	4.60	3.70	7.00	2.80	3.20	2.50	3.50	2.90	2.00	4.90
Maximum			2.89	2.88	2.75	2.90	2.88	2.77	2.79	2.84	2.82	2.92	2.81	2.98	2.77	2.73	2.72	2.74	2.75	2.73	2.86
Minimum			2.55	2.46	2.49	2.54	2.55	2.54	2.57	2.42	2.63	2.54	2.52	2.44	2.60	2.50	2.54	2.47	2.50	2.60	2.51
Specific Gravity	(5 - 0 mm)	min 2.5	2.63	2.62	2.61	2.62	2.64	2.63	2.63	2.63	2.62	2.64	2.65	2.63	2.64	2.63	2.63	2.60	2.60	2.60	2.59
	(20 - 5 mm)	min 2.5	2.62	2.54	2.57	2.61	2.58	2.59	2.62	2.62	2.58	2.61	2.62	2.62	2.59	2.59	2.63	2.59	2.61	2.61	2.61
	(40-20 mm)	min 2.5	2.62	2.56	2.60	2.62	2.56	2.66	2.62	2.62	2.62	2.62	2.62	2.59	2.55	2.61	2.62	2.62	-	2.61	2.61
	(80-40 mm)	min 2.5	2.60	2.58	2.62	2.65	2.59	2.60	2.62	2.62	2.64	2.61	2.61	-	-	-	-	-	-	-	-
Absorption	(5 - 0 mm)	max 3.0	0.52	0.66	0.63	0.32	0.39	0.19	0.35	0.35	0.96	0.22	0.35	0.32	0.26	0.50	0.57	0.45	0.41	0.36	0.50
	(20 - 5 mm)	max 3.0	1.43	1.53	2.14	1.58	2.16	0.98	2.39	1.45	1.44	1.13	1.09	0.75	1.43	1.15	1.21	0.62	1.26	1.08	1.33
	(40-20 mm)	max 3.0	1.17	1.02	1.14	1.26	2.60	1.20	1.18	1.17	1.11	0.76	0.92	1.06	1.05	1.01	0.90	0.89	-	0.77	1.06
	(80-40 mm)	max 3.0	1.01	0.58	0.78	0.54	1.59	1.46	1.12	0.76	0.52	0.57	0.80	-	-	-	-	-	-	-	-
Unit Weight	(5 - 0 mm)	1.5 - 1.85	1.73	1.74	1.72	1.76	1.73	1.75	1.73	1.75	1.73	1.76	1.75	1.78	1.76	1.78	1.76	1.78	1.78	1.79	1.78
(Average)	(20 - 5 mm)	1.5 - 1.85	1.70	1.67	1.68	1.64	1.60	1.64	1.65	1.63	1.64	1.63	1.66	1.65	1.62	1.66	1.66	1.64	1.66	1.67	1.67
	(40-20 mm)	1.5 - 1.85	1.66	1.64	1.65	1.61	1.64	1.62	1.62	1.63	1.63	1.61	1.59	1.57	1.59	1.60	1.61	1.59	1.61	1.58	1.59
	(80-40 mm)	1.5 - 1.85	1.71	1.72	1.69	1.68	1.65	1.70	1.67	1.72	1.73	1.74	1.70	-	-	-	-	-	-	-	-
Washing Loss	(5 - 0 mm)	max 5.0	0.85	0.85	0.83	1.04	2.88	1.74	1.23	1.10	1.67	1.44	0.99	2.06	1.02	1.94	1.95	1.78	2.56	1.87	2.94
(Maximum)	(20 - 5 mm)	max 1.0	0.51	0.39	0.79	0.57	0.30	0.50	0.43	0.73	0.54	0.45	0.78	0.46	0.68	0.90	0.64	0.43	0.48	0.81	0.41
	(40-20 mm)	max 1.0	0.72	0.46	0.61	0.40	0.59	0.75	0.66	0.60	0.42	0.59	0.55	0.52	0.42	0.74	0.41	0.39	0.57	0.44	0.61
	(80-40 mm)	max 1.0	0.65	0.60	0.43	0.55	0.45	0.44	0.37	0.55	0.42	0.45	0.41	-	-	-	-	-	-	-	-
Organic Impurities		Lighter	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok
Abrasion	(20 - 5 mm)	max 40.0	24.12	30.42	26.28	23.70	31.14	14.90	29.50	21.84	24.80	23.48	26.06	27.52	22.44	23.40	24.90	23.76	22.80	24.06	24.24
	(40-20 mm)	max 40.0	24.12	30.42	26.28	23.70	31.14	14.90	29.50	21.84	24.80	23.48	26.06	27.52	22.44	23.40	24.90	23.76	22.80	24.06	24.24
	(80-40 mm)	max 40.0	12.42	18.25	15.20	14.45	14.12	5.88	14.73	8.79	14.43	16.08	16.40	-	-	-	-	-	-	-	-
Soft Rock	(20 - 5 mm)	max 5.0	-	2.07	0.80	0.00	0.85	0.00	0.54	2.05	0.00	0.47	0.00	0.00	0.00	2.90	0.00	0.30	0.05	0.00	0.38
	(40-20 mm)	max 5.0	-	0.57	0.00	0.00	0.00	3.35	0.54	2.05	0.00	0.47	0.00	0.00	0.00	0.48	0.00	0.30	0.05	0.34	0.38
	(80-40 mm)	max 5.0	-	0.00	0.00	0.00	0.00	0.00	0.54	2.05	0.00	0.47	0.00	-	-	-	-	-	-	-	-
Moisture Content	(%)																				
(GENERAL)	Sample nos.		132	155	142	177	186	196	205	214	137	165	158	173	95	71	93	104	89	84	97
	Average		6.7	6.6	7.6	7.6	6.1	5.3	6.3	5.6	4.3	4.9	4.5	4.1	3.0	3.3	2.6	3.2	2.6	3.0	1.6
	Standard dev.		1.7	1.0	2.3	2.1	2.1	1.4	1.5	2.6	1.7	1.6	2.1	1.1	0.8	0.8	0.6	1.0	0.8	1.1	0.6
(DAM)	Sample nos.		195	149	156	200	196	153	132	121	100	89	44	-	-	-	-	-	-	-	-
	Average		7.5	7.6	7.8	8.5	7.4	6.6	6.8	5.8	4.8	4.4	4.2	-	-	-	-	-	-	-	-
	Standard dev.		1.3	1.2	1.7	1.4	1.5	1.2	1.6	1.3	1.3	0.8	1.2	-	-	-	-	-	-	-	-
Concrete Temperature	(°C)																				
GENERAL	Average		28.5	28.4	30.8	30.7	30.9	30.4	29.9	29.7	30.7	30.4	29.2	28.2	27.8	27.7	28.0	28.5	28.7	28.1	27.9
	Standard dev.		1.4	1.4	1.5	1.5	1.4	1.2	1.3	1.4	1.6	1.1	1.4	0.7	0.6	0.5	0.3	0.8	0.6	0.8	0.6
DAM	Average		25.9	26.6	27.8	27.5	27.4	27.6	26.8	26.7	26.8	27.3	27.9	-	-	-	-	-	-	-	-
	Standard dev.		1.2	1.3	1.2	1.1	1.1	1.1	1.0	0.7	0.8	0.9	0.7	-	-	-	-	-	-	-	-

Table 1.5.22 (3) Result of Aggregate Test and Concrete Temperature

Test Item		Specification Requirement	1996 Aug.	Sep.	Oct.	Nov.	Dec.	1997 Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Total
Fitness Modulus (5-0 mm)																
Sample nos.			9	7	3	8	8	6	6	8	8	8	9	9	-	283
Average		2.3 - 3.1	2.69	2.65	2.49	2.45	2.52	2.57	2.59	2.69	2.60	2.66	2.65	2.69	-	2.69
Standard dev.			0.05	0.04	0.03	0.09	0.07	0.06	0.15	0.03	0.05	0.06	0.03	0.04	-	0.09
Variation Coef.			1.80	1.50	1.20	3.60	2.90	2.30	5.90	1.30	1.80	2.40	1.00	1.70	-	3.20
Maximum			2.76	2.68	2.52	2.58	2.60	2.60	2.75	2.74	2.66	2.76	2.70	2.76	-	2.74
Minimum			2.62	2.56	2.46	2.33	2.43	2.45	2.35	2.63	2.54	2.56	2.62	2.60	-	2.57
Specific Gravity	(5 - 0 mm)	min 2.5	2.59	2.59	-	2.59	2.60	2.60	2.60	2.60	2.60	2.60	2.59	2.60	-	2.59
	(20 - 5 mm)	min 2.5	2.62	2.62	-	2.62	2.61	2.61	2.61	2.63	2.61	2.63	2.61	2.61	-	2.54
	(40-20 mm)	min 2.5	2.61	2.62	-	2.62	2.62	2.62	2.62	2.62	2.62	2.62	-	2.61	-	2.55
	(80-40 mm)	min 2.5	-	-	-	-	-	-	-	-	-	-	-	-	-	2.58
Absorption	(5 - 0 mm)	max 3.0	0.49	0.59	-	0.49	0.58	0.58	0.68	0.87	0.59	0.59	0.60	0.67	-	0.27
	(20 - 5 mm)	max 3.0	1.07	1.10	-	1.00	1.05	1.05	1.18	1.36	1.13	1.03	1.06	1.04	-	2.39
	(40-20 mm)	max 3.0	0.80	0.96	-	1.10	1.17	1.17	1.29	1.16	1.25	0.78	2.63	0.85	-	2.63
	(80-40 mm)	max 3.0	-	-	-	-	-	-	-	-	-	-	-	-	-	1.59
Unit Weight	(5 - 0 mm)	1.5 - 1.85	1.78	1.75	1.74	1.78	1.78	1.78	1.76	1.78	1.78	1.78	1.78	1.76	-	1.74
(Average)	(20 - 5 mm)	1.5 - 1.85	1.67	1.67	1.67	1.66	1.66	1.66	1.67	1.68	1.67	1.65	1.67	1.66	-	1.67
	(40-20 mm)	1.5 - 1.85	1.59	1.59	1.61	1.61	1.60	1.59	1.60	1.61	1.62	1.61	1.64	1.60	-	1.64
	(80-40 mm)	1.5 - 1.85	-	-	-	-	-	-	-	-	-	-	-	-	-	1.70
Washing Loss	(5 - 0 mm)	max 5.0	1.54	1.20	1.99	1.66	1.20	1.23	1.97	1.92	2.25	1.68	0.82	1.17	-	3.07
(Maximum)	(20 - 5 mm)	max 1.0	0.67	0.60	0.64	0.58	0.64	0.49	0.97	0.88	0.85	0.80	0.89	0.81	-	1.13
	(40-20 mm)	max 1.0	0.60	0.47	0.57	0.40	0.41	0.60	0.60	0.54	0.60	0.62	0.61	0.68	-	1.19
	(80-40 mm)	max 1.0	-	-	-	-	-	-	-	-	-	-	-	-	-	0.77
Organic Impurities		Lighter	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok	ok
Abrasion	(20 - 5 mm)	max 40.0	24.16	24.50	-	24.80	24.56	24.56	22.48	23.06	23.30	23.30	24.16	24.16	-	30.42
	(40-20 mm)	max 40.0	24.16	24.50	-	24.80	24.56	24.56	22.48	23.06	23.30	23.30	24.16	24.16	-	30.42
	(80-40 mm)	max 40.0	-	-	-	-	-	-	-	-	-	-	-	-	-	21.35
Soft Rock	(20 - 5 mm)	max 5.0	1.16	0.00	-	1.43	0.68	0.68	0.36	0.40	1.55	0.00	0.44	0.40	-	9.28
	(40-20 mm)	max 5.0	0.84	0.00	-	1.43	1.34	1.34	0.70	0.40	0.00	0.00	0.44	0.40	-	22.60
	(80-40 mm)	max 5.0	-	-	-	-	-	-	-	-	-	-	-	-	-	13.78
Moisture Content	(%)															
GENERAL	Sample nos.		76	86	60	72	64	79	41	62	49	79	83	71	12	3,624
	Average		2.7	3.2	1.9	1.7	1.3	0.9	0.6	1.2	1.4	3.4	1.3	1.3	2.0	7.60
	Standard dev.		1.0	0.7	0.7	0.4	0.4	0.5	0.6	0.6	0.6	1.5	1.0	1.1	1.1	1.50
DAM	Sample nos.		-	-	-	-	-	-	-	-	-	-	-	-	-	4,695
	Average		-	-	-	-	-	-	-	-	-	-	-	-	-	4.90
	Standard dev.		-	-	-	-	-	-	-	-	-	-	-	-	-	1.50
Concrete Temperature	(°C)															
GENERAL	Average		27.8	28.4	27.9	28.1	27.9	26.9	27.6	27.1	29.3	29.3	29.4	29.6	29.7	28.85
	Standard dev.		0.7	0.8	0.6	0.5	0.6	0.9	0.9	0.8	0.8	0.7	0.5	0.5	0.6	1.07
DAM	Average		-	-	-	-	-	-	-	-	-	-	-	-	-	27.00
	Standard dev.		-	-	-	-	-	-	-	-	-	-	-	-	-	0.99

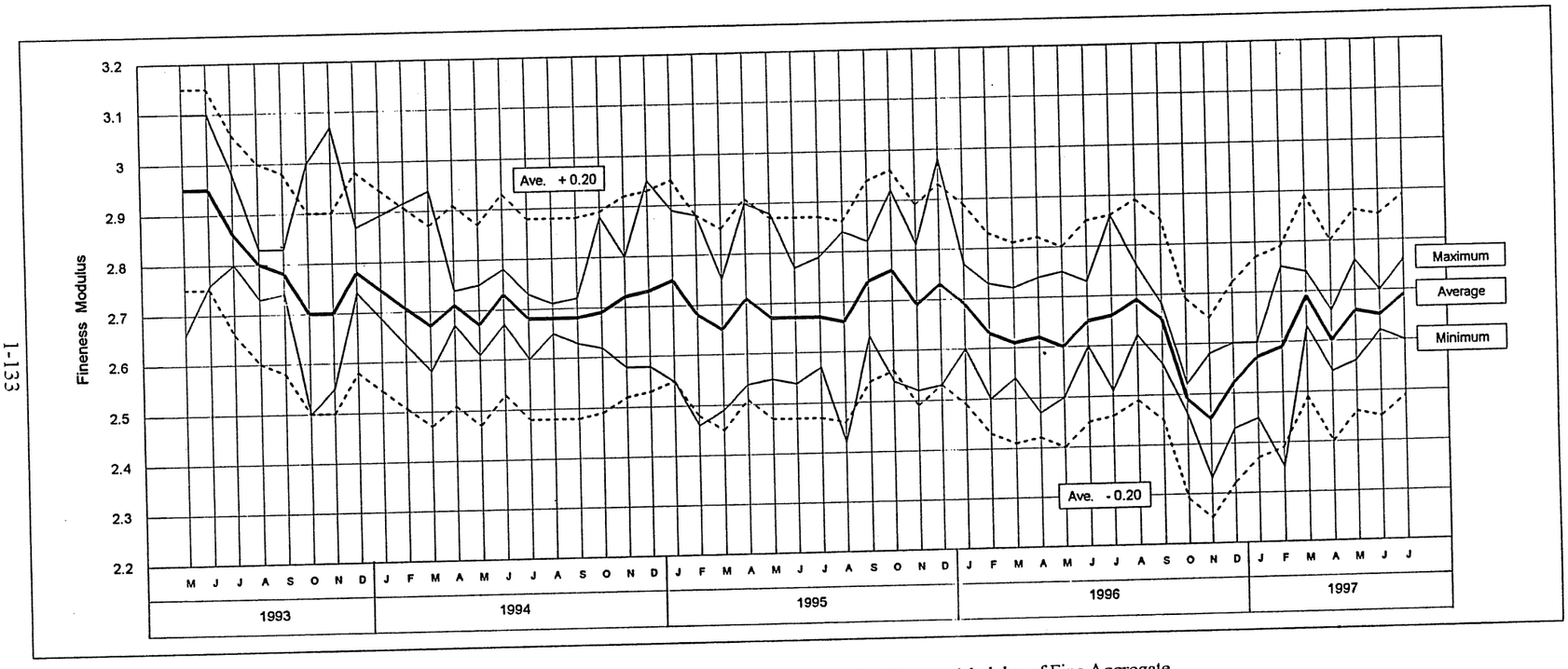
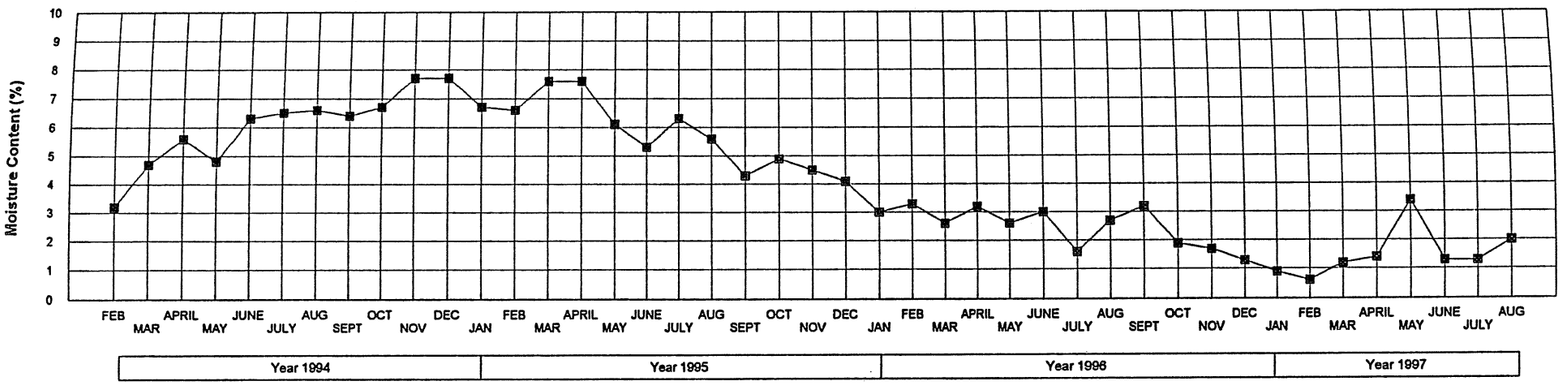


Fig. 1.5.8 Result of Quality Control for Fineness Modulus of Fine Aggregate

Fine Aggregate for General Concrete



Fine Aggregate for Dam Concrete

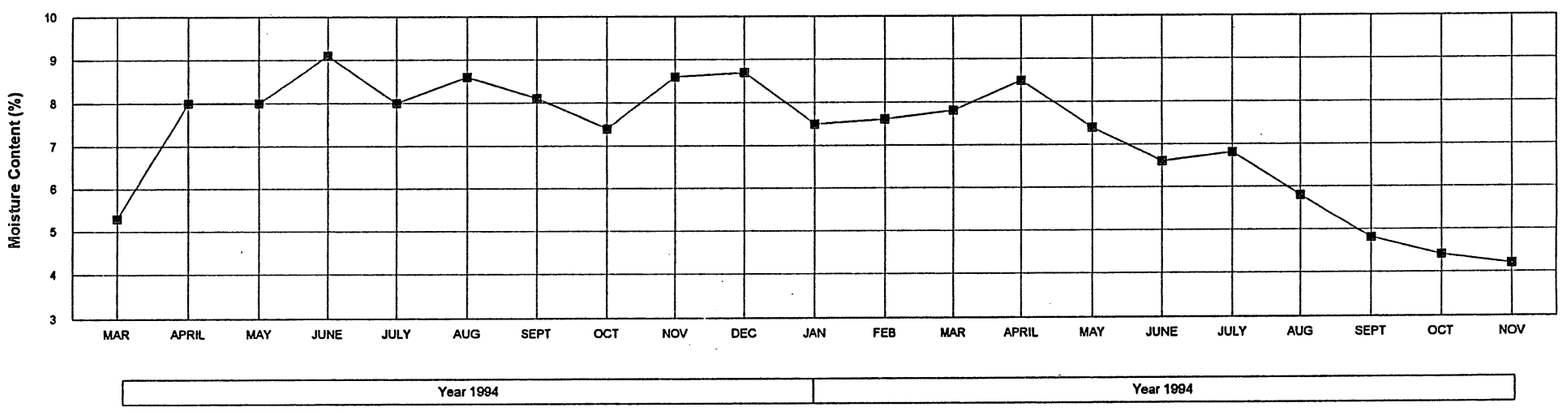


Fig. 1.5.9 Result of Quality Control for Surface Moisture Content of Fine Aggregate

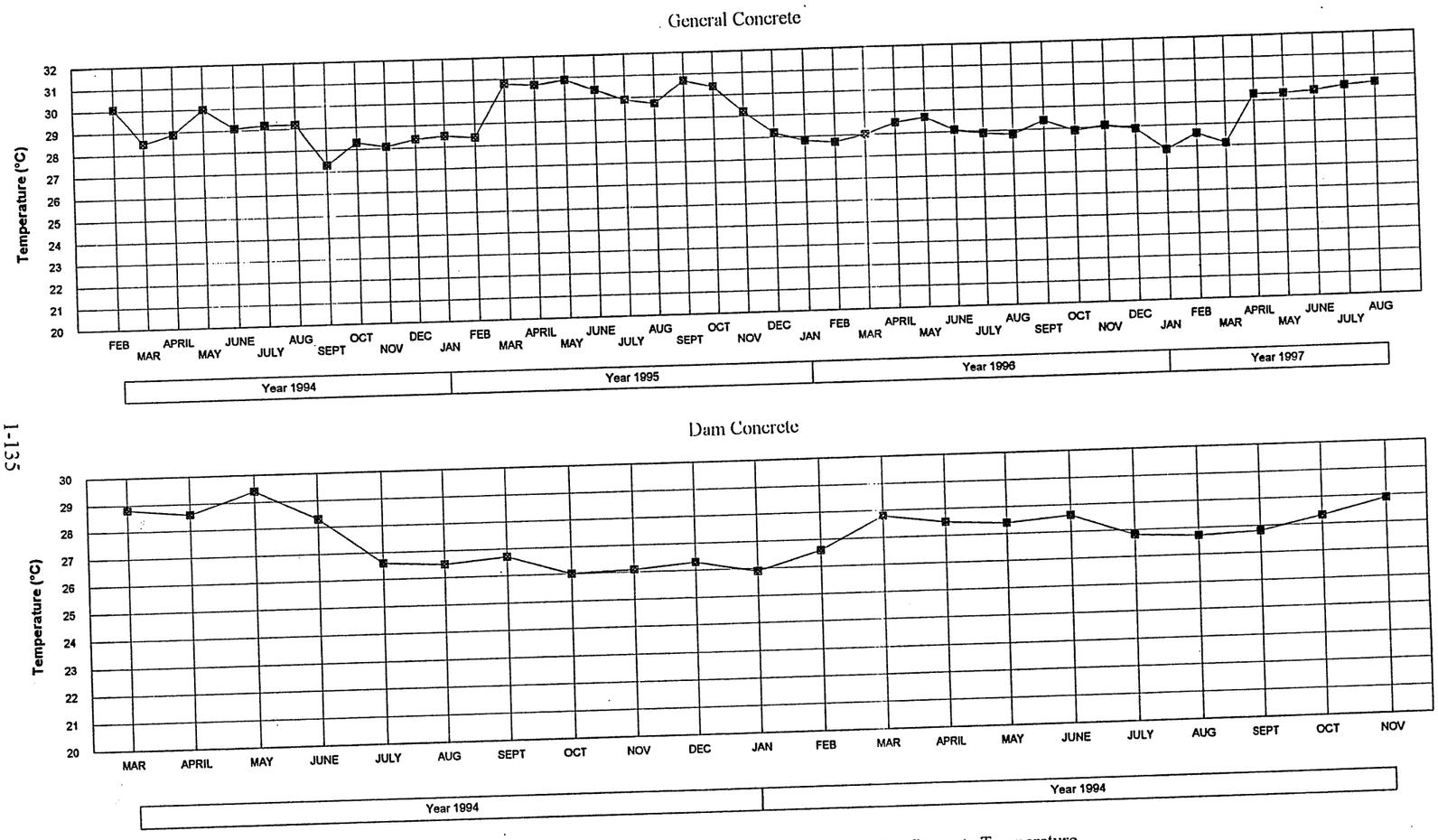


Fig. 1.5.10 Result of Quality Control for Concrete Temperature

Table 1.5.23 (1) Result of Quality Control for Dam Concrete

NO.	DESCRIPTION / CONCRETE TYPE			A1	A1 - R1	A2	A2 - R1	A3	A3 - R1	A3 - R2	A4
1.	Specification of Mix Proportion										
		ck	(Kg/cm²)	330	330	220	220	280	280	280	280
		Slump	(cm)	3 ± 1	3 ± 1	3 ± 1	3 ± 1	8 ± 2	8 ± 2	4 ± 1	12 ± 2
		Air Content	(%)	3 ± 1	3 ± 1	3 ± 1	3 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1
		w/c	(%)	45	47	61	55	49	50	50	48
		s/a	(%)	26	26	29	28	36	36	36	36
		Cement	(Kg)	223	216	166	184	280	270	250	297
2.	Used Period			01/03/94 up to 11/08/94	12/08/94 up to 29/11/94	03/05/94 up to 29/09/95	28/03/94 up to 31/05/94	03/05/94 up to 08/05/95	06/02/95 up to 20/11/95	13/03/95 up to 14/10/95	06/09/94 up to 20/06/95
3.	Result of Qualit Control	1) Slump	NOS	115	337	602	80	215	174	121	7
			X (Cm)	2.68	2.72	2.59	2.55	8.25	7.85	3.71	11.83
		2) Air Content	NOS	115	337	602	80	215	174	121	7
			X (Cm)	3.13	3.1	3.22	3.44	4.47	4.55	4.2	4.69
		3) Compressive Strength									
		a. 28	NOS	115	337	605	80	215	174	121	7
			X (Kg/Cm²)	355	343	250	282	311	308	301	282
			Sn (Kg/Cm²)	31.87	17.34	26.22	32.16	24.26	19.05	13.09	38.83
			Cn (%)	8.98	5.05	10.51	11.39	7.79	6.19	4.35	13.76

Note NOS = Number of Sample X = Arithmetical Mean Value (Average) Sn = Standard Deviation Cn = Coeffiicient of Variation

Table 1.5.23 (2) Result of Quality Control for General Concrete

NO.	DESCRIPTION / CONCRETE TYPE			B1	B1 - R1	B1 - R4	B2	B2 - R1	B2 - R2	B2 - R3	B3	B4	B4 - R1	B4 - R2	B5	B6	B7	B7 - R1	B8 - R1
1.	Specification of Mix Proportion																		
		ck	(Kg/cm²)	280	280	280	260	260	260	260	260	240	240	240	220	210	280	280	120
		Slump	(cm)	8 ± 2	8 ± 2	8 ± 2	12 ± 2	12 ± 2	12 ± 2	12 ± 2	8 ± 2	12 ± 2	12 ± 2	12 ± 2	8 ± 2	16 ± 2	10 ± 2	10 ± 2	8 ± 2
		Air Content	(%)	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1	4.5 ± 1
		w/c	(%)	45	47	49	47	51	49	51	48	51	55	53	53	61	55	51	60
		s/a	(%)	35	35	37	37.5	37.5	36	37	36	37.5	37.5	37	37	45	44	43	45
		Cement	(Kg)	298	290	280	325	300	290	280	286	300	278	268	137	279	287	308	260
2.	Used Period			28/02/94 up to 30/07/94	02/08/94 up to 13/06/95	05/06/95 up to 23/07/96	07/05/93 up to 25/09/93	26/08/93 up to 09/12/95	11/05/94 up to 22/11/95	28/11/95 up to 26/07/97	21/09/94 up to 20/09/95	16/05/93 up to 06/05/93	22/09/93 up to 30/11/93	16/09/94 up to 08/08/97	13/10/94 up to 13/03/97	12/06/95 up to 18/06/96	17/10/94 up to 23/01/95	06/03/96 up to 06/03/96	22/07/95 up to 13/10/95
3.	Result of Qualit Control	1) Slump	NOS	67	226	208	219	65	167	113	62	3	64	389	62	38	5	4	3
			X (Cm)	8.62	8.61	8.26	11.65	11.78	11.68	11.16	8.53	11.5	11.53	11.27	8.18	15.63	9.70	8.80	7.70
		2) Air Content	NOS	67	226	208	219	65	167	113	62	3	64	389	62	38	5	4	3
			X (Cm)	4.11	4.18	4.27	4.64	4.61	4.24	4.46	4.29	4.5	4.43	4.26	4.17	4.18	4.60	4.50	3.80
		3) Compressive Strength																	
		a. 28	NOS	67	226	208	219	65	167	113	62	3	64	389	62	38	5	4	3
			X (Kg/Cm²)	342	320	306	340	316	303	282	302	262	289	267	269	247	286	231	215
			Sn (Kg/Cm²)	30.24	28.66	16.16	46.80	25.70	26.77	11.43	28.52	16.21	28.77	17.75	29.45	17.74	32.24	16.50	18.73
			Cn (%)	8.85	8.97	5.29	13.77	8.14	8.83	4.05	9.43	6.20	9.96	6.65	10.94	7.19	11.29	7.15	8.70

Note NOS = Number of Sample X = Arithmetical Mean Value (Average) Sn = Standard Deviation Cn = Coeffiicient of Variation

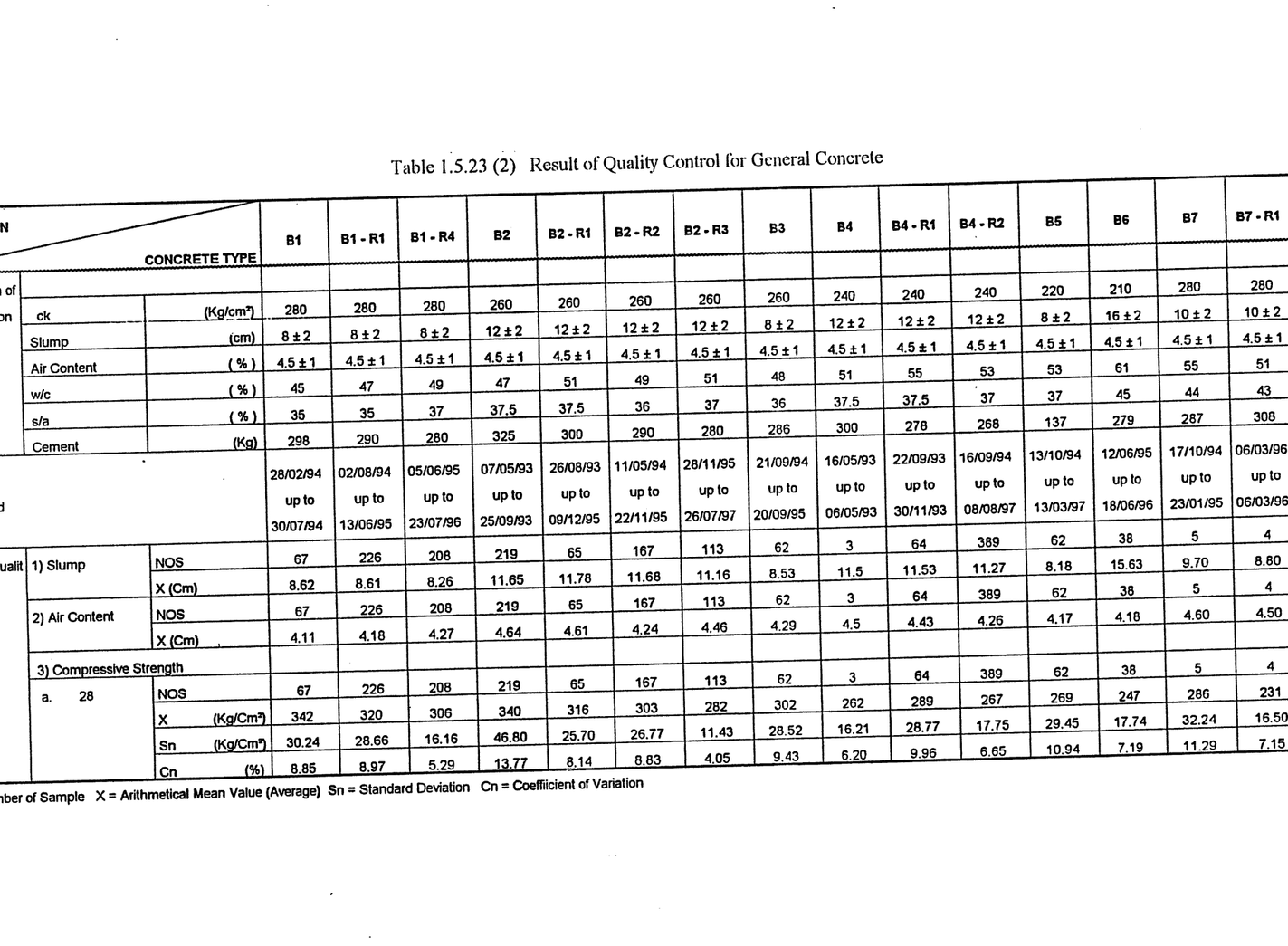


Fig. 1.5.11 (1) Control Chart of Concrete Type A1

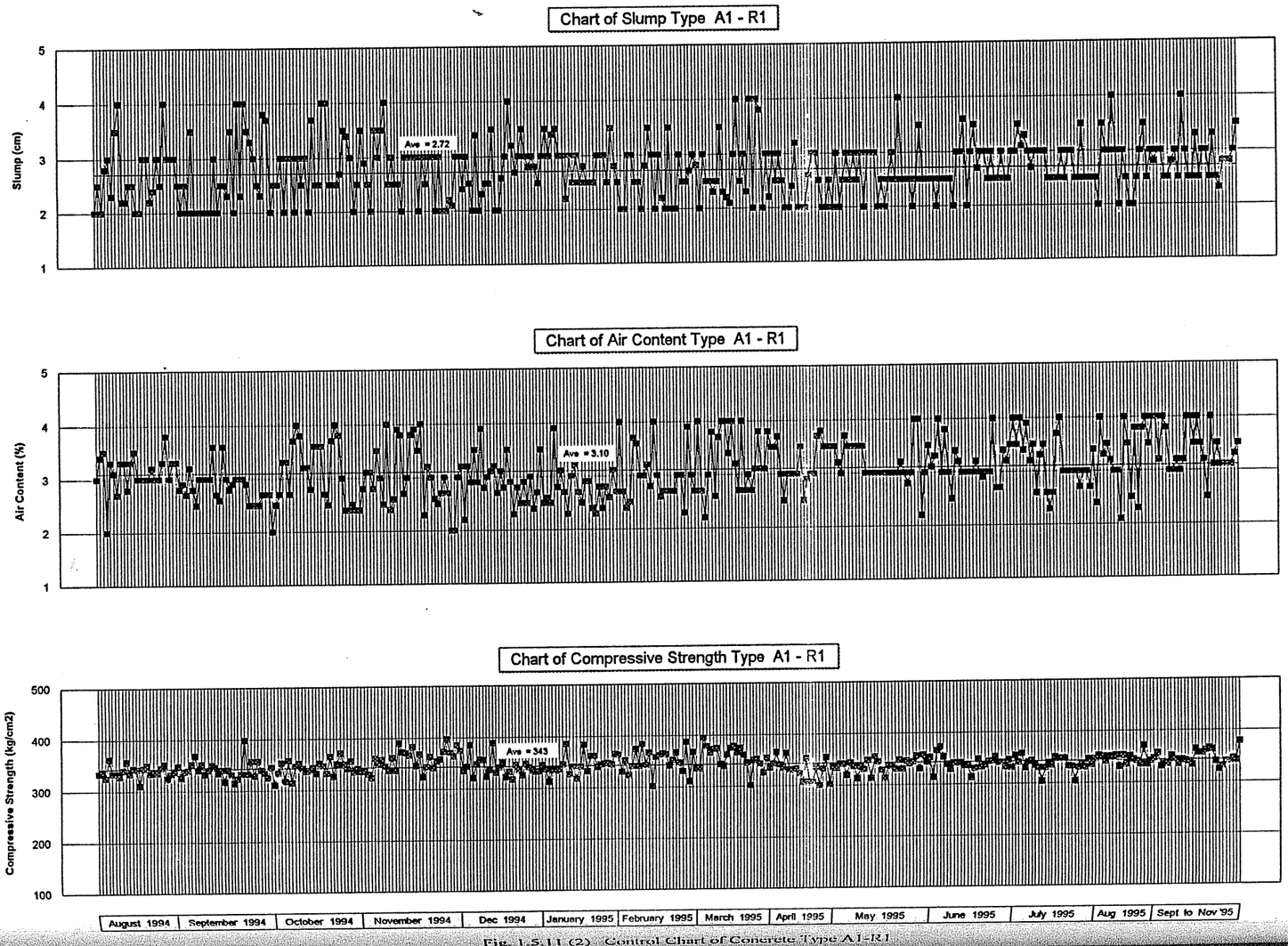


Fig. 1.5.11 (2) Control Chart of Concrete Type A1-R1

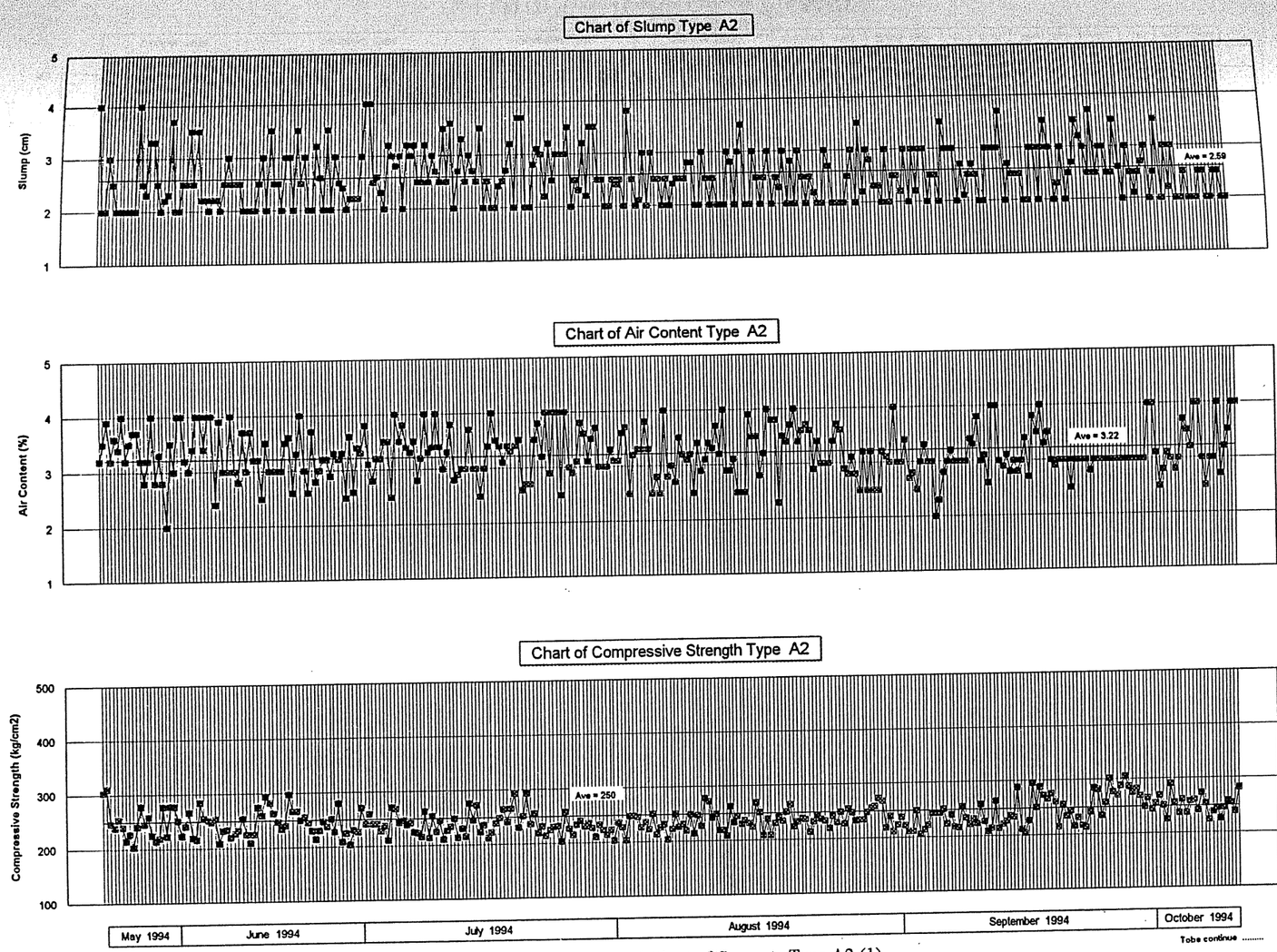


Fig. 1.5.11 (3) Control Chart of Concrete Type A2 (1)

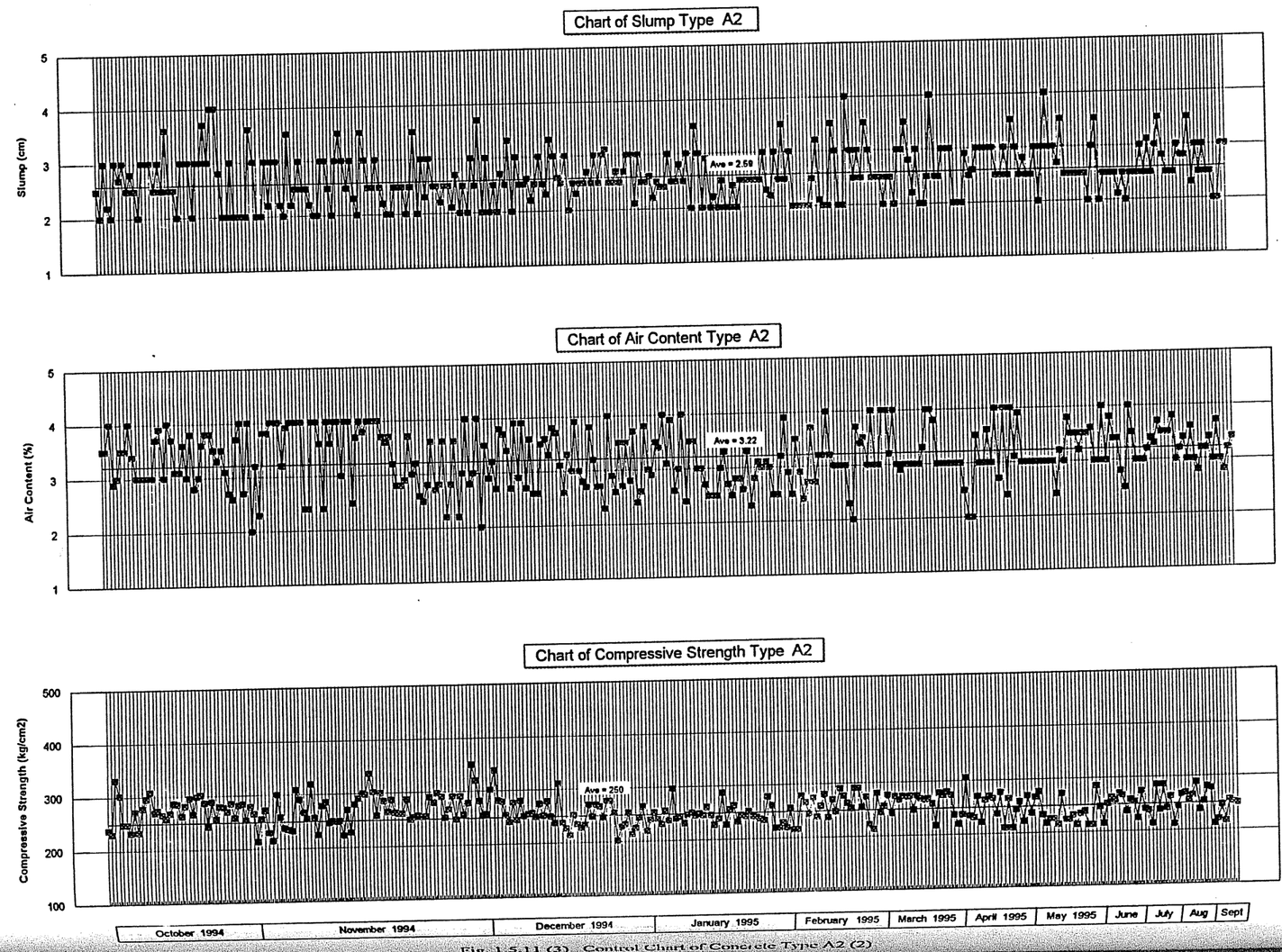
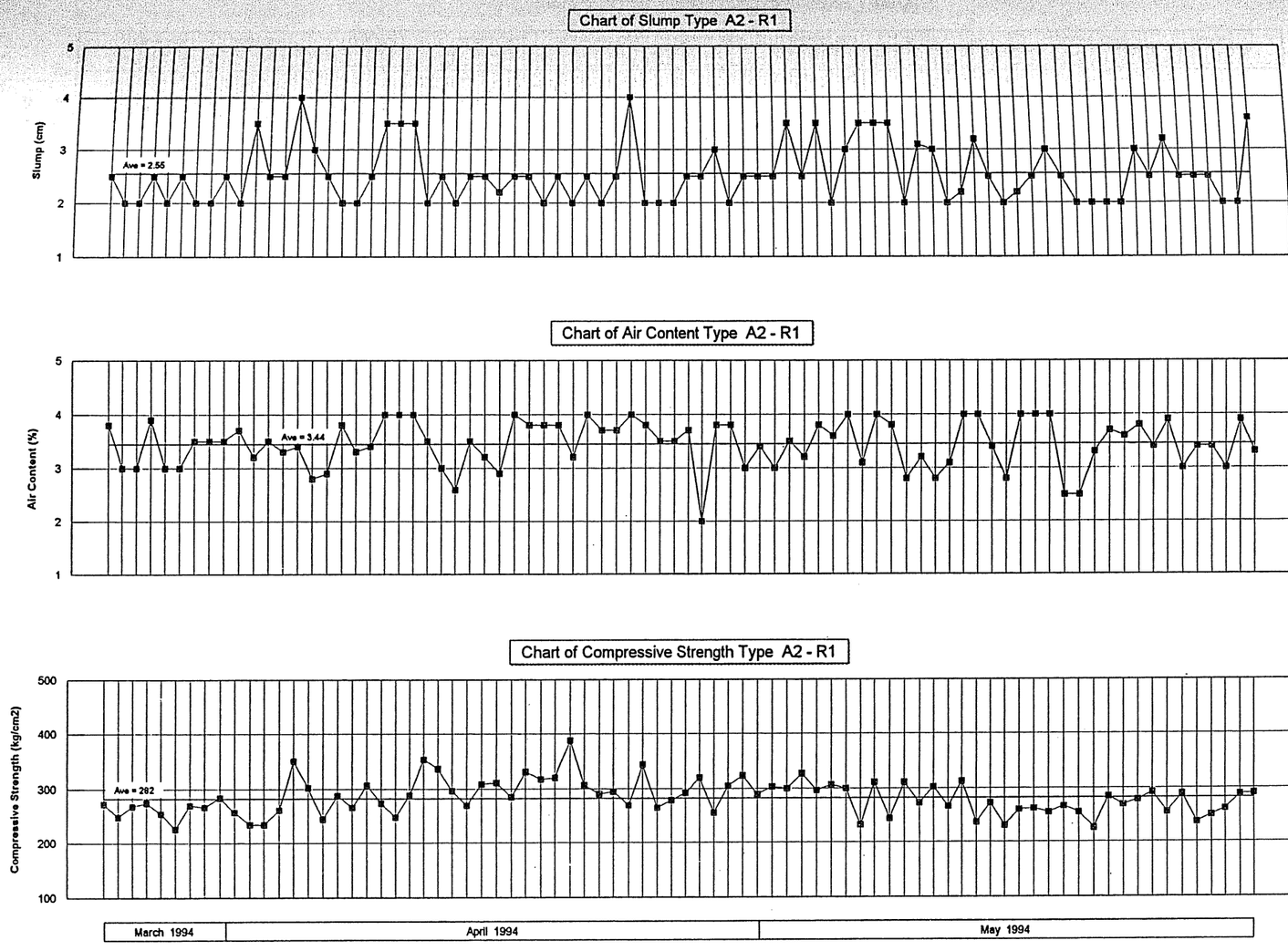


Fig. 1.5.11 (3) Control Chart of Concrete Type A2 (2)



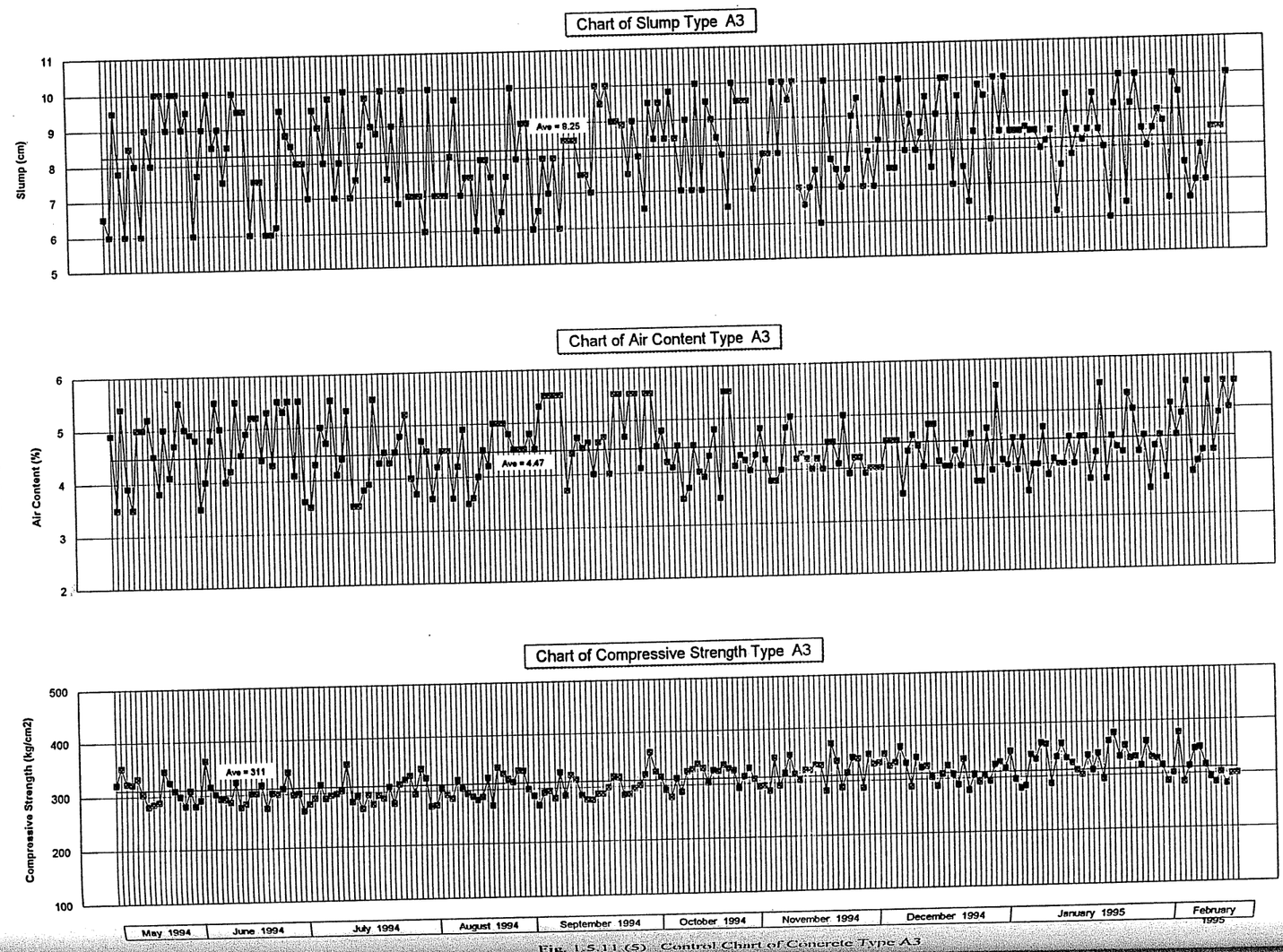


Fig. 1.5.11 (5) Control Chart of Concrete Type A3

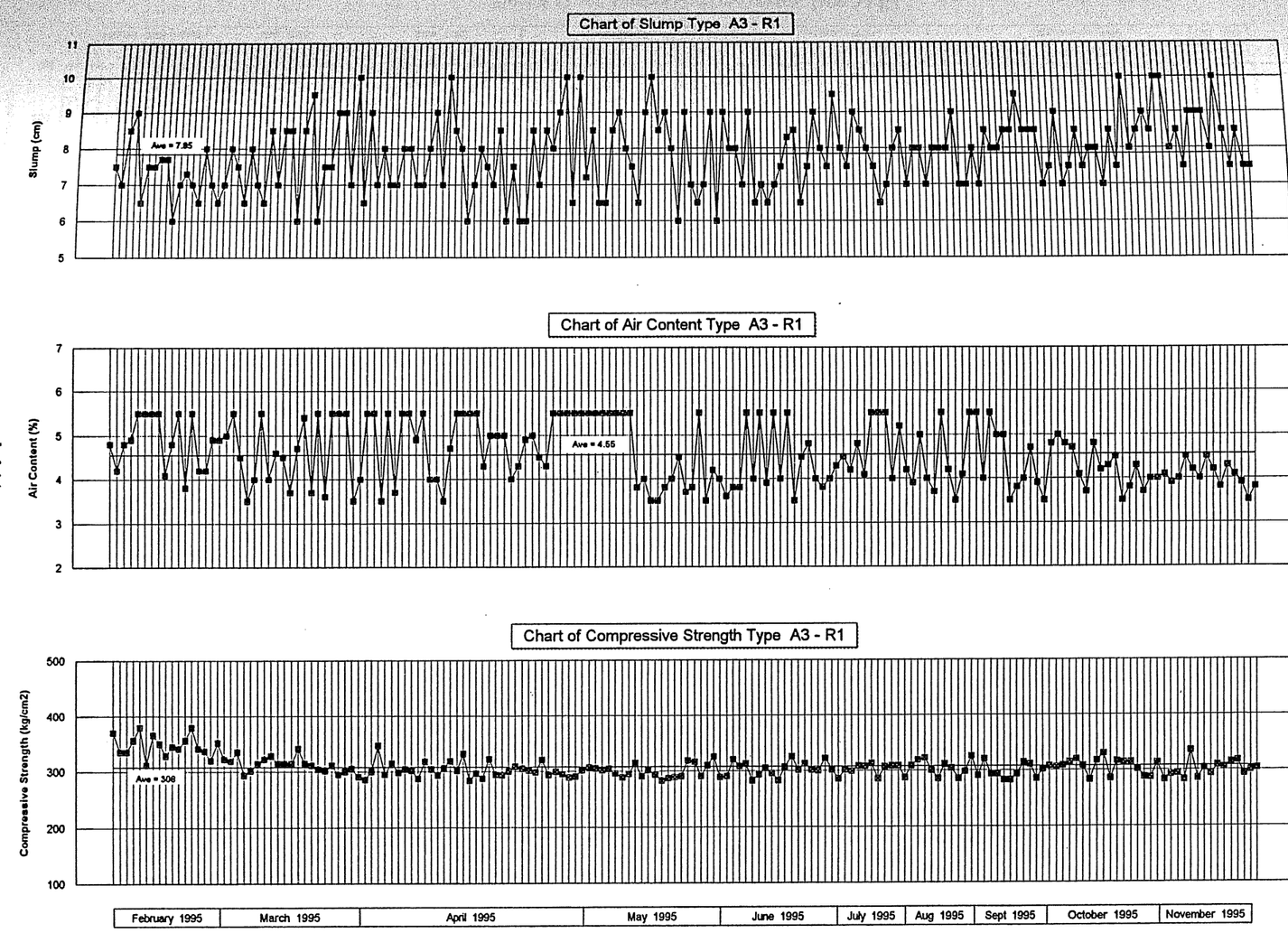
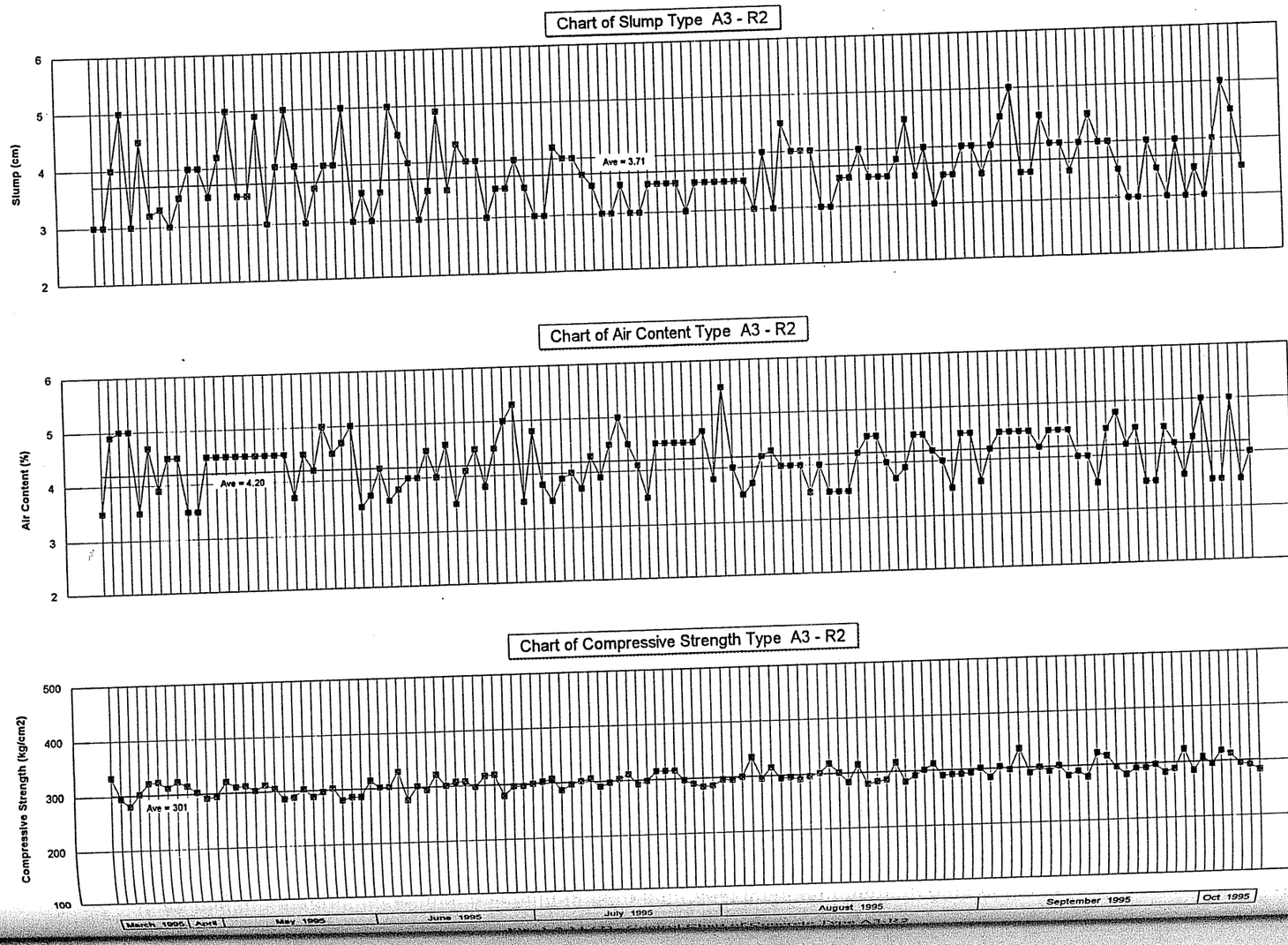


Fig. 1.5.11 (6) Control Chart of Concrete Type A3-R1



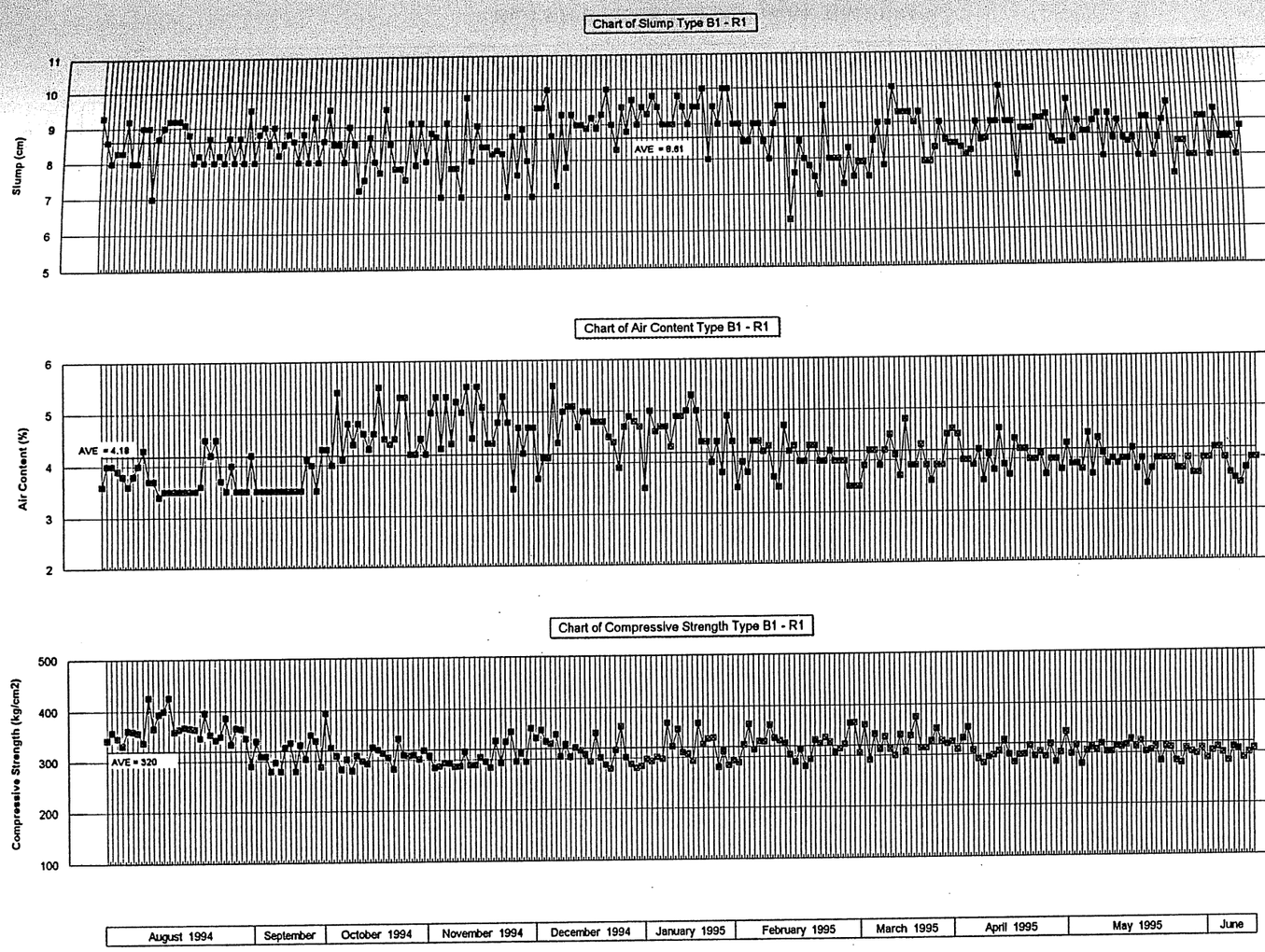


Fig. 1.5.11 (8) Control Chart of Concrete Type B1-R1

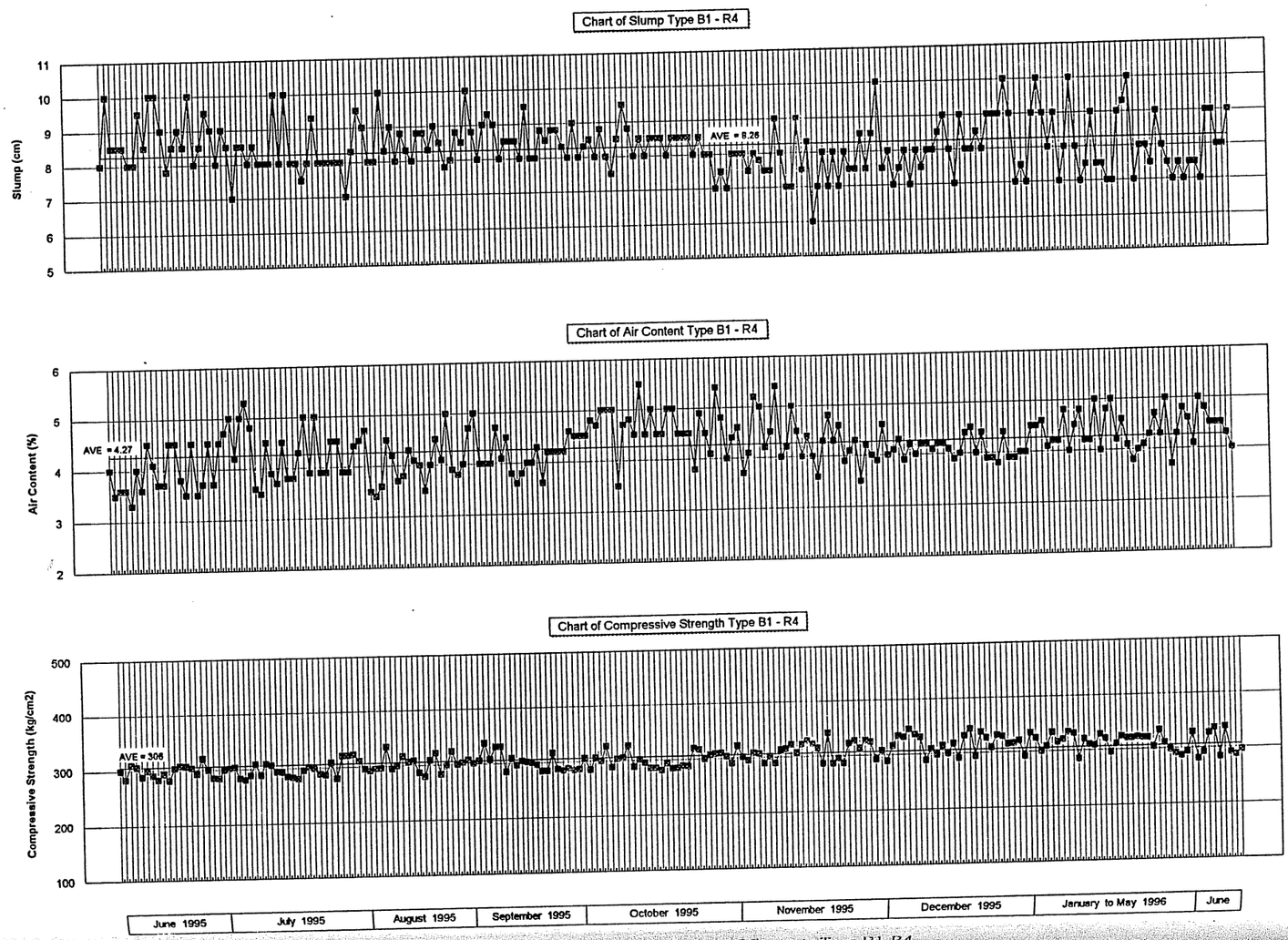


Fig. 1.5.11 (9) Control Chart of Concrete Type B1-R4

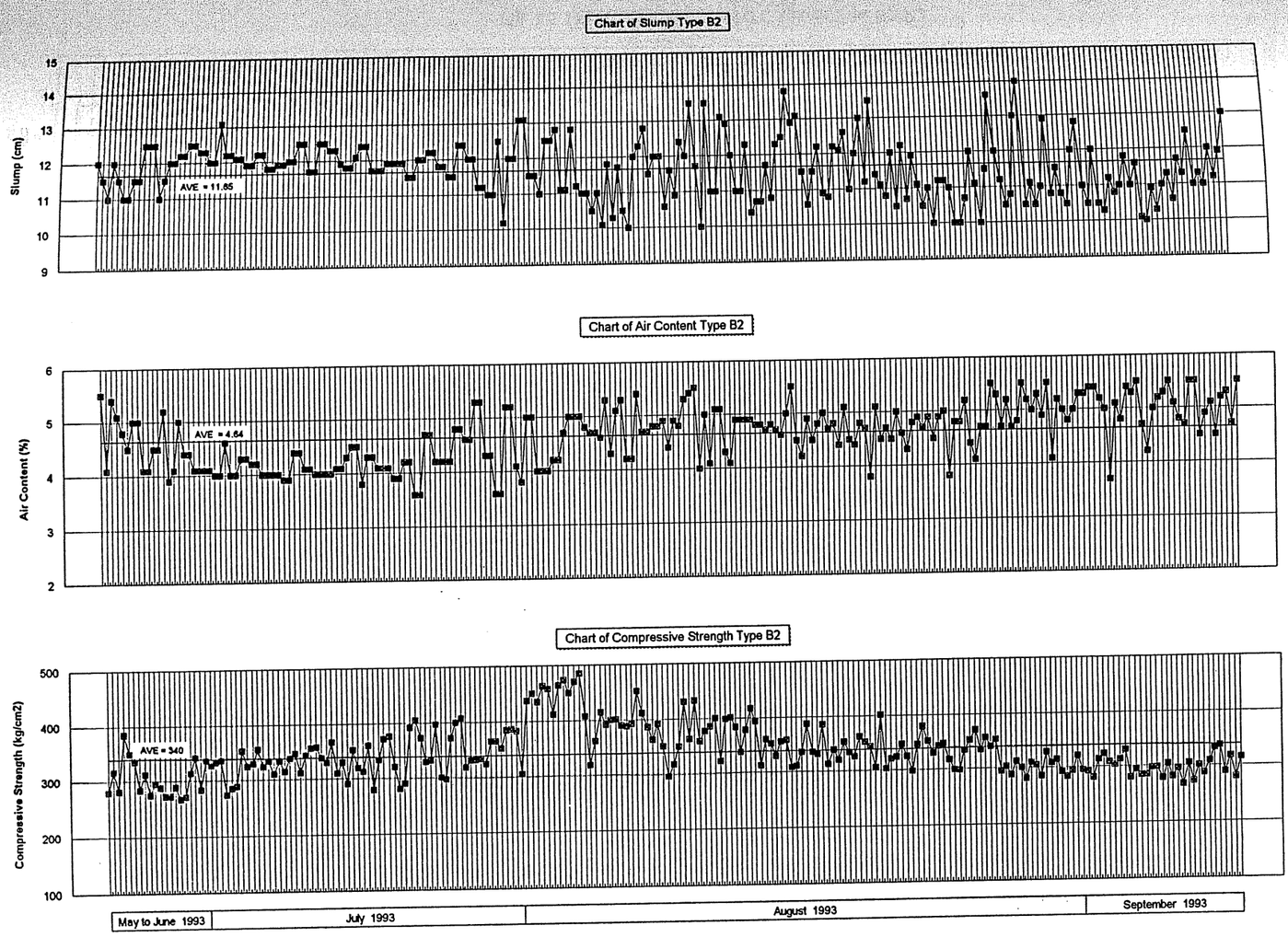


Fig. 1.5.11 (10) Control Chart of Concrete Type B2

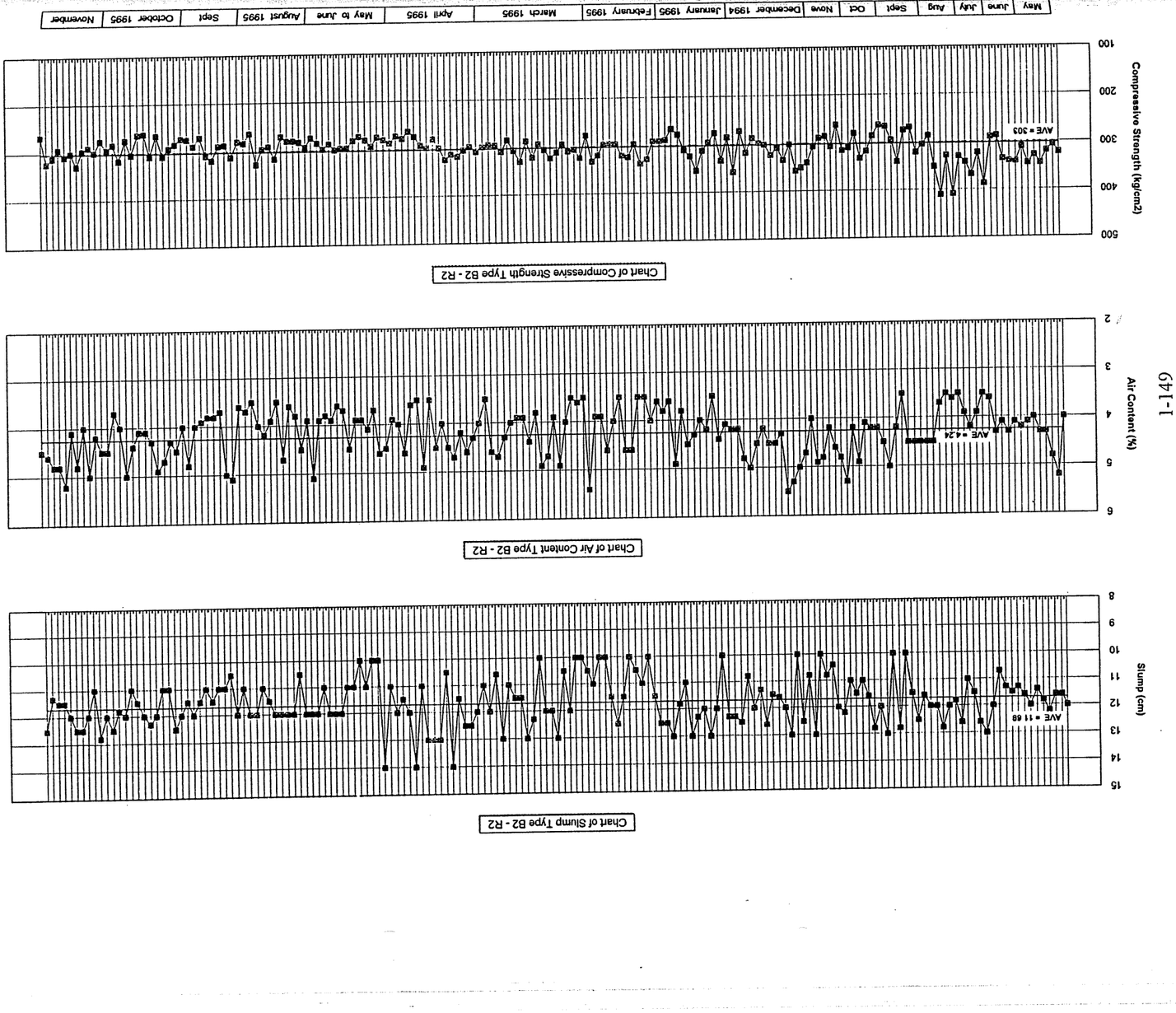


Fig. 1.5.11 (11) Control Chart of Concrete Type B2-R2

Fig. 1.5.11 (12) Control Chart of Concrete Type B2-R3

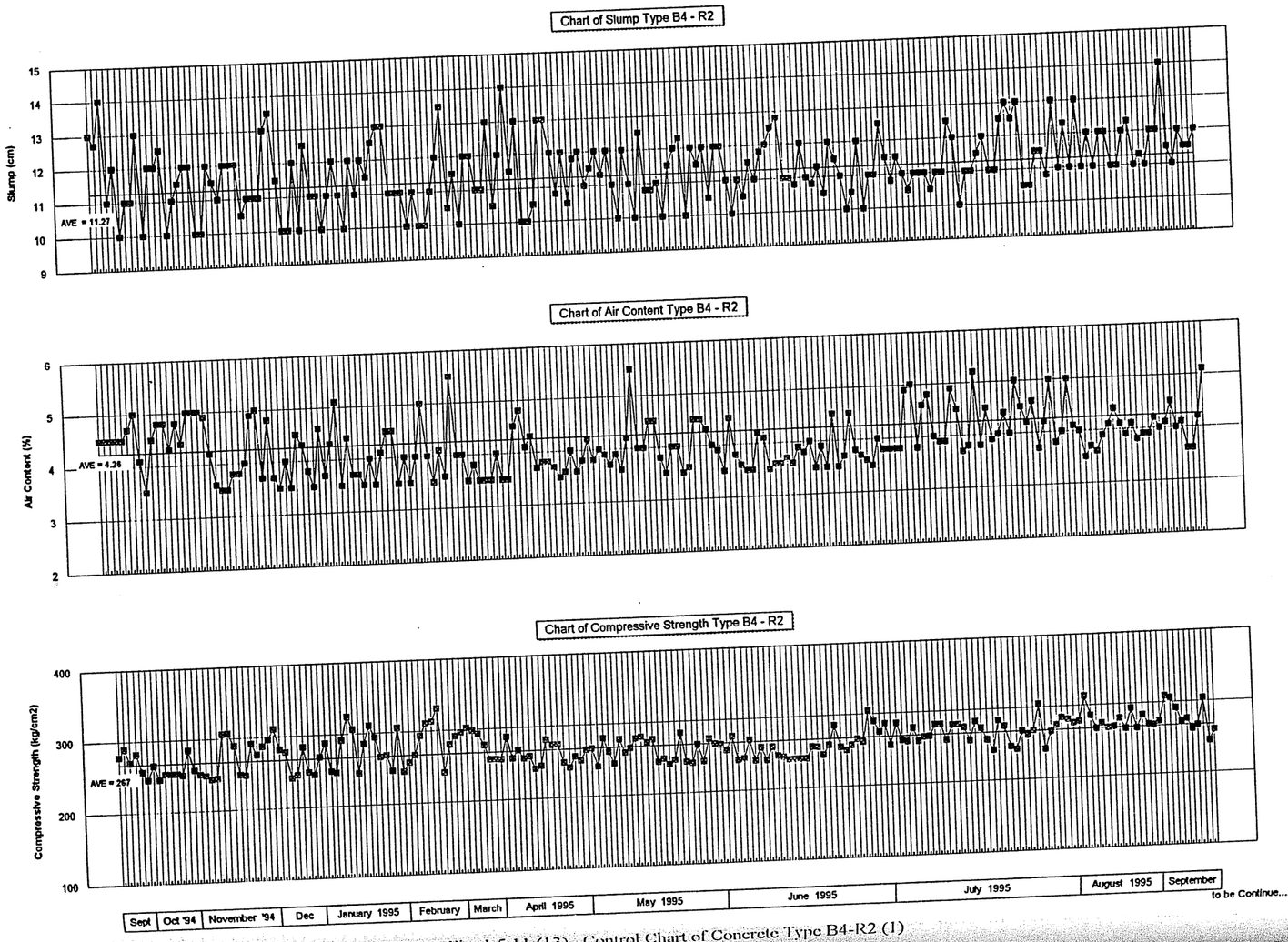


Fig. 1.5.11 (13) Control Chart of Concrete Type B4-R2 (1)

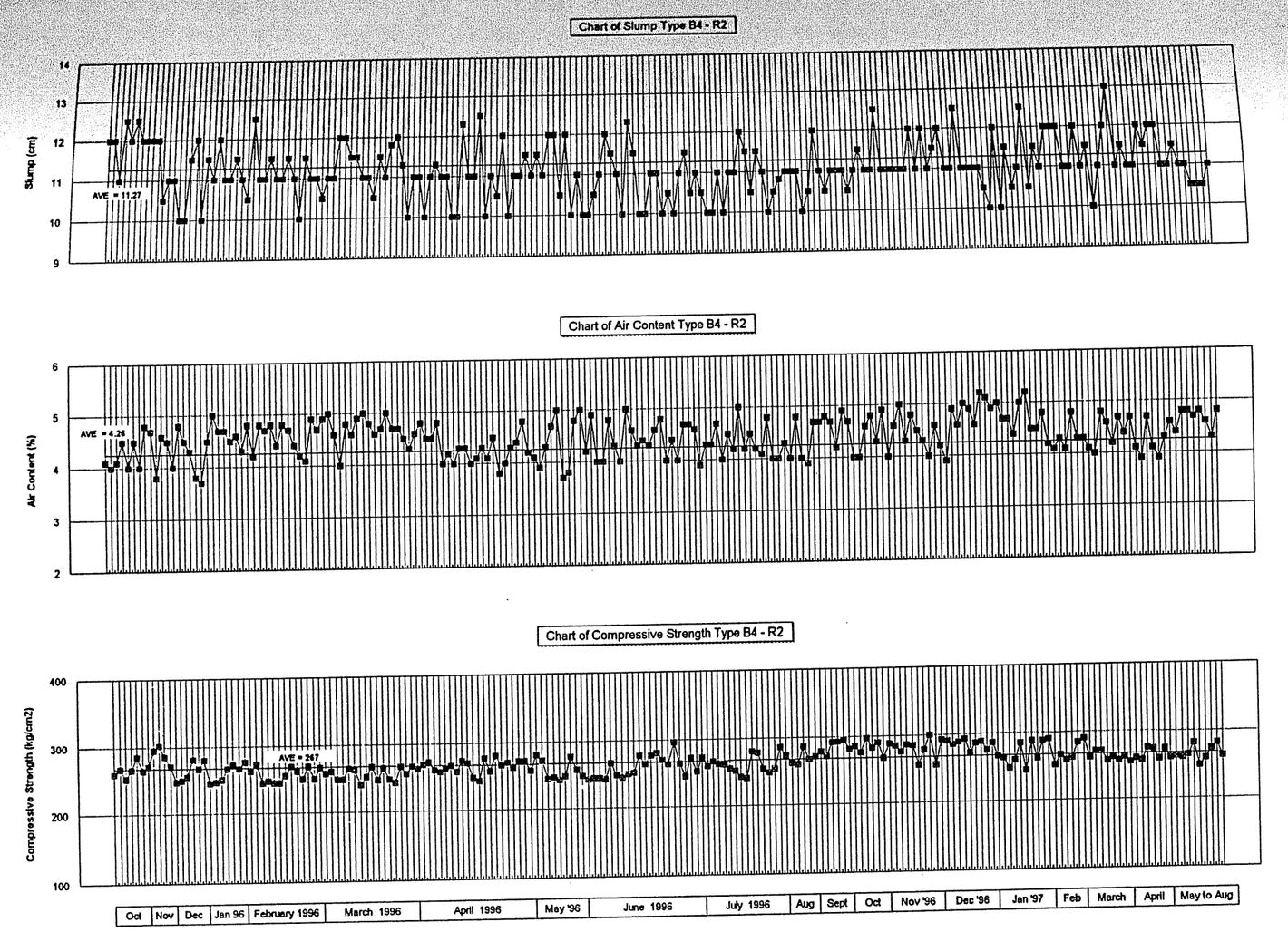


Fig. 1.5.11 (13) Control Chart of Concrete Type B4-R2 (2)

Chapter 6 DESIGN OF MAIN STRUCTURES

6.1 Major Design Change During Construction

Design changes in levels, lines, position and dimensions of some structures including minor change in character or material quality were made during construction to the degree as usually seen in similar works. The major changes during constructions of the dam and power station are summarized as follows :

- Main dam : omission of temporary outlet of dam.
  change in excavation line for dam and stilling basin.
  change in width and design load for spillway bridge.
  change and addition of dam gallery alignment.
  change in mix proportion of concrete on foundation rock.
- Cofferdam : change in cofferdam type
- Foundation treatment : change in target permeability and injection material.
- Tailrace : enlargement of emergency drainage tank and additional pumps.
- Cable duct : change in the alignment of the cable duct from switchyard to powerhouse.
- Muara Takus Buddhist Remains : omission of surrounding protection

Described below are details of the design changes of these. Other minor design changes are described in the Section 6.2 thereafter.

(1) Omission of Temporary Outlet of Dam

(a) General

The original design of the dam incorporated a temporary outlet conduit 1.80 m x 1.5 m wide at EL. 41.50. The purpose of this temporary outlet was to pass water downstream during the initial impounding of the dam. At the time the temporary outlet was included in the design, the proposed date for diversion closure was 1 May 1996, i.e. the start of the traditional "dry" season.

During construction, however, the start of impounding was rescheduled to in the month of November and it was judged that at this time of year a temporary disruption of flow past the dam site would result in a flow in the Kampar River downstream of the junction with Kampar Kiri River, which is not less than the corresponding flow during the dry season. This condition would satisfy the requirement of the Environmental Management Plan of August 1988. There are no major water intakes in Kampar Kanan River between Bangkinang and Kampar Kiri junction.

Therefore the temporary outlet was deleted from the dam on the understanding that closure of the diversion would take place in November 1996, at the start of the traditional "rainy" season.

After that, the Engineer studied the various problems associated with the timing of the lowering and closing the gates of the diversion tunnels in order to fix the starting date of impounding. The following items were the main factors involved.

- Contractor Requirements.
- River Flow and Seasonal Requirements.
- Interaction with work of other Contractor and Public Works.
- Environmental & Social Requirements.
- Proposed sequence for closing of diversion.
- Reservoir storage volume by the controlled impounding.

From the above consideration it was recommended that the impounding of the reservoir commenced in March 1997, i.e. at the end of the rainy season which is the equivalent of the river flow in the month of November.

(b) Consideration of Effect on the Downstream

The river discharge at the junction of the Kampar Kiri in the month of March during cut-off of the riverflow by the initial impounding was checked and compared with the natural river flow before impounding in the dry season. The river discharges in the recession section of the downstream area of the dam was also studied.

Table A The Lowest River Discharge in the Recession Area of Downstream

Site		Catchment Area (km^2)	Discharge (m^3/s)	
			Natural flow in Aug./July (before impounding)	Flow in March after cut off the river (during impounding)
Dam Site		3,337	40.2	134.8
Downstream Remaining Area	Dam to Bangkinang	244	43.1	9.7
	Dam to Danau Bingkuang	721	48.8	28.8
	Dam to Junction of Kampar Kiri	1740	61.1	69.6

Note :
1. The driest season, the month of July or August was chosen in Table.
2. The recent lowest discharges data at dam site from 1989 to 1995 was used.
3. The specific discharge with the lowest value of 1989 to 1995 in the dam catchment area was applied for the downstream remaining area.

Table B The Average River Discharge in the Recession Area of Downstream

Site		Catchment Area (km^2)	Discharge (m^3/s)	
			Natural flow in Aug./July (before impounding)	Flow in March after cut off the river (during impounding)
Dam Site		3,337	91.4	285.6
Downstream Remaining Area	Dam to Bangkinang	244	98.1	20.9
	Dam to Danau Bingkuang	721	111.1	61.7
	Dam to Junction of Kampar Kiri	1740	139.1	148.9

Note : 1. The driest season, the month of July or August was chosen in Table.
2. The recent average discharge at dam site of 1989 to 1995 was used.
3. The specific discharge on the average of 1989 to 1995 in the dam catchment area was applied for the downstream remaining area.

(i) Tidal current intrusion to upstream of the junction of Kampar Kiri River.

Kampar River "lower" downstream area is always affected by the tide in the dry season. The tidal water intrudes as far as 129 km upstream up to Kuala Talam. The river discharge at the junction of Kampar Kiri shown in Table A and B indicates that the cut-off of the river flow during initial impounding of reservoir would not artificially cause the tidal current intrusion to upstream of the junction of Kampar Kiri River and Kampar Kanan River.

(ii) Recession section of the downstream of dam

At present, towns and villages along Kampar Kanan river draw supplies of drinking water from wells located away from the Kampar Kanan River. For bathing and washing, the people in the area of Rantau Berangin to Bangkinang could use the wells existing or newly constructed or water supply services by wagons before releasing water from the reservoir. In the area downstream of Bangkinang, at least 30 m^3/s would be gathered from the surrounding tributaries.

(iii) River flow cut-off period during impounding and water release to downstream

The reservoir capacity at the elevation of EL. 67.50 (Spillway Fixed Crest Level) is 285 mill.m^3. In case the starting date of impounding March 1st, it was expected to take one to two weeks to reach EL. 67.500 based on the past riverflow data. After reaching EL. 67.500, water released to downstream was to be done through the spillway gate.

The impounding of reservoir started actually on March 12th and water released to downstream started on March 30th. It took 18 days to reach up to the fixed crest level on spillway because the year of 1997 was equivalent to a draught year and the inflow was very low at this time of year.

(iv) Water usage of Kampar Kanan River

No irrigation and intake facilities between Bangkinang and Kampar Kiri junction exists and there are irrigation facilities at the tributaries flowing into the Kampar Kanan River. In general, within the "lower" downstream area, there are a lot of rivers or streams, Tuip, Kutup, Sangar, Serkop, Kerumutan, Nilo, Segati and Kampar Kiri River. These rivers could cover the shortage of river flow as an alternative to the Kampar Kanan River for a fixed short time period before water release to downstream is started.

In consideration of the conditions mentioned (i), (ii), (iii) and (iv), the construction of the temporary outlet in the dam body was omitted.

(2) Change in Excavation Line for Dam and Stilling Basin

The foundation rock was required to CM class at river bed and lower portion of the both abutments and CL class at upper portion in order to obtain the adequate shear strength and bearing capacity against the design load for assuring stability of each dam block.

Based on the field inspection, geological condition in the lower portion of left abutment (BL.5-7) and downstream portion of river bed (BL.10-12) were not as good as expected due to the distribution of fracture zones, brecciated zones, river bed gravel deposits, completely weathered rocks and so on.

Therefore, the excavation line at each portion was modified and the weak rock-mass was excavated additionally to obtain the moderately sound rock. The maximum additional excavated depths and total additional excavated volumes at each portion were approximately 10 m, 2,800 m^3 and 1.5 m, 1200 m^3 respectively. At the stilling basin, the additional excavation for foundation rock was required and reached 38000 m^3 with maximum additional excavated depth of 3.2 m. The additional excavation area was replaced with the dam concrete.

Item	Original	Revised
Excavation Vol.		
- Dam	212,490 m^3	285,054 m^3
- Stilling Basin	55,370 m^3	105,722 m^3

(3) Change in Width and Design Load for Spillway Bridge

It became clear by Lot II Metal Works Contractor that an 80 ton class mobile crane was needed to handle the spillway stoplogs in the construction stage. Therefore, the design of the spillway bridge was modified and its dimensions enlarged to provide the total width 6.3 m. The beam's height was increased to 75 cm in order to satisfy required design load, 80 ton mobile crane.

Item	Original	Revised
Total width	5.0 m	6.3 m
Beam height	0.50 m	0.75 m
Number of beams	15	10
Design load	TL-20	80 t mobile crane

(4) Change and Addition of Dam Gallery Alignment

The original alignment of the dam gallery was designed to be along the dam axis between right and left abutment at EL. 34.5 to EL. 79.5. However, various measurement instruments, electric cables, control cables, drain pumps and drain pipes were installed in the gallery and the staff shall enter the gallery quite often in future for the monitoring and inspection for these facilities. Therefore, the additional gallery at EL. 45.5 directed up-downstream, was designed to be used as a connecting passage-way between original gallery and powerhouse control building.

Item	Original	Revised
Horizontal parts along the dam axis	EL.79.5, EL.55.5 and EL.34.4	EL.79.5, EL.55.5, EL.45.5 and EL.34.5
Length of up-downstream direction gallery	0.0 m	26.3 m

(5) Change in Mix Proportion for Concrete on Foundation Rock

After completion of concrete placement on the foundation rock (1st layer), some cracks were found and considered to be caused by an increment of thermal stress due to external restraint. Therefore, the mix proportion of concrete on foundation rock was changed to reduce heat generation of concrete on condition that the concrete was rich enough to ensure filling the cracks and joints in the foundation rock.

Item	Original	Revised
Type of concrete	A-1	A2-R1
Design strength	330 kg/cm^2	220 kg/cm^2
Slump	3± 1 cm	3± 1 cm
Cement content	223 kg/m^3	184 kg/m^3

(6) Change in Cofferdam Type

In original design, the main cofferdam of upstream was designed as a conventional concrete gravity type to prevent damage to the cofferdam by river flow overtopping in flood time. The conventional type, however, had to be replaced with RCC type to accelerate execution of the work. The downstream main cofferdam was also changed from a concrete gravity dam to a fill type dam for the main reason of easy removal after completing construction of the main dam.

Diversion work was to start in April 1993 based on the Tender Design. However, the diversion open works actually started in October 1993 due to the delay of official approval of the contract, which was 6 months delayed from the original plan. As a result, the schedule of the river diversion targeted originally in June or July in dry season 1994 had to be extended to December 1994 amid the rainy season.

In order to avoid the risk of conducting the river diversion in rainy season and to meet the requirements of other contractors schedule (Lot II Metal Works and Lot III Generating Equipment), the work acceleration was required to keep the project time schedule.

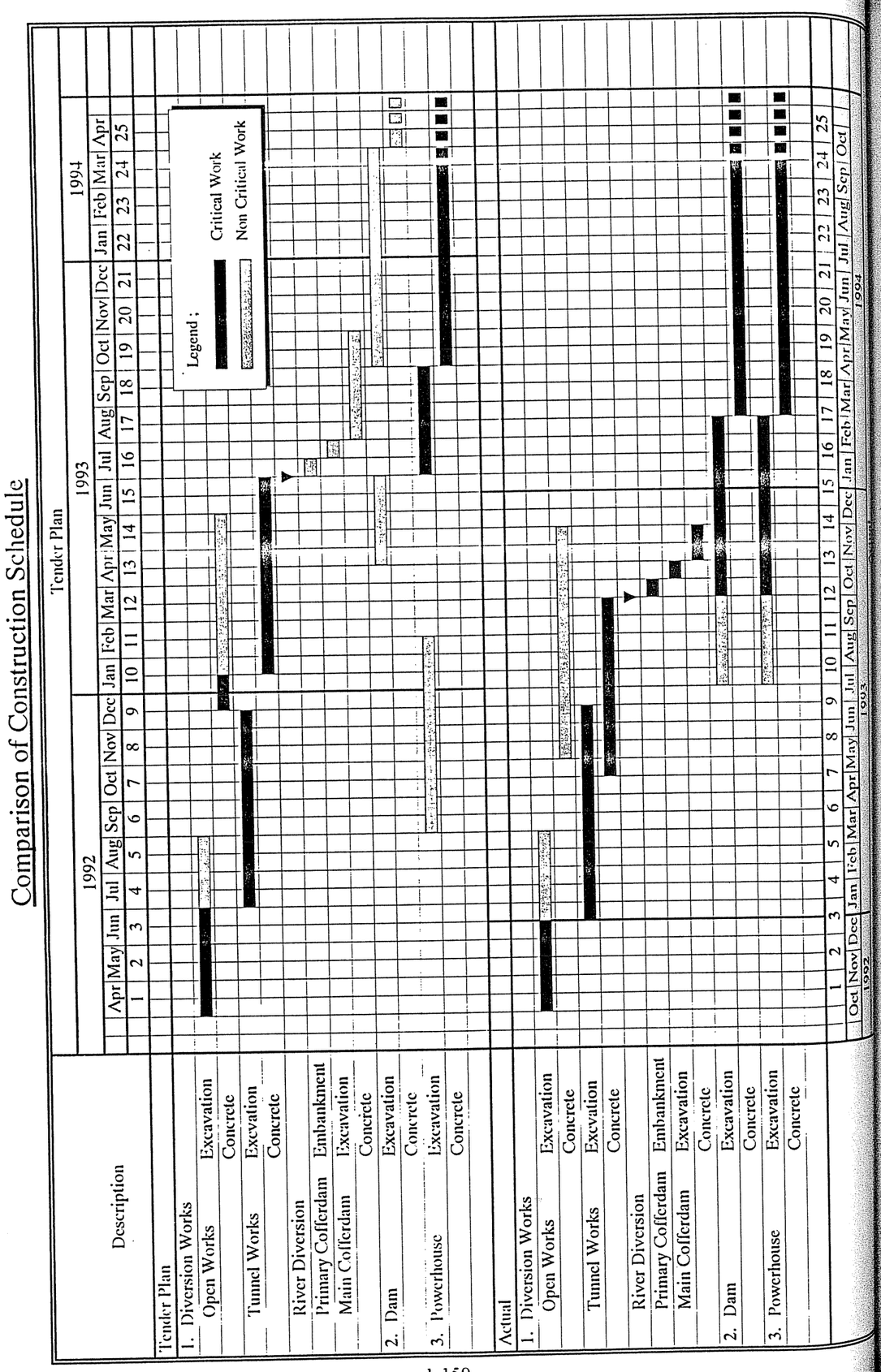
The open works excavation, the tunnel works excavation and concreting were shortened by 3.5m months from original plan by accelerating the execution work time. However, the Contractor could not catch up and work was still behind the time schedule targeted for making the river diversion before rainy season.

Therefore, in order to shorten the construction period without additional construction cost, the type of the upstream cofferdam was revised to RCC (Roller Compacted Concrete) type. The modification of the type of main cofferdam reduced the construction period approximately 1.5 month and saved construction cost. The modification of the downstream cofferdam to a fill type made it easier and cheaper for removal than a concrete gravity type especially in consideration of the timing of the start of impounding and the discharge through spillway.

Some design modification for the diversion tunnel included a change in the dam type mentioned above were done in order to shorten the construction period of the works. The design modification was also made so as to meet the hydraulic design requirement including some minor modified design.

Items	Original	Revised
- Upstream Cofferdam - Downstream Cofferdam	conventional concrete conventional concrete	RCC Fill type
Elevation of Tunnel No.1 Invert EL. at Inlet	EL. 36.0 m	EL. 37.5
Outlet structure	15.0 m	Omission
Work Adit Length	0.0 m	85.0 m

Comparison of Construction Schedule



(7) Change in Target Permeability and Injection Material for Foundation Treatment

Although the grouting work proceeded up to the quaternary and quinary holes, the result of permeability still indicated higher Lugeon than the target value of 2 Lu and a considerable amount of additional holes would be required to achieve the 2 Lu. Therefore, judging from the results of the seepage analysis, piping, uplift pressure and drain discharge, the target permeability of curtain grouting was revised to be 5 Lugeon.

On the other hand, the upper part of dam foundation rock in the river bed indicated to have a rather high permeability and the exist fine fissures would not fill up by normal cement. Therefore, based on test grouting procedure using Ultra Fine cement, the injection material for the quaternary, quinary and additional injection was changed to be ultra Fine Cement.

Item	Original	Revised
Target permeability	2 Lu	5 Lu
Injection material for the quaternary, quinary and additional	Ordinal Portland Cement	Ultra Fine Cement

(8) Emergency Water Storage Tank at Tailrace

On the tender design drawing there was no provision for an extra water storage tank for rainfall runoff. The original design allowed for backfill material to be placed in the 2.5 m space between the powerhouse and tailrace structures at elevations between EL. 37.10 m to EL. 45.30 m.

The backfill material was modified to allow placement of lean mix concrete to take into account the monolith construction required and provision for watertightness adjacent to the powerhouse and tailrace structures. Six hundred cubic metres of backfill concrete was placed from elevation EL. 37.10 to EL. 40.7.

The area above elevation EL. 40.7 was available for the installation of an emergency water storage tank, to provide additional water storage for the excessive drainage runoff that would occur during heavy rainfall around the powerhouse area.

The change in drainage system at the tailrace was as follows :

Items	Original	Revised
Water Tank	Concrete structure length 5.0 m, width 2.0 m, height 1.5 m	The same as original length 9.5 m, width 2.0 m, height 2.3 m
Number of Pump	1 set	2 sets (1 set, contingency)
Drain Pipe Length	25 m (Ø 80 mm)	51 m (25.5 m x 2 lines, Ø 80 mm)
Drainage Gate	Sluice type	Knife type value
Gate Dimension	Width 0.5 m, height 0.5 m	Circle type Ø 0.5 m
Total Weight of Gate	0.12 ton	0.212 ton
Manhole	Steel manhole Ø 800 mm (JIS G 5506)	Concrete precast 1000 mm x 1000 mm

(9) Cable Duct Layout from Switchyard to Powerhouse

An of unfavourable topography of the very steep slope near the dam site was found after the start of excavation for dam access road near the dam there was difficulty constructing the cable duct on the original route between the national road and dam site.

The left bank of the dam site generally consists of steep topography and there is no better site, however, the alternative alignment is topographically rather easier than the other routes because the existing temporally access road to cable crane operation house was available for the upper part of the alignment. Also the length of cable duct is nearly the same as the original alignment. Therefore, the alternative was chosen as an optimum alignment. The water supply pipe from dam site to switchyard is also installed along the cable duct alignment.

In the Tender Design, a steel plate cover was proposed to cover the whole duct except beside the switchyard access road and crossing of national road. It was decided, however, that especially considering risk of theft, a concrete cover be used for the entire length except at each 10m interval where a steel plate stopper was placed in conderation of maintenance and repairing. The thickness of steel plate cover originally 3.6mm was changed to 6.0mm in consideration of a mans weight for maintenance and repairing.

The change in cable duct length was as follows :

Item	Original	Revised
Cable Duct Length	612 m	530 m

(10) Omission of Surrounding Protection for Muara Takus Buddhist Remains

The remains of a Buddhist temple are located about 25 km west from the dam site in a straight line. Since it is important from a historical point of view, study of the reservoir water level was carefully done in accordance with a basic policy for preserving the remains so that they would not be submerged nor be affected by execution of this project.

The survey and study for preservation of the remains were carried out by PLN and Ministry of Education and Culture. It was concluded that special countermeasures for the preservation of the remains would not be required because the high water level of the reservoir was set at EL 85.00 and the temple situated at EL. 86.25 m would not be submerged by reservoir impounding.

It is predicted that the highest water level of the reservoir for the flood occurrence of 200 year return period could reach to EL. 84.70 m according to spillway gate control at the Kotapanjang dam. As for back water arising in the flood time in the critical condition, the water level would rise to EL. 86.10 m that is 0.15 m lower than the elevation of the Temple site.

The following study was done for the riverbank and the temple foundation in order to make certain that the protection work would not be needed.

Riverbank Stability

The calculation was done for the most critical condition of a land slide occurrence possibility that is the ground being saturated with water. In this critical condition, the safety factor show more than 1.2. Therefore, there is no possibility of land slide occurrence of river bank on a large scale after impounding of reservoir.

The fluctuation of reservoir water level is very slow for the required reservoir operation discharge because the reservoir capacity is very large. Also if the reservoir water level is normally operated up to EL. 83.000 m, then there is little chance that reservoir water level will become higher than EL. 85.000 and will not directly impact on the riverbank. From the above consideration, the riverbank protection would not be required at this site.

Temple Foundation

In order to check the likely permeability improvement of the temple foundation if the consolidation grouting was conducted, a seepage analysis was carried out for the temple foundation.

From the calculation result, there was no remarkable improvement by grouting around the temple structure. There is a gravel layer 2 m thick, 1000 Lu under EL. 81.5 m and grouting can not demonstrate its effect. Also, high pressure by grouting injection into the foundation surrounding the temple structure may cause instability under the foundation make the adverse effect on the temple structure. Therefore, it was concluded that foundation improvement of the temple would not be required.

6.2 Main Dam

6.2.1 General

The original shape of the Kotapanjang dam was designed to provide maximum economy in materials based on design loads that were determined and combinated at design stage and geological condition that were expected through investigation results of drilling and test adits. During the construction stage, some minor design modifications and additional stability calculations of the dam were made in consideration of the actual condition after completing the dam foundation excavation and after finalizing the metal structures design.

The main modification items on design conditions and dimensions of structures are summarized as follows ;

- Modification of dam height at both abutments of the dam body caused by modifying the design excavation line on the dam foundation based on the actual foundation condition after excavation and rock classification.

- Modification of dimensions of piers, hoist deck and spillway bridge to suit the actual detail dimensions and weight of steel structures, i.e. gate body, guide frame and hoist for spillway gate, intake gate and each stoplog, trashrack, trashrake and penstock.

Moreover, the final stability calculation of the dam was carried out with the condition of the uplift pressure actually measured acting on the dam body at initial impounding stage. Judging from the results of all calculations carried out on the conditions mentioned above, the stability of dam body was assured.

The following sections describe the process of determination of original dam shape, including theoretical triangular section, the results of the stability calculation based on the design modifications during construction stage with actual uplift pressure measured at initial impounding respectively.

6.2.2 Basic Principles of Dam Design

The gravity dam is normally designed so that its weight and thickness should insure stability against all the imposed forces. Wave action due to wind and /or earthquake should not overflow beyond the crest of the non-over flow section in the dam. Therefore, this section describes the design criteria of the gravity dam that includes both stability criteria of dam body and criteria of dam design details, especially the free board of dam, then, describes the process of determination of the theoretical triangular section of the Kotapanjang dam.

(1) Stability Criteria

For a gravity dam to be stable, the following criteria at any horizontal section and contact face between dam body and bedrock should be satisfied against the external forces and weight of dam body acting on a cantilever element of gravity dam section.

- No tensile stress must be developed along the upstream face in the profile normal to the dam axis. In order to satisfy this requirement, resultant of static forces should be within the middle third line at all sections.

- Safety must be ensured against shearing at the base, the construction joints of the dam and any planes of weakness within the foundation.

- Both compressive and tensile stresses must not exceed their allowable limits.

The stability analysis of a gravity dam is based on the following assumptions,(i) the material in the foundation and in the body of dam is isotropic and homogeneous, (ii) the stresses in the foundation and the body of the dam are within elastic limits,(iii) no movements are caused in the foundations due to transference of the load, (iv) the foundation and the dam behave as one unit, the joint being a perfect one, (v) no loads are transferred to the abutments by beam action and (vi) the stability analysis of the dam is based on considering a slice of the dam one metre thick at the base line and contained between the two vertical planes normal to the base line.

(a) Against overturning

The dam shall resist against overturning caused by the moment acting on the dam base.

$$\sigma_u = V/B(1-6e/B) > 0, \text{ then } e < B/6$$

where, σ_u : stress intensity in the upstream end (t/m^2)
B : the length of the base (m)
V/B : the vertical stress intensity (t/m^2)
e : the distance between the center of base and the acting point of the resultant force (m).

The above condition stipulates the resultant force acting on the dam base must be contained in the region of the middle third of the base. If the above condition is satisfied, any tensile stress does not arise at the front of the dam base.

(b) Against sliding

The shear friction resistance of the contact plane between dam body and bedrock as well as of the weak zone within the bedrock must have a safety factor against the shear force. The stability against sliding on the contact plane between dam body and bedrock was examined by the safety factor required for shear friction resistance as follows :

$$n = (fV + \tau\ell)/H \geq 4$$

where n : safety factor for shear friction
f : coefficient of internal friction
v : normal force per unit width of shearing plane including uplift (t/m)
τ : shear strength (t/m^2)
ℓ : length for shear resistance (m)
H : shearing force per unit width (t/m)

The value of τ in the above formula shall be the smaller of the shear strength value of either bedrock or its contact plane with concrete, and the value of n shall be 4 or more.

(c) Against compressive and tensile stress

The stress analysis of gravity dams is generally made by two dimensional methods applied to cross sections of dams. The conventional method for calculating the concrete stress is follows,

(i) Stress at the upstream and downstream end.

The stress at the upstream and downstream end can be expressed as below :

$$Pd = v/B\ (1+6e/B),\ Pu = v/B\ (1-6e/B)$$

where Pd, Pu : unit stress at downstream and upstream end respectively (t/m^2)
B,v/B,e : Refer to "(a)" of this section

(ii) Maximum stress

$$\sigma_{max} = V/B\ (1+6e/B)\ (1+n^2),\ \tau_{max} = 1/2\,\sigma_{max}$$

where, σ_{max} : the maximum principal stress
τ_{max} : the maximum shearing stress
n : the gradient of the downstream face of the dam

The allowable stress shall be more than σ_{max} and τ_{max}.

(2) Loads to be taken into Consideration

The design is carried out for the condition of the design water level of the reservoir. The external forces to be considered for the design are mainly composed of the following items :

- Hydrostatic pressure at design water level of the reservoir
- Hydrodynamic pressure due to earthquake
- Silt pressure

- Uplift pressure
- Seismic body force
- Dead weight of the dam including superstructure

(a) Hydrostatic pressure at design water level

The design water level is composed of the static head of high water level and the height of the wave due to wind and earthquake. The hydrostatic pressure acts normal to the upstream face of the dam and its resultant force is calculated by the following formula.

$$ps = Wo.(h+hw+he),\ Ps = 1/2.Wo.(h+hw+he)^2$$

where, ps : hydrostatic pressure (t/m^2)
Ps : resultant force of hydrostatic pressure (t/m)
Wo : unit weight of water (t/m^3)
h : water depth from the high water level to each section (m)
hw : wave height due to wind (m)
he : wave height due to earthquake (m)

(b) Hydrodynamic pressure due to earthquake

The hydrodynamic pressure on the dam body due to earthquake must be considered as acting normal to its upstream face, and its value is calculated by the following formula (Westerguard).

$$pd = 7/8.Wo.k.(Hh)^{1/2}$$

where, pd : hydrodynamic pressure (t/m^2)
Wo : unit weight of water (t/m^3)
k : seismic coefficient
H : water depth measured from wave surface of the reservoir to foundation ground (m)
h : water depth from water surface down to a given point (m)

The resultant force of the hydrodynamic pressure between the water surface and the given point can be obtained integrating the above equation.

$$Pd = 7/12.Wo.k.(Hh^3)^{1/2},\ hd = 2/5.h$$

(c) Silt pressure

Silt pressure in the vertical direction must be estimated by the weight of silt measured in water, while the pressure in the horizontal direction must be calculated by the following formula :

$$Pe = Ce.w_1.d\ ,\ Pfe = 1/2\ (Ce.w_1.d).d$$

where, Pe : silt pressure in horizontal direction (t/m^2)
Pfe : horizontal force of silt pressure (t/m)
Ce : coefficient of silt pressure
w_1 : unit weight of silt deposit measured in water (t/m^3)
d : depth of silt deposit (m)

Silt deposit is not suspended particles of silt, and the voids are saturated with water forming a mass. Therefore, the weight of deposited silt is shown by

$$w_1 = w - (1-v)\, wo$$

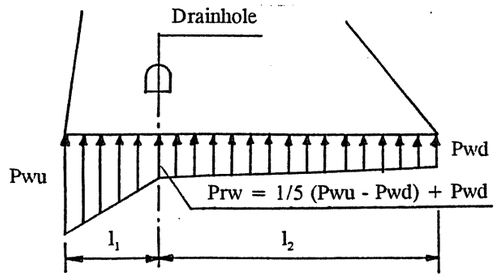
where, wo is unit weight of water (t/m^3), w apparent unit weight of silt (t/m^3) and v porosity. Dynamic silt pressure due to earthquake is normally not taken into consideration.

(d) Uplift pressure

In a stability analysis of the dam, the uplift pressure must be considered to distribute over the entire area of any plane examined, and should be treated as acting normal to that plane. The uplift pressure adopted for the calculation are empirically as follows,

Value of Uplift Pressure Acting on Concrete Gravity Dams

Uplift		
Upstream End (Pwu)	Position of Drain Hole (Prw)	Downstream End (Pwd)
Upstream water pressure (corresponding upstream static head, HWL)	Downstream water pressure plus over 1/5 of the difference between upstream and downstream heads.	Downstream water pressure (corresponding to downstream static head, TWL)



where, Pwu : upstream water pressure
Pwd : downstream water pressure
l_1 : distance between drain holes and upstream
l_2 : distance between drain holes and downstream

Uplift pressure shall not be affected by wave and short period fluctuations of reservoir water level.

(e) Seismic body force

The seismic body force must be taken as the value of the weight of the dam body multiplied by the seismic coefficient for the dam body, and it should be considered as acting horizontally. The seismic coefficient of the dam body is described in detail in Chapter 2, Sec.2.3.5 Seismicity and Seismic Force.

F = W1.k (high water level state), F1 = W1.k/2 (empty state).

where, W1 : weight of concrete dam body
k : horizontally seismic intensity (0.10)

(f) Dead weight of the dam including superstructure

For the design of the dam, the weight of the piers, bridge, gate and hoist etc. are added to the weight of dam body itself.

(3) Fixing Size of Kotapanjang dam Section

Through the feasibility study and the detail design stage, the basic dimensions on Kotapanjang dam and reservoir were determined as follows :

-	Type of dam	Concrete gravity dam
-	Dam foundation elevation	EL. 29.500
-	High water level	EL. 85.000
-	Low water level	EL. 73.500
-	Design sedimentation level	EL. 64.000
-	Crest elevation at overflow	EL. 67.500
-	Center elevation of Penstock	EL. 63.500

(a) Determination of the crest elevation of the dam - freeboard

The crest elevation of the non-overflow section of the dam must be equal to maximum design water surface level plus freeboard. The required freeboard was determined on the basis of experience and results of studies. Wave heights produced by wind and earthquake effects were calculated as follows.

(i) Waves due to wind

$$hw = 0.00086\, V^{1.1}\, F^{0.45} = 1.370\ m\ (S.M.B\ method)$$

where, V : wind velocity (=25 m/sec.)
F : Fetch (=5.000 m)

(ii) Wave due to earthquake

$$he = 1/2k.T/\pi(gHo)^{½} = 0.371m$$

where, k : Design seismic intensity (=0.10)
T : Period of seismic waves in second (=1.0 sec)
Ho: Depth of reservoir water (=55.5m)

(iii) Additional height

Rise of reservoir water level due to unexpected accident may be considered to be in proportion to the flood discharge, duration of accident and number gates etc,. But it is impossible to predict a specific value, therefore the value of 0.5 m is taken in consideration of the above factors and its standard value which may be taken as about 0.5 m.

(iv) Heights of freeboard (hf)

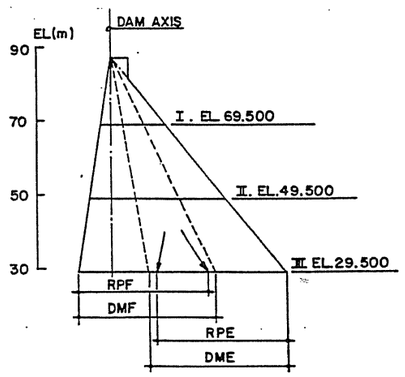
hf > hw (height of wave due to wind)+he (height of wave due to earthquake)+ha (additional height) = 2.241m

From the result of the above calculation (2.241m) is slightly more than standard height of freeboard (2.0m) for a concrete dam of height 50-100m stipulated by the Japanese National Committee on Large Dam. The height of freeboard of 2.5m was selected for the Kotapanjang dam for an additional margin of safety.

Therefore, the crest elevation of the non-overflow section of the dam was determined EL. 87.500 and the height of the dam was 58.000m.

(b) Determination of theoretical triangular section of the dam

Assuming several slopes of upstream and downstream, a theoretical triangular section which provides maximum economy in materials studied on the basis of the middle third condition. Assuming an adequate foundation, the most usual downstream slopes of concrete gravity dams are somewhere around 0.8 (H) : 1 (V) and therefore the slopes are studied based on these figures. Figure of study on upstream and downstream slopes is shown below.



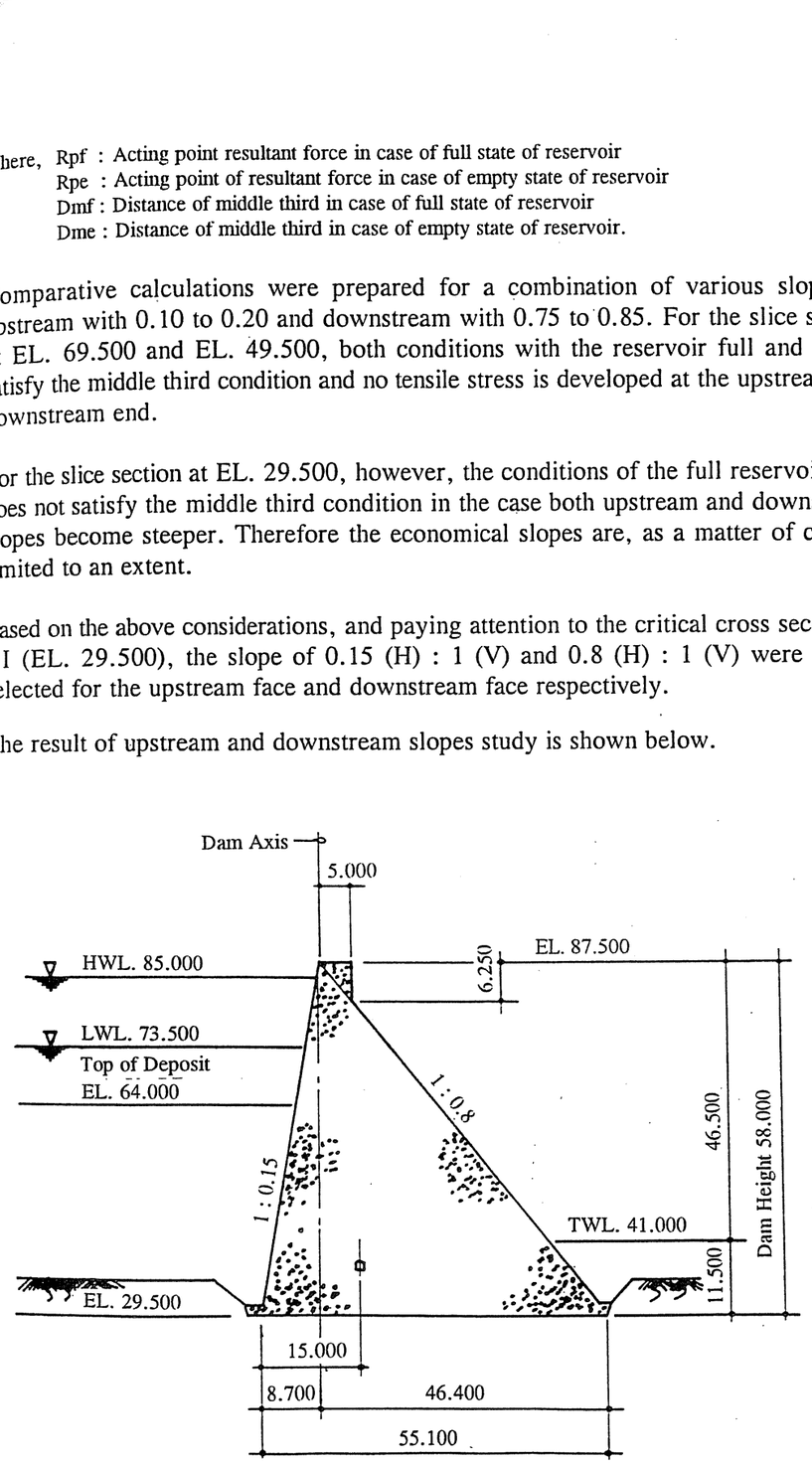
Where, Rpf : Acting point resultant force in case of full state of reservoir
Rpe : Acting point of resultant force in case of empty state of reservoir
Dmf : Distance of middle third in case of full state of reservoir
Dme : Distance of middle third in case of empty state of reservoir.

Comparative calculations were prepared for a combination of various slopes of upstream with 0.10 to 0.20 and downstream with 0.75 to 0.85. For the slice section at EL. 69.500 and EL. 49.500, both conditions with the reservoir full and empty satisfy the middle third condition and no tensile stress is developed at the upstream and downstream end.

For the slice section at EL. 29.500, however, the conditions of the full reservoir state does not satisfy the middle third condition in the case both upstream and downstream slopes become steeper. Therefore the economical slopes are, as a matter of course, limited to an extent.

Based on the above considerations, and paying attention to the critical cross section of III (EL. 29.500), the slope of 0.15 (H) : 1 (V) and 0.8 (H) : 1 (V) were finally selected for the upstream face and downstream face respectively.

The result of upstream and downstream slopes study is shown below.



6.2.3 Stability Analysis of the Dam

In the Kotapanjang dam, the dam body consists of three parts structurally, non-overflow part, overflow part, and intake part. One section at each structural part was selected as the design section for the stability calculation of the dam. The design section has the maximum dam height of 58.0m respectively at the non-overflow, the overflow part and the section J7+10m (BL.8) which is at the center of unit No. 1 penstock. The stability calculation on each design section was carried out and the stability of each design section was assured in the design stage.

During the construction stage and initial impounding of the reservoir, additional stability calculations were carried out to confirm the dam stability due to minor changes of the design condition.

This section describes the results and evaluation of these calculations based on the initial design condition in the design stage, modified design condition in the construction stage and initial impounding stage.

(1) Basic data of dam and reservoir

Dimension of Dam	
Dam crest elevation	EL.87.500 (Non-overflow), EL.67.500 (Overflow)
Dam foundation elevation (Riverbed)	EL. 29.500
Dam crest width	5.000m
Upstream face	1:0.15
Downstream face	1:0.80
Dimension of Reservoir	
High water level	EL. 85.000
Design sedimentation level	EL. 64.000
Wave height by wind	1.370m
Wave height by earthquake	0.371m
Dimension of Downstream	
Tailrace water level	EL. 41.000
Downstream ground elevation (Left abutment only)	EL. 45.300

(2) Design and Loads Condition

(a) Design water level

Normal state : (High water level) + (Wave height by wind)= 86.400 (EL.)
Earthquake state : (High water level) + (wave height by wind) + (wave height by earthquake) = 86.800 (EL.)

(b) Load Condition

Unit weight of dam concrete	: 2.30 t/m³ (Plain), 2.40 t/m³ (Reinforced)
Unit weight of reservoir water	: 1.00 t/m³
Unit weight of silted deposition	: 1.80 t/m³ (in the air), 1.20 t/m³ (in water).

Coefficient of earth-pressure		: 0.5 (in water).
Uplift pressure		: refer 6.2.2.(2).(d)
Horizontal seismic intensity		: 0.10 (water-filled state), 0.05 (empty state).
Allowable stress of concrete		(Normal) (Earthquake)
Compressive stress		: 55 kg/cm² 72 kg/cm²
Tensile stress		: 3 kg/cm² 4 kg/cm²
Shearing stress		: 23 kg/cm² 30 kg/cm²
Coefficient of internal friction for concrete		: 0.80
Shearing strength of rock bed	(General)	: 18 kg/cm²
	(CL)	: 4-10 kg/cm²
	(CM)	: 10-24 kg/cm²
	(CH)	: 24-40 kg/cm²
Load bearing capacity of foundation		: 40 kg/cm².
Coefficient of internal friction for rock bed		: 0.78

(3) Calculation cases

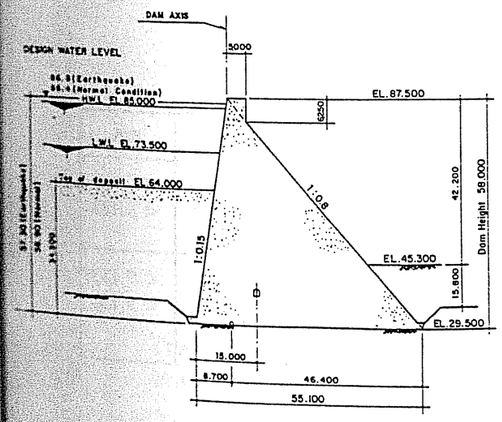
Calculations were carried out on the cross-section of the overflow section, non-overflow section and intake section as follows.

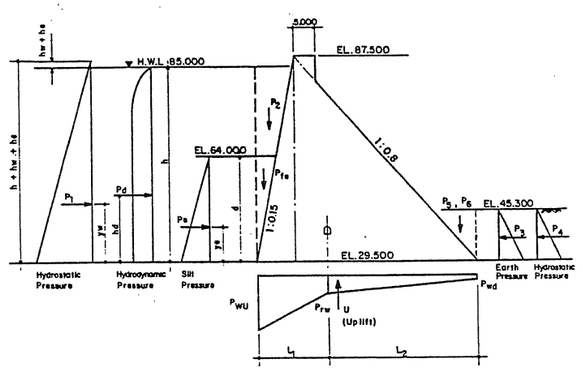
Water Level	Case	Earthquake
High water level state	Normal	-
	Earthquake	k = 0.10 for downstream direction
Empty state	Normal	-
	Earthquake	k/2 = 0.05 for upstream direction

(4) Stability Analysis at Detailed Design Stage

(a) Non-overflow section

(i) Total of external force and dead load.





	Load items	Vertical Force(t/m)	Horizontal Force(t/m)	X (m)	Mx (tm/m)	Y(m)	My (tm/m)
Normal for HWL	Dam weight	3711.108	-	12.477	46304.437	-	-
	Upstream water press	242.821	1618.805	- 5.855	- 1421.717	18.967	30703.874
	Downstream water press	99.856	- 124.820	42.187	4212.918	5.267	- 657.427
	Downstream earth press	99.856	- 62.410	42.187	4212.918	5.267	-328.713
	Silt press	107.123	357.075	- 6.975	- 747.183	11.500	4106.363
	Uplift press	1387.077	-	13.350	- 18517.100	-	-
	TOTAL	2873.687	1788.650		34044.273		33824.139
Earthquake for HWL	Dam weight	3711.108	371.111	12.477	46304.437	19.688	7306.433
	Upstream water press	246.247	1641.645	-5.835	-1436.851	19.100	31355.420
	Downstream water press	99.856	-124.820	42.187	4212.918	5.267	-657.427
	Downstream earth press	99.856	-62.410	42.187	4212.918	5.267	-328.713
	Silt press	107.123	357.075	-6.975	-747.183	11.500	4106.363
	Dynamic water press	-	179.681	-	-	22.200	3988.918
	Uplift press	-1387.077	-	13.350	-18517.100	-	-
	TOTAL	2877.113	2362.282		34029.139		45770.994

	Load Items	Vertical Force(t/m)	Horizontal Force(t/m)	X(m)	Mx (tm/m)	Y (m)	My (tm/m)
Normal for Empty	Dam weight	3711.108	-	12.477	46304.437	-	-
	Downstream water press	99.856	-124.820	42.187	4212.918	5.267	-657.427
	Downstream earth press	99.856	-62.410	42.187	4212.918	5.267	-328.713
	TOTAL	3910.820	-187.230		54730.273		- 986.140
Earthquake for Empty	Dam weight	3711.108	-185.555	12.477	46304.437	19.688	3653.207
	Downstream water press	99.856	-124.820	42.187	4212.918	5.267	-657.427
	Downstream earth press	99.856	-62.410	42.187	4212.918	5.267	-328.713
	TOTAL	3910.820	- 372.785		54730.273		- 4639.347

(ii) Result of stability calculation

Stability against overturning at Non-Overflow Section

		M (tm/m)	V (t/m)	X (m)	a+X (m)	e (m)	B/6
HWL	Normal	67868.412	2873.687	23.617	32.317	4.767	< 9.183
	Earthquake	79800.133	2877.113	27.736	36.436	8.886	< 9.183
Empty	Normal	53744.133	3910.820	13.742	22.442	5.108	< 9.183
	Earthquake	50090.926	3910.820	12.808	21.508	6.042	< 9.183

Note, M : Total of moments
V : Total of vertical forces
X : The distance between the dam axis and acting point of the resultant force (=M/V)
a : The distance between the dam axis and upstream end of the dam

e : The distance between the center of base and acting point of the resultant force [=B/2 - (a+x)]
B : The length of the section

From the above calculations, the acting point of the resultant force is within the middle third so stability against overturning is assured.

Stability against compressive stress at Non-Overflow Section

		V (tm/m)	V/B (t/m²)	e (m)	6.e/B	Pu(t/m²)	Pd(t/m²)
HWL	Normal	2873.687	52.154	4.767	0.519	25.086	79.222
	Earthquake	2877.113	52.216	8.886	0.968	1.671	102.761
Empty	Normal	3910.820	70.977	5.108	0.556	110.440	31.514
	Earthquake	3910.820	70.977	6.042	0.658	117.680	24.274

The largest compressive stress is generated at the downstream end in the high water level state. As the slope of downstream is 1:0.8, the major principal stress (or inclined stress) at a plane at right angles to the downstream face is obtained from the following equation.

$$\sigma = Pd\ (1+n^2) = 102.761\ x\ (1+0.8^2) = 168.528\ t/m^2$$

The foundation bearing stress and the compressive stress in the concrete are within allowable limits for the above values.

Stability against sliding at Non-overflow Section

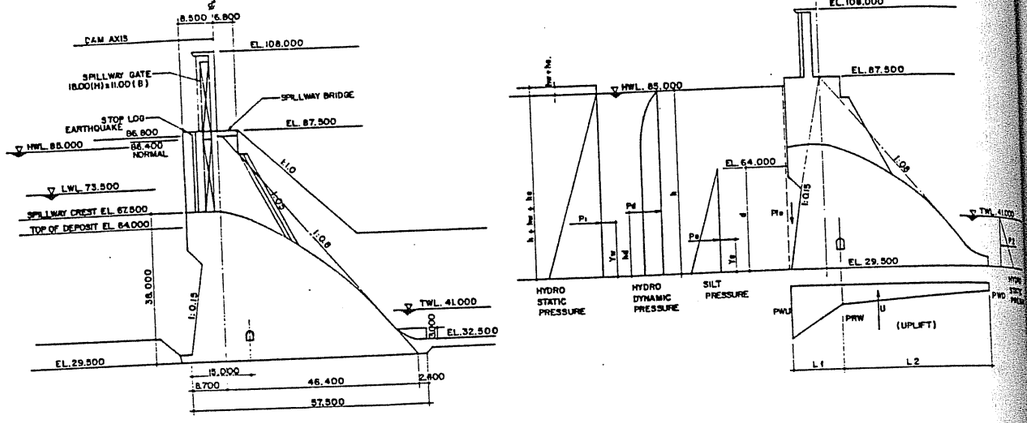
		f.V(t/m)	τ.B (t/m)	H (t/m)	n
HWL	Normal	2241.476	9918.000	1788.650	6.80 > 4
	Earthquake	2244.148	9918.000	2362.282	5.15 > 4
Empty	Normal	3050.44	9918.00	187.23	69.26 > 4
	Earthquake	3050.44	9918.00	372.79	34.79 > 4

Note, f : Coefficient of internal friction (=0.78)
τ : Shearing strength (=180 t/m²)
B : Length of the section
n : Safety factor
H : External horizontal force

The factor of safety n > 4.0 above for all states so the stability against sliding is assured both in the high water level state and the empty state.

(b) Overflow section

(i) Total of external force and dead weight



	Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Mx (tm/m)	Y (m)	My (tm/m)
Normal for HWL	Dam weight	3055.607	-	13.093	40008.049	-	-
	Pier	294.614	-	1.779	524.118	-	-
	Bridge	3.415	-	-2.500	8.538	-	-
	Hoist Deck	11.887	-	2.825	-33.581	-	-
	Gate	8.000	-	-3.565	-28.520	-	-
	Hoist	1.700	-	-3.565	-6.061	-	-
	Upstream water press	153.872	1618.805	-6.597	-1015.097	-	-
	Downstream water press	69.700	-66.125	44.465	3099.204	18.967	30703.874
	Silt press	67.144	357.075	-7.462	501.053	3.833	-253.457
	Uplift press	1244.250	-	12.699	-15800.730	11.500	4106.363
	Total	2421.689	1909.755		26254.867		34556.78
Earthquake for HWL	Dam weight	3055.607	305.561	13.093	40008.049	14.646	4475.246
	Pier	294.614	29.461	1.779	524.118	49.697	1464.123
	Bridge	3.415	0.342	2.500	8.538	57.689	19.730
	Hoist Deck	11.887	1.189	-2.825	-33.581	77.950	92.683
	Gate	8.000	0.800	-3.565	-28.520	47.000	37.600
	Hoist	1.700	0.170	-3.565	-6.061	78.500	13.345
	Upstream water press	155.846	1641.645	-6.590	-1027.006	19.100	31355.420
	Downstream water press	69.700	-66.125	44.465	3099.204	3.833	-253.457
	Silt press	67.144	357.075	-7.462	-501.053	11.500	4106.363
	Dynamic water press	-	179.681	-	-	22.200	3988.918
	Uplift press	-1244.250	-	12.699	-15800.730	-	-
	Total	2423.663	2449.799		26242.958		45299.971

	Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Mx (tm/m)	Y (m)	My(tm/m)
Normal for Empty	Dam own weight	3055.607	-	13.093	40008.049	-	-
	Pier	294.614	-	1.779	524.118	-	-
	Bridge	3.415	-	2.500	8.538	-	-
	Hoist Deck	11.887	-	-2.825	-33.581	-	-
	Gate	8.000	-	-3.565	-28.520	-	-
	Hoist	1.700	-	-3.565	-6.061	-	-
	Total	3375.223	0		40472.543		0
Earthquake for empty	Dam own weight	3055.607	-152.780	13.093	40008.049	14.646	-2237.616
	Pier	294.614	-14.731	1.779	524.118	49.697	-732.087
	Bridge	3.415	-0.171	2.500	8.538	57.689	-9.865
	Hoist Deck	11.887	-0.594	-2.825	-33.581	77.950	-46.302
	Gate	8.000	-0.400	-3.565	-28.520	47.000	-18.800
	Hoist	1.700	-0.085	-3.565	-6.061	78.500	-6.673
	Total	3375.223	- 168.761		40472.543		-3051.343

(ii) Result of stability calculation

Stability against overturning at Overflow Section

		M (tm/m)	V (tm/m)	x (m)	a+x (m)	e (m)	B/6 (m)
HWL	Normal	60811.647	2421.689	25.111	33.811	5.061	< 9.583
	Earthquake	71542.929	2423.663	29.519	38.219	9.469	< 9.583
Empty	Normal	40472.543	3375.223	11.991	20.691	8.059	< 9.583
	Earthquake	37421.200	3375.223	11.870	19.787	8.963	< 9.58

From the above calculations, the acting point of resultant force is within the middle third so stability against overturning is assured.

Stability against compressive stress at Over-flow Section

		V (tm/m)	V/B(t/m²)	e (m)	6.e/B	Pu(t/m²)	Pd(t/m²)
HWL	Normal	2421.689	42.116	5.061	0.528	19.879	64.353
	Earthquake	2423.663	42.151	9.469	0.988	0.506	83.796
Empty	Normal	3375.223	58.700	8.059	0.841	108.067	9.333
	Earthquake	3375.223	58.700	8.963	0.935	113.585	3.816

The largest compressive stress is generated at the downstream end in the high water level state. As the slope of downstream is 1:0.8, the major principal stress (or inclined stress) at a plane at right angles to the downstream face is obtained from the following equation.

$$\sigma = Pd\ (1+n^2) = 83.796\ x\ (1+0.8^2) = 137.425\ t/m^2$$

The foundation bearing stress and the compressive stress in the concrete are within allowable limits for the above values.

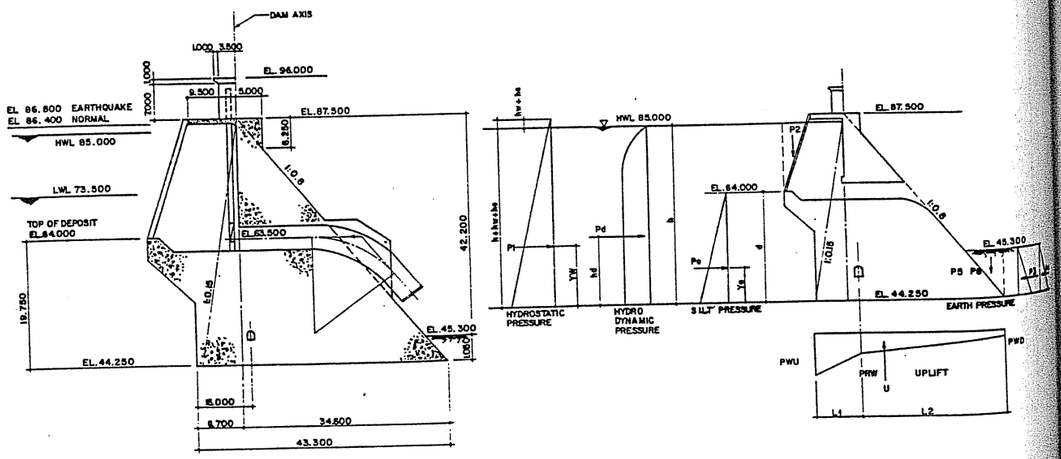
Stability against sliding at Over-flow Section

		f.V(t/m)	τ.B (t/m)	H (t/m)	n
HWL	Normal	1888.917	10350.000	1909.755	6.41 > 4
	Earthquake	1890.457	10350.000	2449.799	5.00 > 4
Empty	Normal	2632.674	10350.000	0	-----
	Earthquake	2632.674	10350.000	168.761	76.93 > 4

The factor of safety, n > 4.0 above for all states so the stability against sliding is assured both in the high water level state and the empty state.

(c) Intake section

(i) Total of external force and dead load



	Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Mx (tm/m)	Y (m)	My (tm/m)
Normal for HWL	Dam weight	2294.200	-	7.927	18186.123	-	-
	Control structure	8.933	-	-1.960	-17.509	-	-
	Trashrake	1.750	-	-9.500	-16.625	-	-
	Intake Gate	2.000	-	-1.600	-3.200	-	-
	Hoisting motor	1.000	-	-1.600	-1.600	-	-
	Upstream water press.	277.261	888.311	-5.189	-1438.707	14.050	12480.770
	Soil press	14.043	117.019	-5.500	-77.237	6.583	770.336
	Downstream earth press.	0.441	-0.276	34.320	15.135	0.350	-0.097
	Downstream water press.	0.441	-0.551	34.320	15.135	0.350	- 0. 193
	Centrifugal force	-28.583	13.728	23.683	-676.931	19.250	264.264
	Frictional force	0.057	0.244	23.683	1.350	19.250	4.697
	Uplift press	-515.116	-	2.586	-1332.130	-	-
	Total	2056.427	1018.475		14653.804		13519.776
Earthquake for HWL	Dam own weight	2294.200	229.420	7.927	18186.123	15.392	3531.233
	Control structure	8.933	0.893	-1.960	-17.509	49.308	44.032
	Trashrake	1.750	0.175	-9.500	-16.625	43.250	7.569
	Intake Gate	2.000	0.200	-1.600	-3.200	47.950	9.590
	Hoisting motor	1.000	0.100	-1.600	-1.600	51.750	5.175
	Upstream water pressure.	280.252	905.251	-5.231	-1465.998	14.183	12839.175
	Soil pressure	14.043	117.019	-5.500	-77.237	6.583	770.336
	Downstream earth pressure.	0.441	-0.276	34.320	15.135	0.350	-0.097
	Downstream water pressure.	0.441	-0.551	34.320	15.135	0.350	-0.193
	Dynamic water pressure.	-	105.613	-	-	17.020	1797.533
	Centrifugal force	-28.583	13.728	23.683	-674.931	19.250	264.264
	Frictional force	0.057	0.244	23.683	1.350	19.250	4.697
	Uplift pressure	-515.116	-	2.586	-1332.130	-	-
	Total	2059.418	1371.816		14628.513		19273.314

	Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Mx (tm/m)	Y (m)	My (tm/m)
Normal for Empty	Dam own weight	2294.200	-	7.927	18186.123	-	-
	Control structure	8.933	-	-1.960	-17.509	-	-
	Trashrake	1.750	-	-9.500	-16.625	-	-
	Intake Gate	2.000	-	-1.600	-3.200	-	-
	Hoisting motor	1.000	-	-1.600	-1.600		-
	Downstream earth press	0.441	-0.276	34.320	15.135	0.350	-0.097
	Downstream water press	0.441	-0.551	34.320	15.135	0.350	-0.193
	Total	2308.765	- 0.827		18177.459		- 0.290
Earthquake for Empty	Dam own weight	2294.200	-114.710	7.927	18186.123	15.392	-1765.616
	Control structure	8.933	-0.447	-1.960	-17.509	49.308	-22.041
	Trashrake	1.750	-0.088	-9.500	-16.625	43.250	-3.806
	Intake Gate	2.000	-0.100	-1.600	-3.200	47.950	-4.795
	Hoisting motor	1.000	-0.050	-1.600	1.600	51.750	-2.588
	Downstream earth pressure	0.441	-0.276	34.320	15.135	0.350	-0.097
	Downstream water press	0.441	-0.551	34.320	15.135	0.350	-0.193
	Total	2308.765	- 116.222		18177.459		-1799.136

(ii) Result of stability calculation

Stability for overturning at Intake section

		M (tm/m)	V (tm/m)	x (m)	a+x (m)	e (m)	B/6 (m)
HWL	Normal	28173.580	2056.427	13.700	22.400	0.750	< 7.217
	Earthquake	33901.827	2059.418	16.462	25.162	3.512	< 7.217
Empty	Normal	18177.169	2308.765	7.873	16.573	5.077	< 7.217
	Earthquake	16378.323	2308.765	7.094	15.794	5.856	< 7.217

From the above calculations, the acting point of resultant force is within the middle third so stability against overturning is assured.

Stability against compressive stress at Intake Section

		V (tm/m)	V/B (t/m^2)	e (m)	6.e/B	Pu(t/m^2)	Pd(t/m^2)
HWL	Normal	2056.427	47.493	0.750	0.104	42.554	52.432
	Earthquake	2059.418	47.562	3.512	0.487	24.399	70.725
Empty	Normal	2308.765	53.320	5.077	0.704	90.857	15.783
	Earthquake	2308.765	53.320	5.856	0.811	96.563	10.077

The largest compressive stress is generated at the downstream end in the high water level state. As the slope of downstream is 1:0.8, the major principal stress (or inclined stress) at a plane at right angles to the downstream face is obtained from the following equation.

$$\sigma = Pd\ (1+n^2) = 70.725 \times (1+0.8^2) = 115.989\ t/m^2$$

The foundation bearing stress and the compressive stress in the concrete are within allowable limits for the above value.

Stability against sliding at Intake Section

		f.V(t/m)	τ.B(t/m)	H(t/m)	n
HWL	Normal	1604.013	7794.0	1018.475	9.22 > 4
	Earthquake	1606.346	7794.0	1371.816	6.85 > 4
Empty	Normal	1800.837	7794.0	0.827	11,601.97 > 4
	Earthquake	1800.837	7794.0	116.222	82.55 > 4

The factor of safety, n > 4 above for all states so the stability against sliding is assured both in the high water level state and the empty state.

(5) Stability Analysis at Construction Stage

(a) Modified item on design condition

Modification on design condition for dam stability analysis is as follows:

(i) Shearing strength of dam foundation rock

From the results of the geological investigation for the dam foundation rock after completion of excavation, the design value of shearing strength of rock was modified as shown in the following table. The values were obtained for each dam concrete block, block-by-block in consideration of the actual ratio of rock grade distribution on the surface of dam foundation.

Shearing Strength and Ratio of Rock Grade

BL. No.	Shearing Strength (t/m^2)	Ratio of Rock Grade (%)			
		CH	CM	CL	D or Clay
1	112	0	38	62	0
2	166	0	87	13	0
3	166	9	75	14	2
4	205	61	30	9	0
5	163	33	36	29	2
6	160	31	41	18	10
7	158	14	62	19	5
8	157	24	47	22	7
9	199	60	27	11	2
10	193	58	23	19	0
11	171	18	67	12	3
12	168	6	80	13	1
13	187	47	36	15	2
14	214	68	26	6	0
15	192	55	27	16	2
16	125	0	50	50	0

Note; CM include CH (n CM), CM (n CH)
CL include CM (n CL) , CM-CL, CL (n CM)

(ii) Dimension of concrete and steel structures

On the basis of the detailed design of steel structures by the Lot II Contractor, some superstructures required re-design and their dimensions were modified accordingly. Modified items and their reasons are shown in the following table.

Modified Item	Back Ground
Overflow section • Location and size of grooves of spillway gate and stoplog • Weight of gate, stoplog and hoist motor • Size of openings in the spillway hoist deck • Crest width of spillway section and design load for spillway bridge	 After final design of spillway gate and stoplog ditto After final weight of spillway hoist motor After final methods for installation of spillway gate and stoplog
Intake section • Height of dam block (elevation of dam block foundation) • Weight of gate, stoplog, trashrack, trashrake, hoist motor and thickness of penstock	 Additional excavation due to removal for weathered rock. After design of each structure.

(b) Non-overflow section

(i) Total of external force and dead load

The total of external forces and dead loads were the same as those determinated at the design stage. Therefore, the checking and studying for stability of the non-overflow section was carried out only against sliding.

(ii) Result of stability calculation

Stability against sliding at Non-Overflow Section

		f.V(t/m)	τ.B(t/m)	H(t/m)	n
HWL	Normal	2241.476	8650.7	1788.650	6.08 > 4
	Earthquake	2244.148	8650.7	2362.282	4.61 > 4
Empty	Normal	3050.440	8650.7	187.230	62.49 > 4
	Earthquake	3050.440	8650.7	372.785	31.38 > 4

Note ; Shearing strength : 157 t/m^3 (BL-8)

The factor of safety, n > 4.0 above for all states so the stability against sliding is assured both in the high water level state and the empty state.

(c) Overflow Section

(i) Total of external force and dead weight

	Load Items	Vertical force (t/m)	horizontal force (t/m)	X (m)	Mx (tm/m)	Y (m)	My(tm/m)
Normal for HWL	Dam own weight	3055.607	-	13.093	40008.049	-	-
	Pier	295.776	-	1.819	538.017	-	-
	Bridge	9.054	-	3.150	28.520	-	-
	Hoist Deck	11.508	-	-2.825	-32.510	-	-
	Gate	6.147	-	-3.565	-21.914	-	-
	Hoist	1.007	-	-3.565	-3.590	-	-
	Upstream water press	153.872	1618.805	-6.597	-1015.097	18.967	30703.874
	Downstream water press	69.700	-66.125	44.465	3099.204	3.833	-253.457
	Silt press	67.144	357.075	-7.462	-501.053	11.500	4106.363
	Uplift press	-1244.250	-	12.699	-15800.730	-	-
	Total	2425.565	1909.755		26298.896		34556.780
Earthquake for HWL	Dam own weight	3055.607	305.561	13.093	40008.049	14.646	4475.246
	Pier	295.776	29.578	1.819	538.017	49.920	1476.533
	Bridge	9.054	0.905	3.150	28.520	57.550	52.083
	Hoist Deck	11.508	1.151	-2.825	-32.510	77.950	89.720
	Gate	6.147	0.615	-3.565	-21.914	47.000	28.905
	Hoist	1.007	0.101	-3.565	-3.590	78.500	7.929
	Upstream water press	155.846	1641.645	-6.590	-1027.006	19.100	31355.420
	Downstream water press	69.700	-66.125	44.465	3099.204	3.833	-253.457
	Silt press	67.144	357.015	-7.462	-501.053	11.500	4106.363
	Dynamic water press	-	179.681	-	-	22.200	3988.918
	Uplift press	-1244.250	-	12.699	-15800.730	-	-
	Total	2427.539	2450.127		26286.987		45327.660

	Load Items	Vertical force (t/m)	horizontal force(t/m)	X (m)	Mx (tm/m)	Y (m)	My(tm/m)
Normal for Empty	Dam own weight	3055.607	-	13.093	40008.049	-	-
	Pier	295.776	-	1.819	538.017	-	-
	Bridge	9.054	-	3.150	28.520	-	-
	Hoist Deck	11.508	-	-2.825	-32.510	-	-
	Gate	6.147	-	-3.565	-21.914	-	-
	Hoist	1.007	-	-3.565	-3.590	-	-
	Total	3379.099	0		40516.572		0
Earthquake for Empty	Dam own weight	3055.607	-152.780	13.093	40008.049	14.646	-2237.616
	Pier	295.776	-14.789	1.819	538.017	49.920	-738.267
	Bridge	9.054	-0.453	3.150	28.520	57.550	-26.070
	Hoist Deck	11.508	-0.575	-2.825	-32.510	77.950	-44.821
	Gate	6.147	-0.307	-3.565	-21.914	47.000	-14.429
	Hoist	1.007	-0.050	-3.565	-3.590	78.500	-3.925
	Total	3379.099	168.954		40516.572		3065.128

(ii) Result of stability calculation

Stability against overturning at Overflow Section

		M (tm/m)	V (tm/m)	x (m)	a+x (m)	e (m)	B/6 (m)
HWL	Normal	60855.676	2425.565	25.089	33.789	5.039	< 9.583
	Earthquake	71614.647	2427.539	29.501	38.201	9.451	< 9.583
Empty	Normal	40516.572	3379.099	11.990	20.690	8.060	< 9.583
	Earthquake	37451.444	3379.099	11.083	19.783	8.967	< 9.583

From the above calculations, the acting point of the resultant force is within the middle third so stability against overturning is assured.

Stability against compressive stress at Overflow Section

		M (tm/m)	VB(t/m²)	e (m)	6.e/B	Pu(t/m²)	Pd(t/m²)
HWL	Normal	2425.565	42.184	5.039	0.526	19.995	64.373
	Earthquake	2427.539	42.218	9.451	0.986	0.591	83.849
Empty	Normal	3379.099	58.767	8.060	0.841	108.190	9.344
	Earthquake	3379.099	58.767	8.967	0.936	113.773	3.761

The largest compressive stress is generated at the downstream end in the high water level state. As the slope of downstream is 1 : 0.8, the major principal stress (or inclined stress) at a plane at right angles to the downstream face is obtained from the following equation.

$$\sigma = Pd\,(1+n^2) = 83.849 \times (1+0.8^2) = 137.512\ t/m^2$$

The foundation bearing stress and the compressive stress in the concrete are within allowable limits for the above values.

Stability against sliding at Overflow Section

		f.V(t/m)	τ.B (t/m)	H (t/m)	n
HWL	Normal	1,891.941	9,660.000	1,909.755	6.04 > 4
	Earthquake	1,893.480	9,660.000	2,450.127	4.71 > 4
Empty	Normal	2,635.697	9,660.000	0	----
	Earthquake	2,635.697	9,660.000	168.954	72.77 > 4

Note ; f : Coefficient of internal friction = 0.78
τ : hear strength = 168 t/m² (BL-12)

The factor of safety, n > 4 above for all states so the stability against sliding is assured both in the high water level state and the empty state.

(d) Intake section

(i) Total of external force and dead load

	Load Items	Vertical force (t/m)	Horizontal force(t/m)	X (m)	Mx (tm/m)	Y (m)	My(tm/m)
Normal for HWL	Dam weight	2366.457	-	7.827	18522.259	-	-
	Control structure	8.933	-	-1.960	-17.509	-	-
	Trashrake	1.025	-	-9.500	-9.738	-	-
	Intake Gate	2.035	-	-1.942	-3.951	-	-
	Hoisting motor	0.665	-	-1.600	-1.064	-	-
	Upstream water press	271.532	906.102	-5.214	-1415.768	14.190	12857.594
	Soil press	12.815	122.049	-5.542	-71.026	6.723	820.533
	Downstream earth press.	0.864	-0.540	34.544	29.846	0.490	-0.265
	Downstream water press.	0.864	-1.080	34.544	29.846	0.490	-0.529
	Centrifugal force	-28.583	13.728	23.683	-676.931	19.670	270.030
	Frictional force	0.057	0.244	23.683	1.350	19.670	4.799
	Uplift press	-535.130	-	3.025	-1618.534	-	-
	Total	2101.534	1040.503		14768.780		13952.162
Earthquake for HWL	Dam weight	2366.457	236.646	7.827	18522.259	15.651	3703.747
	Control structure	8.933	0.893	-1.960	-17.509	49.308	44.032
	Trashrake	1.025	0.103	-9.500	-9.738	43.670	4.498
	Intake Gate	2.035	0.204	-1.942	-3.951	41.859	8.539
	Hoisting motor	0.665	0.067	-1.600	-1.064	52.170	3.495
	Upstream water press	274.405	923.210	-5.257	-1442.547	14.323	13223.143
	Soil press	12.815	122.049	-5.542	-71.026	6.723	820.533
	Downstream earth press.	0.864	-0.540	34.544	29.846	0.490	-0.265
	Downstream water press.	0.864	-1.080	34.544	29.846	0.490	-0.529
	Dynamic water press	-	98.873	-	-	16.468	1628.241
	Centrifugal force	-28.583	13.728	23.683	-676.931	19.670	270.030
	Frictional force	0.057	0.244	23.683	1.350	19.670	4.799
	Uplift press	-535.130	-	3.025	-1618.534	-	-
	Total	2104.407	1394.397		14742.001		19710.263

	Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Mx (tm/m)	Y (m)	My(tm/m)
Normal for Empty	Dam own weight	2366.457	-	7.827	18522.259	-	-
	Control structure	8.933	-	-1.960	-17.509	-	-
	Trashrake	1.025	-	-9.500	-9.738	-	-
	Intake Gate	2.035	-	-1.942	-3.951	-	-
	Hoisting motor	0.665	-	-1.600	-1.064	-	-
	Downstream earth pressure.	0.864	-0.540	34.544	29.846	0.490	-0.265
	Downstream water pressure	864.000	-1.080	34.544	29.846	0.490	-0.529
	Total	2380.843	- 1.620		18549.689		- 0.794
Earthquake for Empty	Dam own weight	2366.457	-118.323	7.827	18522.259	15.651	-1851.873
	Control structure	8.933	-0.447	-1.960	-17.509	49.308	-22.041
	Trashrake	1.025	-0.051	-9.500	-9.738	43.670	-2.227
	Intake Gate	2.035	-0.102	-1.942	-3.951	41.859	-4.270
	Hoisting motor	0.665	-0.033	-1.600	-1.064	52.170	-1.722
	Downstream earth pressure.	0.864	-0.540	34.544	29.846	0.490	-0.265
	Downstream water pressure.	0.864	-1.080	34.544	29.846	0.490	-0.529
	Total	2380.843	- 120.576		18549.689		- 1882.927

(ii) Result of stability calculation

Stability for overturning at Intake Section

		M (tm/m)	V (tm/m)	x (m)	a+x (m)	e (m)	B/6 (m)
HWL	Normal	28720.942	2101.534	13.667	22.367	0.549	< 7.273
	Earthquake	34452.264	2104.407	16.371	25.071	3.253	< 7.273
Empty	Normal	18548.895	2380.843	7.791	16.491	5.327	< 7.273
	Earthquake	16666.762	2380.843	7.000	15.700	6.118	< 7.273

From the above calculations, the acting point by resultant force is within the middle third so against overturning is assured.

Stability against compressive stress at Intake Section

		V (tm/m)	VB(t/m²)	e (m)	6.e/B	Pu(t/m²)	Pd(t/m²)
HWL	Normal	2101.534	48.161	0.549	0.075	44.549	51.773
	Earthquake	2104.407	48.226	3.253	0.447	26.669	69.783
Empty	Normal	2380.843	54.561	5.327	0.732	94.500	14.622
	Earthquake	2380.843	54.561	6.118	0.841	100.447	8.675

The largest compressive stress is generated at the downstream end in the high water level state. As the slope of downstream is 1:0:8, the major principal stress (or inclined stress) at a plane at right angles to the downstream face is obtained from the following equation.

$$\sigma = Pd\,(1+n^2) = 69.783 \times (1+0.8^2) = 114.444\ t/m^2$$

The foundation bearing stress and the compressive stress in the concrete are within allowable limits for the above values.

Stability against sliding at Intake Section

		f.V(t/m)	τ.B (t/m)	H (t/m)	n
HWL	Normal	1639.197	7112.668	1040.503	8.41 > 4
	Earthquake	1641.437	7112.668	1394.397	6.27 > 4
Empty	Normal	1857.066	7112.668	1.620	5.53 > 4
	Earthquake	1857.066	7112.668	120.576	74.39 > 4

Note ; f : Coefficient of internal friction = 0.78
τ : shear strength = 163 t/m² (BL-5)

The factor of safety, n>4 above for all states so the stability against sliding is assured both in the high water level state and the empty state.

(e) Other dam sections

From the result of the stability calculations for the three design sections mentioned above, both the state of high water level and earthquake showed the minimum safety factor. The stability of each dam block was assured at construction stage, as shown in the following table.

Result of Stability Calculation at Construction Stage

BL. No	Section (m)	Compressive Stress (t/m²)		Major Principal Stress (t/m²)	Safety Factor against sliding
		Upstream end	Downstream end		
1	J0+6.5	8.109	20.195	33.120	21.01
2	J1+7.5	10.025	39.975	65.559	12.22
3	J2 +7.5	12.147	61.919	101.547	7.89
4	J3 + 7.5	14.001	68.845	112.906	8.45
5	J4 + 10.0	26.669	69.783	114.444	6.27
6	J5 + 10.0	23.268	91.351	149.816	5.50
7	J6 + 10.0	18.313	93.015	152.545	4.73
8	J7 + 9.0	1.671	102.761	168.528	4.61
9	J8 + 7.5	0.591	83.849	137.512	5.44
10	J9 + 7.5	0.591	83.849	137.512	5.30
11	J10 + 7.5	0.591	83.849	137.512	4.78
12	J11 + 7.5	0.591	83.849	137.512	4.71
13	J12 + 7.5	0.591	83.849	137.512	5.16
14	J13 + 7.5	6.389	98.359	161.308	6.18
15	J14 + 7.5	13.863	67.685	111.003	8.13
16	J15 + 8.25	7.538	42.384	69.510	9.53

(6) Stability Analysis after Initial Impounding

The distribution of actual uplift pressure acting on each dam block was found to be larger than the design value except at the over flow-sections. (refer to Part 2, chapter 9). Therefore, the stability analysis was carried out again based on this actual distribution of uplift pressure. This uplift pressure on the design water level (Normal : EL.86.400, earthquake EL.86.800) was estimated on the basis of measurement records during initial impounding because the reservoir water level did not reach HWL.85.0 m in the period of initial impounding.

Result of Stability Calculation after Initial Impounding

BL. No	Section (m)	Compressive Stress (t/m²)		Major Principal Stress (t/m²)	Safety Factor against sliding
		Upstream end	Downstream end		
1	J0+6.5	7.853	21.287	34.911	21.06
2	J1+7.5	12.058	39.143	64.195	12.25
3	J2 +7.5	11.158	61.064	100.145	7.86
4	J3 + 7.5	11.369	68.137	111.745	8.41
5	J4 + 10.0	34.714	66.200	108.568	6.33
6	J5 + 10.0	17.684	88.844	145.705	5.42
7	J6 + 10.0	17.936	92.782	152.162	4.72
8	J7 + 9.0	18.243	105.861	173.612	4.79
9	J8 + 7.5	8.001	88.395	144.968	5.52
10	J9 + 7.5	5.740	87.586	143.641	5.38
11	J10 + 7.5	3.191	86.705	142.196	4.92
12	J11 + 7.5	5.236	87.440	143.402	4.79
13	J12 + 7.5	16.416	81.592	133.811	5.28
14	J13 + 7.5	2.573	100.363	164.595	6.15
15	J14 + 7.5	8.660	66.318	108.762	8.03
16	J15 + 8.25	9.904	39.866	65.380	9.53

6.2.4 Stress Analysis of Dam Body

The stability of the dam against overturning, sliding and the stresses arising at the upstream and downstream ends of the dam section was assured in the stability calculation in Section 6.2.3. However, the assumptions of the calculation method are only substantially correct except for horizontal planes near the base of dam where the effect of foundation yielding are reflected in the stress distribution. At these locations, therefore, the stress changes occurring due to foundation yielding need to be analyzed by the finite element method.

Based on the geological investigation the stress changes were assumed to be small in this dam and foundation rock, but it is important to check the safety of a high dam like the Kotapanjang dam. This section estimates the tensile and compressive stresses arising in the dam body and foundation and also the safety against the shear force of the dam base.

(1) Condition of Analysis

The same dam sections in Section 6.2.3 "Stability Analysis" were selected, which are shown in Fig.1.6.2-1. The cases of analysis are (i) Full state of reservoir in normal condition and (ii) Empty state of reservoir in normal condition.

(a) Load Conditions

The load was applied by converting one concrete block into per-unit length at each calculation section. The conditions of the loads were described in detail in Section 6.2.2.(2) "Loads to be Taken into Consideration".

(b) Physical Constant

Respective physical constants are given as follows,

	Modulus of elasticity (kg/cm²)	Shear strength (kg/cm²)	Poisson`s ratio	Coefficient of internal friction
Concrete	230,000	23.0	0.17	0.8
Rock bed (CH)	60,000	24.0	0.20	1.0
Rock bed (CM)	30,000	18.0	0.25	0.78

The above material properties of CM class were based on the result of shear strength test of the rock and loading test which were carried out at dam site and the average value was taken in this analysis for CM class. However, the values of CH class were determined referring to "the relation between rock classification and modulus elasticity" and "the relation of rock classification and the results of shearing test" (source : Geology Investigation for Dam Foundation, classified by Kikuchi and Saito) since the rock test for CH class was not carried out. The average value was also taken for CH class based on the above date.

(2) Result of calculation

(a) Stress and Displacement

The distribution of principal stress and displacement of each design section are shown in Fig.1.6.2-2 to 1.6.2-3. Table 1.6.2-1 shows the maximum compressive and tensile stresses being produced in the foundation rock and dam body near the dam base for each section, the principal stresses in the foundation rock and the dam body and the maximum displacement.

(i) Stress in x-Direction and y-Direction

In the foundation rock, the maximum tensile stress in the x direction (horizontal direction) was 6.56 kg/cm^2 at just the upstream of the dam base in the overflow section, while for the same element the tensile stress in the y direction (vertical direction) was negligibly small at 0.03 kg/cm^2. The maximum tensile stress in the y direction was 1.19 kg/cm^2 at the upstream of the dam base in the overflow section, while the compressive stress was maximum 11.21 kg/cm^2 at the upstream end of the dam base in the intake section.

On the other hand, in the dam body, the maximum tensile stress in the x direction was 2.48 kg/cm^2 at the inlet part of the intake section, which is somewhat high value. At the other sections, the tensile stress was 1.0 kg/cm^2 or less so was not likely to cause any troubles.

For the compressive stress, the maximum stress was 20.03 kg/cm^2 at the downstream end of the dam body in the non-overflow section. The stress in the y direction showed all compression and was 18 kg/cm^2 the at maximum value. The values, accordingly, are lower than allowable stresses.

(ii) Principal Stress

In the foundation rock, the tensile stress σ_1 was maximum 6.865 kg/cm^2 at just upstream of the dam base in the overflow section, while for the compressive stress was maximum 3.575 kg/cm^2 at the upstream of the dam base in the non-overflow section. The tensile stress of σ_3 was maximum 0.309 kg/cm^2 at the upstream of the dam base in the overflow section, while for the compression stress was maximum 17.341 kg/cm^2 at just the downstream of the dam base in the non-overflow section. Therefore, σ_1 and σ_3 were not considered likely to cause any special trouble.

In the dam body, the tensile stress σ_1 was maximum 2.494 kg/cm^2 at the inlet part of the intake section, while for the compressive stress at full state, σ_3 showed all compression and it became 31.625 kg/cm^2 even at maximum value at the downstream end of the dam body in the non-overflow section. It was below the allowable stress and would not caused any special troubles.

(iii) Displacement

In the foundation rock the displacement in the horizontal direction is maximum 0.651 cm at the overflow section in the full state on the plus side, while the maximum 0.897 cm at the overflow section, at the empty state appeared on the minus side in the vertical direction. These values are less than 1 cm, and therefore considered negligible.

In the dam body, the displacement was maximum 1.475 cm at the overflow section, in the full state on the plus side in the horizontal direction. Since the overflow section was higher than the other sections, about 1.5 cm displacement was thought to have appeared. The displacement in both directions was about 1.0 cm more or less on other sections.

Table 1.6.2-1 Maximum Stress and Displacement

Unit : kg/cm^2, cm

			Non-over flow	Over-flow	Intake
Foundation rock	σx	tensile side	4.967	6.557	5.113
		compressive side	13.974	11.714	5.004
	σy	tensile side	0.957	1.185	0.803
		compressive side	11.161	10.748	11.209
	τxy	positive side	6.212	5.007	5.182
		negative side	4.935	4.517	3.208
Dam body	σx	tensile side	0.999	0.807	2.480
		compressed side	20.031	13.770	5.223
	σy	tensile side	not produced	not produced	17.523
		compressed side	17.812	17.797	17.523
	τxy	positive side	14.056	12.065	8.453
		negative side	6.365	6.409	4.445
Foundation rock	σ1	tensile side	5.012 (F)	6.865 (F)	5.262 (F)
		compressive side	3.575 (E)	3.455 (E)	3.182 (E)
	σ3	tensile side	0.278 (F)	0.309 (F)	0.201 (F)
		compressive side	17.341 (E)	13.673 (E)	11.624 (E)
Dam body	σ1	tensile side	1.014 (E)	1.185 (F)	2.494 (E)
		compressed side	6.590 (F)	6.377 (F)	4.961 (F)
	σ3	tensile side	not proceed	not produced	- 0.288 (F)
		compressed side	31.625 (F)	24.975 (F)	18.812 (F)
Foundation rock	δx	positive side	0.580 (F)	0.651 (F)	0.489 (F)
		negative side	0.068 (E)	0.078 (E)	0.033 (E)
	δy	positive side	0.023 (F)	0.029 (F)	0.650 (F)
		negative side	0.871 (E)	0.897 (E)	0.570 (E)
Dam body	δx	positive side	1.120 (E)	1.475 (F)	0.918 (F)
		negative side	0.562 (F)	0.878 (E)	0.377 (E)
	δy	positive side	not produced	not produced	- 0.133 (F)
		negative side	0.991 (E)	1.081 (F)	0.824 (E)

Note : (F) At full state, (E) At empty state

(b) Shear Failure

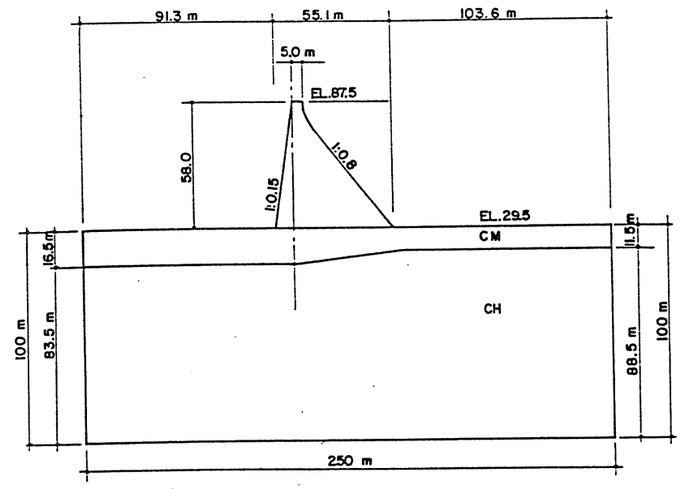
The local safety factor was found to be a minimum 2.466 near the downstream end of the contact surface between the dam body and foundation rock as shown in Table 1.6.2-2 and Fig. 1.6.2-4. The shear strength is generally expressed as the fracture strength. The rock bed has the yield point, and if load is applied repeatedly over the yield point, the rock bed will fracture. This singular point arised at about 60% of the fracture point. From this consideration, the point safety factor will be required at least 1.7 or more.

In consideration of the uncertainty of strength evaluation and uncertainty of the stress analysis, the local safety factor of 2 or above is recommended. Since the minimum safety factor in this analysis showed 2.46 which was larger than 2, shear failure will not occur in the Kotapanjang dam.

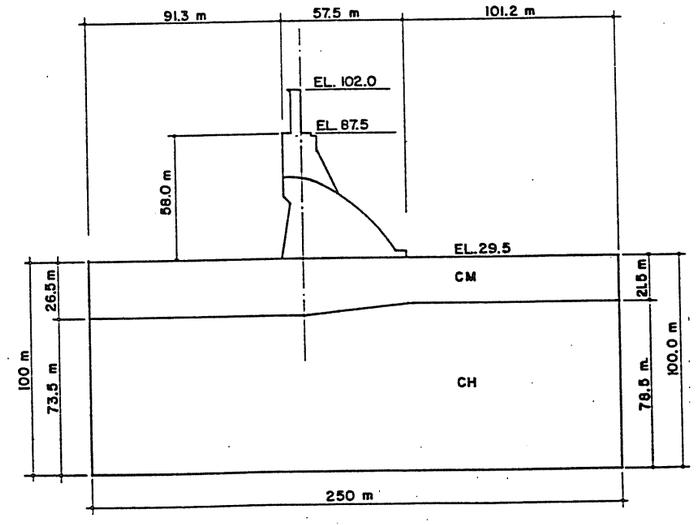
Table 1.6.2-2 Minimum Point of Safety Factor

Section No.	Analytical case	Analytical location	Minimum point safety factor
Non-overflow section	Full state	In foundation rock In dam body	4.731 5.852
	Empty state	In foundation rock In dam body	3.636 2.466
Overflow section	Full state	In foundation rock In dam body	4.365 3.719
	Empty state	In foundation rock In dam body	5.085 5.810
Intake section	Full state	In foundation rock In dam body	4.567 5.260
	Empty state	In foundation rock In dam body	8.328 6.506

Non-over flow section



Over flow section



Intake section

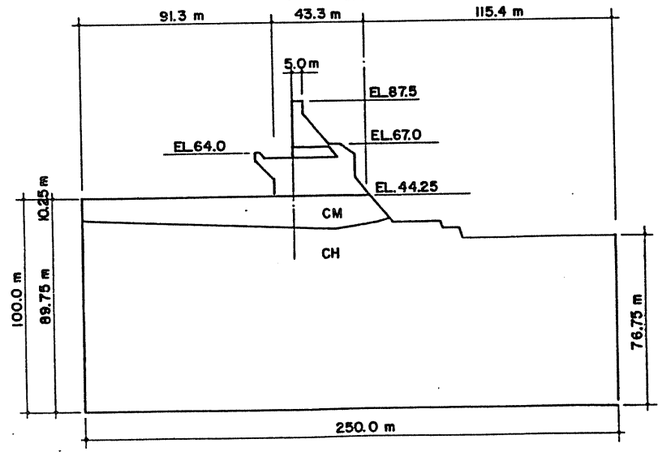
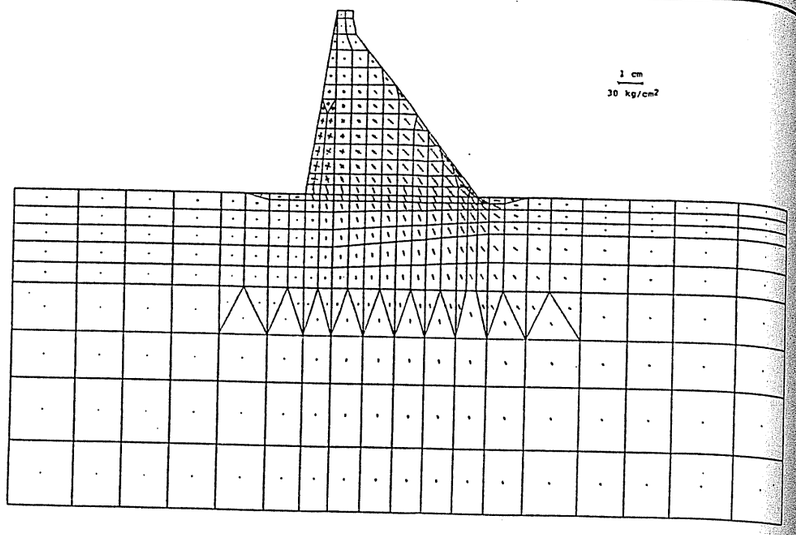
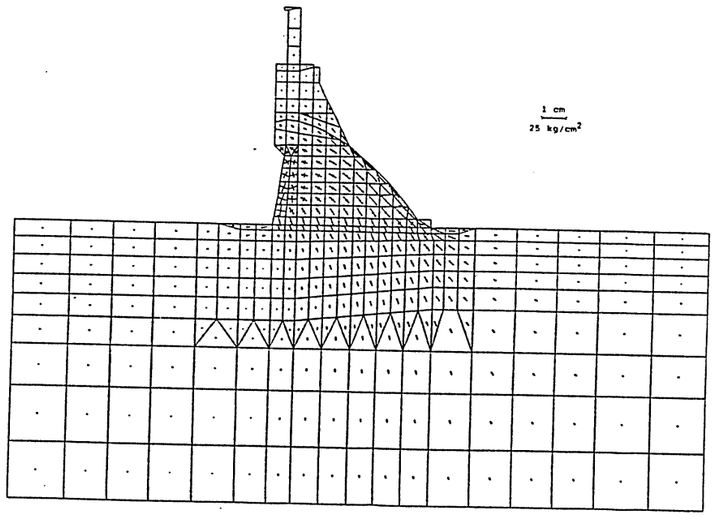


Fig. 1.6.2-1 Analysis Model

Non-over flow section



Overflow section



Intake section

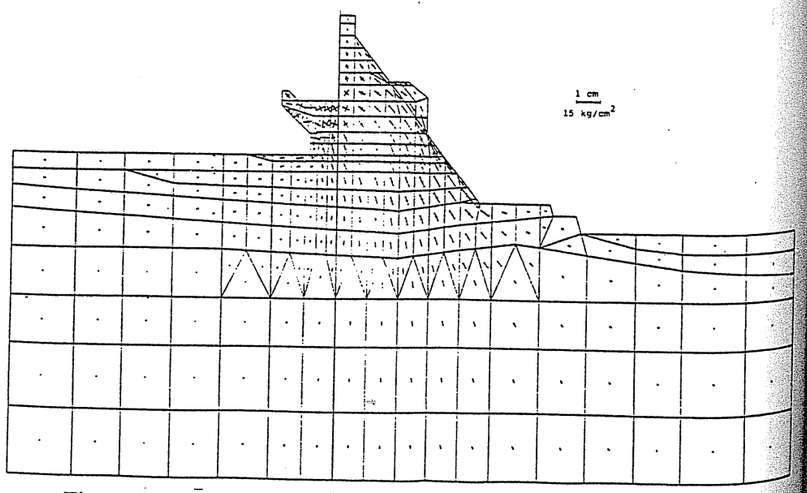
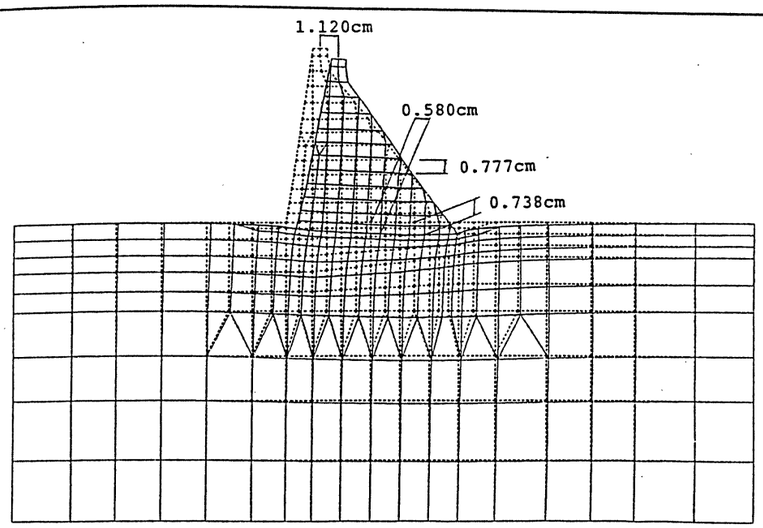
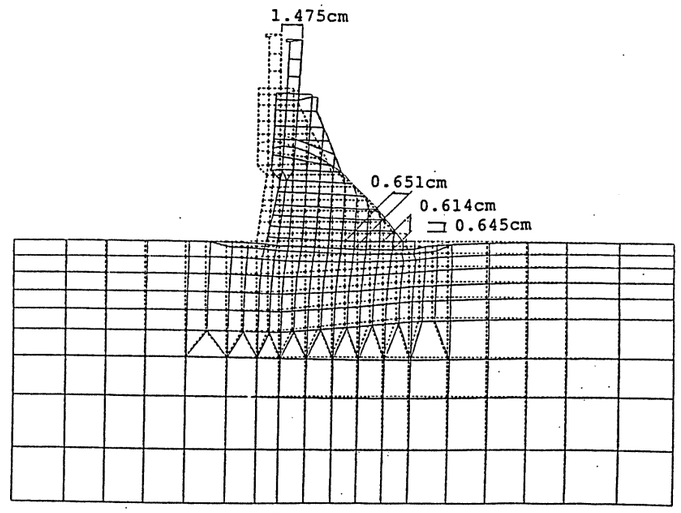


Fig. 1.6.2-2 Distribution of Principal Stress (Full State)

Non-Overflow section



Overflow section



Intake section

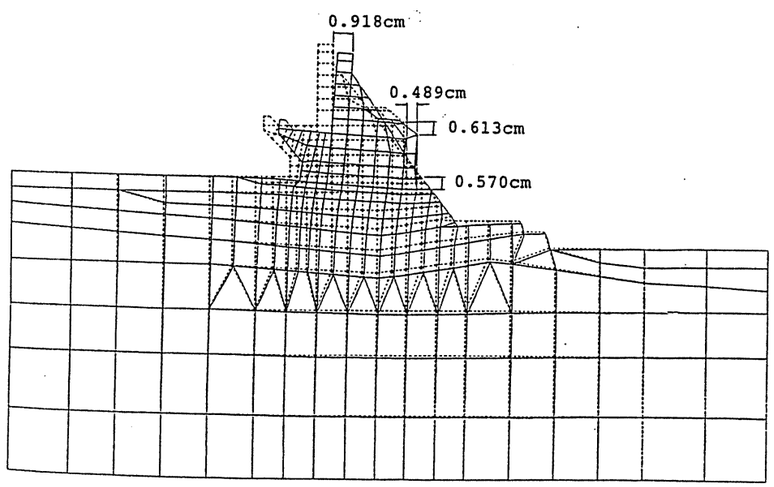


Fig. 1.6.2-3 Displacement (Full State)

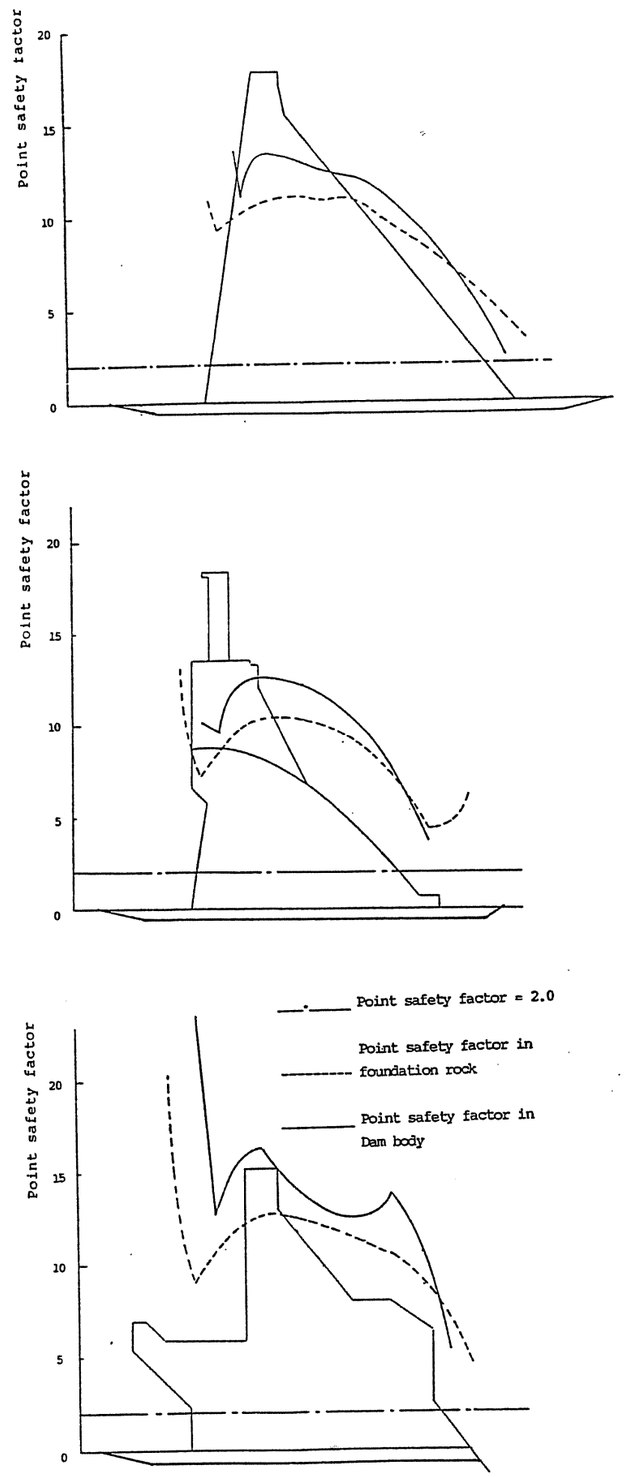


Fig. 1.6.2-4 Distribution of Local Safety Factor (Full State)

6.2.5 Dam Structural Details and Associated Civil Works

(1) Contraction Joints

During the process of hardening of concrete, the rise in temperature of concrete takes place by the heat of hydration. In massive concrete as in dams, the heat of hydration of cement does not radiate rapidly and therefore the process of setting produces temperatures in the interior of the dam which are well above those in the adjacent atmosphere. After concrete has hardened, it gradually cools down to the mean ambient temperature due to heat transfer.

In this process of falling temperature, the internal and external restraints prevent the contraction displacement of concrete due to the changing temperature and therefore tensile stresses generate in the concrete. When the tensile stresses develop and exceed the tensile strength of concrete, cracks occur in the concrete.

Vertical cracks transverse and parallel with the dam axis cause leakage from the reservoir, stress concentrations and discontinuities which prevent the structure from carrying loads as designed respectively.

Therefore, as one method in crack control for dam concrete, transverse joints and longitudinal joints are provided in concrete dams for the purpose of reducing random cracking transverse and parallel with dam axis respectively by confining crack opening to predetermined planes. From the result of thermal stress studies and past experience, longitudinal joints were not required in the Kotapanjang dam.

(a) Transverse Joints

(i) Spacing

The transverse joints are set normal to the dam axis from foundation to the top of dam and are continuous from the upstream face to the downstream face. The spacing of transverse joints is determined by features of the dam foundation, the detail of structure associated with the dam, the result of temperature studies and the probable capacity of the batching plant.

For the Kotapanjang dam, transverse joints were generally spaced 15.0m apart except the concrete block with penstock (20.0m) and both abutment dam blocks in consideration of past experiences of dams in Japan and the result of the structural calculations for the dam block with penstock.

(ii) Detail Design of Transverse Joint

- Keys : Vertical keys in transverse joints were generally set to transmit the loads, especially shearing forces at the joints.

- Waterstop : By reason of preventing leakage of water from the reservoir, waterstops were set across the transverse joints at both upstream side and downstream side. At the upstream side, two sheets of waterstop were set in order to get reliable water tightness under the condition of ungrouped joints. The material of the waterstop used in the concrete dam was a PVC seal 40 cm wide as shown in Fig. 1.6.2-5.

(iii) Drainage Holes at Joints

Drainage holes were set behind the waterstops for the purpose of monitoring watertightness of waterstops and draining the leakage of water through the waterstops.

(iv) Arrangement of Waterstops and Drainage Holes

Waterstops were set more than 1m distant from the upstream face of the dam in order to get reliable water tightness. The arrangement of waterstops and drainage hole at Kotapanjang are shown below.

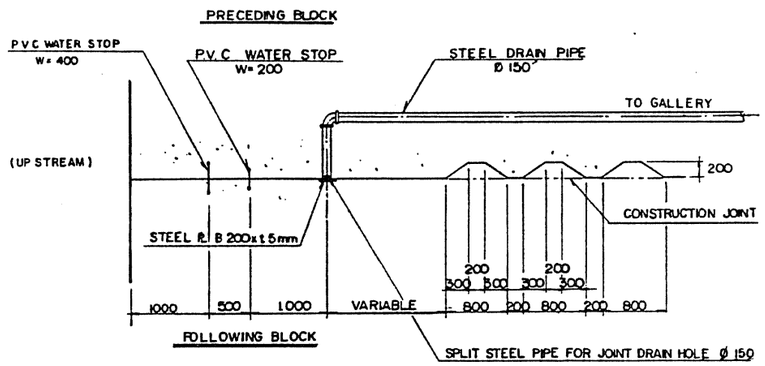


Fig. 1.6.2-5 Detail Design of Vertical Joint (Plan)

(2) Crest Details

Structures on the dam crest mainly consist of the crest road, spillway bridges, handrails, drainage system and lighting facility.

(a) Crest Road

The crest road was used for the installation of the spillway and intake gates and hoists during the construction period and will be used for future inspection, maintenance and repair of the dam. The crest width was determined to suit the various sizes of machines used for the above works, especially the size of mobile crane for installation of gates, stoplogs of spillway and intake.

The original crest width of 5m was determined at the design stage in consideration of the installation method of gates, using a 20 ton mobile crane, which was selected by previous experience. The crest width of the spillway section, however, was required to be widened 1.3m and became 6.3m in total width because a 80 ton mobile crane may be required for lowering stoplogs to allow repair of the spillway gate if some gate damage was found there during future operations.

Asphalt 5cm thickness was used as the surface treatment on the crest road for reason of maintenance and repair.

(b) Handrail

A handrail was installed on both sides of the crest road to insure vehicle and pedestrian safety. The handrail height of 1.2m consists of concrete parapet 60 cm high and a steel pipe with 60 cm high was selected based on experience in Japan. The design was made against horizontal force 250 kg/m which is equivalent to a crowd load expected.

(c) Drainage system and lighting facility

The steel drainage pipes 100 mm diameter were installed at 15 m intervals on the upstream side of the crest road to drain the bridge deck and crest road.

The lighting poles were installed at 15 m intervals on the downstream side and their electrical wires were installed in the concrete parapet of the handrail.

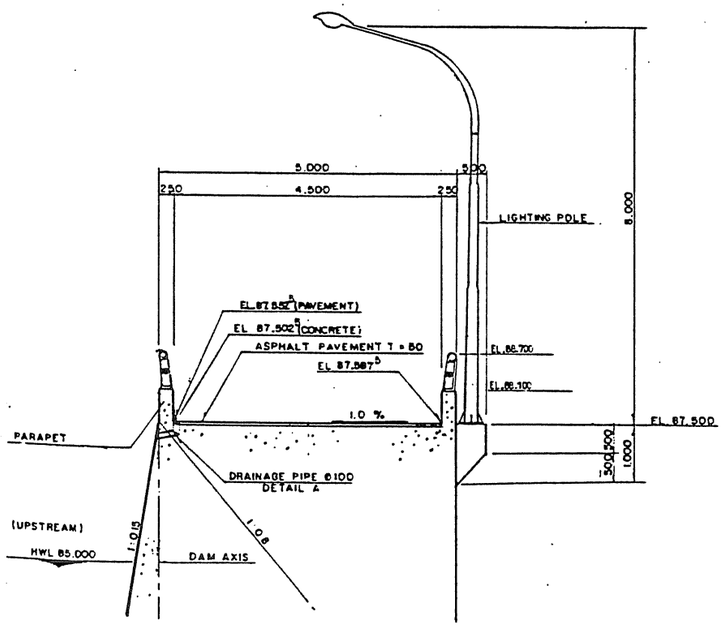


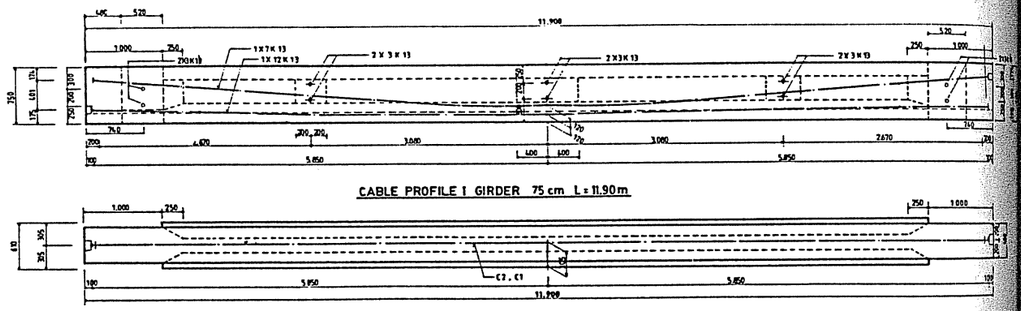
Fig. 1.6.2-6 Dam Crest Detail

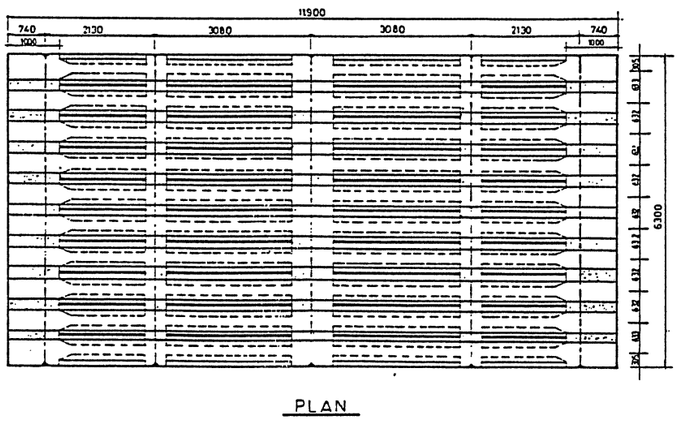
(3) Spillway Bridge

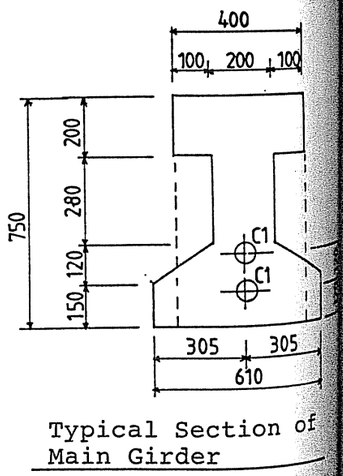
(a) General.

The spillway bridge was designed at the dam crest level to provide access over the spillway to the right abutment of the dam. The bridge is also used for installation and maintenance of spillway gates, spillway stoplogs, hoist deck and hoisting motors by mobile crane.

In the design stage, the bridge was originally designed with prestressed concrete girders 55 cm deep over a total width of 5.0m on condition that the live load TL-20 was acting on the bridge. In the construction stage, however, it became clear by Lot II Metal Works Contractor that a 80 ton class mobile crane was needed to handle the spillway stoplogs. Therefore, the design of the bridge was modified and its dimensions were enlarged to provide a total width of 5.8m. The girder's depth of 75cm in order to satisfy the required design loads of a 80 ton mobile crane. The modified structual shape is shown below. The original design and the modified design are compared in the table below.







Structural Shape of Spillway bridge

Comparison of Original and Modified Design

	Original Design	Modified Design
Type	Pre-tension simple slab bridge	Post-tension simple slab bridge
Beam Length (m)	11.900	11.900
Effective Span (m)	11.500	11.500
Effective Width (m)	4.500	5.800
Total Width (m)	5.000	6.300
Beam Height (m)	0.500	0.750
Inclined Angle (^{0})	90	90
Number of Beam	15	10

(b) Design Condition

(i) Design load

A 80 ton mobile crane generates much larger loads and stresses in the bridge members than that of the original design load TL-20. The 80 ton mobile crane, however, should normally be used on the spillway bridge only once in several years, at the time of handling the spillway stoplogs for future spillway gates maintenance.

Therefore, as the design live load, the 80 ton mobile crane was applied for the service stress condition, while the load of TL-20 for the ultimate stress condition because the 80 ton mobile crane load was considered to be temporary and short period load. The load condition at original design and modified design are shown in table below.

Load Condition of Original and Modified Design

	Original Design	Modified Design
Live load - Service stress condition	TL - 20	80 ton mobile crane at lifting stoplog.
- Ultimate stress condition	TL - 20	TL - 20
Dead load - Reinforced concrete - Concrete - Asphalt pavement - Handrail	2500 kg/m^3 2350 kg/m^3 2300 kg/m^3 50 kg/m	2500 kg/m^3 2350 kg/m^3 2300 kg/m^3 50 kg/m
Impact factor (i) - Uniform load - Concentrated load	i = 10 / (25+L) i = 20 / (50+L) L : Effective span (m)	i = 10 / (25 + L) i = 20 / (50 + L)
Moment at ultimate stress condition.	1.30 x Md + 2.50 x Ml or 1.70 + (Md + Ml) Md : moment due to dead load Ml : moment due to live load	1.30 x Md + 2.50 x Ml or 1.70 x (Md + Ml)

(ii) Allowable stresses intensity

Concrete

(Unit : kg/cm²)

Items	Design Value	
	Main Girder	Cast in-site
Concrete design strength (σck)	500	250
- Initial condition	345.8	160.9
- Service condition	432.3	201.1
Initial allowable stress		
- Compressive stress	190.2	88.5
- Tensile stress	14.8	0.0
Service allowable stress (σca)		
- Compressive stress	172.9	80.4
- Tensile stress	33.1	22.6
Allowable diagonal tension stress (σla)	12.0	-
Allowable mean shear stress (τma)	6.5	-
Allowable ultimate shear stress (τua)	60.0	-
Modulus of elasticity (Ep)	3.07×10^5	2.18×10^5

Prestress cable and reinforcement steel

Items	Design Value	
	Prestress Cable	Reinforcement steel
Type or grade	Uncrated stress relieved seven wire strand. ASTM A416 Grade 270 Low relaxation	U 39 (Deformed)
Diameter	12.7 mm	13 mm
Effective sectional area	0.9871 cm²	1.33 cm²
Modulus of elasticity	1.96×10^6 kgf/cm²	2.1×10^6 kgf/cm²
Ultimate tensile strength	19,000 kgf/cm²	3,000 kgf/cm²

(iii) Calculation Method

Load distribution

The load distribution across the bridge girders was calculated by the anisotropic slab theory of T. Guyon & C. Massonnet.

Bending stress (σ_c) in service stress condition

(Top of Beam) (Bottom of Beam)

$$\sigma_{cu'} = \frac{Pe}{Ac} + \frac{(M + Pe \times ep)}{Ic} yu, \quad \sigma_{cu} = \frac{Pe}{Ac} + \frac{(M + Pe \times ep)}{Ic} yb$$

Where, σ_{cu}, $\sigma_{cu'}$: Concrete stress (kg/cm²)
Pe : Effective Prestress (kg)
M : Bending moment due to live load and dead load (kg.cm)
ep : Eccentricity of PC cables (cm)
Ac : Sectional area of concrete (cm²)
Ic : Moment of inertia of area (cm²)
yb : Distance of neutral axis from the bottom of beam
yu : Distance of neutral axis from the top of beam

Shear stress in service stress condition

- Diagonal tension stress (σ_1) and Mean shear stress (τm)

$$\sigma_1 = -\frac{1}{2}(\sigma_c - \sqrt{\sigma c^2 + 4\tau^2}), \quad \tau = \frac{(Shd - Sp).Q}{bw.I}, \quad \tau m = \frac{shd - sp}{bw.d}$$

where , τ : Shear stress (kg/cm²)
σc : Concrete stress (kg/cm²)
Q : Geometrical moment of area (cm³)
I : Moment of inertia of area (cm⁴)
Shd: Shear force due to design load (kg)
Sp : Vertical component of prestress (kg)
bw : Width of the web of beam (cm)
d : Effective height of the beam (cm)

Resisting bending moment for rupture (Mu) and safety factor against rupture (Fs)

$Mu = Aps.fps\ (d - kx),\ Fs = Mu / Mn > 1.0$

where, Mn : Actual moment calculated under load combination in ultimate stress condition (t.m).
Aps : Area of tensile steel (cm²)
fps : Stress of tensile steel (kg/cm²)
d : Effective height of the section (cm)
kx : Distance between centroid of compression area and compressive edge of the section (cm) (k = 0.4 : in case of rectangular section)

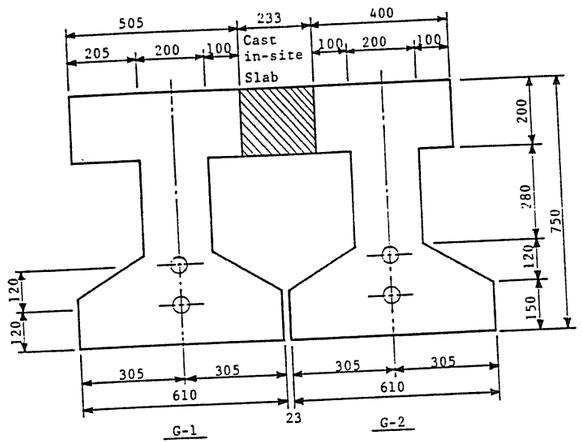
Shear stress in ultimate stress condition (τu)

$\tau u = (shd - sp) / bw.d < \tau ua$

where, shd : Shear force due to ultimate stress condition (kg)
sp : Vertical component of prestress (kg)
bw : Width of the beam (cm)

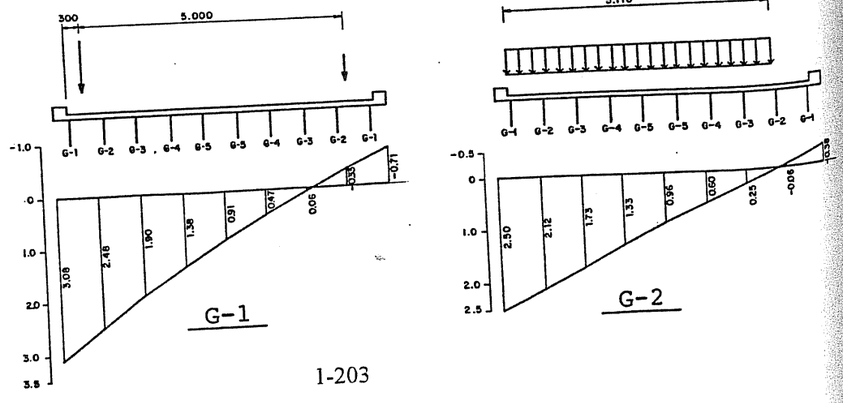
(c) Structural Calculation of Main Girder

(i) Shape of Cross Section



(ii) Calculation of bending moment and shear force

The load condition, bending moment and shear force due to each design load are shown in Table 1.6.2-3. Based on the load distribution, the bending moment and shear force acting on each main girder are shown in Table 1.6.2-4. From this calculation, the bending stress and shearing stress of the main girder was studied for girder No.1 and No.2 under the service stress condition and ultimate stress condition respectively. The load condition and influence line diagram of the girder No.1 and No.2 are shown in the following figures.



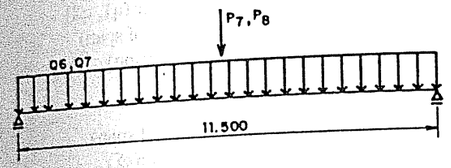
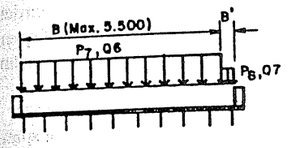
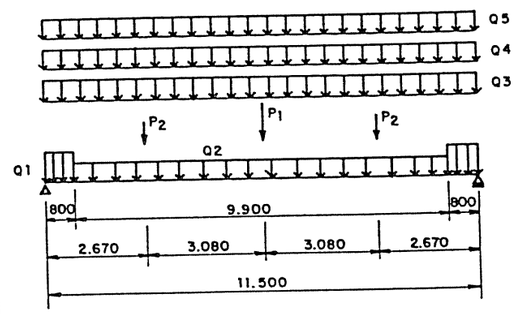
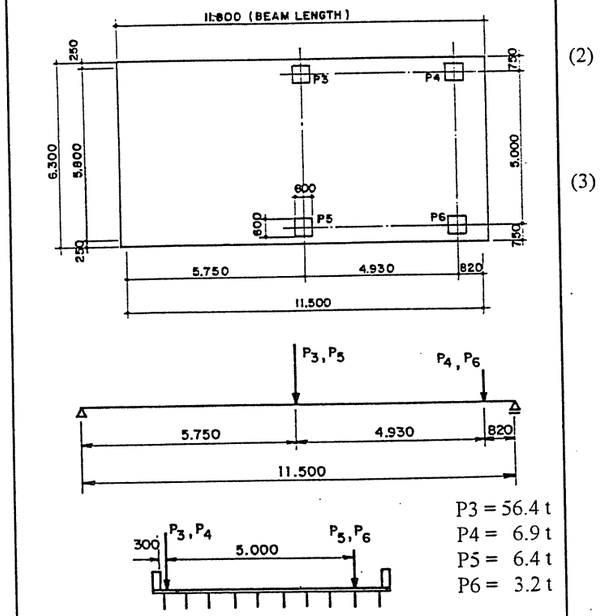
Load Condition	Mean bending moment and shear force (per 1 girder)
3. Live Load on Ultimate Stress Condition   P7 = 5.0 t/m, P8 = 2.5 t/m, q6 = 0.35 t/m², q7 = 0.175 t/m²	(1) Impact factor i = 10/(25+l) = 0.274 (2) Primary line load (P7) Mm = 1.831 x B t.m (center of span) Sm = 0.319 x B t (supporting) (3) Secondary line load (P8) Mm = 0.916 x B' tm (center of span) Sm = 0.159 x B' t (supporting) (4) Primary uniform load (q6) Mm = 0.737 x B tm (center of span) Sm = 0.256 x B t (supporting) (5) Secondary uniform load (q7) Mm = 0.369 x B' tm (center of span) Sm = 0.128 x B' t (supporting) where B, B' : Loading width B (m) / B' (m) G-1: 4.60 / 0.00 G-2: 5.11 / 0.00 G-3: 5.50 / 0.30 G-4: 5.50 / 0.30 G-5: 5.50 / 0.30

Table 1.6.2-4 Bending Moment and Shear Force

		G-1	G-2	G-3	G-4	G-5
Bending moment (t/m	Dead Load	17.188	17.046	16.926	16.855	16.797
	Live Load (1)	57.250	51.395	41.001	33.230	26.116
	Live Load (2)	13.351	15.356	14.869	14.855	14.687
	Total on Service	74.438	68.441	57.927	50.085	42.913
	Total on Ultimate	55.722	60.550	59.176	59.049	58.554
Shear force at center of span (t)	Dead Load	0.282	0.282	0.282	0.282	0.282
	Live Load (1)	19.581	17.568	14.003	11.335	8.890
	Live Load (2)	3.311	3.808	3.688	3.684	3.643
	Total on Service	19.863	17.850	14.285	11.617	9.172
	Total on Ultimate	8.644	9.887	9.587	9.577	9.474
Shear force at support (t)	Dead Load	5.899	5.850	5.808	5.783	5.763
	Live Load (1)	11.913	10.758	8.656	7.105	5.691
	Live Load (2)	2.987	3.438	3.328	3.324	3.287
	Total on Service	17.812	16.608	14.464	12.888	11.454
	Total on Ultimate	15.136	16.200	15.870	15.828	15.709

Note : Live Load (1) : 80 ton mobile crane
Live Load (2) : TL-20

Table 1.6.2-3 Load Condition and Mean Bending Moment and Shear Force.

Load Condition	Mean bending moment and shear force (per 1 girder)
1. Dead Load  q1 = 0.750 t/m, q2 = 0.690 t/m, q3 = 0.109 t/m, q4 = 0.115 t/m, q5 = 0.429 t/m, p1 = 0.282 t, p2 = 0.141 t	(1) Own weight of main girder (q1, q2) Mm = 11.437 t.m (center of span) Sm = 4.028 t (supporting) (2) Own weight of slab (q3) Mm = 1.802 t.m (center of span) Sm = 0.627 t (supporting) (3) Own weight of transverse girder (P1, P2) Mm = 1.187 t.m (center of span) Sm = 0.282 t (center of span and supporting (4) Own weight of pavement (q4) Mm = 1.103 t.m (center of span) Sm = 0.384 t (supporting) (5) Own weight of curb & handrail (q5) Mm = 0.70 tm (center of span) Sm = 0.247 t (supporting)
2. Live Load on Service Stress Condition  P3 = 56.4 t P4 = 6.9 t P5 = 6.4 t P6 = 3.2 t	(1) Impact factor i = 20/(50+l) = 0.325 (2) Primary load (P3, P4) Mm = 21.866 tm (center of span) Sm = 4.585 t (supporting) Sm = 7.473 t (center of span) (3) Secondary load (P5, P6) Mm = 2.612 tm (center of span) Sm = 0.818 t (supporting) Sm = 0.848 t (center of span)

(iii) Effective Prestress Force (Pe)

$Pe = Aps \times (\sigma pt - \Delta \sigma pr - \Delta \sigma pc) = 181,583\ kg/cm^2$

Where , Aps : Area of steel (= 18.7549 cm^2)
σpt : Tendon stress during prestress transfer (= 11400 kg/cm^2)
Δσpr : Stress loss by relaxation (= 285 kg/cm^2)
Δσpc : Stress loss by creep and dry shrinkage (= 1433 kg/cm^2)

(iv) Structural Actual Calculation of Main Girder

Service stress condition

The calculation result of bending stress diagonal tension stress (σl) and mean shear stress (τm) are shown in the table below. It is shown that the stresses of the main girder No. 1 under the service stress condition when the 80 ton mobile crane is acting on the girder, are within the allowable stress in all locations except for the mean shear stress at section No. 7 which is in the center of the girder.

From above calculation result, the mean shear stress at section No. 7 exceeds the allowable mean shear stress, 6.5 kg/cm^2. Therefore, re-arrangement of stirrups was necessary at the center of the main girder.

U-shape stirrups fabricated by reinforcement bar D13 were designed and the required spacing of the stirrup was 15.2 cm, which satisfied the actual spacing of 14.0cm.

Calculation of Bending Stress

Section No.	No.1	No.2	No.3	No.4	No.5	No.6	No.7
Distance	0.00	0.375	1.05	2.30	3.45	4.60	5.75
Ac (cm^2)	3787.5	3787.5	2971.0	2971.0	2971.0	2971.0	2971.0
Ic ($x10^6\ cm^4$)	1.775	1.775	1.823	1.823	1.823	1.823	1.823
yu (cm)	37.50	37.50	[illegible]	39.60	39.60	39.60	39.60
yb (cm)	37.50	37.50		35.40	35.40	35.40	35.40
Pe (kg)	181583	181583		181583	181583	181583	181583
PC Position (cm)	31.85	30.41		23.11	22.77	17.31	16.42
ep (cm)	5.65	7.09		12.29	12.63	18.09	18.98
Moment due to dead load (t.m)	0.000	2.133		10.889	14.299	16.378	17.188
Moment due to live load (t.m)	0.000	4.402		24.017	35.090	46.161	57.250
Total moment (t.m)	0.000	6.535		34.906	49.389	62.539	74.438
Moment due to prestress (t.m)	-10.259	-12.87		-22.317	-22.934	-32.848	-34.464
Pe/Ac (kg/cm^2)	47.94	47.94		61.12	61.12	61.12	61.12
Bend stress of girder top (kg/cm^2)	26.27	34.5		88.47	118.59	125.62	147.95
Bend stress of girder bottom (kg/cm^2)	69.61	61.3		36.67	9.75	3.46	-16.50

Calculation of Diagonal Tension Stress and Mean Shear Stress

Section No.	No.1	No.2	No.3	No.4	No.5	No.6	No.7
Distance (m)	0.00	0.375	1.05	2.30	3.45	4.60	5.75
Shd (ton)	17.812	17.275	14.489	13.242	12.132	11.020	19.863
Sp (ton)	6.898	6.898	6.808	6.579	5.123	2.998	1.100
bw (cm)	50.5	50.5	20.0	20.0	20.0	20.0	20.0
d (cm)	70.0	70.0	70.0	70.0	70.0	70.0	70.0
I ($x10^6$ cm^4)	1.775	1.775	1.823	1.823	1.823	1.823	1.823
Q ($x10^4$ cm^3)	3.551	3.551	3.372	3.372	3.372	3.372	3.372
τ (kg/cm^2)	4.27	4.11	7.10	6.16	6.48	7.42	17.35
σc (kg/cm^2)	47.94	47.94	61.55	62.57	64.17	64.54	65.73
σl (kg/cm^2)	0.38	0.35	0.81	0.60	0.65	0.84	4.80
τm (kg/cm^2)	3.09	2.94	5.49	4.76	5.01	5.73	13.40

Ultimate stress condition

The calculation result of the resisting bending moment and safety factor against rupture at the center of the girder G-2 is shown in the table below. The safety factor are satisfied with design safety factor 1.0. The calculation result of ultimate shear stress on supporting point and center of the girder G-2 is shown in the table below. It is shown that the stress of the main girder G-2 in the ultimate stress condition is within the allowable stress, 60 kg/cm^2, in both locations.

	Center of girder (Section No. 7)	Remarks
Maximum Bending Moment : Mn (t.m)	60.550	Refer to Table 1.6.2-4
Area of PC steel : Aps (cm^2)	18.7549	
Stress of PC steel : fps (kg/cm^2)	16531	
Effective height of section : d (cm)	58.58	
Distance from compressive edge to neutral axis : x(cm)	22.80	
Resisting bending moment : Mu (t.m)	153.344	
Safety factor : Mu/Mn	2.5	

	Supporting Point (section No. 1)	Center of girder (section No. 7)	Remarks
Shear force due to ultimate load condition : shd (ton)	16.200	9.887	Refer to Table 1.6.2-4
Vertical component of prestress : sp (ton)	6.898	1.100	
Width of web : bw (cm)	40.0	20.0	
Effective height of girder : d (cm)	70.0	70.0	
Ultimate shear stress : τu (kg/cm^2)	3.4	6.3	

(d) Structural Calculation of Transverse Girder.

(i) Calculation of bending moment

The bending moment due to concentrated load, Q and uniform load, W, on the transverse girder is calculated by the following equations.

$MQ = q.b.\Sigma\mu a\ (2Q/\ell)\ \sin(\pi d/\ell)\ \sin(\pi x/\ell),\quad Mw = q.b.\Sigma\mu a\ (4w/\ell)\ \sin(\pi x/\ell)$

where ; q : Interval of transverse girder
b : 1/2 of reaction width
ℓ : Effective span
x : Distance between supporting point of main girder and calculating transverse girder.
d : Distance between supporting point of main girder and loading point.
μa : Coefficient of load distribution

The load condition and bending moment due to each design load and bending moment acting on the center transverse girder is shown in Table 1.6.2-5 to 1.6.2-6 respectively.

Effective width of web and parameters of section.

	Against Bending Moment	Against Axis Force
Width of upper flange (cm)	209.6	268.0
Width of web (cm)	80.0	80.0
Height of girder (cm)	75.0	75.0
Sectional area (cm^2)	8592.0	9760.0
Moment of inertia of area ($x10^6$ cm^4)	4.268	4.686
Distance of neutral axis from the top of girder (cm)	29.20	26.91
Distance of neutral axis from the bottom of girder (cm)	45.80	48.09

Table 1.6.2-5 Load Condition and Bending Moment

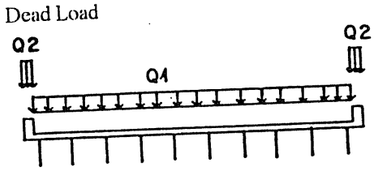
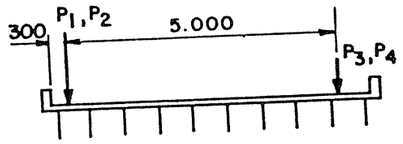
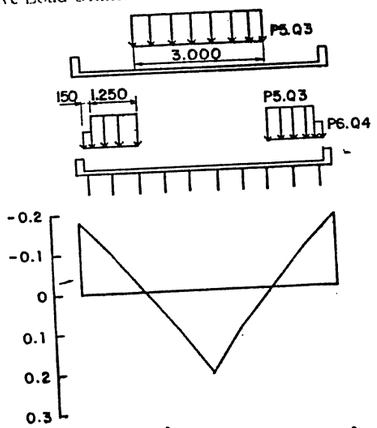
Load Condition	Mean Bending Moment
1. Dead Load  q1 = 0.115 t/m², q2 = 0.858 t/m	(1) Own weight of pavement (w1) Mm = 1.421 tm (2) Own weight of Curb and Handrail (w2) Mm = 10.599 tm
2. Live Load on Service Stress Condition  P1 = 56.4t, P2 = 6.9t, P3 = 4t, P4 = 3.2t	(1) Impact factor i = 20/(50 +ℓ) = 0.325 (2) Wheel load P1 : Mm = 126.092 tm P2 : Mm = 3.411 tm P3 : Mm = 14.308 tm P4 : Mm = 1.583 tm
3. Live Load Ultimate stress Condition  P5 = 5.0 t/m, P6 = 2.5 t/m, q3 = 0.35 t/m², q4 = 0.175 t/m².	(1) Impact factor i = 10/(25+ℓ) = 0.274 (2) Primary line load (P5) Mm = 10.748 tm (3) Secondary line load (P6) Mm = 5.374 tm (4) Primary uniform load (q3) Mm = 5.508 tm (5) Secondary uniform load (q4) Mm = 2.754 tm

Table 1.6.2-6 Bending Moment for Center Transverse Girder

	Max. moment	Min. moment
Dead Load	-1.731	-1.731
Live Load (1)	0	-17.534
Live Load (2)	4.925	-3.944
Total on Service	-1.731	-19.265
Total on Ultimate	10.644	-11.566

(ii) Effective prestress force (Pe)

Pe = 6 pieces x 0.9871 cm² x 9680 kg/cm² = 57331 kg

(iii) Structural Calculation of Transverse Girder

Service stress condition

The calculation result of the bending stress is shown in the table below. The stresses of the center transverse girder in the service stress condition with the 80 ton mobile crane acting on the girder, are within the allowable stresses.

	Max. Bending Moment	Min. Bending Moment
Ac (cm²)	9760.0	9760.0
Ic (x10⁶cm⁴)	4.268	4.268
Yu (cm)	29.20	29.20
Yb (cm)	45.80	45.80
Pe (kg)	57331	57331
ep (cm)	5.80	5.80
Moment due to Load (t.m).	-1.731	-19.265
Moment due to Prestress (t.m).	-3.325	-3.325
Pe /AC (kg/cm²)	5.87	5.87
Bending stress of top of girder (kg/cm²).	2.41	-9.58
Bending stress of bottom of girder (kg/cm²)	11.30	30.11

Ultimate stress condition

The calculation result of the resisting bending moment against rupture in the center transverse girder is shown in the table below, and the safety factor is satisfied with design safety factor 1.0.

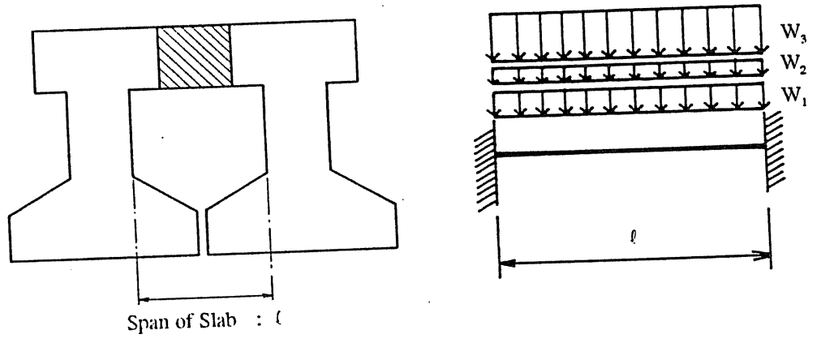
	Maximum bending moment	Minimum bending moment	Remarks
Max. Bend Moment : Mn (t.m)	10.644	-12.085	Refer to Table 1.6.2-6 The former : upper PC. The latter : lower PC
Area of PC steel : APS (cm²)	5.9226 5.9226	5.9226 5.9226	
Stress of PC steel : fps (kg/cm²)	17670 17670	16090 17291	
Effect height of section : d (cm)	25.0 45.0	50.0 30.0	
Distance from compressive edge to neutral axis : x (cm)	5.87 5.87	14.54 14.54	
k	0.4 0.4	0.4 0.4	
Resist bend moment : Mu (t.m)	70.180	-68.293	
Safety factor : Mu/Mn	6.5	5.6	

(e) Structural Calculation of Slab

(i) Structural calculation in service stress condition.

Calculation model

The maximum wheel load of 56.4 ton due to the 80 ton mobile crane acts on the slab through the bearing plate sized 60cm x 60cm, while the span of the slab is 43.2cm. Therefore, the bending moment in the service stress condition would be modeled under the condition that the uniform load caused the 80 ton mobile crane acted on the constrained beam with boths end fixed. The calculation model and load condition are shown below.



Span of Slab : ℓ

Where,
W1 : Own weight of slab (= 0.300t/m)
W2 : Own weight of pavement (= 0.069t/m)
W3 : Uniform load due to live load (= 124.55 t/m)
ℓ : Span of slab (= 0.432 m)

Structural calculation

The calculation results of the bending stress at the supporting point and the center of span of the slab are shown in the table below, which shows that the stresses of the reinforced concrete slab are within the allowable stresses in both locations.

Location	Center of Span	Supporting Point
Moment due to dead load (t.m).	0.002	0.006
Moment due to live load (t.m).	0.969	-1.937
Total moment (t.m).	0.971	-1.943
Width of slab (cm).	60.0	60.0
Effective height (cm).	15.0	15.0
Distance between upper steel bar and compressive edge (cm).	5.0	5.0
Sectional area of lower steel bars (cm^2).	9.05	9.05
Sectional area of upper steel bars (cm^2).	9.05	9.05
Elastic modulus ratio of concrete to reinforcement bar.	9.65	6.82
Compressive stress for concrete σc (kg/cm^2).	45.54	102.97
Tensile stress of lower steel bar σs (kg/cm^2).	814	1549
Stress for upper steel bar σs' (kg/cm^2).	-21.72 (compressive)	48.02 (Tensile)

(ii) Structural Calculation in Ultimate stress condition

Calculation method

The bending moment in the slab under the ultimate stress condition is calculated by the following equations, on the condition that the load of T-20 acts on the continuous slab.

- Bending moment in the direction of the span

	(Dead Load)	(Live Load)
Supporting point	$w\ell^2/10$	$(0.12\ \ell+0.07) \times P \times 0.8$
Center of span	$-w\ell^2/10$	$-(0.15\ell+0.125) \times P$

- Bending moment in the direction at right angle with the span

	(Dead Load)	(Live Load)
Center of span	take no account	$(0.10\ell+0.04) \times P \times 0.8$

where, w : Uniform load due to dead loads
P : Design load of rear wheel of T-20 (P = 8 ton)
ℓ : Span of slab (m)

Structural calculation on ultimate stress condition

The calculation results of the maximum bending moment under the ultimate stress condition and the resisting bending moment for rupture on the slab are shown in the table below, and the safety factors are satisfied exeeceding the design safety factor 1.0 in all locations.

	In direction of span		In direction at right angle with span
	Center of span	Supporting point	Center of span
Moment due to dead load(tm).	0.011	-0.011	0
Moment due to live load (tm).	0.780	-1.518	0.532
Moment on ultimate stress condition Mn(t.m).	1.965	-3.810	1.330
Width of slab (cm)	100.0	100.0	100.0
Height of slab (cm)	20.0	20.0	20.0
Distance of upper steel bar to compressive edge (cm).	5.0	5.0	5.0
Sectional area of lower steel bar (cm^2)	9.050	9.050	9.050
Sectional area of upper steel bar (cm^2)	9.050	9.050	9.050
Design strength of concrete (kg/cm^2)	250	500	250
Ultimate tensile strength of steel bar (kg/cm^2)	3.000	3.000	3.000
Resisting bending moment Mu (t.m)	4.736	-5.083	5.279
Safety factor Mu/Mn	2.41	1.33	3.96

(4) Gallery

The gallery system was designed for the purpose of (i) draining the water percolating from the upstream face and seeping through the foundation rock, (ii) access to the interior of the structure for inspecting and observing the dam behavior after completion, (iii) access through the dam for control cables and power cables, (iv) space for drilling drain holes during construction, (v) space for drilling and grouting additional holes for the upstream grout curtain and for drilling additional drain holes for drainage of foundation in future as a countermeasure against unusual leakage or uplift pressure arising and (vi) access to the penstock man-hole for inspecting and maintenance of the intake gates.

(a) Location and Size

The location of the dam gallery was designed such that it was located above the one-third point of the base width from the upstream face, at a distance of 5m from the foundation rock surface at river bed. It extends the length of the dam near the foundation rock surface, conforming in elevation to the transverse profile of the canyon, in plan it was approximately parallel to the dam axis. And the penstock gallery was also designed to be along the dam axis between the dam gallery with EL. 53.50 left abutment and No. 3 penstock for connecting the passage way to each penstock man-hole.

The size of the dam gallery was determined to be 2.0m (width) x 2.5m (height) vault type in consideration of the size of a drilling machine. And the penstock gallery was also designed as a 2.0m (width) x 2.5m (height), rectangular type.

The original alignment of the dam gallery mentioned above, was designed to be along the dam axis between the right and left abutments at EL. 34.50 to EL. 80.00. However, various measurement instruments, electricity cable, control cables, drainage pump and drain pipes were installed in the gallery. For the monitoring and inspection for these facilities and dam body its self, the staff shall enter the gallery quite often in the future.

Therefore, the additional gallery at EL. 45.50 was designed to be used as the connecting passage-way between the original gallery and the powerhouse control building during the construction stage.

(b) Drainage system

The drainage system was provided at floor level in the dam gallery by installing drain holes drilled into the foundation rock and drain ditches 0.3m (depth) x 0.5m (width) along the upstream edge of the gallery, and the drain water is led to the sump pit located at the lowest elevation of EL. 34.50 of the gallery.

Two (2) drainage pumps each with capacity of 800 l/min were installed in the sump pit to pump the water up to the drain ditch located just downstream of the dam.

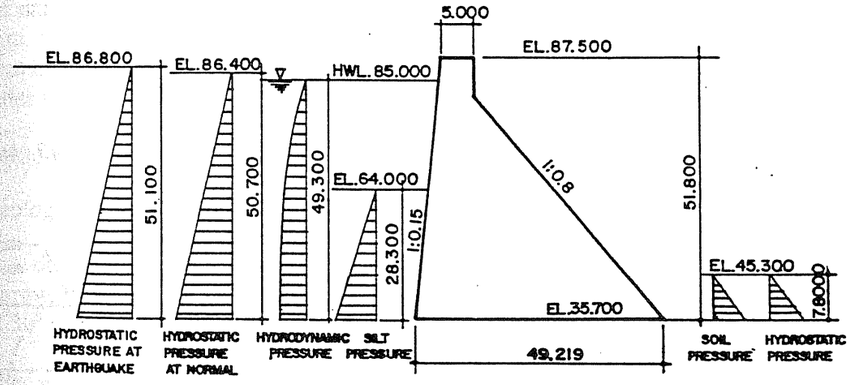
(c) Drain holes

Drain holes were installed in the floor of the dam gallery at the contact between the foundation rock and the dam concrete in order to reduce uplift pressure and to drain seepage water. The drain holes comprise drilling a hole with 66 mm diameter 5.0m deep to the rock foundation and a 50mm diameter steel pipe placed at approximately 5m intervals along the dam axis, in the floor of the dam gallery.

(d) Structural calculation

Reinforcement is usually required around the gallery in the dam because concentrations of stresses will occur around the gallery. The stresses around the gallery can be determined using "Opening in thin plates" (stress concentration at hole). The distribution of reinforcement was designed as follows. The detail of the calculation method is referred to section 6.7.3 "Penstock".

Calculation case is (i) Normal condition with H.W.L, and (ii) Earthquake condition with H.W.L. Calculation of loads are shown below.



Dam own weight		:	2967.386
Hydrostatic pressure	(normal)	:	50.70 t/m²
	(earthquake)	:	51.10 t/m²
Hydrodynamic pressure		:	4.58 t/m²
Silt pressure		:	15.57 t/m²

Calculation result of stress

The values and figures given in the design condition introduce the following stress equations.

For normal state,

$\sigma_z = 70.458 - 0.249y,\ \sigma_y = 45.093 - 0.243y + 0.03y^2,\ \tau_{zy} = 56.366 - 0.901y - 0.004y^2$

For earthquake state,

$\sigma_z = 91.980 - 1.121y,\ \sigma_y = 58.867 - 1.189y + 0.068y^2,\ \tau_{zy} = 73.584 - 1.253y - 0.003y^2$

Principle stresses σ_1, σ_2 at the point of Y = 35.140m (center of the gallery) are given below :

unit:t/m²

	Normal Condition	Earthquake Condition
σ_z σ_y σ_{zy}	61.71 73.60 19.77	52.59 101.05 25.85
σ_1 σ_2	88.30 47.01	112.25 41.39

Required reinforcement

The cross section of the gallery is conveniently modeled as a circumcircle as shown below, thereby the required reinforcement against the tensile force Ft produces around the circular opening was calculated as follows.

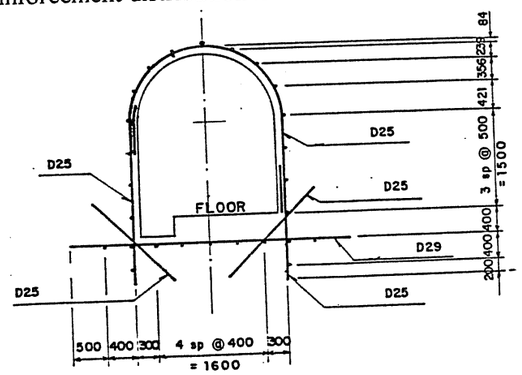
r = *1.720 + 0.100 = 1.820m* *where, 0.100 : cover of reinforcement*

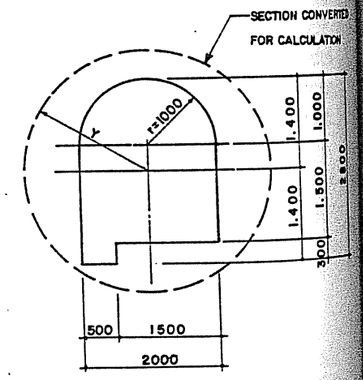
Ft = $0.2\, r\sigma = 0.364\,\sigma$ *(t)*

Required reinforcement is given As = Ft x 1000/σsa cm² (per 1.0m - depth)

Normal condition : *As1 = 0.364 x 88.30 x 1000/2000 = 16.07 cm²*
Earthquake condition : *As2 = 0.364 x 112.25 x 1000/(2000 x 1.5) = 20.43 cm²*

Reinforcement distribution is shown below.





(e) Miscellaneous

The lighting is provided by a fluorescent lamp 40W installed at 6m intervals on the downstream side wall of the gallery. Cable racks were installed on both side walls of the gallery to carry control cables and power cables from the powerhouse to the dam crest. They were mainly for electrical-mechanical equipments such as the spillway and intake gate, water supply facility, lighting facility and monitoring instruments installed on each hoist deck, the dam left abutment, dam crest and in the dam body respectively.

6.3 Foundation Treatment

6.3.1 General

The consolidation grouting, curtain grouting and foundation drainage were provided for improvement of the dam foundation.

The foundation is an integral part of the dam and requires as much attention as the dam itself. From the result of the geological condition after dam excavation, the dam foundation was judged to be good and capable of withstanding the design loads imposed by the dam, reservoir and appurtenant structures.

The foundation itself, however, was well jointed and some fracture zones with clay layer in the river bed and left abutment. Consolidation grouting and curtain grouting was carried out to establish an effective barrier for seepage under the dam through its foundation and consolidate the surface of its foundation loosened by excavation.

The consolidation grouting was injected to a depth of 7-10m with 3m hole spacing on the dam concrete. A single line of curtain grouting was carried out in the heel of the dam for the depth of 50m in the river bed and 30m in both banks. The target permeability was revised from the tender design value of 2 Lugeon to 5 Lugeon in order to reduce the numbers of additional holes.

6.3.2 Consolidation and Curtain Groutings

(1) Geological Condition of the Dam Site

The dam site features an asymmetrical V-shaped valley consisting of a relatively narrow gorge 80 to 85 m wide at the normal river water level, with a moderate slope of 30° on the left bank, and a steep cliff of 50° on the right bank.

The whole area of the dam site is underlined by bedrock of dacitic tuff of Carboniferous to Early Permian age. There are many discontinuities of rock mass occurring such as joints, cracks, fissures and fracture zones. Some of the discontinuities are filled with iron-oxide and/or quarts veins due to hydro thermal alteration. In general, the rock condition is described as hard, massive, homogeneous, sound rock, and/or well-jointed dacitic tuff.

A Talus deposit covered the dam right bank and powerhouse foundations, river deposit (sand & gravel) covered the riverbed dam foundation. The thicknesses of these materials were around 2m to 4m, while on the left dam abutment, residual soil of dacitic tuff formed an overburden 2m to 3m thick.

Dam site geology is summarized as follows.

(i) The foundation of the dam site is hard, massive homogeneous and a well jointed dacitic tuff.

(ii) The fractured zones exist in the river bed and the left bank.
(iii) The joints in the dam site show generally north-south and east-west direction with steep dip.
(iv) Seepage of ground water was found in the left bank, river bed and right bank. The ground water level is high except in the low water level area in the upper left bank.

(2) Consolidation Grouting

The grouting area covered the heel to toe area of the dam and was spaced 3m apart based on the split spacing in the river bed area. In both the banks, holes were drilled on each upstream-downstream line, because of the small concrete cover in the higher side of the dam concrete block.

In the river bed area, the drilling depths of holes were 10m for the primary, 7m for secondary and tertiary holes. In both the left and right banks, the drilling depth was 7m for all sequences, as shown in Fig. 1.6.3-1. The drilling direction was vertical to the concrete surface except at the higher part of both banks where the drilling direction was perpendicular to the rock surface.

The grouting method was downstage for two stages. Permeability of the final sequence of injection was 10 Lu with a probability equaled or exceeded at 15%, taking into account the rock condition and dam height (referred to section 4.3.2 Plan, Design and Construction of Grouting of chapter 25 "Construction of Multi Purpose Dam" by Ministry of Construction Japan).

(3) Curtain Grouting

A single line of curtain grouting was placed on the upstream heel of the dam concrete block to impede the flow of water around the dam foundation.

The grouting sequence was based on the split spacing method from the primary holes for every 12m apart to the tertiary holes for every 3m apart, as shown in Fig. 1.6.3-1. Depth of drilling was estimated by the empirical formula taking into account the dam height and permeability condition as follows ;

Depth of Curtain

Estimation	Required Depth
d = H/3 + C H : dam height (58 m) C : 8 - 25 m	27 - 52 m
d = α . H H : dam height (58 m) α : 0.5 - 1.0	29 - 58 m
Design depth	50 m

The design line of curtain grouting was determined to be 50m in the river bed area and 30m in both banks. Grouting areas in both abutments were planned to reach the ground water level as shown Fig. 1.6.3-2.

The target permeability of the curtain grouting was originally 2 Lugeon at the early stage of the grouting work in 1994. Although the grouting work proceeded up to the quaternary and quinary holes, the results of permeability still indicated higher Lugeon than the target permeability of 2 Lugeon. It was judged that a considerable numbers of additional holes would be required to achieve the 2 Lugeon target, so the target Lugeon value was studied again.

In order to minimize the additional curtain grouting, seepage analysis by the three dimensional Finite Element Method (FEM) was done to the examine application of the target permeability of 5 Lugeon, as shown in Fig. 1.6.3-3.

The results of the calculation show;

(a) Piping

The maximum velocity around the drain holes is 88.2 x 10^{-5} cm/s, which is well below the Justin's critical velocity, 1.02 cm/s, for 0.01mm particles. (Refer to Fig. 1.6.3-4).

Justin's basic equation is based on the ideal single particle and the safest value of particle diameter 0.01 mm for critical flow velocity 1.02 cm/sec (the smallest particle) was selected to evaluate the safety against piping.

Diameter Soil Particle and Critical Flow Velocity

Diameter of particle (mm)	Critical flow velocity (cm/sec)
5.00	22.86
3.00	17.71
1.00	10.22
0.80	9.14
0.50	7.23
0.30	5.60
0.10	3.20
0.08	2.89
0.05	2.29
0.03	1.77
0.01	1.02

Source : The construction of multi-purpose dam Vol. 3 by Ministry of Construction Japan.

(b) Uplift Pressure

Uplift pressure is smaller than the dam design assumption for the dam stability as shown in Fig. 1.6.3-5.

(c) Drain from Dam Gallery

The drain discharge from the lower dam gallery is estimated at 701 m^3/day which is within the maximum drainage capacity of 1000 m^3/day provided by the pump capacity of 800 l/min installed in the lower dam gallery. (Refer to Fig. 1.6.3-6, Fig. 1.6.3-7 and Table 1.6.3-1).

Judging from the above results, the target permeability of the curtain grouting was revised to be 5 Lugeon.

6.3.3 Foundation Drainage

Although grouting of the foundation has been carried out with meticulous care, the possibility is always there that some channel ways of underground water may remain and flow through these channels will accelerate as the reservoir fills up. Such flows can be intercepted by drainage holes. The drainage holes also serve to reduce uplift pressure even in an otherwise homogenous foundation by intercepting the streamlines. The magnitude of pressure relief depends on the depth, diameter and spacing of the drains. According to current practice, the following standard patterns are provided by the Dam Technology Center Ministry of Construction, Japan.

Diameter : commonly 46 mm, however, 66 mm is also used considering the holes clogging.

Depth : 5 m in rock foundation

Spacing : Depend on the foundation condition. In the case where drainage holes are provided downstream of the grout curtain, 5 m is adopted.

According to the seepage analysis, the flow volume and flow velocity is very small. Therefore, referring to the above standard pattern, the drainage holes 66 mm in diameter are provided immediately downstream of the grout curtain at a spacing of about 5 m and extended to about a further 5 m from dam foundation as shown in Fig. 1.6.3-7.

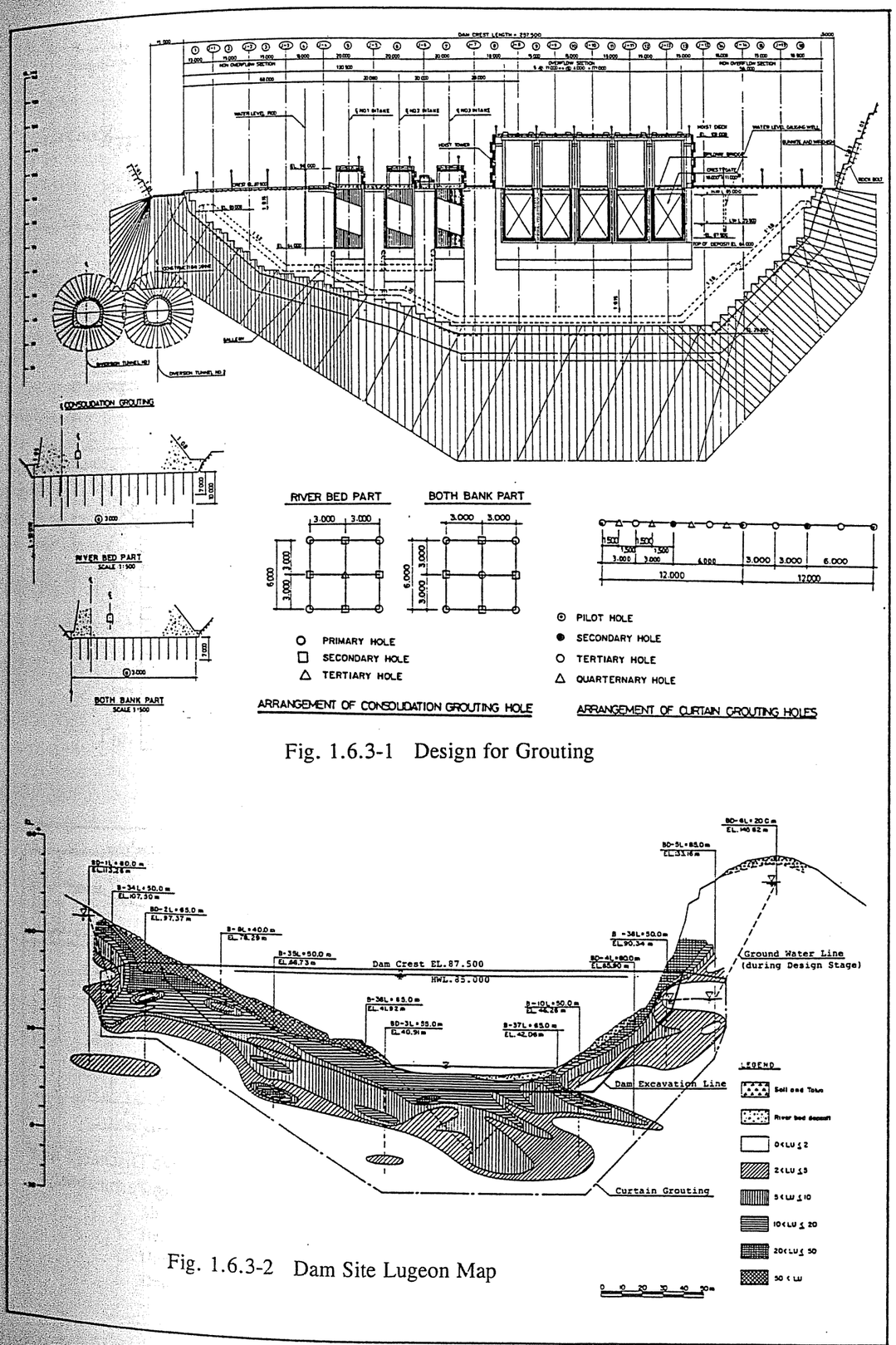


Fig. 1.6.3-1 Design for Grouting

Fig. 1.6.3-2 Dam Site Lugeon Map

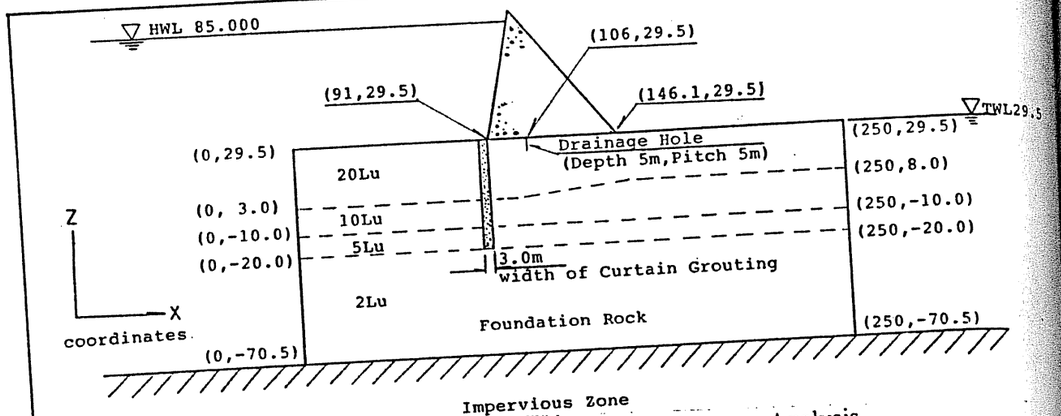


Fig. 1.6.3-3 Model of Seepage Analysis

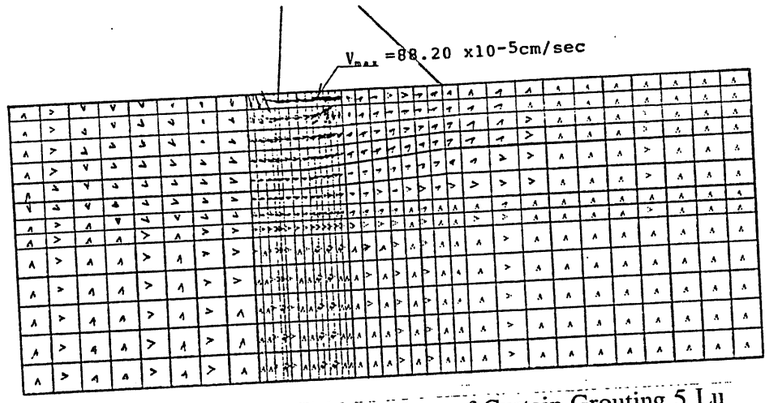


Fig. 1.6.3-4 Flow Velocity Chart of Curtain Grouting 5 Lu

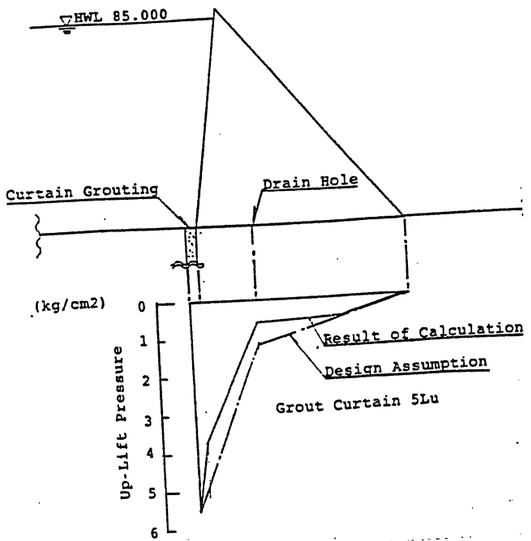


Fig. 1.6.3-5 Distribution of Uplift Pressure

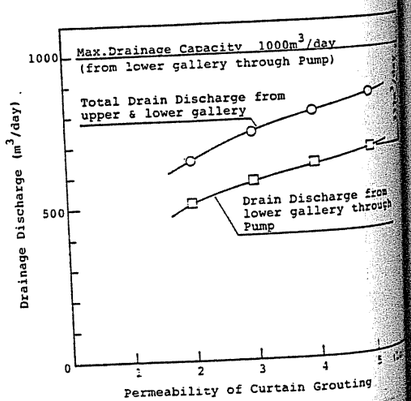
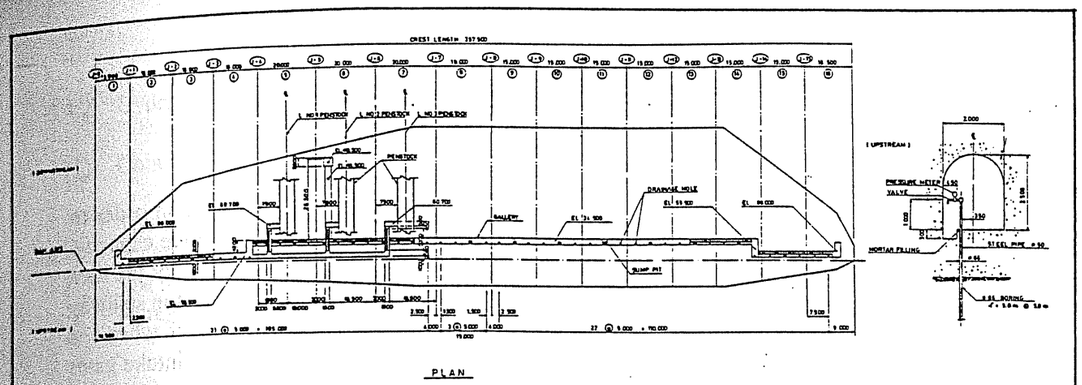


Fig. 1.6.3-6 Drainage Discharge a
Grouting Permeabili



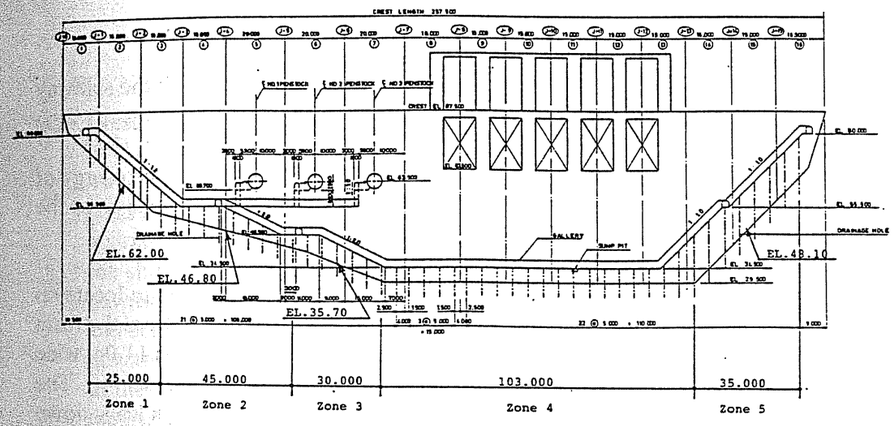


Fig. 1.6.3-7 Zoning in Seepage Analysis and Design for Drainage System

Table 1.6.3-1 Drain Flow Volume from Dam Gallery

*1 Zone	Length	*2 Center EL.	*3 Head (m)	2 Lu		3 Lu		4 Lu		5 Lu	
				*4 Unit Discharge (m^3/day/m)	*5 Drain (m^3/day)	Unit Discharge (m^3/day/m)	*5 Drain (m^3/day)	Unit Discharge (m^3/day/m)	*5 Drain (m^3/day)	Unit Discharge (m^3/day/m)	*5 Drain (m^3/day)
1	25.0	62.0	23.0	1.3888	34.7	1.5839	39.6	1.7140	42.9	1.8077	45.2
2	45.0	46.8	38.2	2.3066	103.8	2.6306	118.4	2.8468	128.1	3.0023	135.1
				from gallery	138.5		158.0		171.0		180.3
3	30.0	35.7	49.3	2.9768	89.3	3.3950	101.9	3.6740	110.2	3.8747	116.2
4	103.0	29.5	55.5	3.3512	345.2	3.8220	393.7	4.1360	426.0	4.3620	449.3
5	35.0	48.1	36.9	2.2281	78.0	2.5411	88.9	2.7499	96.2	2.9001	101.5
				Through Pump	512.5		584.5		632.4		667.0

Note : 1. Drain flow from zone 3,4 and 5 shall be pumped up
2. The center EL. is the elevation of each part of gallery
3. Head means (HWL 85.0 - center EL.)
4. Unit discharge equals to (q x head/55.5), where q is unit discharge at Zone 4
5. Drain flow is q' x length

6.4 Dam Discharge Facility

6.4.1 Hydraulic Design Requirements.

A spillway of the center over flow type with five crest gates was provided in order to release surplus or flood water which cannot be contained in the allotted storage capacity. In addition, there was an intake structure with three gates at the left bank side of the dam for the purpose of releasing plant discharge to the power station. Therefore the spillway flow will occur mostly during floods or periods of sustained high run off when the capacity of the intakes are exceeded or when releasing discharge through the intake gates stops.

The basic hydraulic design requirements are given as follows ;

- The design flood of 8000 m^3/s can be released at HWL.85.00 by all the spillway gates opening.

- The stilling basin is designed to have enough capacity and dissipate the energy of flushing flow for the design flood discharge so as not to scour or undermine the downstream toe and the dam abutment.

To satisfy the above hydraulic design requirements, the spillway was planned to locate on the river bed six (6) blocks of the dam in consideration of geology, topography and stream line direction. This over flow type spillway has five (5) fixed wheel type gates 11.0m wide and 18.0m high which can regulate and control the flood flow from the reservoir. The spillway chute was constructed on the downstream surface of the dam which leads down to a stilling basin 35.0m below, 71.0 m wide, 29.0m high and 90.0 m long.

The discharge capacities, water surface profiles, and other requirements of the over flow spillway were studied through a hydraulic model test. The type and dimensions of the energy dissipator were determined also by the model studies.

For the selection of spillway gate numbers, the total weight comparison was made for the design flood discharge of 8000 m^3/s. The following table shows the number of spillway gates and their weight for different conditions. Five (5) gates 18.0 m high were found to be the most economical and was selected for the final design.

Item	Number of spillway gates		
	3 units	4 units	5 units
Crest Elevation	EL.60.50	EL.65.00	EL.67.50
Height of gate	25.0 m	20.5 m	18.0 m
Width of gate	11.0 m	11.0 m	11.0 m
Gate own weight per unit	220.4 ton	139.3 ton	10.39 ton
Total gate weight	661.2 ton	557.2 ton	519.5 ton

6.4.2 Hydraulic Design of Overflow Spillway

(1) Control Structure

The control structure was designed to release the design flood discharge safely and not generate negative pressures on the spillway surface.
The basic dimensions of the control structure configuration were determined as follows, by referring to design flood discharge of 8000m^3 / s.

Design flood water level	:	H.W.L. 85.000
Crest level	:	E.L. 67.500
Overflow width	:	B = 11 m x 5 spans = 55.0 m
Total pier width	:	b = 4 m x 4 units = 16.0 m
Width of control structure	:	W = B + b = 71.0 m
Design head	:	Hd = 85.0 m - 67.5 m = 17.5 m

(a) Crest Configuration.

The standard type of Harrold's shape which follows the ideal locus of flow without generating negative pressures on the overflow surface of the dam, was used for the basic hydraulic study of overflow discharge capacity. Therefore, in the crest configuration of the spillway, the standard type of Harrold's shape was applied for the upstream end to downstream face in the range $x < 0.5$ Hd and a circle curve line was inserted between the end of Harrold's shape and downstream face of the dam as Harrold's shape is used for the downstream face for $x < 0.5$ Hd. Based on the above basic shape, the final configuration of the overflow crest was determined by a comparative study considering the results of the hydraulic model test.

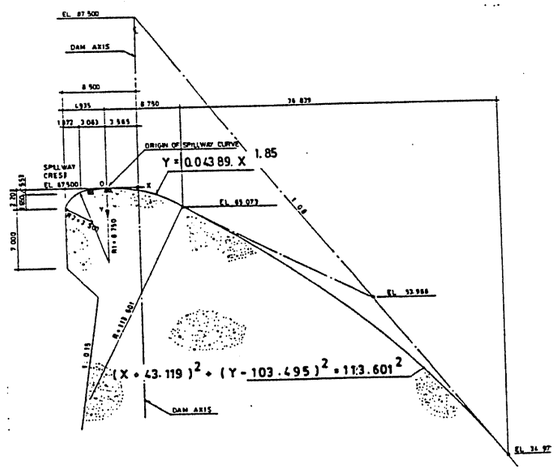
- The Harrold's shape is given as follows :

$$Y = 0.5 / Hd^{0.85} X^{1.85} = 0.043892\, X^{1.85} \qquad (0 \le X \le 8.75\ m)$$

- The circle curve line which passes through the downstream end of Harrold's curve and downstream face of dam is given as follows,

$$(X + 43.119)^2 + (Y-103.495)^2 = 113.601^2 \qquad (8.75m \le X \le 45.588m)$$

The crest configuration based on Harrold's shape is given as follow.



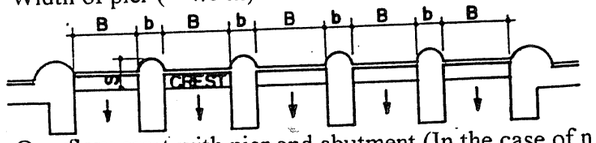
The crest configuration based on Harrold's shape

(b) Overflow Discharge Capacity

The overflow discharge capacity taking account of the influence of piers was calculated by means of the Ishii-Fujimoto's formula and Iwasaki's formula which are adopted by the River Bureau, Ministry of Construction of Japan.

$$Cd = 2.200 - 0.0416\,(Hd/W)^{0.990} = 2.1873$$
$$a = (Cd - 1.6) / (3.2 - Cd) = 0.5799$$
$$C = 1.60\,[1+1.1598\,(H/Hd)] / [1+0.5799\,(H/Hd)] = 2.1873$$
$$Md = 0.0756\,(Hd/B)^{0.5}\,\{1/n + 1.465\,(n-1)/n\,(b/s')^{1.7}\} = 0.0501$$
$$C' = C\,\{1-Md\,(H/Hd)^{1.5}\} = 2.0997$$

where,
- n : Number of spans (= 5)
- C' : Coefficient of overflow discharge taking into consideration the influence of the pier.
- B : Width of overflow at of each span (= 11.0 m)
- H : Overflow head above the crest (= Hd = 17.5 m)
- Hd : Design head (= 17.5 m)
- Md : Amount of compensation of the coefficient of overflow discharge in the case of design head, taking into consideration the influences of the pier and abutment.
- S' : Horizontal distance from the crest of dam to the upstream end of the pier (= 8.5 m)
- b : Width of pier (= 4.0 m)



Overflow crest with pier and abutment (In the case of n = 5)

- W : Dam height (= 58.0 m)
- a : Constant
- C : Coefficient of discharge
- Cd : Coefficient of discharge when H = Hd

As result, the discharge in the case of H= Hd is given as follows.

$$Q = n\,C'\,B\,H^{3/2} = 8365\ m^3/s > 8000\ m^3/s$$

Therefore, the design head of 17.5 m is sufficient to cope with the design flood discharge of 8000 m^3/s.

(2) Discharge Carrier.

The guide wall height corresponding to the design flood discharge 8000 m^3/s was determined from the profile of the gradually varied flow in the discharge carrier.
The pure water depth without air mixture at a point with an arbitrary depth y from the crest and the mean velocity (V = q/h) are calculated by integrating the differential equation of gradually varied flow.

$$dh/dx = (So-S_f) / \{As + d(v^2/2g)/dh\}$$

where,
- h : The depth of flow section
- dx : Elementary length of the channel
- So : The slope of the channel bottom (So = sin ⊖ = -dz/dx z : the vertical distance of the channel bottom).
- S_f : The energy slope (Sf = -dh/dx h: the total load)
- ⊖ : The bottom - slope angle
- As : The energy coefficient (assume = 1.0)
- V : The mean velocity of flow through the section

It is necessary to determine the height of the guide wall by taking into consideration the influence of factors such as the mixture of air in the channel in jet flow state, the influence of the splashes, etc. In this case, the freeboard △ h is calculated by means of the empirical formula of the U.S. Bureau of Reclamation as follows.

$$\triangle h\ (m) = 0.61 + 0.037\,Vh^{1/3}$$

where,
- V : Cross - sectional mean velocity (m/s)
- h : Water depth (m)

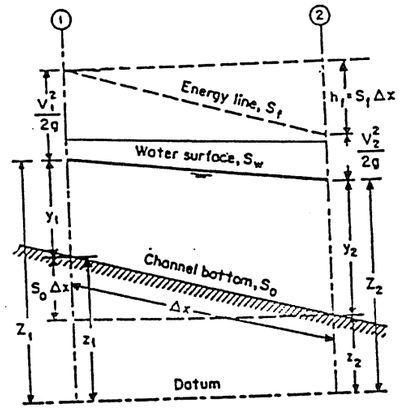
The direct step method was applied to the channel. The figure given below illustrates a short channel reach length △ x. Equating the total heads at the two end sections ① and ②, and solving for △x,

$$So\,\triangle x + y_1 + v_1^2/2g = y_2 + v_2^2/2g + S_f\triangle x$$
$$\triangle x = (E_2 - E_1) / (So - S_f) = \triangle E / (So - S_f)$$

In the above equations, y is the depth of flow, V is the mean velocity, So is the bottom slope, and S_f is the friction slope. When the Manning formula is used, the friction slope, Sf, is expressed by

$$S_f = n^2 v^2 / R^{4/3}$$

where , n : Manning's coefficient (0.013)
R : Hydraulic radius



As the bottom of the discharge carrier consists of three different shapes. The first derivative of the bottom line equations become as follows .

$0 \le x \le 8.75\ m$: $y = 0.04389\ x^{1.85}$, $dy/dx = 0.0812\ x^{0.85}$

$8.75 < x < 45.588\ m$: $y = 103.495 - \sqrt{113.601^2 - (x + 43.119)^2}$

$$dy/dx = (x + 43.119) / \sqrt{113.601^2 - (x + 43.119)^2}$$

$45.588\ m < X$: $y = 1.25\ x - 24.456$, $dy/dx = 1.25$

Simplifying the discharge used in this calculation, the discharge per meter width of the spillway was taken as given below.

$$q = Q / n.B = 145.45\ m^3/s$$

where, q : Discharge per a meter width
Q : Design discharge (= 8000 m^3/s)
n : Number of openings (=5)
B : Width of opening between 2 piers. (=11.0m)

The piers are not continuous to the lower part of the discharge carrier and the flow will not be uniform all over the bottom. In this calculation however, it is assumed that the discharge per a meter width q is equal throughout the discharge carrier.

The water depth at the top of the spillway crest is equal to the critical depth and calculated as follows.

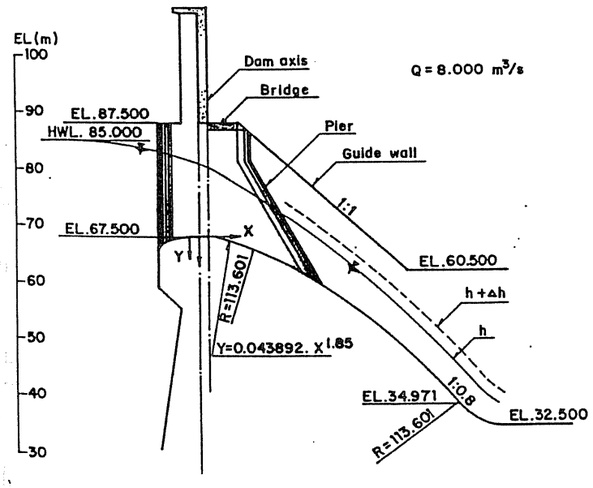
$$hc = (q^2/g)^{1/3} = 12.92m$$

where , hc : Critical depth (m)
q : Discharge per meter width (= 145.45 m^3/s)

The results of the calculation are shown in the following table and figure. In the result however, the water depth above of the crest (hc = 12.92 m) does not take into consideration the influence of the pier and other relevant factors.

Flow Profile at the Discharge Carrier for Design Flood 8000 m^3/s

Height from the crest y (m)	Water depth h (m)	Velocity V (m/s)	Free board Δ h (m)	h + Δ h (m)
0.0	12.92	11.26	1.59	14.52
5.0	7.91	18.38	1.96	9.88
10.0	6.75	21.55	2.12	8.87
15.0	6.05	24.04	2.23	8.28
20.0	5.50	26.45	2.34	7.48
25.0	5.19	28.03	2.41	7.60
30.0	4.85	29.99	2.49	7.34
35.0	4.72	30.82	2.52	7.24



Water surface Configuration at the Discharge Carrier.

The mixture of air due to the development of a boundary layer of turbulent flow was examined next. The occupance of white turbidity in the flow beyond a certain point downstream of the dam indicates that the boundary layer of turbulent flow has reached the water surface at this point and is therefore carrying air in the flow. The equation given by USCE WES was used for the calculation of the development process of a boundary layer in the flow section. In general, a roughness height of the order of k = 0.2 mm is assumed to be appropriate for the discharge carrier.

$$\delta = 0.08\, L\,(L/K)^{-0.233} = 0.25\ m$$

where , δ : Boundary layer depth
L : Flowing distance from start point of overflow crest surface (=59m)
K : Roughness height (= 0.2 mm)

The water depth at the downstream end of the discharge carrier is 4.72 m when the discharge is 8000 m^3/s and therefore the boundary layer does not appear on the surface before the flow reaches the downstream of the discharge carrier. As a consequence, it is clear that there is no flow mixed with air in this case.

It is concluded that a minimum side wall height of about 7.5 m is required at around the end of the straight section of the discharge carrier.

(3) Energy dissipator

The hydraulic design of the energy dissipator was made based on a typical hydraulic jump type dissipator with horizontal apron which is commonly used for high-dam spillways and for large canal structures.

As the structure of energy dissipator would not be destroyed by the design flood discharge of 8000 m^3/s, the hydraulic design was done for the 80% of the design flood discharge that is 6400 m^2/s, by referring to the "Criteria for River Control Structure Bureau, Ministry of Construction of Japan, Construction of Multi Purpose Dams, Vol.III Addenda, 1987". However, the guide wall height of the chute way and stilling basin were designed for the design flood 8000 m^3/s in order to have adequate height for overtopping.

(a) Calculation of the Hydraulic Dimensions at Upstream end of apron and Hydraulic jump on horizontal apron.

The calculations of each inflow discharge and hydraulic jump on the horizontal apron were carried out by assuming that the jet flow continues at the upstream end of the apron, instead of becoming submerged.

The rapid - flow depth h_1 at the upstream of the apron was calculated by the following equation.

$$h_1 = q/\sqrt{2g(W+H-h_1-h_f)} = 2.99\ m$$

$$hf = 0.02\ W\,(W/H + 1) = 2.10\ m.$$

where , W : Height from the apron to the crest of the dam (35.0m)
H : Water head of overflow (85.0 - 67.5 = 17.5 m)
q : Discharge per unit width (=8000 x 0.8 / 71.0 = 91 $m^3/s/m$)
h_f : Frictional loss

The following equation is applicable in the case of a hydraulic jump on the horizontal apron.

$$h_2/h_1 = 1/2.(\sqrt{1+8F_1^2}-1)$$

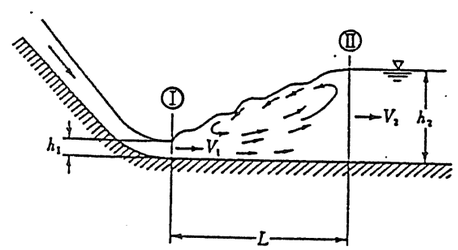
where , h_1 : Jet flow depth (2.99 m)
h_2 : Ordinary flow depth (sequent depth) (m)
F_1 : Fraud number of jet flow ($F_1 = V_1/\sqrt{g_{h1}} = 5.62$)
V_1 : Mean velocity of jet flow before hydraulic jump (30.43 m/s)

Then the ordinary flow depth h_2 was obtained.

$$h_2 = h_1/2\ (\sqrt{1+8F_1^2}-1) = 22.32\ m$$

The length of jump L_1 is given by the type of stilling basins classified by USBR. (See Table 1.6.4-1).

For the type I, the length become $L_1 = 6.06\ h_2 = 135$ m



Hydraulic jump of stilling basin

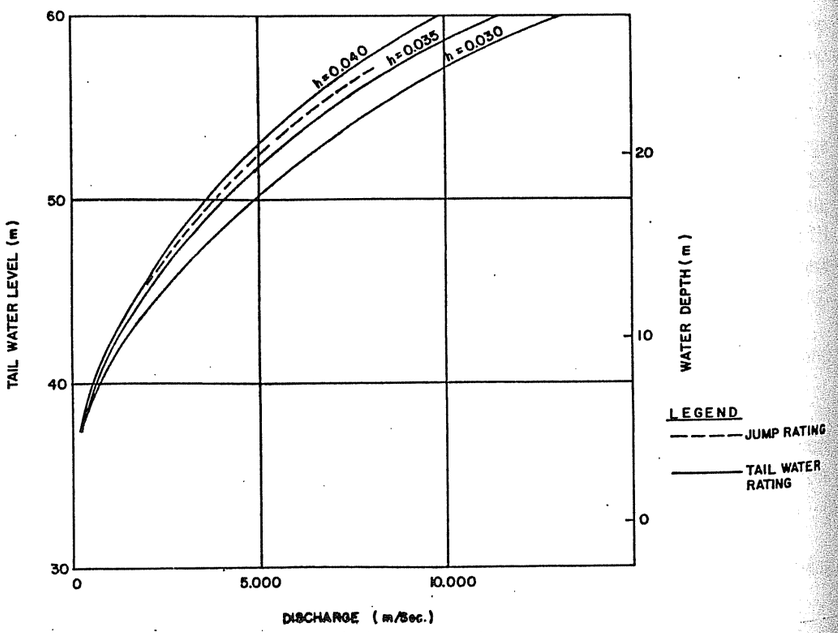
(b) Relation between Tailwater and Jump Ratings

The relationships "Water depth versus water area" and " Water depth versus wetted perimeter" for the most representative cases were calculated from the river cross section diagrams of 6 sites in the vicinity of the dam site. The roughness coefficient of the river is assumed about n = 0.035, however, the tail water rating curves for roughness coefficient n = 0.03, 0.035 and n = 0.04 were also calculated.

The obtained results are shown in the table and figure below. From the above study, the rating curve of n = 0.035 practically coincides with the hydraulic jump curve and therefore the stilling basin provided with an apron proved to be appropriate.

Relation between Water Level and Discharge of Natural River.

Water level	Water depth	Water area	Wetted peri-meter	Hydraulic radius	n = 0.030		n = 0.035		n = 0.040	
					V	Q	V	Q	V	Q
H (m)	H (m)	A (m)	P (m)	R (m)	(m/s)	(m^3/s)	(m/s)	(m^3/s)	(m/s)	(m^3/s)
37.5	5.0	200	74	2.76	1.67	334	1.43	286	1.25	250
40	7.5	350	80	4.38	2.30	806	1.97	691	1.73	605
42	9.5	480	85	5.65	2.73	1,310	2.34	1,123	2.05	983
44	11.5	650	91	7.14	3.19	2,074	2.73	1.778	2.39	1,556
46	13.5	810	98	8.26	3.52	2,849	3.01	2.441	2.64	2,136
48	15.5	1,000	106	9.43	3.84	3,842	3.29	3,292	2.88	2,881
50	17.5	1,200	116	10.34	4.08	4,902	3.50	4,200	3.06	3,676
52	19.5	1,420	128	11.09	4.28	6,078	3.67	5,208	3.21	4,558
54	21.5	1,670	145	11.52	4.39	7,331	3.76	6,283	3.29	5,498
56	23.5	1,970	160	12.31	4.59	9,039	3.93	7.747	3.44	6,780
58	25.5	2,280	172	13.26	4.82	10,993	4.14	9,421	3.61	8,245
60	27.5	2,590	182	14.23	5.05	13,090	4.33	11,217	3.79	9,817



Hydraulic Jump Curve and Rating Curve

(c) Type of Stilling Basin

From intensive studies of many existing structures and laboratory investigations, various types of generalized designs of stilling basins have been developed by the U.S Bureau of Reclamation. The basin's details and characteristics are described in Table 1.6.4-1. USBR basin I is the basin created by a jump occurring on a flat floor with no appurtenance. This can be designed easily but it is not very practical because of its expensive length and its lack of control.

USBR basin III indicates that the jump and basin length can be reduced about 60% with the addition of chute blocks, baffle piers and a solid end sill. Therefore, the basin is shorter and more economical, but in consequence, has a lower safety factor. This type is also usually used on small spillways. Basin IV is not suitable for this site as described in the table.

From the above reason, USBR basin II was selected for the stilling basin in common use for high dam spillways and for large canal structures.

The basin contains chute blocks at the upstream end and a dentated sill near the downstream end. No baffle piers are used because the relatively high velocities entering the jump might cause cavitation on the piers.

Referring to Fig. 1.6.4-1 we have $L = 4.0\ h_2$ with $F_1 = 5.62$ ($Q = 6400\ m^3/s$) and therefore the length of the horizontal apron is calculated as follows :

$$L_{II} = 4.0 \times 22.32 = 89.28\ m$$

The dimensions of the energy dissipators for this dam are calculated in conformity with the figure below and are shown in Fig. 1.6.4-2.

Recommended Proportions (Basin II)

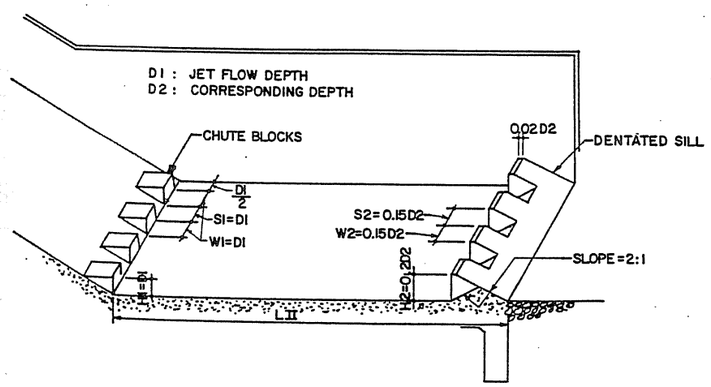
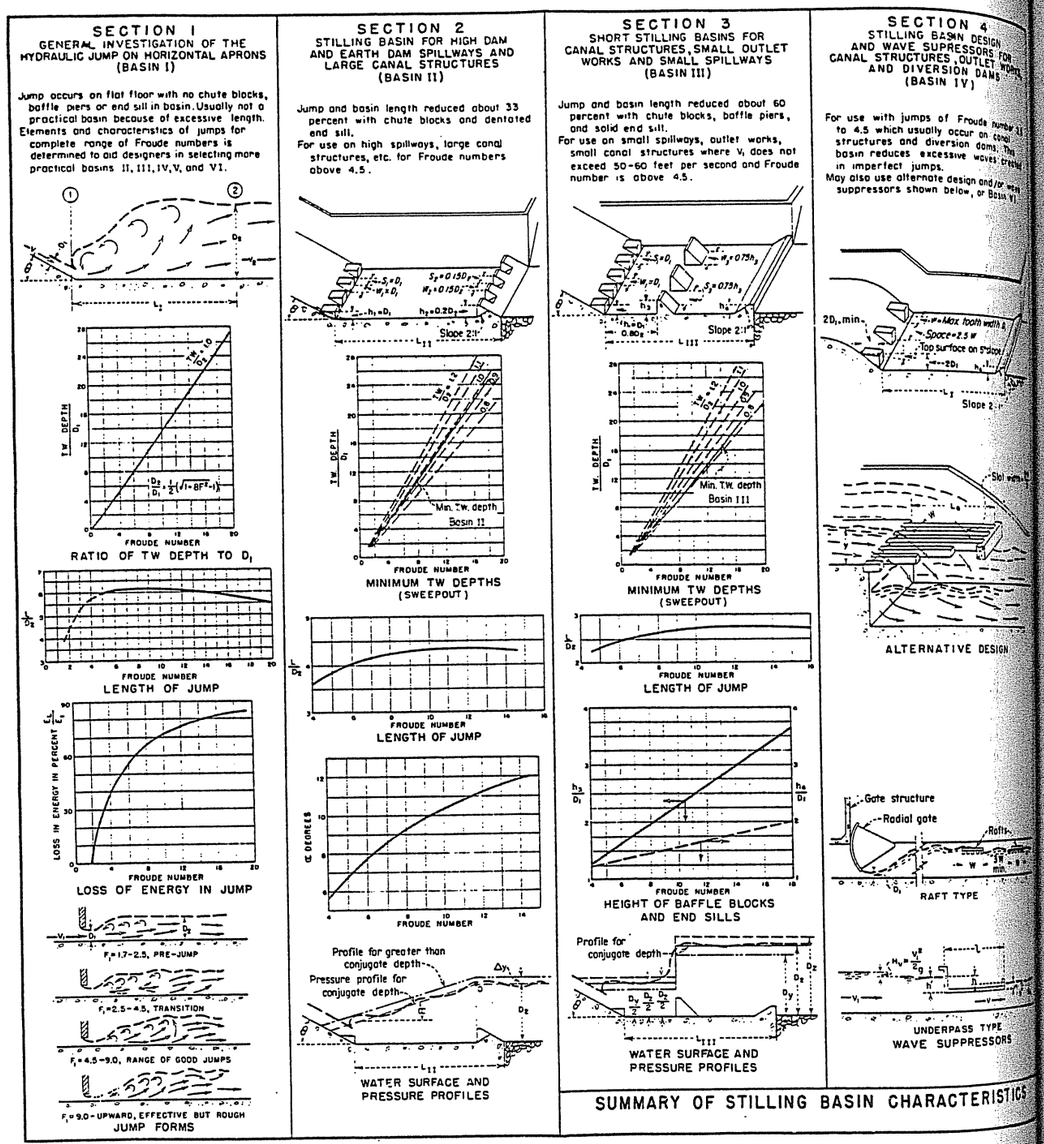


Table 1.6.4-1 Type of Stilling Basin



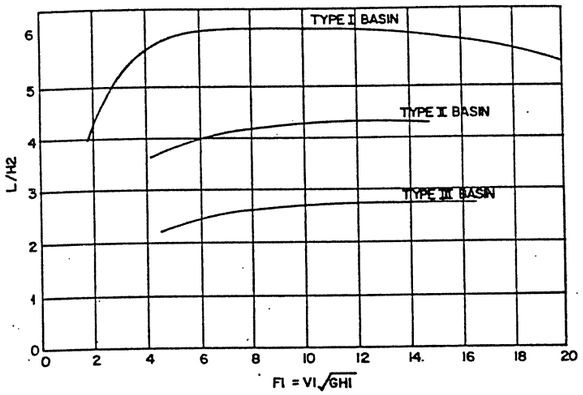


Fig. 1.6.4-1 Length of Jump on Horizontal Channel

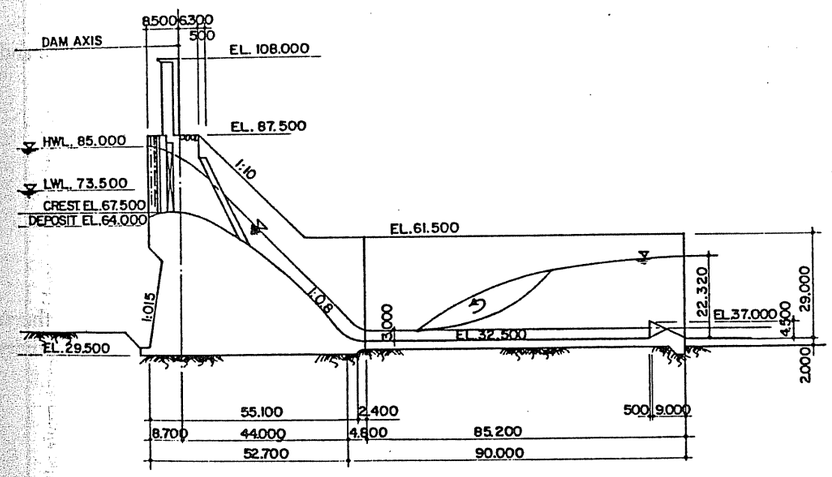


Fig. 1.6.4-2 Energy Dissipator

(d) Guide wall Height (Retaining wall)

The freeboard of the guide wall of the energy dissipator Δh is given by means of the following empirical formula of the US Bureau of Reclamation.

$$\Delta h = 0.1 (V_1 + h_2)$$

The required guide wall height Hs is calculated by the following expression, which is obtained by applying the aforesaid empirical formula to the case of Q = 6400 m^3/s.

$$H_S = h_2 + \Delta h = 27.6 = 28m$$

Flow depth of energy dissipator for 8000 m/s

The rapid - flow depth h_1 at the upstream of the apron is calculated as follows.

$$h_1 = q/\sqrt{2g(W+H-h_1-h_f)} = 3.72\ m$$

where ; W : Height from the apron to the crest of the dam (35.0 m)
H : Water head of overflow (85.0 - 67.5 = 17.5 m)
q : Discharge per unit width (q = 8,000/71.0 = 112.6 $m^3/s/m$)
h_f : Frictional loss (2.10m)

From the following equation and V_1 = 30.27 m/s, F_1 = 5.01, the water depth after hydraulic jump h_2 was calculated as follows.

$$h_2/h_1 = 1/2(\sqrt{1+8F_1^2}-1)\ ,\ h_2 = 24.56\ m$$

Therefore, the length of horizontal apron L_{II}, the freeboard of the guide wall Δh and the required guide wall height Hs was calculated as follows.

L_{II} = $3.9 \times h_2 = 3.9 \times 24.56 = 95.8\ m$
Δh = $0.1\ (V_1 + h_2) = 5.48\ m$
H_s = $h_2 + \Delta h = 30\ m$

6.4.3 Selection of Gate Type.

A comparative study of gate types for the spillway was made to select the most economical gate taking account of hydraulic performance, safety, productivity of construction work and cost.

Design Conditions.

Design water level	EL 85.000
Spillway crest elevation	EL 67.500
Design flood discharge	8000 m^3/s
Design head	17.5 m
Number of gate	5 no
Dimension (width X height)	11m x 18m
Freeboard of gate	0.500m

Alternative types of gate.

The three gate types (i) Roller Gate with Single Leaf, (ii) Roller Gate with Double Leaf and (iii) Radial Gate were selected for the comparative study.

Result of Comparative Study.

Each type was examined for various aspects such as cost, structural characteristics, quantity of work, hydraulic performance, construction and safety. As explained in Table 1.6.4-2, the results show that the roller gate with a single leaf is suitable for the spillway gate at the Kotapanjang dam.

Table 1.6.4-2 Comparative Study of Gate Type

Item	Roller Gate (Single Leaf)	Roller Gate (Double Leaf)	Radial Gate
1. Cost (Ratio)	US $ 7,816.6 (100%)	US $8,509.8 (109 %)	US $ 10,845.3 (138 %)
2. Structure	Pier height is the highest among the alternatives. Length of the pier along stream is short. Loads to piers are transferred with a number of rollers. Gate structure is simple.	Upper piers are 8m shorter than single leaf roller gate. Grooves and gate structure is more complicated than single leaf.	Upper piers not required. Longer piers length along P.C rods is required to transfer loads from supporting pins to piers. Needs mucH attention to the construction joint of civil works and metal works.
3. Quantity Metal Works Civil Works Concrete Forms Reinforcement bar	 850 t 7,188m^3 5,965 m^2 720 t	 925 t 6,576 m^3 5,200 m^2 690 t	 1,100 t 14,472 m^3 9,720 m^2 1,200 t
4. Hydraulic Performance	Good	Good, Needs attention for water proof at grooves.	Excellent
5. Construction	Grooves are made by block out. Fabrication of gates can be carried out separately from installation.	Construction of grooves is much complicated.	Needs adjustment of civil work and foundation materials and needs much attention to prevent damage to the P.C. Sheath during concrete placement.
6. Gate Dimension and Limitation	Adopted for long span.	Adopted for high gate.	
7. Hoisting	Large motor is necessary as operation load is heavy.	Large motor is necessary as operation load is heavy.	Althought load and motor are relatively small, differential pulley block system is not applicable from structural condition of radial gate. Therefore wire rope becomes thick and hoisting equipment becomes heavy.
8. Safety	Reliable, as load is supported by a number of rollers.	Reliable, as load is supported by a number of rollers.	As entire load passes through pins, there is problems of distribution of loads uniformal to piers. When overtops, arms of gates will be damaged.
9. Result of Evaluation	Consisting a simple and economical structure. There are not many construction joints of civil works and metal works, therefore productivity of construction is high. Overtopping by misoperation of gates will not cause serious damage to the gate structure.	Structure is simple as a single leaf type, however, requires consideration of water proofing leaves. Therefore productivity of construction is lower than of a single leaf type. Cost is about 15% higher than of a single leaf type.	Quantity of construction work is higher and more complicated work is required. Cost is about 22% higher than that of a roller gate with a single leaf, so cost is the highest among this study. Overtopping by misoperation of gates shall be strictly avoided since it causes serious damage to the gate structure.

6.4.4 Modification by Hydraulic Model Test

The Hydraulic model test was made at the Hydraulic Laboratory of PLN LMK. The main purpose of the hydraulic model test was to verify whether the spillway energy dissipator which had been designed based on the hydraulic calculation demonstrated the expected effects of energy dissipation, and should any portion be corrected to obtain an optimum design of the spillway.

The following items were observed or measured in the model test.

(i) Energy dissipating effect of the stilling basin.
(ii) Scouring of river bed downstream of the apron.
(iii) Reservoir water level.
(iv) Water profile on the dam crest and the chute way.
(v) Water profile, velocity distribution and length of hydraulic jump in the stilling basin.
(vi) Pressure acting on the chute blocks and the dentated sills arranged on the apron.
(vii) Velocity distribution of downstream of the apron
(viii) Water level fluctuation in the tailrace.

For the determination of the similarity ratio between model and prototype Froude's law was employed, and the scale of the model was selected at 1 to 80.

Tests were carried out mainly for the discharge of 6400 m^3/s and 8000 m^3/s. However in the case of some items tests were also carried out for a discharge less than 6400 m^3/s.

The results of experiment show that the original design of spillway performs basically its function. The length of the stilling basin designed for a discharge of 6400 m^3/s was sufficient even for a discharge of 8000 m^3/s. However, the design should be modified on the basis of the results of the original and modified model tests.

(a) Guide wall height of chute way

The water profile along the left guide wall of the chute way for a discharge of 8000 m^3/s is shown in Fig. 1.6.4-3. It was decided necessary to raise the height of the chute 0.50m.

(b) Pier tail

The pier tail thickness caused disturbance in flow behind the tails. To reduce that disturbance it was decided necessary to modify the pier tail.

(c) Guide wall height of stilling basin

The water profile in the stilling basin for a discharge of 8000 m^3/s is shown in Fig. 1.6.4-4. It was decided necessary to increase the height of guide wall of the stilling basin 1.0 m.

(d) Protection of downstream of stilling basin

The results of the scouring test for the original design are shown in Fig. 1.6.4-5. The results show that quite deep scouring occurs just downstream of the stilling basin near the end of tailrace wall. Moreover there was the deepest scouring along the projection at the right side of the river.

Therefore, the scouring test was carried out for a modified model that had additional armored concrete just downstream of the stilling basin and additional crib along the projection of the right side of river or the projection cutting. The results of the scouring test for this modified model are as shown in Fig. 1.6.4-5. From this results, the cutting for the hydraulical projection would be best for preventing scouring along the right side of the river.

Based on the above consideration, the optimum dimensions of spillway were determined.

After the hydraulic model test, it was decided the countermeasures recommended to prevent scouring would not to be carried out for the following reasons.

(i) For tailrace foundation.

The geological features of the river bed at the downstream of the stilling basin were investigated and finding that the bed rock at the river bed has sufficient durability against scouring as followings.

- Solidity : hard to moderately hard.
- Discontinuity spacing : more than 5 m
- Tightness : Closed - open (predominantly)
- Rock - classification : CH (mostly) - CH (localized).

(ii) For right side of the river.

- This kind of scouring did not seem to endanger the main structure of this project, (the dam or tailrace) because the scoured places were distant from these main structures.

- The geological condition of the projection at the right bank of the river was investigated to be hard or moderately hard in solidity. Then it should be difficult to scour the projection itself and become unstable.

- Construction costs of the countermeasures were considered too expensive for the permanent structure against a design flood expected once in several hundred years.

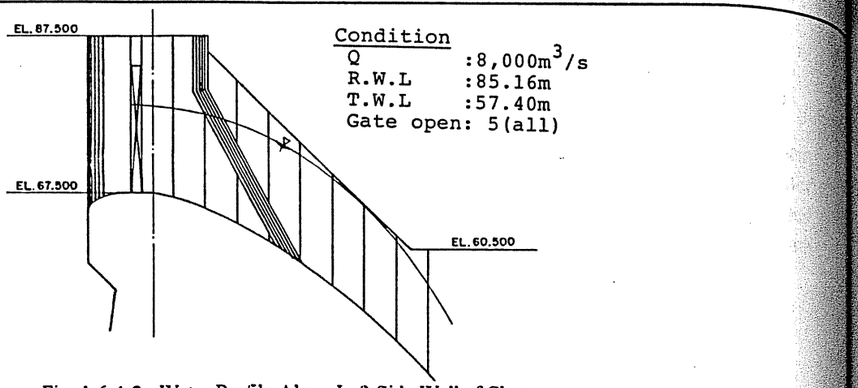


Fig. 1.6.4-3 Water Profile Along Left Side Wall of Chute way

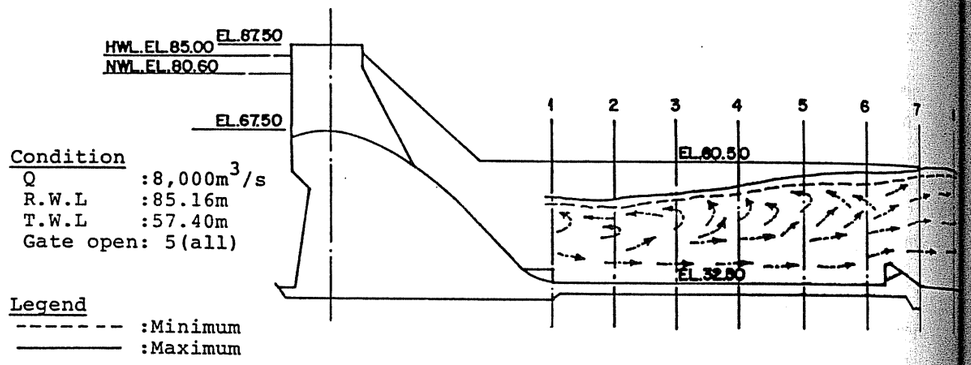


Fig. 1.6.4-4 Water Profile in Stilling Basin

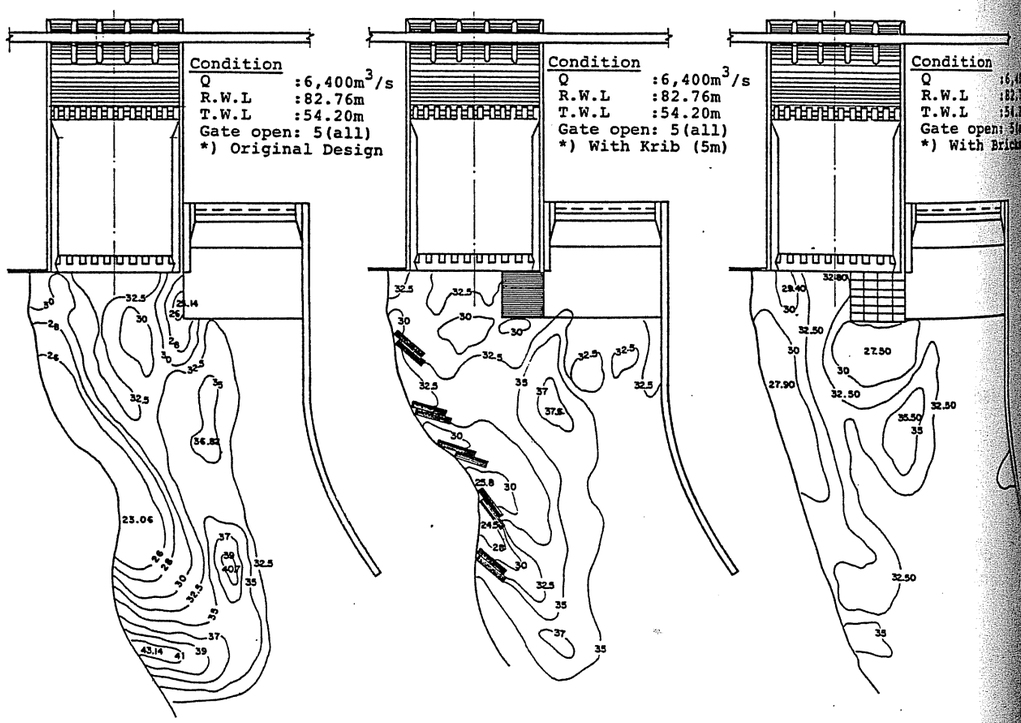


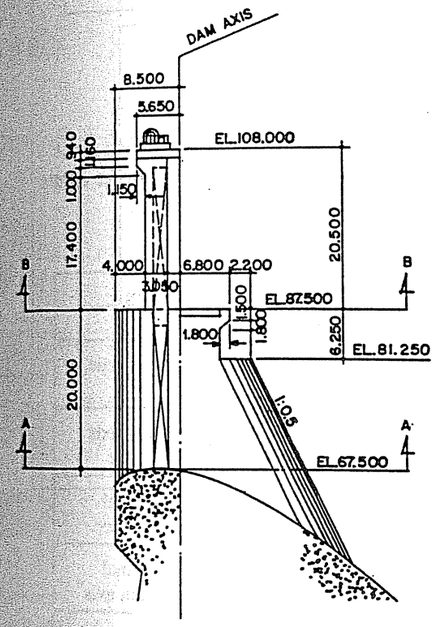
Fig. 1.6.4-5 Scouring Pattern

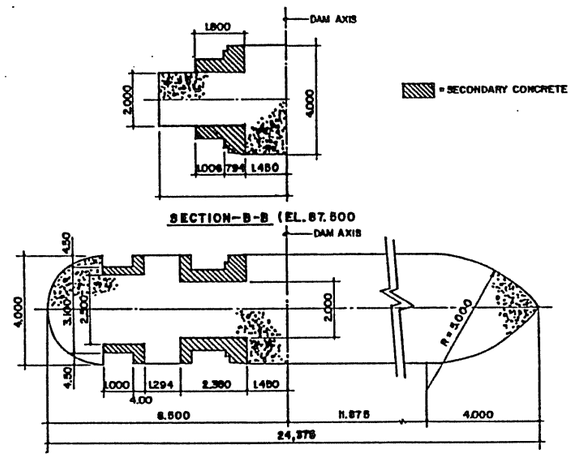
6.4.5 Structural Design

(1) Spillway Piers

(a) Design condition

General view of structure





Design Load

Unit weight	Reinforced concrete	2.40 t/m³
	Water	1.00 t/m³
Earthquake force	Horizontal seismic intensity	0.10
Water level	High water level	EL. 85.000
	Low water level	EL. 73.500
Working Load - Spillway bridge	Own weight (Normal)	109.7t/one support
	(Earthquake)	55.7 t/one support
- Hoist deck	Own weight	83.5 t/one support
	Hoisting machine weight	15.1 t/one unit
	Roller turning and resistance of friction	46.15 t/one unit
	Spillway Gate weight	76.7t/one gate
- Water pressure	Gate open condition	1287.51 t
	Gate close condition	680.63 t

Allowable stress intensity

(Unit : Kg/cm²)

	Item		Long Period	Short Period
Concrete	Allowable compressive stress intensive due to bending	σ ca	60	90
	Allowable shear stress intensity	τ a	8	12
	Allowable bond stress intensity	τ oa	14	21
Reinforcement	Allowable tensile stress intensity	σ sa	2000	3000

(b) Structural Calculation

Calculations were carried out for one of the centre piers as a representative case since both side piers were more stable than the centre piers. The calculation was done as a cantilever.

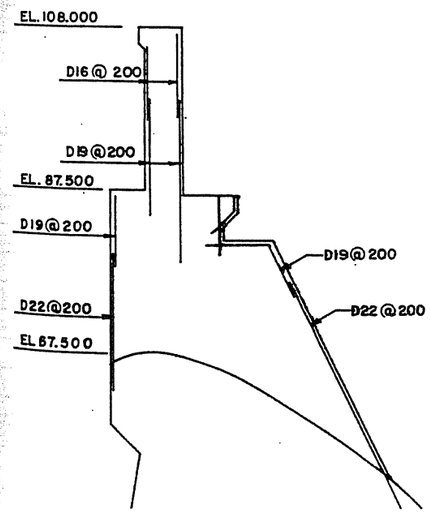
Calculation case

Case No	Cross section	Water level	Earthquake load	Gate condition	Remark
1	A - A	H.W.L	None	One gate open One gate close	Stability of dam axis direction
2		H.W.L	Along dam axis	One gate open One gate close	Stability of dam axis direction
3		L.W.L	To upstream	Two gates hoisting	Stability of normal direction to dam axis
4		H.W.L	To downstream	Two gates hoisting	Stability of normal direction to dam axis
5	B- B	L.W.L	Along dam axis	Two gates hoisting	Stability of dam axis direction
6		L.W.L	To downstream	Two gates hoisting	Stability of dam axis direction

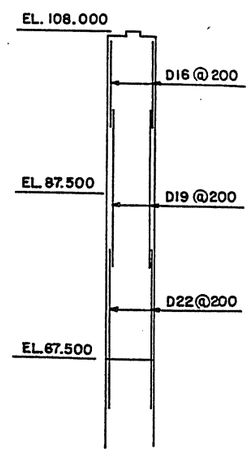
Calculation of reinforcement

Cross Section	Case	Cross sectional force	Required Reinforcement (per unit width)	Distribution of Reinforcement	Stress Intensity (kg/cm²)	Remark
A - A	1	M = 1818.91 t.m Q = 606.88 t N = 3984.76 t	1.00 cm² below	D 22 @ 200 (As=19.4 cm²)	σ s = 1587 σ c = 43.6 τ = 1.3 τ o = 3.7	< σ s a < σ c a < τ a < τ o a
	2	M = 7398.05 Q = 1003.79 N = 3930.76	8.54 cm²			
	3	M = 17652.59 Q = 324.91 N = 3969.11	1.00 cm² below	D 19 @ 200 (As=14.3 cm²)	σ s = 1585 σ c = 58.1 τ = 0.7 τ o = 2.4	< σ s a < σ c a < τ a < τ o a
	4	M = 1454.97 Q = 1269.13 N = 3969.11	1.00 cm² below			
B - B	5	M = 1068.01 Q = 89.50 N = 881.63	5.3 cm² below	D19 @ 200 (As=14.3 cm²)	σ c = 1585 σ c = 58.1 τ = 0.7 τ o = 2.4	< σ s a < σ c a < τ a < τ o a
	6	M = 718.15 Q = 81.83 N = 881.63	1.00 cm² below	D19 @ 200 (As=14.3 cm²)		

Reinforcement distribution



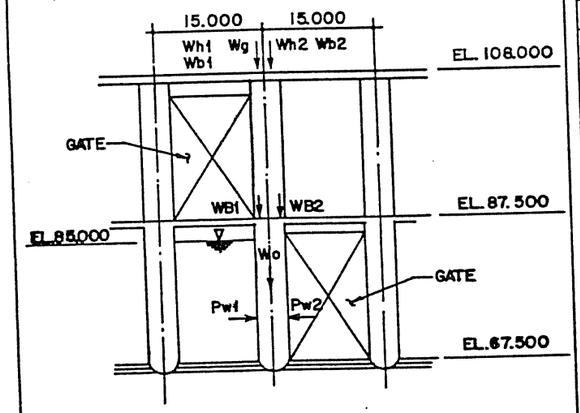
Normal direction to dam axis



Dam axis direction

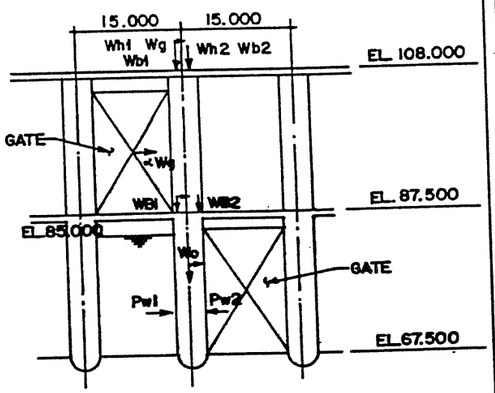
Load Condition	Total of external force and dead load

Case 1



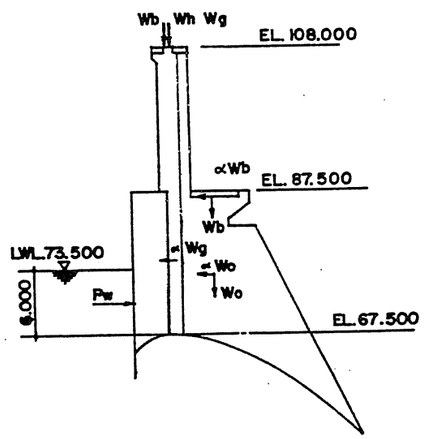
	W(t)	H(t)	X (m)	Y (m)	W.X (t.m)	H.Y(t.m)
Wg	38.35		-0.750		-28.76	
Wh1	7.55		-0.750		-5.66	
Wh2	7.55		0.750		5.66	
Wb1	83.50		-0.750		-62.63	
Wb2	83.50		0.750		62.63	
WB1	109.70		-1.750		-191.98	
WB2	55.70		1.750		97.48	
Pw1		1287.51		4.416		5681
Pw2		-680.63		5.500		-3743
Wo	3598.91		0		0	
Total	3984.76	606.88			-123.26	1937

Case 2



	W(t)	H(t)	X (m)	Y (m)	W.X (tm)	H.Y(tm)
Wg	38.35	7.67	-0.750	29.000	-28.76	221.1
Wh1	7.55	1.51	-0.750	40.500	-5.66	61.6
Wh2	7.55		0.750		5.66	
Wb1	83.50	16.70	-0.750	39.350	-62.63	657.0
Wb2	83.50		0.750		62.63	
WB1	55.70	11.14	-1.750	19.327	-97.48	215.3
WB2	55.70		1.750		97.48	
Pw1		1287.51		4.416		5681
Pw2		-680.63		5.500		-3743
Wo	3598.91	359.89	0	12.022	0	4326
Total	3930.76	1003.79			-28.76	7438

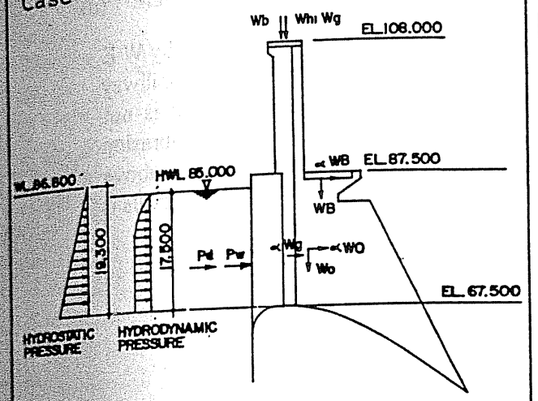
Case 3



	W(t)	H(t)	X' (m)	Y'(m)	W.X'(tm)	H.Y'tm)
Wg	76.70	7.67	6.421	9.000	492.49	69.0
Wh	15.10	1.51	6.421	40.500	96.96	61.1
Wb	167.00	16.70	6.596	39.350	1101.53	657.1
WB	111.40	11.14	1.271	19.327	141.59	215.3
Pw		-72.00		2.000		-144.0
Wo	3598.91	359.89	2.955	12.022	10634.78	4326.6
Total	3969.11	324.91			12467.35	5165.0

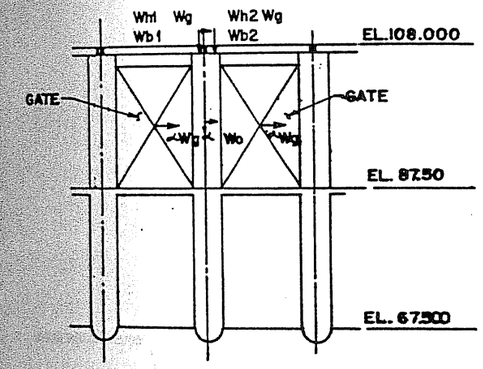
Load Condition	Total of external force and dead load

Case 4



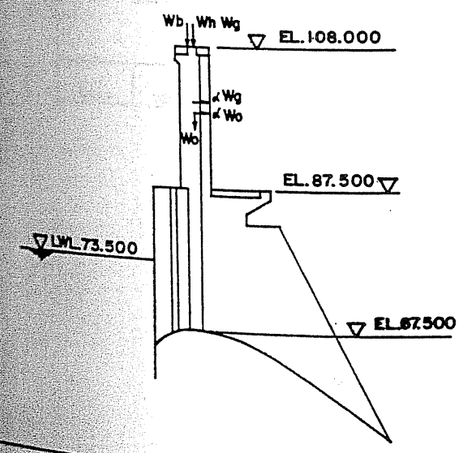
	W(t)	H(t)	X' (m)	Y' (m)	W.X'(tm)	H.Y'(tm)
Wg	76.70	-7.67	6.421	9.000	492.49	-69.03
Wh	15.10	-1.51	6.421	40.500	96.96	-61.16
Wb	167.00	-16.70	6.596	39.350	1101.53	-657.15
WB	111.40	-11.14	1.271	19.327	141.59	-215.30
Pw		-744.98		6.433		-4792.46
Pd		-127.24		7.000		-890.68
Wo	3598.91	-359.89	2.955	12.022	10634.78	-4326.60
Total	3969.11	1269.13			12467.35	-11012.38

Case 5



	W(t)	H(t)	X (m)	Y (m)	W.X(tm)	H.Y(tm)
Wg	76.70	15.34	0	9.000	0	138.06
Wh1	46.15	2.90	-0.750	20.500	-34.61	59.45
Wh2	46.15		0.750		34.61	
Wb1	83.50	16.70	-0.750	19.350	-62.63	323.15
Wb2	83.50		0.750		62.63	
Wo	545.63	54.56	0	10.032	0	547.35
Total	881.63	89.50			0	1068.01

Case 6



	W(t)	H(t)	X'(m)	Y'(m)	W.X'tm)	H.Y'tn)
Wg	76.70	-7.67	0.679	9.000	52.08	-69.03
Wh	92.30	-2.90	0.679	20.500	62.67	-59.45
Wb	167.00	-16.70	0.854	19.350	142.62	-323.15
Wo	545.63	-54.56	0.043	10.032	23.46	-547.35
Total	881.63	-81.83			280.83	-998.98

Wg : Spillway gate weight
Wh1, Wh2 : Load of hoisting machine or roller turning and resistance friction
Wb1, Wb2 : Load of hoist deck
WB1, WB2 : Load of spillway beridge
Pw1, Pw2 : Water pressure
Wo : Body own weight
Pd : Hydrodynamic pressure

(2) Reinforcement of Overflow Section

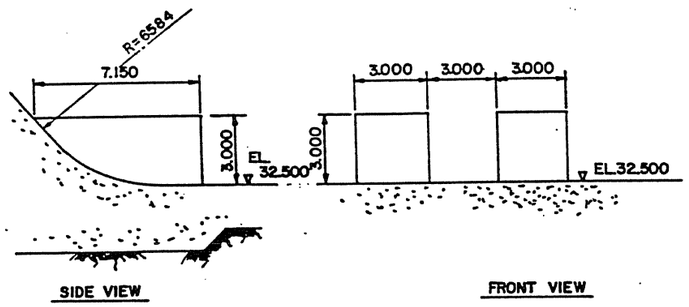
On the basis of the study data carried out both theoretically and experimentally by W E S (US Army Engineers) and U S B R, negative pressures on the Kotapanjang spillway crest under the design pressure was very small and therefore reinforcement is not necessary. However, reinforcement is provided in order to protect the scaling and abrasion due to the impact of the fluid flow considering safety design. From the above consideration, the distribution of reinforcement bar of 19 mm dia at 300 mm intervals is provided.

(3) Chute block and Dentated sill

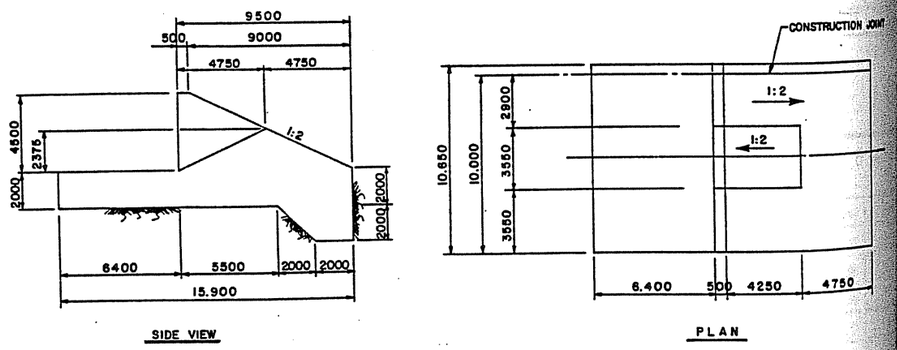
(a) Design condition

(i) General view of structure.

Chute blocks



Dentated sill



(ii) Design Load

Own weight	:	Reinforced concrete	2.40 t/m^3
		Water	1.00 t/m^3
Foundation	:	Shear strength	180 t/m^2
Property	:	Coefficient of internal friction	0.78
Water level	:	Flood state	EL. 54.82

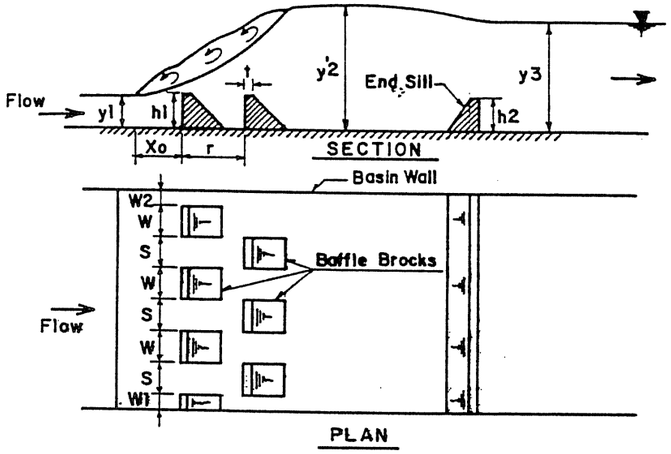
(b) Stability of end sill.

(i) Fluid water pressure.

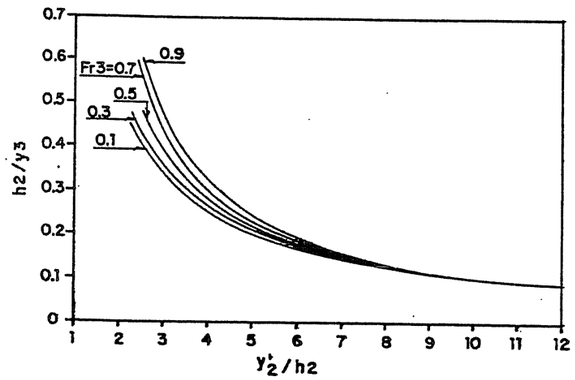
Fluid water pressure acting on the dentated sill is given in the following equation.

$$FB_2 = [(y_2'/y_3) - 1 - 2\,(Fr_3)^2\,(1 - y_3/y_2')] \,.\, F_3 = 4.198 \text{ t/m.}$$

where, FB_2 : Fluid water pressure per unit width acting on the dentated sill
F_3 : $F_3 = 1/2\ r y_3^2$ (= 285.605 t/m)
r : Unit water weight (=1.0 t/m^3)
y_3 : Water depth at downstream of dentated sill (=23.9 m)
y_2' : Water depth at upstream of dentated sill after hydraulic jump (=24.3 m from the variation of y_2'/h_2 with h_2/y_3 for entated sill)
Fr_3 : Froude number (= 0.249)



The source : Journal of Hydraulic Research 18 (1980) No.3



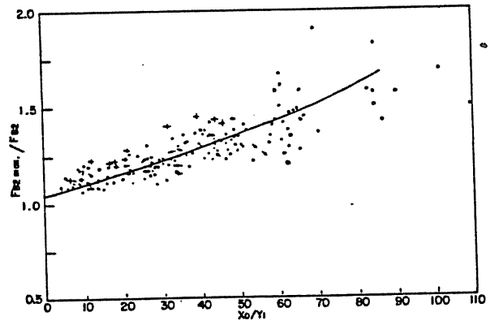
Variation of y2'/h2 with h2/y3 for end sill

From the figure below,

Xo/y1 = (85.20 - 9.50) / 2.99 = 25.3 then

F_{B2} *max* / F_{B2} = *1.2*

$\therefore$ F_{B2}*max = 1.2 x 198 = 5.0 t/m*



Relationship for maximum force on a trapezoidal baffle block

Hence, the fluid water pressure acting on the dentated sill is estimated to be 5.0 t/m. The acting point of fluid water pressure was taken conservatively at 60 % height of the dentated sill as the velocity distribution is not uniform.

(ii) Stability calculation

Total external force and own weight

	Vertical force (t)	Horizontal Force (t)	Distance from Coordinate (m)	Moment due to force (t/m)	Moment of Resistance (t/m)
Own weight	1351.62	-	6.622	-	8950.43
Fluid flow	-	-50.00	6.700	-335.00	-
Buoyancy	-563.18	-	6.622	-3729.38	-
Total	788.44	50.00		-4064.38	8950.43

Safety factor against overturning = Mr/Mo = 2.2 > 1.2
Safety factor against sliding = (v.f + τ ℓ) /H = 584 > 4.0

The calculation value of safety factor is satisfied with the minimum required safety factor.

(c) Structural calculation of chute block and dentated sill

(i) Design condition

Fluid water pressure acting on the chute block is given in the following.

$P = K.V^2.A$

where ; P : Fluid water pressure
K : Coefficient depend on the shape of structure (=0.07)
V : Maximum velocity (V = 30.43 m/s is given from hydraulic study. Refer to chap 6.4.3)
A : Vertical projection area in fluid flow

(The source : Road Bridge Guidance by Japan Road - Association)

Fluid water pressure acting on the end sill was obtained in the stability calculation taken here as the design load.

Allowable stress intensity

Concrete allowable compressive stress due to bending	σca = 90 kg/cm²
Steel tensile stress	σsa = 3,000 kg/cm²

(ii) Structural calculation

For the chute block, the fluid water pressure will act parallel the downstream face with slope 1 : 0.8, the pressure P acting on one block and the horizontal component of force Ph is,

$P = K.V^2.A = 747\ t$
$Ph = P \times \cos 51.34^o = 467\ t$

For the dentated sill, the force acting on one dentated sill and the bending moment are,

$F1 = 5.0\ t/m \times 3.55\ m = 17.75\ t.m$
$M = 17.75 \times 2.70 = 47.93\ t/m$
$N = 0$
$Q = 17.75\ t$

(iii) Reinforcement distribution

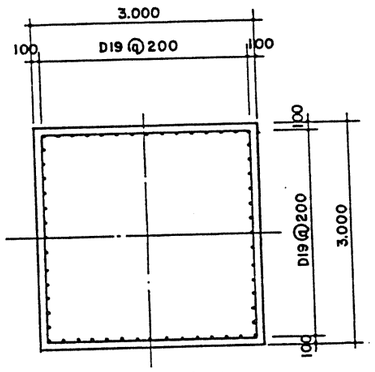
- Chute block

Reinforcement will be provided for the tensile force 467 t.
Required reinforcement As = 467x10³/3000 = 155.67 cm².

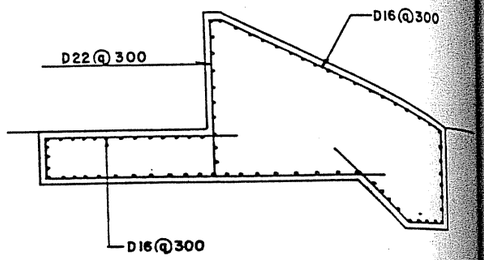
Therefore, the distribution is calculated that in the case of D19 @ 200, total reinforcement As = 2.865 cm² / piece x 56 pieces = 160.44 cm² > 155.67 cm².

- Dentated sill

From the section force obtained in the proceeding calculation, the reinforcement is not necessary for the dentated sill. However, the following reinforcement was provided considering past construction experience to be on the safe side.



Chute blocks

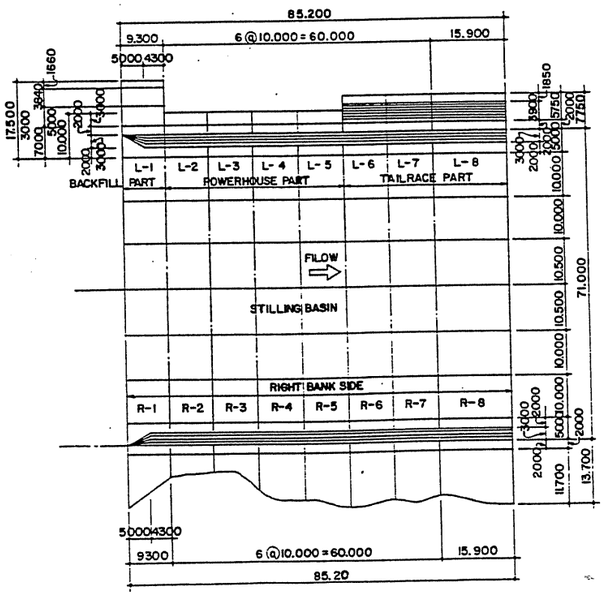


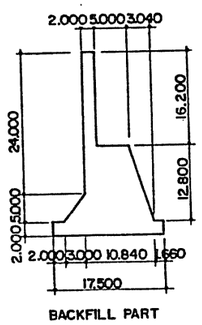
Dentated sill

(4) Guide Wall of Stilling Basin

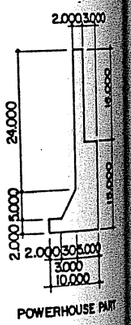
(a) Design Condition

(i) General Plan

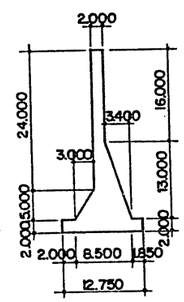




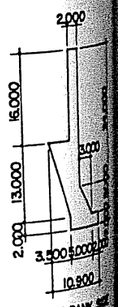
BACKFILL PART



POWERHOUSE PART



TAILRACE PART



RIGHT BANK SIDE

(ii) Design Load

Unit weight	:	Reforced concrete	2.40 t/m³
		Water	1.00 t/m³
		Soil	1.80 t/m³
Eartquake force	:	Horizontal seismic intensity (to act on left bank to right bank)	0.10
Water level	:	Normal and Earthquake	41.00 m
		Flood	54.82 m
Working Load	:		
- Earth pressure	:	Normal and Flood state	53.01 t
		(Horizontal component)	43.42 t
		(Vertical component)	30.41 t
		Earthquake state	62.84 t
		(Horizontal component)	47.63 t
		(Vertical component)	40.99 t
- Surcharge load	:	Unifrom load	1.00 t/m²

Note : Coulomb`s method is used be evaluate the earth pressure.

- Normal condition

Active earth pressure $P_A = 1/2\ K_A.r.H^2$, $P_H = P_A \cos\varphi$, $P_V = P_A \sin\varphi$

where, K_A : Coefficient of active earth pressure.

$$\cos^2\varphi/\cos\alpha[\ 1+\sqrt{\sin(\varphi+\delta)\ \sin(\varphi-\theta_o)/\cos\alpha.\ \cos\theta_o)}]^2$$

r : Unit weight of soil of backfill material (=1.8 t/m²)
H : Assumed vertical height of soil (= 15.36m)
α : Angle between earth pressure and back face of the wall (=35°)
δ : Friction angle between soil and back of wall (=35°)
φ : Internal friction angle of soil (=35°)
θ_o : Inclined angle of grand surface to horizon (=0)

- Earthquake condition

$P_E = (1 - kv).1/2.k_{AE}.r.H^2$, $P_{EH}=P_E \cos(\varphi + \theta)$, $P_{EV}=P_E \sin(\varphi+\theta)$

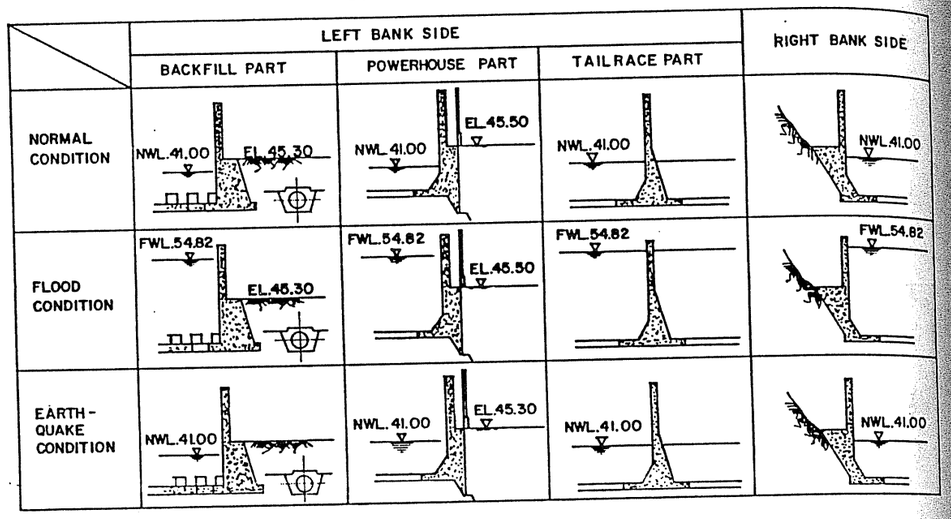
where, K_{AE} : Coefficient of active earth pressure at earthquake.

$$= \cos^2(\varphi-\theta)/\cos\theta.\cos(\delta+\theta)\ [1+\sqrt{\sin(\varphi+\delta)\ \sin(\varphi-\theta-\theta_o)/\cos(\delta+\theta)\ \cos\theta_o)}]^2$$

kv : Vertical seismic intensity (=0)
θ : $\mathrm{Tan}^{-1} K$ (= Tan^{-1} 0.10 = 5.7106)
k : Resultant seismic intensity [= kh/ (1-kh) = 0.10]
kh : Horizontal seismic intensity (=0.10)

(b) Stability of guide wall.

Calculation cases.



Result of stability calculation

Part	Condition	Vertical Force (t)	Horizontal Force (t)	Total Moment (t.m)	e (m)	Safety Factor for Sliding	Bearing Stress (t/m²)
Back fill	Normal	509.80	-11.71	5296.14	1.639 < 17.5/6 = 2.916	60.8 > 1.5	45.50 < 180
	Flood	447.48	-252.31	6839.46	6.534 < 17.5/2 = 8.750	2.6 > 1.1	12.76 134.62 < 180
	Earth-quake	509.88	36.08	4935.94	0.931 < 17.5/3 = 5.833	19.7 > 1.2	0.00 38.43 < 180 19.00
Power house	Normal	281.30	-	1844.09	1.556 < 10.0/6 = 1.666	-	54.39 < 180
	Flood	281.30	-	1786.29	1.351 < 10.0/2 = 5.000	-	1.87 50.93 < 180
	Earth-quake	281.30	29.88	1523.66	0.416 < 10.0/3 = 3.333	67.6 > 1.2	5.33 35.15 < 180 21.11
Tail race	Normal	209.93	-	1319.93	0.088 < 12.75/6 = 2.125	-	17.15 < 180
	Flood	201.60	-	1374.66	0.449 < 12.75/2 = 6.375	-	15.78 19.12 < 180
	Earth-quake	206.93	34.35	1012.02	1.484 < 12.75/3 = 4.250	71.5 > 1.2	12.51 27.56 < 180 4.90
Right Bank	Normal	249.89	-	1520.15	0.633 < 10.90/6 = 1.816	-	30.91 < 180
	Flood	270.73	-	1580.15	0.387 < 10.90/2 = 5.450	-	14.94 30.13 < 180
	Earth-quake	249.89	25.16	1215.76	0.585 < 10.90/3 = 3.633	57.8 > 1.2	19.55 30.31 < 180 15.54

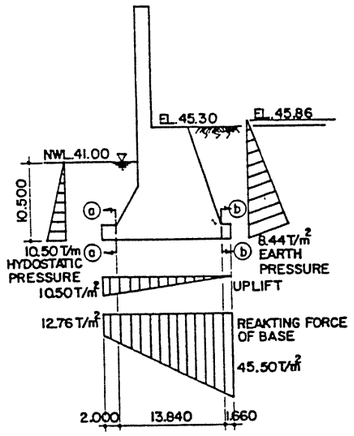
(c) Structural Calculation and Reinforcement

Bending Moment and Shearing Force

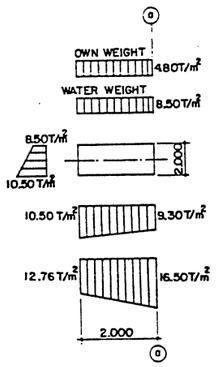
Part	Condition	Section	Bending moment (t.m)	Axial Force (t)	Eccen-vicity (m)	Shearing force (t)
Backfill	Normal	a - a	150.96	22.35	0.180	65.68
		b - b	9.21	9.47	-0.092	17.65
	Flood	a - a	- 63.96	-25.46	2.806	- 31.38
		b - b	176.52	34.52	0.700	154.02
		c - c	143.80	77.76	0.000	45.32
		d - d	17.98	32.06	0.000	11.33
	Earthquake	a -a	222.63	29.15	0.371	90.98
		c - c	62.99	77.76	0.000	7.78
Power - house	Normal	a - a	3.47	19.00	0.035	6.54
	Flood	a - a	7.90	46.64	0.014	9.32
		b - b	134.93	76.80	0.000	43.43
	Earthquake	a - a	37.37	24.88	0.263	37.33
		b - b	61.44	76.80	0.000	7.68
Tail-race	Normal	a - a	26.03	19.00	0.023	26.17
		b - b	28.83	19.00	0.034	28.50
	Flood	a - a	20.11	46.64	0.014	20.46
		b - b	32.00	46.64	0.014	31.68
	Earthquake	a - a	47.15	24.32	0.246	45.97
		b - b	6.39	15.43	-0.300	8.49
Right bank	Normal	a - a	24.23	19.00	0.023	24.21
	Flood	a - a	30.16	46.64	0.014	28.49
	Earthquake	a - a	51.21	26.06	0.296	49.31

Backfill Part

Normal Condition



a - a section



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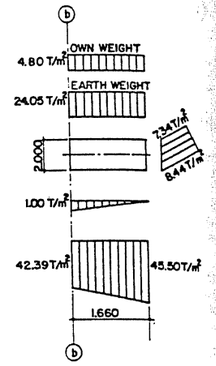
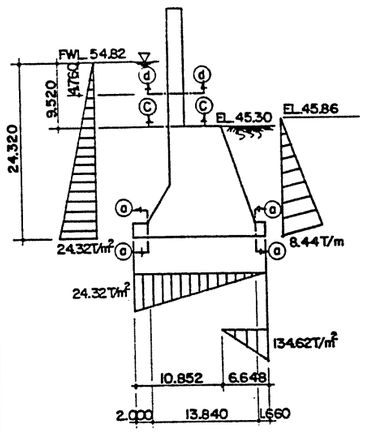
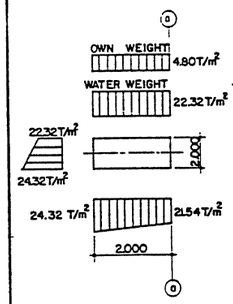


Flood Condition



a - a section



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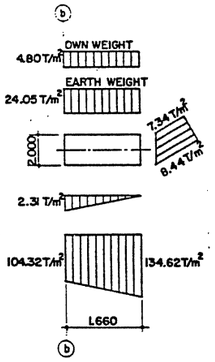
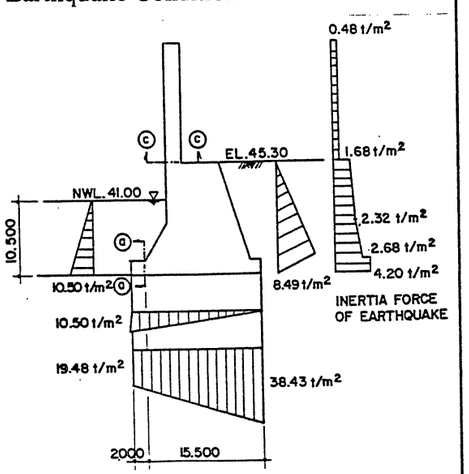


c - c section

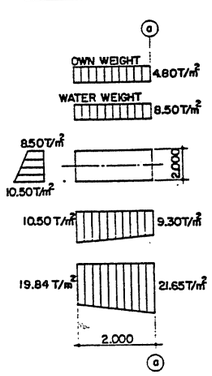


d - d section

Earthquake Condition



a - a section



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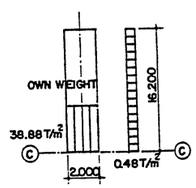
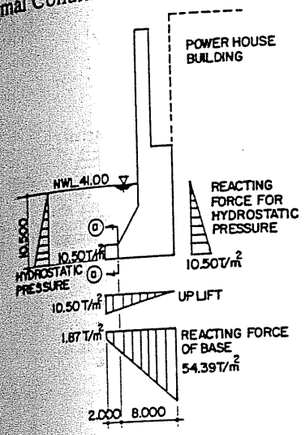
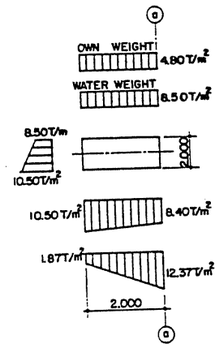


Powerhouse Part

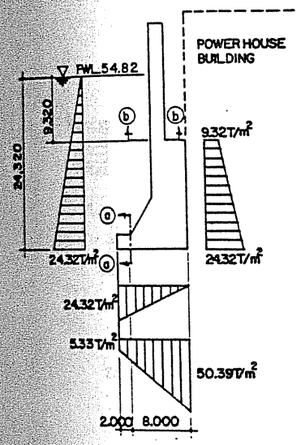
Normal Condition



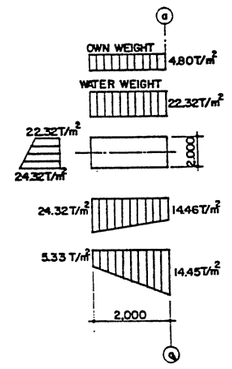
a - a section



Flood Condition



a - a section



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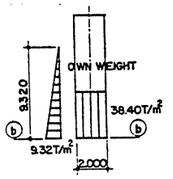
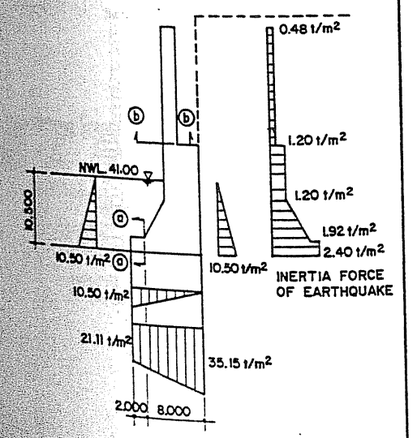


Earthquake Condition



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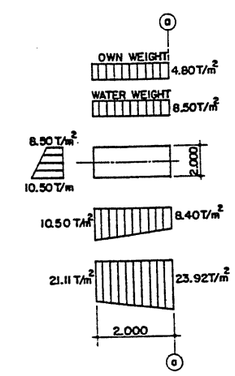

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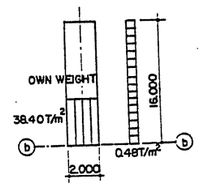
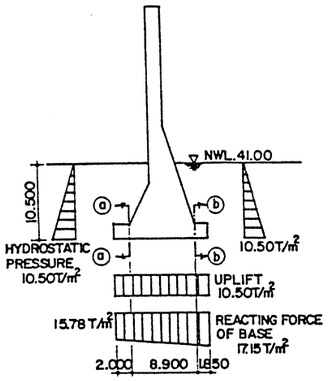
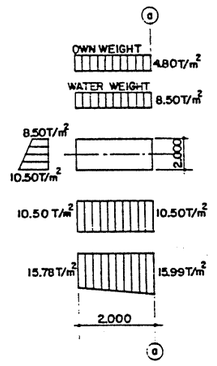


Tailrace Part

Normal Condition



a - a section



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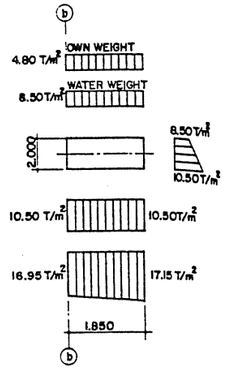
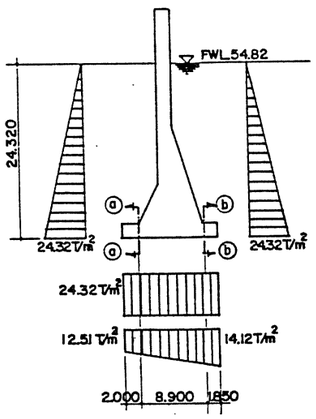
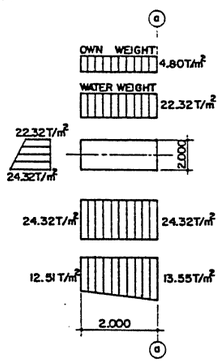


Flood Condition



a - a section



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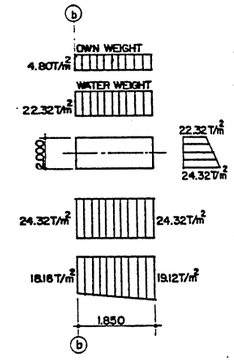
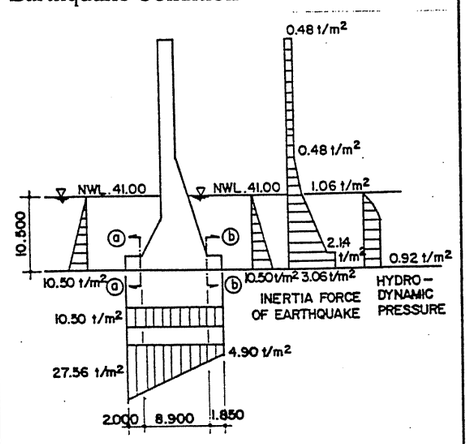
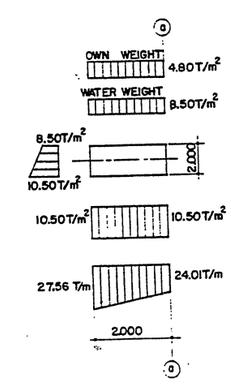


Earthquake Condition



a - a section



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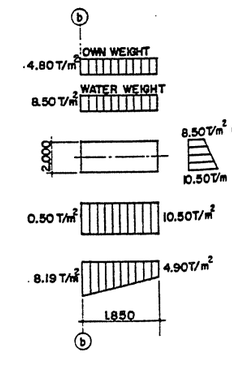
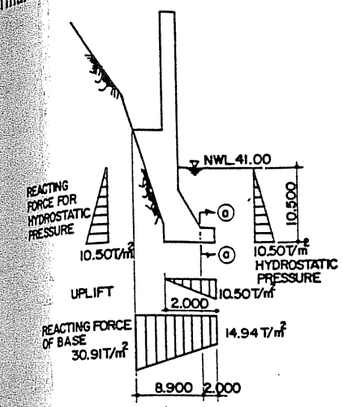
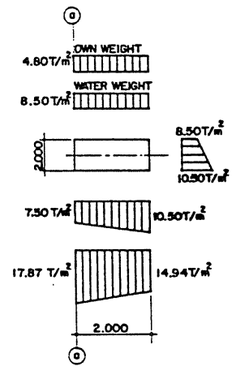


Rightbank Part

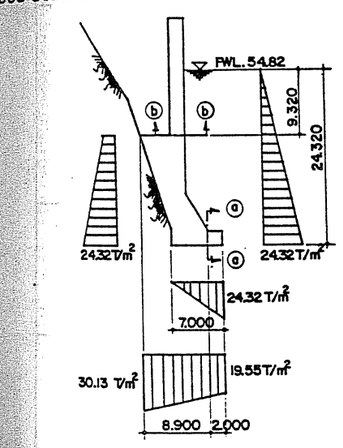
Normal Condition



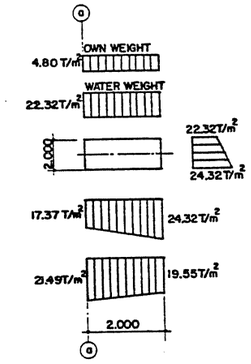
a - a section



Flood Condition



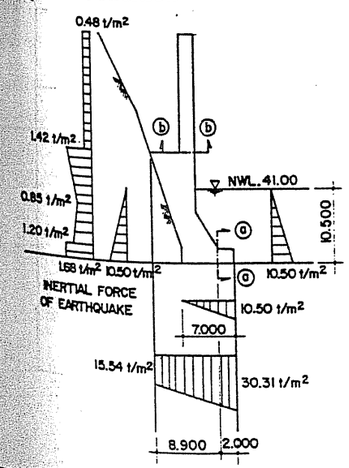
a - a section



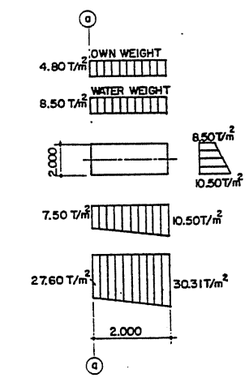
b - b section

Same as Powerhouse part

Earthquake Condition

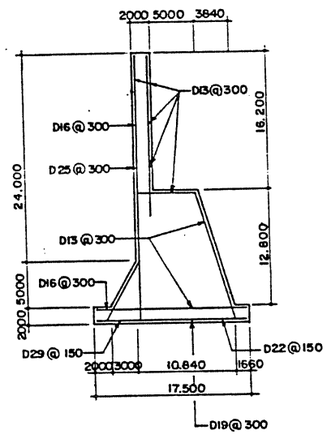


a - a section

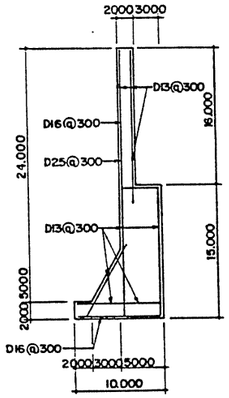


b - b section

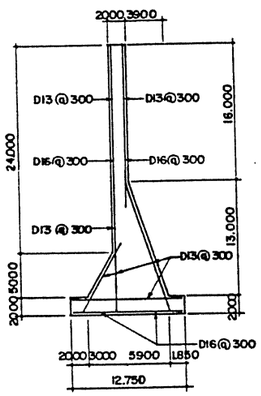
Same as Powerhouse part



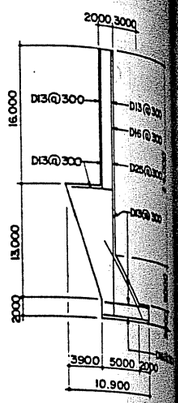
Backfill Part



Powerhouse Part



Tailrace Part



Rightbank Part

Distribution of reinforcement

Reinforcement distribution

Calculation case			Required reinforcement	Reinforcement distribution	Stress intensity
Backfill	a - a section	Normal Flood Earthquake	36.60 cm² below 9.85 cm² 35.58cm²	Lower part : D 29 @ 150 (As = 42.9 cm2) Upper part : D 16 @ 300	σ s = 1715 kg/cm² σ c = 38.7 kg/cm² τ = 3.7 kg/cm² τ o = 6.2 kg/cm²
	b - b section	Normal Flood Earthquake	below 1.00 cm² 23.27 cm² –	Lower part : D 22 @ 150 (As = 25.8 cm²) Upper part : D 13 @ 300	σ s = 2692 kg/cm² σ c = 49.5 kg/cm² τ = 8.6 kg/cm² τ o = 18.4 kg/cm²
	c - c section	Normal Flood Earthquake	14.60 cm² below 1.00 cm²	Stilling basin side D 25 @ 300 (As = 16.9 cm²) Backfill side D 13 @ 300	σ s = 2617 kg/cm² σ c = 53.5 kg/cm² τ = 2.4 kg/cm² τ o = 9.1 kg/cm²
	d -d section	Normal Flood Earthquake	below 1.00 cm² 1.00 cm² below 1.00 cm²	Stilling basin side D 16 @ 300 (As = 6.61 cm²) Backfill side : D 13 @ 300	σ s = 29 kg/cm² σ c = 4.7 kg/cm² τ = 0.1 kg/cm² τ o = 0.4 kg/cm²
Power-house	a - a section	Normal Flood Earthquake	below 1.00 cm² below 1.00 cm² below 1.42 cm²	Lower part : D 16 @ 300 (As = 6.61 cm²) Upper part : D 13 @ 300	σs = 797 kg/cm² σc = 14.5kg/cm² τ = 1.8 kg/cm² τo = 11.0 kg/cm²
	b - b section	Normal Flood Earthquake	13.76 cm² below 1.00 cm²	Stilling basin side D 25 @ 300 (As = 16.9 cm²) Powerhouse side : D 13 @ 300	σs = 2361kg/cm² σc = 49.9 kg/cm² τ = 2.3 kg/cm² τo = 8.7 kg/cm²

Calculation Case			Required Reinforcement	Reinforcement Distribution	Stress Intensity
Tailrace	a - a section	Normal Flood Earthquake	below 2.23 cm² below 1.00 cm² 3.44 cm²	Lower part : D 16 @ 300 (As = 6.61 cm²) Upper part : D 13 @ 300	σs =1635 kg/cm² σc = 21.7 kg/cm² τ = 2.4 kg/cm² τo = 14.5 kg/cm²
	b - b	Normal Flood Earthquake	2.96 cm² below 1.00 cm² below 1.00 cm²	Lower part : D 16 @ 300 (As = 6.61 cm²) Upper part : D 13 @ 300	σs = 963 kg/cm² σc = 14.4 kg/cm² τ = 1.5 kg/cm² τo = 14.7 kg/cm²
	c- c section	Normal Flood Earthquake	- - below 1.00 cm²	Stilling basin side D 16 @ 300 Tail race side : D 16 @ 300	
Rightbank side	a - a section	Normal Flood Earthquake	1.73 cm² 1.00 cm² 3.59 cm²	Lower part : D 16 @ 300 (As = 6.60 cm²) Upper part : D 13 @ 300	σs = 1703 kg/cm² σc = 22.8 kg/cm² τ = 2.6 kg/cm² τo = 18.5 kg/cm²
	b - b section	Normal Flood Earthquake	13.76 cm² below 1.00 cm²	Stilling basin side bank side : D 25 @ 300 Right bank side : D 13 @ 300	σs = 2361 kg/cm² σc = 49.9 kg/cm² τ = 2.5 kg/cm² τo = 8.9 kg/cm²

Allowable stress intensity

Unit ; kg/cm²

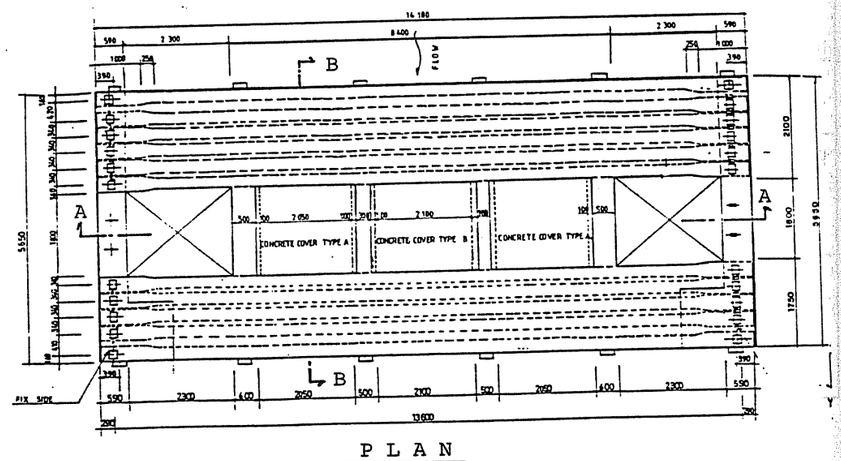
	Type		Long period	Short period
Concrete	Allowable compressive stress intensity due to bending Allowable shear stress intensity Allowable bond stress intensity	σca τa τoa	60 8 14	90 12 21
Reinforcement	Allowable tensile stress intensity	σsa	2000	3000

(5) Hoist Deck

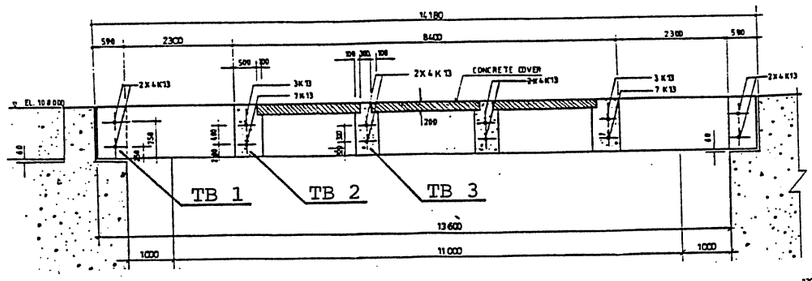
The original design of the hoist deck was a slab bridge structure consisting of pre-stressed concrete beams with three hollow parts. However, the hoist deck was modified to be a slab bridge structure with the transverse beams and precast concrete slab panels placed between the transverse beams in consideration of the design loads acting on the hoist deck which were newly proposed by the Lot II Contractor of Metal Works.

(a) Structural analysis

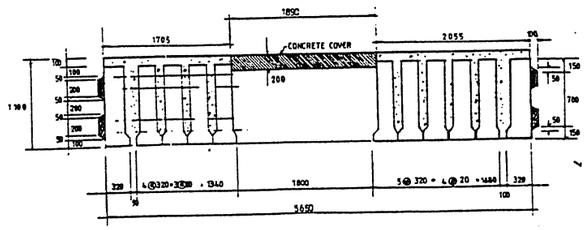
(i) Structural shape of the modified hoist deck



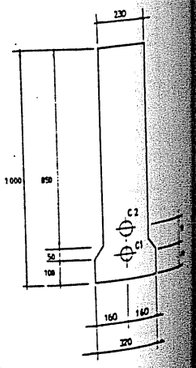
PLAN



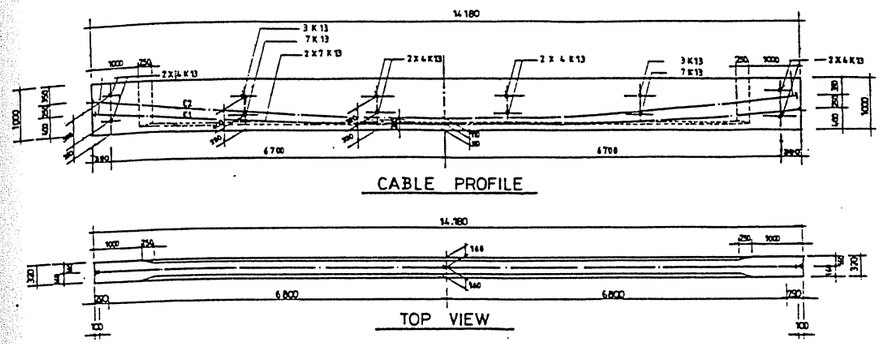
SECTION A-A



SECTION B-B



TYPICAL SECTION OF MAIN BEAM

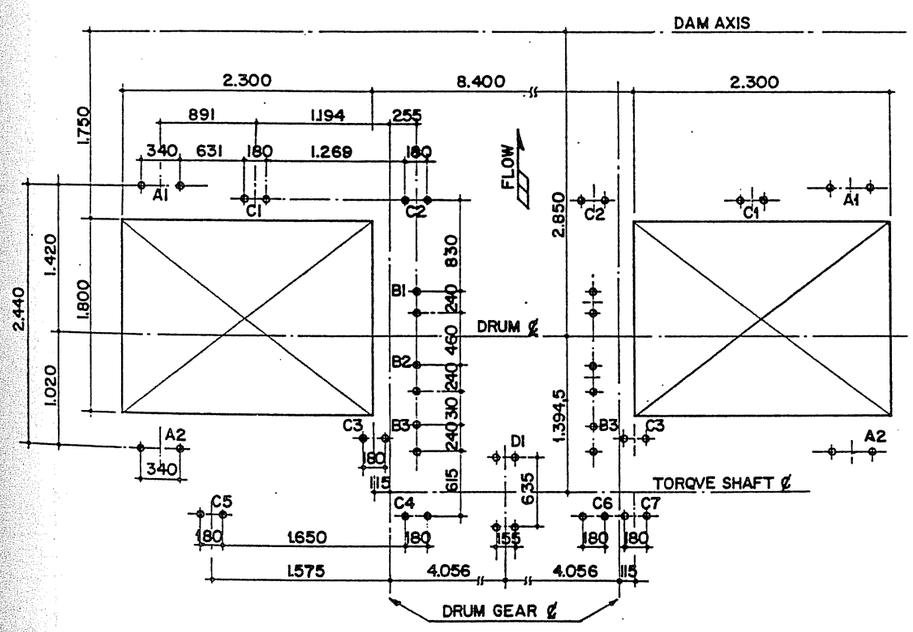


(ii) Design load

The load acting on the hoist deck due to the operation of the spillway gate hoist was determined by the Lot II Contractor, as shown in the figure below. Then, the load of the rated motor torque was applied for the design load for the service stress condition, while the load of the maximum motor torque was applied for the ultimate stress condition.

(unit : ton)

	A1	A2	B1	B2	B3	C1	C2	C3	C4	C5	C6	C7	D1
Load of Rated Motor Torque	21.4	30.2	19.2	14.6	6.2	7.3	1.0	7.7	3.8	1.0	2.7	1.8	1.0
Load of Max. Motor Torque	52.0	74.0	46.4	34.9	13.9	16.8	1.0	17.9	7.3	1.0	5.4	3.2	1.0



(iii) Allowable stress and design parameters

The tables below show the allowable stress and design parameters of the hoist deck structure.

Allowable Stress and Design Parameters for Main Beam

Items	Design Value
Concrete Design Strength (σck)	500 kg/cm²
Initial condition	345.8 kg/cm²
Service condition	432.3 kg/cm²
Service Allowable Stress (σca)	
Compressive	172.9 kg/cm²
Tensile	33.1 kg/cm²
Allowable Mean Shear Stress (σ_{la})	12 kg/cm²
Allowable Mean Shear Stress (τ_{ma})	6.5 kg/cm²
Allowable Ultimate Shear Stress (τ_{ua})	60 kg/cm²
Modulus of Elasticity (Ep)	3.07x10⁵ kg/cm²
Modulus of Rupture	41.2 kg/cm²

Allowable Stress and Design Parameters for PC Cable and Reinforcement

Items	Design Value
Prestress Cable	
Diameter	12.7mm
Effective section area	0.9871 cm²
Modulus of elasticity	1.96x10⁶ kg/cm²
Ultimate tensile strength	19000 kg/cm²
Reinforcement steel	
Diameter	13 mm
Effective section area	1.33 cm²
Grade	U 39 (deformed)
Modulus of elasticity	2.1x10⁶ kg/cm²

Allowable Stress and Design Parameters for Transverse Beams

Items	Design Value
Concrete ultimate strength	430.92 kg/cm²
Mild Reinforcement Field Strength	4081.6 kg/cm²
Steel's Modulus of elasticity	2.04x10⁶ kg/cm²
Prestressing/Post-Tension System	Grouted
Tendon Type	Stress relieved
Tendon's Ultimate Strength	18,979 kg/cm²
Tendon's Effective Stress	10.204 kg/cm²
Tendon's Field Stress	16,063 kg/cm²
Tendon's Modulus of Elasticity	1.969x10⁶ kg/cm²
Strength Reduction Factor (μ)	0.9

(iv) Structural Calculation of Main Beam

A FRAME analysis was applied to calculate the bending moment and shear force for the service stress condition and the ultimate stress condition. The calculation model and loads conditions are shown in Fig. 1.6.4-6 and the table below respectively. The calculation for bending stress and shear in the service and ultimate stress conditions are discussed in Section 6.2.5 (3) "Spillway Bridge".

Condition of Analysis

Calculation Condition	Load Combination
Service Stress Condition	Dead Load + Design Load (Rated Motor Torque)
Ultimate Stress Condition	1.3xDead Load+Max. Load (Max. Motor Torque)

Effective prestress force (Pe)

$$Pe = Aps\ x\ (\sigma pt - \Delta\sigma pr - \Delta\sigma pc) = 133{,}361\ kg$$

where, Aps : Area of steel (= 13.8194 cm²)
σpt : Tendon stress during prestress transfer (= 11400 kg/cm²)
Δσpr : Stress loss by relaxation (= 285 kg)
Δσpc : Stress loss by creep and dry shrinkage (= 1464.8 kg/cm²)

Service stress condition

The calculation result of bending stress shown below show that the stresses of the main beam under the service stress condition are within the allowable stress in all locations.

The calculation results of diagonal tension stress (σI) and mean shear stress (τm) shown below show that the stress in the main beam under service stress condition are within the allowable stress in all locations.

Section No.	No.1	No.2	No.3	No.4	No.5	No.6	No.7
Distance (m)	0.000	0.71	1.43	2.85	4.18	5.50	6.80
Ac (cm²)	3520.0	3520.0	2642.6	3642.6	2642.6	2642.6	2642.6
Ic (10⁶ cm⁴)	3.549	3.549	2.839	2.839	2.839	2.839	2.839
Yu (cm)	55.0	55.0	57.2	57.2	57.2	57.2	57.2
Yb (cm)	55.0	55.0	52.8	52.8	52.8	52.8	52.8
Pe (kg)	132,823	132,823	132,823	132,823	132,823	132,823	132,823
Position of PC (cm)	52.50	45.35	38.99	28.65	21.86	16.50	16.50
ep (cm)	2.50	9.65	13.81	24.15	30.94	36.30	36.30
Moment due to Dead load (t.m)	-0.35	8.74	16.41	20.64	23.08	4.50	4.50
Moment due to Live load (t.m)	-4.98	15.49	24.73	21.95	23.05	9.52	9.52
Total moment (t.m)	-5.33	24.23	41.14	42.59	46.13	14.02	14.02
Moment due to Prestress (tm)	-3.32	-12.82	-18.34	-32.08	-41.10	-48.21	-48.21
Pe/Ac (kg/cm²)	37.73	37.73	50.26	50.26	50.26	50.26	50.26
Bending stress of top of beam (kg/cm²)	24.33	55.41	96.20	71.44	60.39	-18.63	-18.63
Bending stress of Bottom of beam (kg/cm²)	51.13	20.05	7.86	30.71	40.91	113.85	113.85

Calculation of diagonal tension stress and mean shear stress

Section No.	No.1	No.2	No.3	No.4	No.5	No.6	No.7
Distance (m)	0.00	0.71	1.43	2.85	4.18	5.50	6.80
Shd (ton)	29.51	29.51	21.49	21.33	10.67	7.24	1.06
Sp (ton)	13.94	13.94	13.94	13.94	0.00	0.00	0.00
bw (cm)	32.0	32.0	23.0	23.0	23.0	23.0	23.0
d (cm)	95	95	95	95	95	95	95
I ($x10^6$ cm^4)	2.667	2.667	2.134	2.134	2.134	2.134	2.134
Q ($x10^3$ cm^3)	40.000	40.000	28.750	28.750	28.750	28.750	28.750
τ (kg/cm^2)	7.3	7.3	4.4	4.3	6.3	4.2	0.6
σc (kg/cm^2)	37.7	38.0	52.0	51.1	50.7	50.3	47.6
σl (kg/cm^2)	1.4	1.4	0.4	0.4	0.8	0.3	0.01
τm (kg/cm^2)	5.1	5.1	2.5	2.4	305	2.4	0.3

- Ultimate stress condition

The calculation result of resisting bending moment and safety factor against rupture are shown in the table below, and the safety factor exceeds the minimum design value 1.0.

The calculation result of ultimate shear stress is shown in the table below, and show that the stress under the ultimate stress condition is within the allowable stress 60 kg/cm^2.

Maximum bending moment : Mn (t.m)	83.08
Area of PC steel : Aps (cm^2)	13.8194
Stress of PC steel : fps (kg/cm^2)	16731
Effective height of section : d (cm)	82.18
Distance from compressive edge to neutral axis : x (cm)	29.57
Resisting bending moment : Mu (t.m)	162.66
Safety factor : Mu/Mn	1.9 > 1.0

Shear due to ultimate load condition : Shd (ton)	63.12
Vertical component of prestress : S_p (ton)	13.94
Width of web : bw (cm)	32.0
Effective height of girder : d (cm)	95.0
Ultimate shear stress : τu (kg/cm^2)	16.2 < 60.0

(v) Structural calculation of transverse beams

- Service stress condition

The calculation results of bending stress, diagonal tension stress and mean shear stress in the transverse beams are shown below and show that the stresses of the transverse beams under service stress condition are within the allowable stresses in all locations.

Transverse Beam - 1

Section No.	No.1	No.2	No.3	No.4
Distance (m)	0.00	0.00	12.00	18.00
Ac (cm^2)	6490.0	6490.0	6490.0	6490.0
Ic (10^6 cm^4)	6.554	6.544	6.544	6.544
Yu (cm)	55.0	55.0	55.0	55.0
Yb (cm)	55.0	55.0	55.0	55.0
Pe (kg)	88,640	88,640	88,640	88,640
Moment due to Dead Load (t.m)	-0.06	-0..8	-0.09	-0.05
Moment due to Live Load (t.m)	-3.89	17.06	24.75	11.67
Total moment (t.m)	3.83	16.98	24.66	11.72
Moment due to Prestress (t.m)	-4.43	-4.43	-4.43	-4.43
Pe/Ac (kg/cm^2)	13.66	13.66	13.66	13.66
Bending stress of top of beam (kg/cm^2)	13.16	24.21	30.66	19.79
Bending stress of Bottom of beam (kg/cm^2)	14.16	3.11	-3.34	7.53
Shd (ton)	0.40	1.34	1.23	0.48
Sp (ton)	0.00	0.00	0.00	0.00
bw (cm)	59.0	59.0	59.0	59.0
d (cm)	95.0	95.0	95.0	95.0
I (x 10^6 cm^4)	4.917	4.917	4.917	4.917
Q (x 10^3 cm^3)	73.750	73.750	73.750	73.750
τ (kg/cm^2)	0.10	0.34	0.31	0.12
γc (kg/cm^2)	13.66	13.66	13.66	13.66
σl (kg/cm^2)	0.00	0.01	0.01	0.00
τm (kg/cm^2)	0.07	0.24	0.22	0.09

Transverse Beam - 2

Section No.	No.1	No.2	No.3	No.4
Distance (m)	0.00	6.00	12.00	18.00
Ac (cm^2)	6400.0	6400.0	6400.0	6400.0
Ic (10^6 cm^4)	6.231	6.231	6.231	6.231
Yu (cm)	56.4	56.4	56.4	56.4
Yb (cm)	53.6	53.6	53.6	53.6
Pe (kg)	110.800	110.800	110.800	110.800
Moment due to Dead Load (t.m)	1.26	2.48	2.47	1.24
Moment due to Live Load (t.m)	3.89	17.06	24.75	11.67
Total moment (t.m)	5.15	19.54	27.22	12.91
Moment due to Prestress (t.m)	-16.62	-16.62	-16.62	-16.62
Pe/Ac (kg/cm^2)	17.31	17.31	17.31	17.31
Bending stress of top of beam (kg/cm^2)	6.93	20.00	26.90	13.95
Bending stress of Bottom of beam (kg/cm^2)	27.18	14.80	8.19	20.50

Shd (ton)	18.47	16.96	20.71	22.97
Sp (ton)	0.00	0.00	0.00	0.00
bw (cm)	60.0	60.0	60.0	60.0
d (cm)	95.0	95.0	95.0	95.0
I (x 10^6 cm^4)	5.000	5.000	5.000	5.000
Q (x 10^3 cm^3)	75.000	75.000	75.000	75.000
τ (kg/cm^2)	4.62	4.24	5.18	5.74
σc (kg/cm^2)	17.06	17.40	17.55	17.23
σl (kg/cm^2)	1.17	0.98	1.41	1.74
τm (kg/cm^2)	3.24	2.98	3.63	4.03

Transverse Beam - 3

Section No.	**No.1**	**No.2**	**No.3**	**No.4**
Distance (m)	0.00	6.00	12.00	18.00
Ac (cm^2)	5100.0	5100.0	5100.0	5100.0
Ic (10^6 cm^4)	4.659	4.659	4.659	4.659
Yu (cm)	58.5	58.5	58.5	58.5
Yb (cm)	51.5	51.5	51.5	51.5
Pe (kg)	88.640	88.640	88.640	88.640
Moment due to Dead Load (t.m)	2.18	3.89	3.93	2.31
Moment due to Live Load (t.m)	6.51	7.73	8.96	10.18
Total moment (t.m)	8.69	11.62	12.89	12.49
Moment due to Prestress (t.m)	-6.20	-6.20	-6.20	-6.20
Pe/Ac (kg/cm^2)	17.38	17.38	17.38	17.38
Bending stress of top of beam (kg/cm^2)	20.51	24.19	25.78	25.28
Bending stress of Bottom of beam (kg/cm^2)	14.63	11.39	9.98	10.43
Shd (ton)	4.63	2.59	12.89	12.49
Sp (ton)	0.00	0.00	0.00	0.00
bw (cm)	50.0	50.0	50.0	50.0
d (cm)	95.0	95.0	95.0	95.0
I (x 10^6 cm^4)	4.167	4.167	4.167	4.167
Q (x 10^3 cm^3)	62.500	62.500	62.500	62.500
τ (kg/cm^2)	1.39	0.78	3.87	3.75
σc (kg/cm^2)	17.57	17.79	17.88	17.86
σl (kg/cm^2)	0.11	0.03	0.80	0.76
τm (kg/cm^2)	0.97	0.55	2.71	2.63

Ultimate stress condition

The resisting bending moment for rupture of the beams were derived form the ultimate stress conditions for the transverse beams TB 2 and TB 3, since TB1 was located on the supporting point of the pier, the resisting bending moment calculation was omitted. The safety factor of the TB2 and TB3 are shown in the table below. Safety factors for TB2 and TB3 exceeds the minimum safety factor 1.0.
The calculation results of ultimate shear stress for the transverse beams TB1, TB2 and TB3 are shown in the table below, and show that the ultimate shear stress in all the beams is within the allowable ultimate shear stress 60 kg/cm^2.

Resisting Moment and Safety Factor

	Resisting Moment	Moment due to ultimate stress condition	Safety Factor
TB2	189.61 tm	61.98 tm	3.1
Tb3	101.11 tm	26.68 tm	3.7

	TB1	TB2	TB3
Shear force due to ultimate load condition : Shd (ton)	1.13 0.0	52.31 0.0	7.47 0.0
Vertical component of prestress : sp (ton)	59.0	60.0	50.0
Width of web : bw (cm)	95.0	95.0	95.0
Effective height of beam : d (cm)	0.2 < 60	9.2 < 60	1.6 < 60
Ultimate shear stress : τu (kg/cm^2)			

(vi) Structural calculation of precast concrete slab

Design Condition

Design load	Unit weight of concrete	2.4 t/m^3
	Live load	0.5 t/m^3
Allowable stress intensity	(Concrete) Compressive stress	93.3 kg/cm^2 9.0 kg/cm^2
	Shearing stress	
	(Reinforcement bar) Tensile stress	2000 kg/cm^2

	Type I	Type II	Remarks
Span (m)	2.250	2.300	
Thickness (cm)	20.0	20.0	
Maximum bending moment (t.m/m)	0.620	0.648	
Maximum shearing force (t/m)	1.103	1.127	
Concrete cover for steel (cm)	5.0	5.0	
Reinforcement content (cm^2)	13.24	13.24	
	D16ctc 150mm	D16ctc 150mm	
Compressive stress of concrete (kg/cm^2)	16.0	16.7	$<\sigma ca=93.3\ kg/cm^2$
Tensile stress of concrete (kg/cm^2)	360.1	376.3	$<\sigma ca=2000\ kg/cm^2$
Shearing stress of concrete (kg/cm^2)	0.85	0.87	$<\tau oca=9.0\ kg/cm^2$

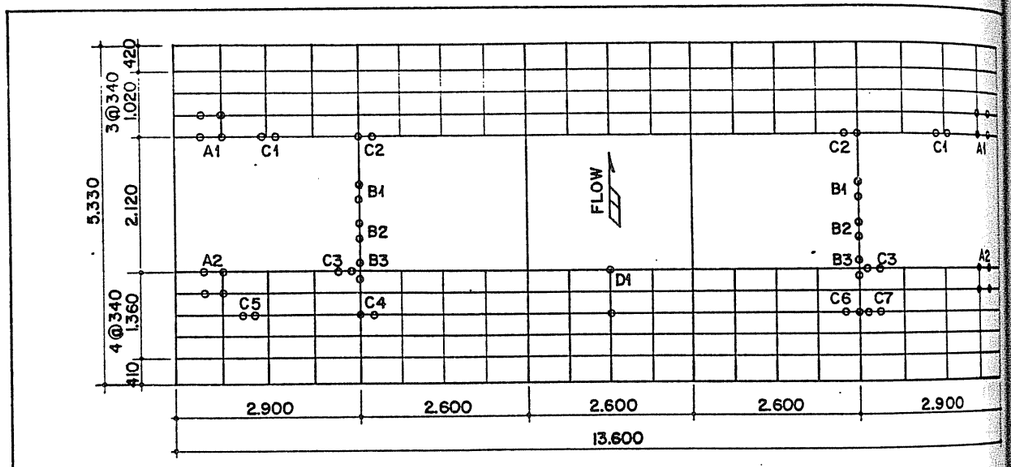


Fig. 1.6.4-6 Calculation Model and Load Conditions.

6.5 Dam Diversion Tunnel

6.5.1 Diversion Method and Floods.

(1) General .

The river diversion is essential for isolating the construction area for foundation work i.e, dewatering, excavation, base preparation and laying concrete in the dam section. In this project, two diversion tunnels with cofferdams were adopted because the dam site valley is too narrow topographically so is not suitable for a open channel diversion or sluice diversion. In addition, the river flow direction and the good rock foundation in the abutments showed that the diversion tunnel method was favorable at this site.

A three (3) years return period flood of 1300 m^3/s was adopted for the diversion design flood while the annual average discharge at the dam site was 600m^3/s, that is, a dry-season flood based on 10 years discharge data. Therefore the diversion tunnels and cofferdams were designed following the two conditions :

(i) To enable a flow of at least 600 m^3/s in free water surface in the tunnel with temporary cofferdam during construction of main cofferdam.

(ii) To allow the design flood of 1300 m^3/s flow in pressure flow condition in the tunnel with main cofferdam during construction of main dam.

(2) Original Plan of River Diversion

The original plan for the river diversion consisted of two lined tunnels with 10m diameter and inlet and outlet portals on the left abutment of Kampar Kanan River, because good tunneling and portalling conditions were anticipated in the left abutment and the tunnel length was rather shorter than that of the right abutment.

The tunnel alignment and location of the inlet and outlet portals as well as those of the cofferdams were determined by the river flow characteristics, topography, the abutment rocks, and the dried open space required for dam construction. The center distance of the two diversion tunnels was determined to be 30m to avoid stress concentration caused by each tunnel.

The upstream cofferdam was planned as a two-stage construction. A first stage cofferdam (hereinafter refered to as the primary cofferdam) was planned to be located upstream of the dam and designed to be EL.44.50 in order to provide sufficient diversion capacity for a dry-season flood based on the past 10 years river flow data. The second-stage cofferdam (hereinafter refered to as the secondary cofferdam) was planed to be located just downstream of the primary cofferdam and designed to be EL. 51.50 in order to provide sufficient diversion capacity for the design flood of 1300 m^3/s, which is equivalent to a three (3) years return period, and also to avoid the submergence of existing national road running through the dam site left bank. The construction program showed that the secondary cofferdam was to be completed before the wet-season.

A concrete dam was selected for the secondary cofferdam because enough field space was secured for the construction works at the dam site and no major damage would occur in the event of for overtopping during flood.

The downstream cofferdams with crest elevation of EL.43.60 m and EL.45.3m for the primary and secondary cofferdam respectively, were designed to be adequately high for the estimated tailwater level for the design floods mentioned above.

The cost comparison on the height of secondary cofferdam and diameter of diversion tunnel was made for the diversion design flood of 1300 m^3/s including the relocation cost of the existing road which would be submerged by the construction of the cofferdam.

From the result of the study, the combination of the dam height 17.5m (H.W.L. 50.50 m) and the tunnel diameter 10.0m was the most economical and selected for final design. In this case, the national road would not be submerged as the elevation of the lowest part of the road is higher than of H.W.L 50.50.

(3) Design Modification during Construction

The original construction schedule was to commence the river diversion works on 1st April 1992 and divert the river water in the dry season of 1993. However the commencement of the works was actually delayed until 16th October 1992. This meant the river diversion had to start at the middle of January 1994 and the original cofferdam construction had to be completed in the middle of May 1994. This period was in the middle of the rainy season and it seemed to be very dangerous for the works.

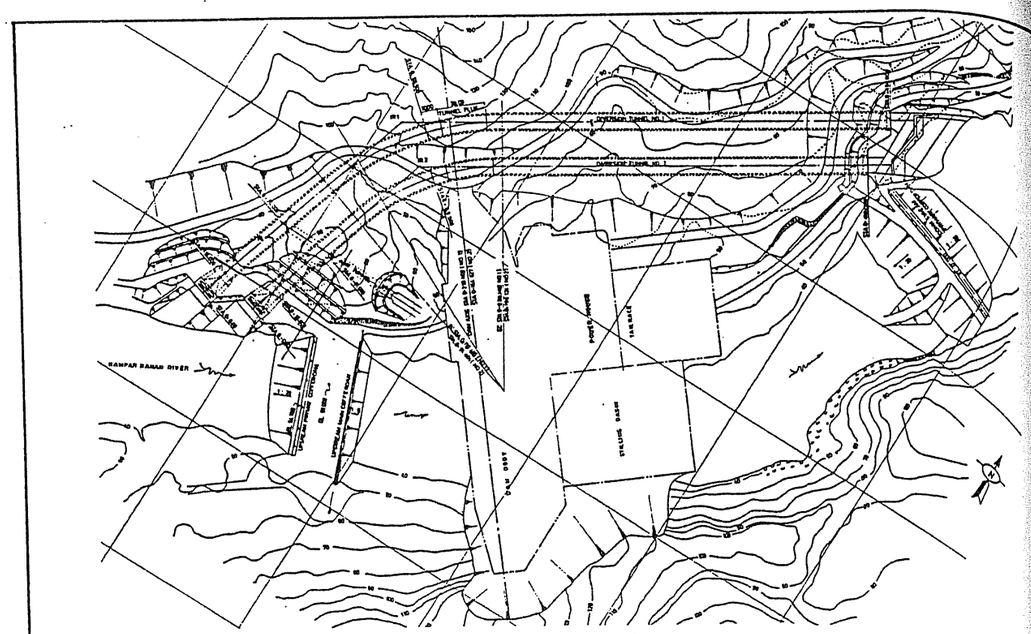
Therefore, some design modifications of the diversion plan mentioned below, were done during the construction period in order to shorten the construction period of the diversion works. The design modification was made so as to meet the hydraulic requirement even in some modificated items.

(i) First proceeding in river diversion No. 2 Tunnel
(ii) Modification of construction method or dam type of the secondary cofferdam.
(iii) Cancellation of construction for outlet structure.
(iv) Installation of additional temporary work adit.

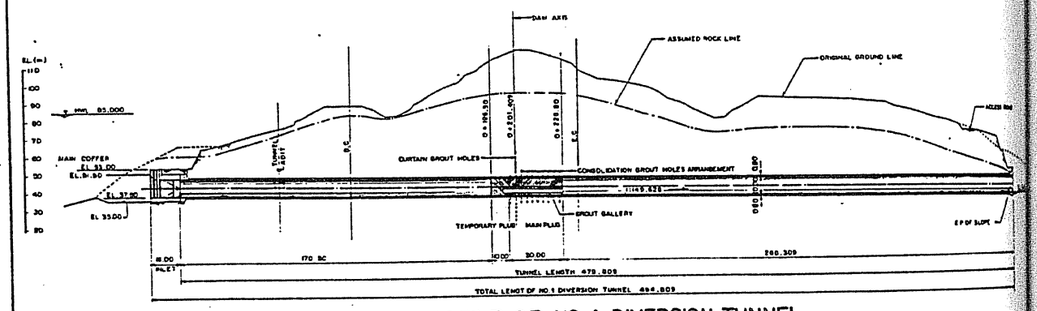
The design modification of each structure is illustrated in detail in the following sections for each structure. The original design and the modified design are compared in the following table. The plan and profile of the modified diversion tunnels are shown in Fig. 1.6.5-1.

Design modification results.

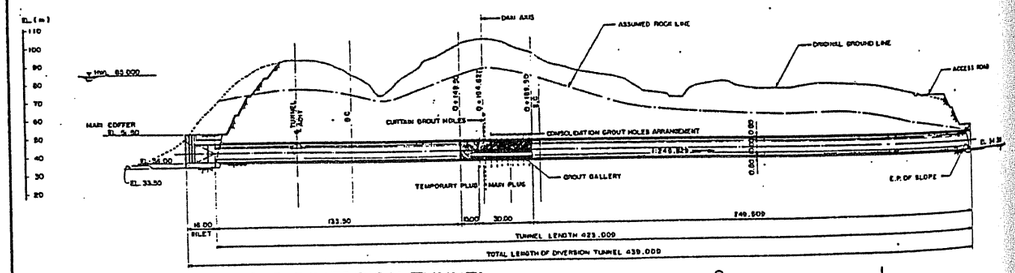
Item		Original Design	Modified Design
1. Tunnel			
* Number		Two (2) unit	Two (2) unit
* Diameter		10.0 m	10.0 m
* Shape		Standard Horse-shoe.	Standard Horse-shoe
* Lining		80 cm - thick reinforced concrete.	80 cm - thick reinforced concrete.
* Tunnel Length	No. 1	481.3 m	478.809 m
	No. 2	425.5 m	423.009
* Gradient	No. 1	1:283	1:149.628
	No. 2	1:250	1:248.829
2. Inlet Portal			
* Invert EL.	No. 1	EL. 36.00	EL. 37.50
	No. 2	EL. 36.00	EL. 36.00
* Length	No. 1 & 2	15.0 m	16.0 m
* Width & Height	No. 1 &2	17.5 x 18.0 m	17.5 x 18.0 m
3. Outlet Portal			
* Invert EL.	No. 1 & 2	EL. 34.30	EL. 34.30
* Length	No. 1 & 2	15.0 m	0.0 m
* Width & Height	No. 1 & 2	13.0 x 16.0 m	-
4. Upstream cofferdam			
* Primary			
Type		Earth and Rock fill	Earth and Rock fill
Crest EL.		EL. 45.50	EL. 46.50
Crest Length		80.0 m	84.8 m
Dam Height		10.5 m	12.5 m
* Secondary			
Type		Concrete Gravity	Concrete Gravity (by RCC)
Crest EL.		EL. 51.50	EL. 51.50
Crest Length		125.0 m	132 m
Dam Height		17.5 m	19.5 m
5. Downstream cofferdam			
* Primary			
Type		Earth and Rock fill	Earth and Rock fill
Crest EL.		EL. 43.60	EL. 44.00
Crest Length		103.0 m	101.5 m
Dam Height		8.1 m	9.2 m
* Secondary			
Type		Concrete Gravity	Earth and Rock fill
Crest EL.		EL. 45.30	EL. 45.30
Crest Length		105.0 m	100.3 m
Dam Height		10.8 m	10.5 m



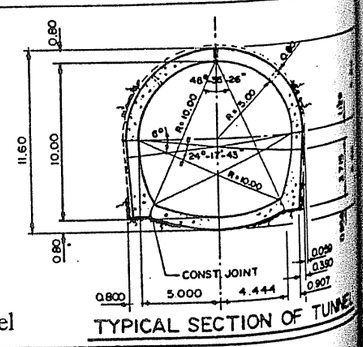
DIVERSION PLAN



PROFILE OF NO. 1 DIVERSION TUNNEL



PROFILE OF NO. 2 DIVERSION TUNNEL



TYPICAL SECTION OF TUNNEL

Fig. 1.6.5-1 Profile and Plan of Diversion Tunnel

6.5.2 Hydraulic Design

(1) Design modification of Tunnel plan. (Preceding in river diversion No. 2 Tunnel)

In the original design, the length of tunnel No.2 455.50m (excluding inlet and outlet portals) is shorter than that of the No. 1 tunnel 511.30m. Therefore, the construction period of No. 2 tunnel could be 1 month shorter than No. 1.

To divert the river through the No. 2 tunnel first made the it necessary for following matters to be considered for the purpose of early commencement of cofferdam construction.

(i) Discharge Capacity

In the case of making the river diversion in the dry-season of August, the river discharge was estimated 70 m^3/s, which can flow through No. 2 tunnel itself with the water height of 3.5 m.

(ii) Cofferdam

The secondary cofferdam was designed for the discharge of 1300 m^3/s and both diversion tunnels would be piped flow. This meant that the modification of the crest elevation did not affect the design of the secondary cofferdam. However, the primary cofferdam was designed for the discharge of 600m^3/s, so that the height of the primary cofferdam would be increased 0.7m and the cost increased proportionately.

(iii) Inlet Portal Excavation

The open excavation could be executed downward to EL. 36.50 from the dam access road which elevation was about EL. 80.00 , while the original elevation of the excavation for diversion inlet was EL. 35.00. This meant the cost saving born by the reduction of excavation, rock bolt, gunite works due to the reduction of slope area of excavation

(iv) Geological Condition

The No. 1 tunnel was designed to pass through the rocks classified as CM and CH. To raise the inlet by 1.5m did not affect the geological conditions of the tunnel.

(v) Coffering for No. 1 Tunnel

Coffering was needed in front of the No. 1 tunnel. If the inlet elevation of No. 1 tunnel was raised by 1.5m, this coffering height would be about 2m compared to 4m in case of the original elevation. This is an important advantage of the design modification.

(vi) Cost Comparison

From the cost comparison, some construction costs of the diversion works can be saved.

From the result of the studying mentioned above, the gradient of the No. 1 tunnel, elevation of inlet invert and crest elevation of upstream primary cofferdam were modified to shorten the construction period of the works.

(2) Hydraulic Design

(a) Discharge Calculation of Diversion Tunnel

Calculations were made based on the final dimension of the No.1 tunnel and No.2 tunnel. For the flow conditions through the diversion tunnels, the free flow and the piped flow were studied for the dry-season flood of 600 m^3/s for the primary cofferdam and diversion design flood of 1300 m^3/s for the secondary cofferdam respectively. The basic calculation conditions are shown below.

	No. 1 Tunnel	No. 2 Tunnel
Cross section	Standard horse-shoe shape, lined	
Diameter	10.0 m	10.0 m
Inlet Elevation	EL. 37.50	EL. 36.00
Outlet Elevation	EL. 34.30	EL. 34.30
Tunnel Length	478.8 m	423.0 m
Tunnel Gradient	1:150	1:249

(i) Free flow

In this case, discharge volume was calculated by the uniform flow formula of Manning as shown in follows ;

$$Q = A.V, \quad V = 1/n \,.\, R^{2/3} \,.\, I^{1/2}$$

Where, Q : Discharge (m^3/s)
A : Water Area (m^2)
V : Velocity (m/s)
n : Coefficient of Roughness (= 0.014)
R : Hydraulic mean depth (R=A/P; m)
P : Wetted perimeter (m)
I : Tunnel Gradient

The calculation result is shown in Table 1.6.5-1. In the case of a discharge of 600 m^3/s, downstream water level in the tunnel was calculated as follows.

	Water depth		EL.of Outlet	Downstream Water level
No. 1 Tunnel	3.3 m	+	EL. 34.3 m	= EL. 37.60
No. 2 Tunnel	4.8 m	+	EL. 34.3 m	= EL. 39.10

The downstream water level in the each tunnel calculated above, was lower than the downstream water level of 42.15m that was calculated by the Rating Curve at Dam site, Fig. 1.6.5-2. So, the water surface of the free flow in the tunnel was influenced by the downstream water level at the outlet. However from the calculation result of water surface of the free flow in each tunnel as mentioned in the latter part of this section, it was clarified that the upstream water depth was not influenced by the downstream water level at the outlet because a flow with critical depth occurred in the tunnel. Therefore, the two diversion tunnels can cope with the diversion flood of 600 m^3/s in free flow as shown in the table below.

Table 1.6.5-1 Calculation Result of Hydraulic Characteristic

Upstream water level EL. (m)	Water Depth (m)		Discharge (m^3/s)			Remarks
	No. 1T	No. 2 T	No. 1 T	No.2 T	Total	
36.5	0	0.5	0	4.5	4.5	Q = 600 m^3/s
37.0	0	1.0	0	18.2	18.2	EL. 40.8 m
37.5	0	1.5	0	46.8	46.8	Water depth
38.0	0.5	2.0	5.8	80.2	86.0	T1 : 3.3 m
38.5	1.0	2.5	23.4	119.4	142.8	T2 : 4.8 m
39.0	1.5	3.0	60.2	163.2	223.4	
39.5	2.0	3.5	103.3	211.1	314.4	
40.0	2.5	4.0	153.8	262.1	415.9	
40.5	3.0	4.5	210.3	315.7	526.0	
41.0	3.5	5.0	272.0	371.2	643.2	

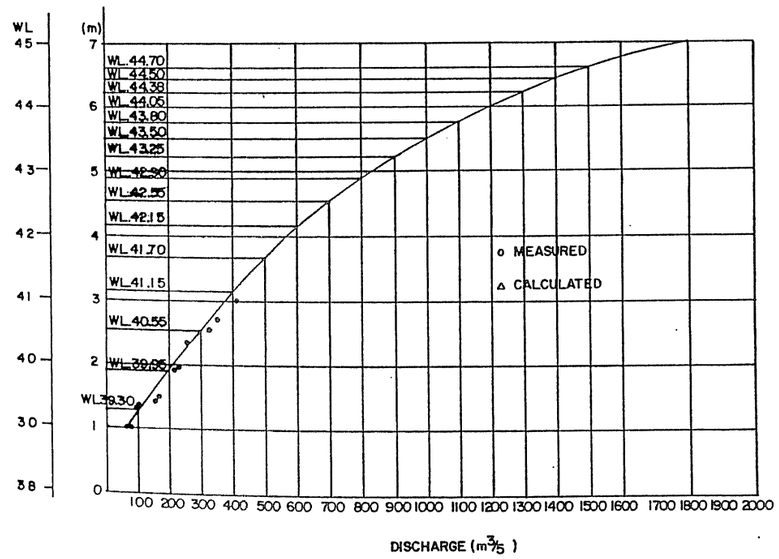


Fig. 1.6.5-2 Rating Curve at Dam Site

(ii) Piped Flow

For a discharge of Q=1300 m^3/s, the flow in the tunnel was the piped flow, which was governed by the water head difference in the upstream and downstream water levels. Discharge volume was calculated by the following equation.

$$Q = A.V, \quad V = \sqrt{2gh \,/\, (fe + fo + fb + f' L/R)}$$

Where ;Q :Discharge
A :Area of water (= 82.925 m^2)
V :Velocity
h :Design water head difference from upstream water level to downstream river course.
fe :Coefficient of Entrance Loss (= 0.20)
fo :Coefficient of Outlet Loss (=1.00)
fb :Coefficient of Loss in Bend (= $f_{b1} \times f_{b2}$)
fb1 :$0.131 + 0.1632\ (D/c)^{7/2}$
fb2 :$(\theta/90^0)^{1/2}$
D :Tunnel Diameter (=10.0m)
C :Radius of curvature (No. 1 T : = 180.0m), (No. 2 T : = 150.0m)
θ :Inter Angle (No. 1 T : = 40.0^0), (No. 2 T : = 40.0^0)
f'.L/R :Friction Loss ($=2gn^2.L/R^{4/3}$)
L :Length of tunnel (No. 1 T : 478.8m), (No. 2 T : 423.0m)
R :Hydraulic mean depth
f' :Coefficient of Friction Loss (=2.538m)

The calculation result is shown in the following table. From the result of the calculation, the discharge of 1300 m^3/s would flow through the two tunnels in the piped flow condition and the upstream and downstream water level would be at EL. 49.90 and EL. 44.30 respectively. Therefore, the secondary cofferdam with crest elevation EL. 51.50 and EL. 45.30 for upstream and downstream and the two diversion tunnels could cope with the design diversion flood of 1300 m^3/s.

Calculation result of upstream water surface in piped flow

Discharge (m^3/s)	Downstream water level (EL.m)	Velocity (m/s)		h (m)	Upstream water level (EL.m)
		No. 1T	No.2T		
600	42.15	3.586	3.649	1.19	43.34
650	42.35	3.885	3.954	1.40	43.75
700	42.55	4.184	4.258	1.62	44.17
750	42.75	4.483	4.562	1.86	44.61
800	42.90	4.781	4.866	2.12	45.02
850	43.10	5.080	5.170	2.39	45.49
900	43.25	5.379	5.474	2.68	45.93
950	43.40	5.569	5.778	2.99	46.39
1000	43.50	5.976	6.082	3.31	46.81
1050	43.65	6.276	6.387	3.66	47.31
1100	43.80	6.575	6.691	4.01	47.81
1150	43.90	6.873	6.995	4.38	48.28
1200	44.05	7.172	7.299	4.77	48.82
1250	44.20	7.471	7.603	5.18	49.38
1300	44.30	7.769	7.907	5.60	49.90

(b) Upstream and Downstream Water Level

(i) Free flow

In general, the water level on the surface of free flow in the tunnel was calculated by the following equation,

$$dh/dx = (\sin\theta - If) \,/\, [\cos\theta - Q^2/(g.A^3)\ .\ dA/dh]$$

Where; h : Water depth (m)
θ : Tunnel or channel gradient (degree)
If : Hydraulic gradient ($= n^2.Q^2/(R^{4/3}.A^2)$: by Manning Formula)
Q : Discharge (m^3/s)
A : Water area (m^2)
n : Coefficient of Roughness (=0.014)
R : Hydraulic mean depth

From this equation, critical depth (Hc) and depth of uniform flow (Ho) were calculated by following condition respectively.

Critical depth : $n^2.Q^2 / (R^{4/3} - A^2) = \sin\theta \fallingdotseq i$
Depth of uniform flow : $Q2 / (g.A3).(dA/dh) = \cos\theta \fallingdotseq 1$

The calculation result is shown in the table below. From the result of the calculation, in the case that discharge was between 50 m^3/s and 600 m^3/s, the critical depth (Hc) was always higher than the depth of uniform flow (Ho).

On the other hand, in the case of discharge of 600 m^3/s, the downstream water level of free flow in the tunnel was lower than the downstream water level at the outlet. So, the water level in the tunnel was influenced by the downstream water level at the outlet.

Critical Depth and Depth of Uniform Flow

Q (m^3/s)	No. 1 Tunnel		No. 2 Tunnel	
	Hc (m)	Ho (m)	Hc (m)	Ho (m)
50	1.845	1.364	1.845	1.554
100	2.725	1.963	2.725	2.262
150	3.440	2.464	3.440	2.257
200	4.064	2.913	4.064	3.392
250	4.627	3.326	4.627	3.889
300	5.145	3.717	5.145	4.361
350	5.623	4.091	5.623	4.817
400	6.066	4.452	6.066	5.262
450	6.478	4.804	6.478	5.702
500	6.865	5.151	6.865	6.145
550	7.225	5.491	7.225	6.598
600	7.560	5.833	7.560	7.076

Note, Q : Discharge in tunnel (m^3/s)
Hc : Critical depth (m)
Ho : Uniform flow depth (m)

- Water surface in the tunnel

The following table shows the calculation result of the water depth of the free flow in the tunnel that was calculated under the condition that the downstream water level in the tunnel was equal to the downstream water level at the outlet.

Water depth in the tunnel

Distance from outlet (m)	Water depth in the tunnel (m)		Remarks
	No. 1Tunnel	No. 2 Tunnel	
0	7.85	7.85	Outlet water level : EL. 42.15
40	7.56	7.68	
80	7.26	7.51	
120	6.95	7.34	
160	6.62	7.16	
200	6.29	6.97	
240	5.90	6.77	
280	5.45	6.56	
320	5.27	6.32	
360	4.71*	6.02	*No. 1 Tunnel : under critical depth
380	-	5.78 *	*No. 2 Tunnel : under critical depth

Calculation Condition

	No. 1 Tunnel	No. 2 Tunnel
Discharge (m^3/s)	250	350
Critical depth (m)	4.627	5.623
Depth of Uniform flow (m)	3.326	4.817

From the result of the calculation with discharge of 600 m^3/s, flow with a critical depth occurred in the tunnel. Therefore, the free flow in the tunnel was not influenced by the downstream water level at the outlet. And the inlet water level also was not influenced by the downstream water level at the outlet.

- Upstream water level

From the calculation result mentioned above, the inflow discharge at the inlet was governed only by the critical depth. Then the upstream water level was calculated by the following equation.

$$W.L = Z + Hc + Vc^2/(2g) + fsc.Vc^2/(2g)$$

Where; W.L : Upstream water level (m)
Z : Elevation of inlet base (No.1 T : EL. 37.50, No.2 T : EL. 36.00)
Vc : Critical velocity (m/s)
fsc : Coefficient of entrance loss (=0.2)

Now, the relation between critical velocity and critical depth can be presented in the following equation.

$$Vc/\sqrt{g.Hc} = 1.0$$

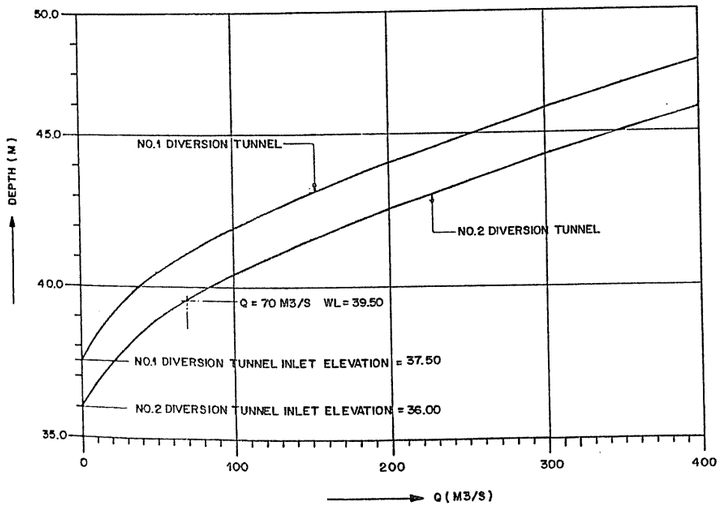
Therefore, equation for W.L can be represented in the following equations.

$$No.\ 1\ Tunnel\ :\ WL_1 = Z_1 + 1.6\,Hc_1 = EL.\ 37.5 + 1.6\,Hc_1$$
$$No.\ 2\ Tunnel\ :\ WL_2 = Z_2 + 1.6\,Hc_2 = EL.\ 36.0 + 1.6\,Hc_2$$

The result of calculation using by the critical depth and above equations, is shown in the table and figure below. From the calculation result, the upstream water level at the inlet for a discharge of 600 m^3/s was estimated to be EL. 45.00.

Tunnel discharge and upstream water level

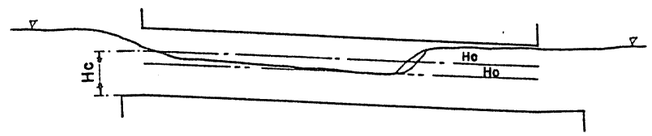
Upstream water level EL. (m)	Discharge (m^3/s)			Remarks
	No. 1 Tunnel	No. 2 Tunnel	Total	
39.0	20	53	73	
40.0	40	84	124	
41.0	65	128	193	
42.0	104	175	279	
43.0	148	225	373	
44.0	200	286	486	
45.0	258	350	608	D/S WL. = 42.15 m



WL and Discharge

- Shape of water depth in the tunnel

From the result of the above calculation the shape of water depth in the tunnel was estimated to be as shown in the following figure.



(ii) Pipe Flow

From the calculation result for pipe flow, the upstream and downstream water level was estimated to be EL. 49.90 and EL. 44.30 respectively when the design flood of 1300 m^3/s would flow through the two tunnels.

(3) Crest Elevation of Cofferdam

(a) Primary cofferdam

The crest level of the upstream and downstream primary cofferdam was considered respectively from the result of the hydraulic design and the rating curve at dam site.

The height of the cofferdams should have 1.5m free board because they were build up by rock and soil.

	Diversion capacity (m^3/s)	Water level EL. (m)	Crest level EL. (m)
Upstream cofferdam	600	45.00	46.50
Downstream cofferdam	600	42.15	44.00

(b) Secondary cofferdam

The secondary cofferdam would be constructed immediately after the completion of the primary cofferdam. The structure of the upstream and downstream cofferdam would concrete gravity dam and rockfill dam with 1.5m and 1.0m free board from the water level of 1300 m^3/s respectively in consideration of the importance of the structure and the retention effect by upstream cofferdam.

	Diversion capacity (m^3/s)	Water level EL (m)	Crest level EL (m)
Upstream cofferdam	1300	49.90	51.50
Downstream cofferdam	1300	44.30	45.30

6.5.3 Structural Design of Diversion Tunnel

(1) Tunnel

(a) Design Condition

(i) Design Calculation

The Otto-Frey-Baer equation which is widely used for the design of the pressure tunnel is used for the calculation of the stress occuring in the concrete lining due to the inner pressure. In this study, however, the calculation is conveniently simplified by using the graphs prepared Nishikawa-Fukuda et al. (Source : Water Power Engineering Nov. 1972, Jan. 1973 and Jan. 1974, Japan). Assumptions in the Otto-Fley-Baer Equation are as followings.

- The bedrock extends infinitely
- Both concrete and bedrock are homogeneous and isotropic
- Both concrete and bedrock are continuous
- The cover depth of the reinforcement is not taken into account (d=d'=0)
- The reinforcing bars are converted to a thin cylinder
- Any tangential tensile stresses don't occur by the crack produced in the lining concrete.

(ii) Calculation cases

The calculation was made for three cases. Both inner and external pressures were applied for each case.

	Rock class	Modulus of elasticity
Case 1	CH - Class	Er = 40,000 kg/cm^2
Case 2	CM - Class	Er = 20,000 kg/cm^2
Case 3	CL - Class	Er = 10,000 kg/cm^2
Case 4	D -Class	Er = 7,000 kg/cm^2

(iii) Loads

Item	Design value	Remarks
Inner pressure	2.7 kg/cm^2	Highest water level : EL.67.50 Lowest level of tunnel : EL.41.00
External pressure	2.0 kg/cm^2 5.0 kg/cm^2	Mortar injection (crown only) Consolidation and curtain grouting (uniformly around entire section)

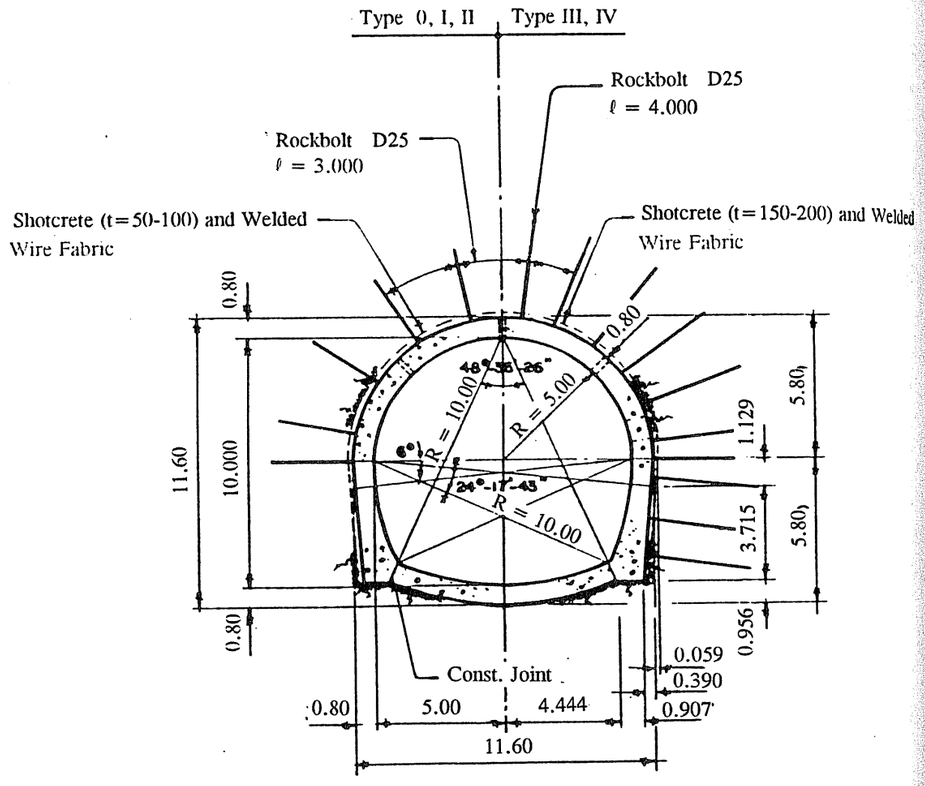
(iv) Allowable stresses

In order to make the design of the tunnel lining more economical, allowable stresses in the calculation were modified based on the Standard Specification for Concrete, Japan Society of Civil Engineering, 1986, as follows during construction.

(Unit : kg/cm²)

	Original Design	Modified Design	Remarks
Allowable compressive stress of concrete	60	96	Concrete B-2 type σck = 260 kg/cm².
Allowable tensile stress of concrete	0	20.4	σcta = 0.5 σck$^{2/3}$
Allowable tensile stress of reinforcement	2000	2000	SD.30

(v) Standard section of the tunnel



(b) Structural Calculation

(i) Calculation against inner pressure

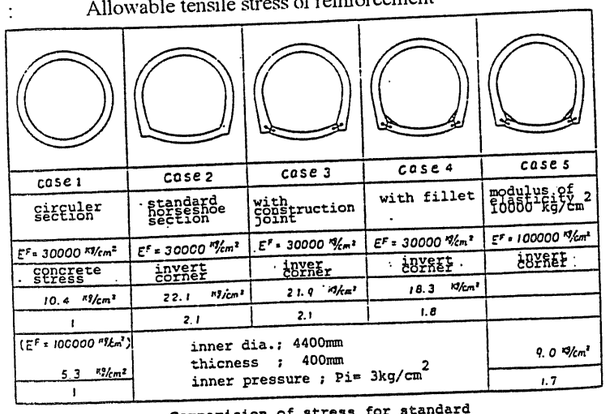
Calculation condition

Inner diameter of tunnel	(D)	10.0 m
Thickness of lining	(Ct)	0.8 m
Inner pressure	(Pi)	2.7 kg/cm²

Calculation result

	Modulus of Elasticity of Rock (Er) (kg/cm²)				Remarks
	40,000	20,000	10,000	7,000	
Ct/D σct σct1 σct2 Evaluation	0.08 3.2 kg/cm² 8.7 kg/cm² 18.2 kg/cm² < σcta	0.08 4.4 kg/cm² 11.9 kg/cm² 25.0 kg/cm² > σcta	0.08 5.4 kg/cm² 14.6 kg/cm² 30.7 kg/cm² > σcta	0.08 5.7 kg/cm² 15.4 kg/cm² 32.4 kg/cm² > σcta	From Fig.1.6.5-3 σct1 = σct x Pi σct2 = σct1 x 2.1
Fe (cm²/cm) Fe/D x 10³ σst σst1 σst2 Evaluation	No Reinforcement - - - - -	0.096 D19 @ 300 0.096 140 kg/cm² 378 kg/cm² 794 kg/cm² < σsta	0.096 D19 @ 300 0.096 240 kg/cm² 648 kg/cm² 1361kg/cm² < σsta	0.162 D19 @ 300 0.162 340 kg/cm² 918 kg/cm² 1928kg/cm² < σsta	From Fig.1.6.5-4 to 1.6.5-5 σst1 = σst x Pi σst2 = σst1x 2.1

Note : σct : Tangential stress of concrete acting unit inner pressure.
σct1 : Tangential stress of concrete acting design inner pressure.
σct2 : Tangential stress of invert corner of concrete in the standard horse-shoe section, which was expected approximately 2.1 times larger than that of a circular section. (Refer to the table below).
σcta : Allowable tensile stress of concrete.
Fe : Tangential reinforcement content.
σst : Tangential stress of reinforcement acting unit inner pressure.
σst1 : Tangential stress of reinforcement acting design inner pressure.
σst2 : Tangential stress of invert corner of reinforcement of the standard horse-shoe section.
σsta : Allowable tensile stress of reinforcement



Comparision of stress for standard horse-shoe section and circular section

(ii) Calculation against external pressure caused by mortar injection.

- Calculation Condition

The external pressure by mortar grouting was assumed to act only around the crown part of tunnel section because the tunnel excavation employed NATM and there was no opening between concrete lining and tunnel support except the crown part.

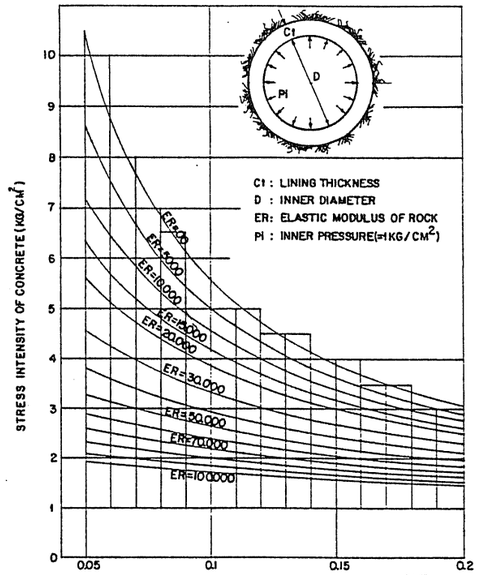


Fig. 1.6.5-3 Stress of Concrete Generated by Inner Pressure

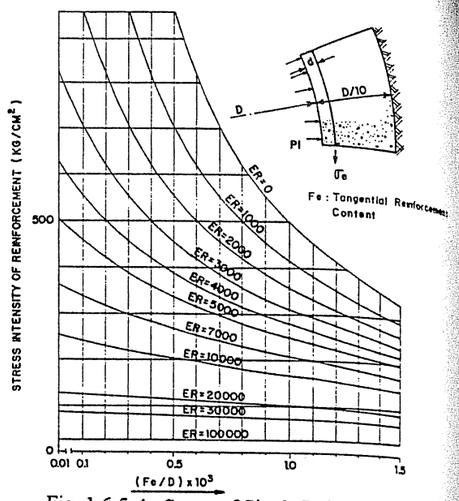


Fig. 1.6.5-4 Stress of Single Reinforcement Generated by Inner Pressure

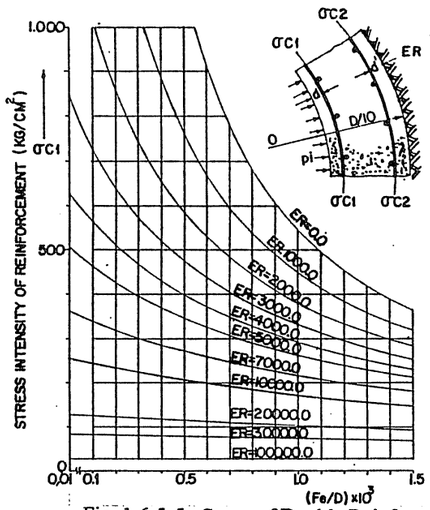


Fig. 1.6.5-5 Stress of Double Reinforcement Generated by Inner Pressure

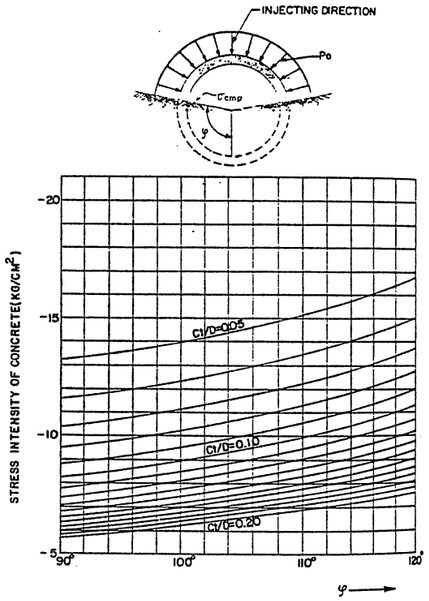


Fig. 1.6.5-6 Stress of Concrete Generated by Ununiformed External Pressure

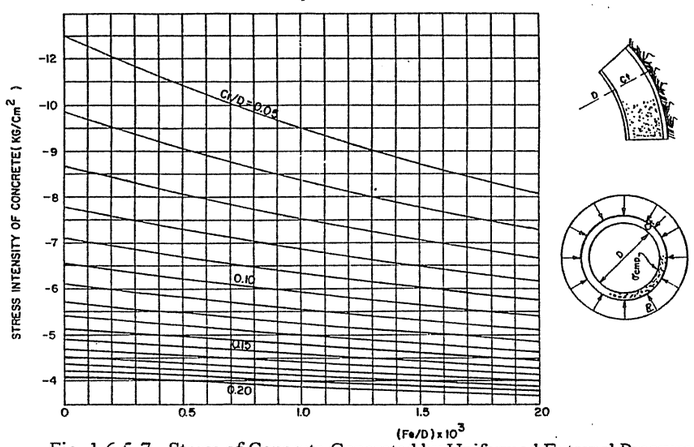


Fig. 1.6.5-7 Stress of Concrete Generated by Uniformed External Pressure

- External pressure P_{0-1} = 2.0 kg/cm²
- Non acting range of external pressure φ = 90°

- Calculation Result

From Fig. 1.6.5-6, the compressive stress of concrete generated by the external pressure of 1 kg/cm² (σcmp) was determined to be of 9.5 kg/cm². Then the compressive stress of concrete generated by the design external pressure (σcc) was as follows.

$$\sigma cc = \sigma cmp \times P_{0-1} = 9.5 \times 2.0 = 19.0\ kg/cm^2 < \sigma cca = 96\ kg/cm^2$$

Therefore, there was not need for some reinforcement to strengthen against the external pressure caused by mortar injection.

(iii) Calculation against external pressure caused by grouting.

- Calculation Condition

The external pressure caused by the high pressure grouting was assumed to act uniformly around the entire circle and the compressive stresses were generated in the lining.

- External pressure P_{0-2} = 5.0 kg/cm²
- Reinforcement D19 @ 300 (Fe = 0.096 cm²/cm)

- Calculation result

From Fig. 1.6.5-7 and Ct/D = 0.08, Fe/D x 10^3 = 0.096, compressive stress of concrete generated by the unit external pressure (σcmp) was calculated to be 7.7 kg/cm².

Then, the compressive stress of concrete generated by the design external pressure (σcc1) was as follows.

$$\sigma cc1 = \sigma cmp \times P_{0-2} = 7.7 \times 5.0 = 38.5\ kg/cm^2 < \sigma cca = 96\ kg/cm^2$$

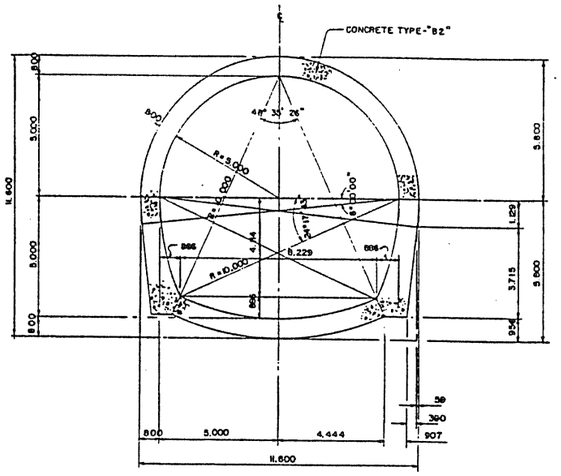
And the compressive stress at the invert corner of concrete in the standard horse-shoe section (σcc2) was as follows.

$$\sigma cc2 = \sigma cc1 \times 2.1 = 38.5 \times 2.1 = 80.9\ kg/cm^2 < \sigma cca = 96\ kg/cm^2$$

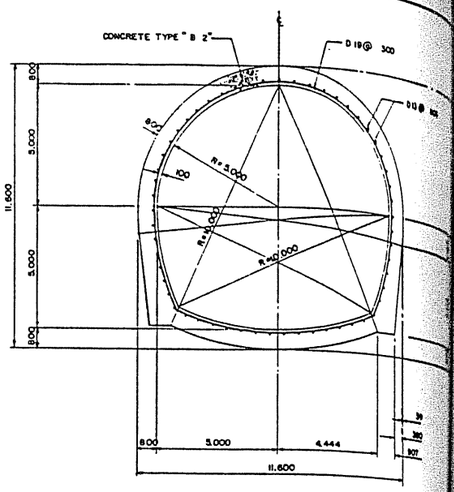
Therefore, there was not need for some reinforcement to strengthen against the external pressure caused by high pressure grouting.

(iv) Result of Calculation

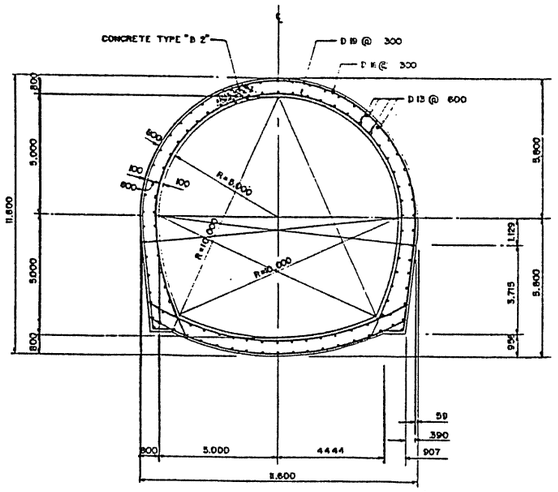
The summarized results of these calculations and the arrangement of reinforcement is shown in Fig. 1.6.5-8. The original design and the modified design are compared in the following table.



TYPE - I



TYPE - II



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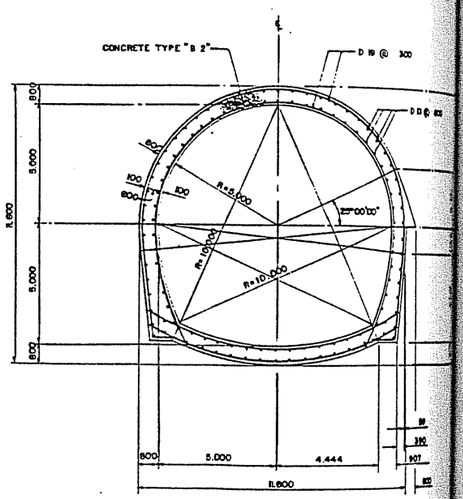


TYPE - IV

Rock classification (Modulus of Elasticity)	Arch and side wall		Invert		Remarks
	Main bar	Distribution bar	Main bar	Distribution bar	
CH (40,000 kg/cm²)	None	None	None	None	Type - I
CM-CL (20,000~10,000kg/cm²)	D19@ 300	D13@600	D19@ 300	D13@ 600	Type - II
D (7,000 kg/cm²)	D19@300 D16@300	D13@600 D13@600	D19@300 D16@300	D13@600 D13@600	Type-III
Tunnel Entrance	D19@300 D16@300	D13@600 D13@600	D19@300 D16@300	D13@600 D13@600	Type-IV

Note : For the distribution bars, Fe x 1/5

Fig. 1.6.5-8 Tunnel Reinforcement Pattern

Tunnel Reinforcement, Original & Modified

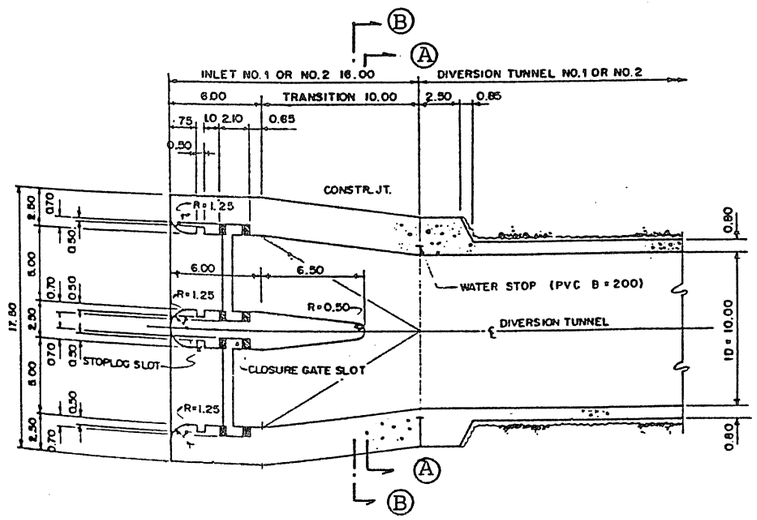
Type	Original (Tender)				Modified			
	Arch and Side wall		Invert		Arch and Side Wall		Invert	
	Main Bar	Distribution Bar	Main Bar	Distribution Bar	Main Bar	Distribution Bar	Main Bar	Distribution Bar
I	D 13 @ 300 (Double)	D 13 @ 600	D 13 @ 300 (Double)	D 13 @ 600	None	None	None	None
II	D 16 @ 300 (Double)	D 13 @ 600	D 16 @ 300 (Double)	D 13 @ 600	D 19 @ 300 (Single)	D 13 @ 600	D 19 @ 300 (Single)	D 13 @ 600
III	D 19 @ 300 (Double)	D 13 @ 600	D 19 @ 300 (Double)	D 13 @ 600	D 19 @ 300 D 16 @ 300 (Double)	D 13 @ 600	D 19 @ 300 D 16 @ 300 (Double)	D 13 @ 600
IV	D 19 @ 300 (Double)	D 13 @ 600	D 22 @ 300 (Double)	D 16 @ 600	D 19 @ 300 (Double)	D 13 @ 600	D 19 @ 300 (Double)	D 13 @ 600

(2) Inlet

(a) Design Condition

Unit weight	Reinforced concrete	2.4 t/m³
	Soil	1.8 t/m³
	Water	1.0 t/m³
Working loads	Concrete own weight	6.0 t/m
	Earth weight	2.4 t/m
	Water weight	19.0 t/m
	Live load	3.6 t/m
	Hydrostatic pressure	27.0 t/m
Allowable stress intensity	Compressive stress of concrete	96 kg/cm² (σck = 260 k g/cm²)
	Tensile stress of reinforcement bar	3000 kg/cm² (short period)

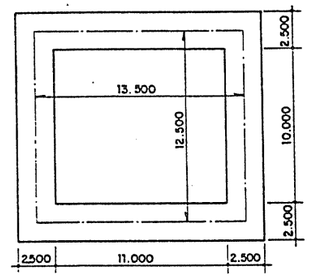
(b) Calculation Section

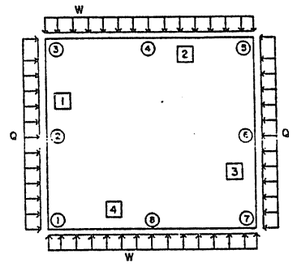


(c) Calculation

Section A - A

(1) Calculation Model and Local Condition





Vertical Load W = 31.0 t/m
Horizontal Load Q = 27.0 t/m

(2) Bending Moment

Member	Node	Length (m)	Moment (t.m)
1	1	0.000	-413.4
	2	6.250	114.0
	3	12.500	-413.4
2	3	0.000	-413.4
	4	6.750	293.0
	5	13.500	-413.4
3	5	0.000	-413.4
3	6	6.250	114.0
	7	12.500	413.4
4	7	0.000	413.4
	8	6.750	293.0
	1	13.500	-413.4

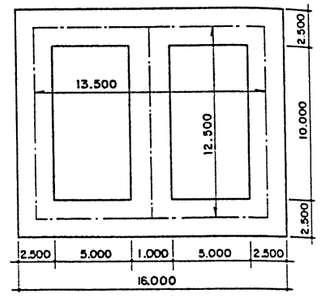
(3) Calculation for Stresses of Reinforced concrete

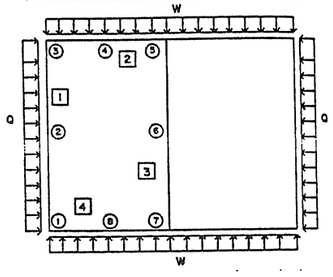
M (t.m)	B (cm)	T (cm)	D (cm)
413.4	100	310	300
293.0	100	250	240

As (cm2)	P	σc (kg/cm²)	σs (kg/cm²)
64.24	0.0021	44	2263
46.45	0.0019	51	2744

Section B - B

(1) Calculation Model and Local Condition





Vertical Load W = 31.0 t/m
Horizontal Load Q = 27.0 t/m

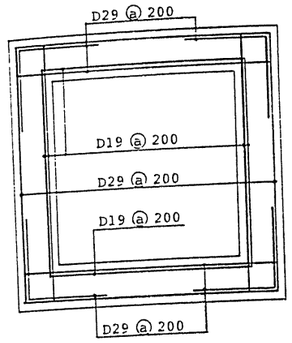
(2) Bending Moment

Member	Node	Length (m)	Moment (t.m)
1	1	0.000	-368.9
	2	6.250	217.1
	3	12.500	-368.9
2	3	0.000	-368.9
	4	3.750	201.8
	5	7.500	-421.0
3	5	0.000	0.000
	6	6.250	0.000
	7	12.500	0.000
4	7	0.000	-421.0
	8	3.750	201.8
	1	7.500	-368.9

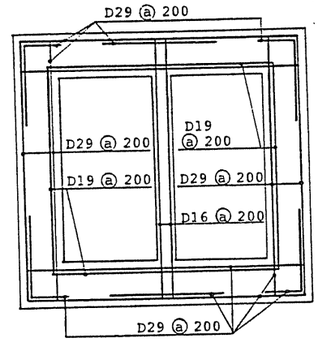
(3) Calculation for Stresses of Reinforced concrete

M (t.m)	B (cm)	T (cm)	D (cm)
421.0	100	310	300
217.1	100	250	240

As (cm2)	P	σc (kg/cm²)	σs (kg/cm²)
64.24	0.0021	45	2312
46.45	0.0019	38	2048



Section A-A



Section B-B

Arrangement for Reinforcement

6.5.4 Tunnel Support

(1) Original Design

(a) Geological Condition

From the result of the geological investigation, the geological condition along the route of the tunnel was presumed as follows.

The diversion tunnel was planned in the left river bank. The bed rock at the dam site consisted of diacidic tuff. The fresh part had a uniaxial strength of 400 - 700 kgf/cm² and the elastic wave velocity was approximately 3.2 km/s. However, it was weathered in the left bank of dam site with marked presence of joints and fissures.

Along the tunnel axis, solid rock, class CH, was found below the elevation of 40m. The surface layers up to 20m below the surface were markedly weathered and rocks of classes CL-D were present.

(b) Original design of tunnel support

The original tunnel support pattern was determined based on the standards of the Japanese Road Association and that of Cirata Hydro Power Project. The pattern consisted of four (4) types, namely of Type - I, II, III and IV, that were applied to the CH class in the center part of tunnel, CH-CM class rock with little overburden and tunnel entrance/large sized fracture zones respectively as shown in Table 1.6.5-2 and Fig. 1.6.5-9.

(2) Modified Design during Construction

An engineering classification which was developed by Bieniawski to show a geomechanics classification guide for excavation and support in rock tunnels, was performed to observe the geological condition in the excavation of the Temporary Adit, so as to examine the geological condition in the diversion tunnel and the support system for its validity.

From the result of the observation in the Tunnel Adit, the rock mass rating (RMR) by Bieniawski were within the range 61 to 87 and the rock was classified into Class 1 (Very Good Rock) or Class 2 (Good Rock) by the Geomechanical Classification Guide by Bieniawski as shown in Table 1.6.5-3. In accordance with this guide, the tunnel support of Class 1 and Class 2 were recommended to be generally no support and rock bolts of 3m long in crown spaced at 2.5m with shotcrete of 50mm thick without wire mesh respectively.

A fifth type of tunnel support Type 0 was determined additionally to be applied for class 2 rock - good rock, namely of rock bolts of 3m long in crown spaced at 2.0m with shotcrete 50mm thick without wire mesh which was a lighter support than Type I.

The modified tunnel support pattern is shown in Table 1.6.5-2 and Fig. 1.6.5-9.

Table 1.6.5-2 Tunnel Support Pattern

Rock class	Support type	Rook Bolt D25mm			Shotcrete		Steel support	Remarks
		Length (m)	Spacing (m)		Thick -ness (mm)	Wire mesh		
			Ring Dir	Progressive Dir				
1	0	3.00	2.28	2.00	50	None	-	Additional
2	I	3.00	2.28	2.00	50	With	-	Original
3	II	3.00	2.28	1.50	100	With	-	Original
4	III	4.00	1.25	1.20	150	With	-	Original
5	IV	4.00	1.25	1.00	200	With	H-200 x 200 @ 1.00m	Original

Geomechanics Classification by Bieneawski

The Geomechanics Classification or the rock mass rating (RMR) system was developed by Bieniawski in 1973. This engineering classification of rock masses, utilizes the followings six (6) parameters, all of which are measurable in the field. (refer to Table 1.6.5-3).

- Uniaxial compressive strength of intact rock material
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Ground water conditions
- Orientation of discontinuities

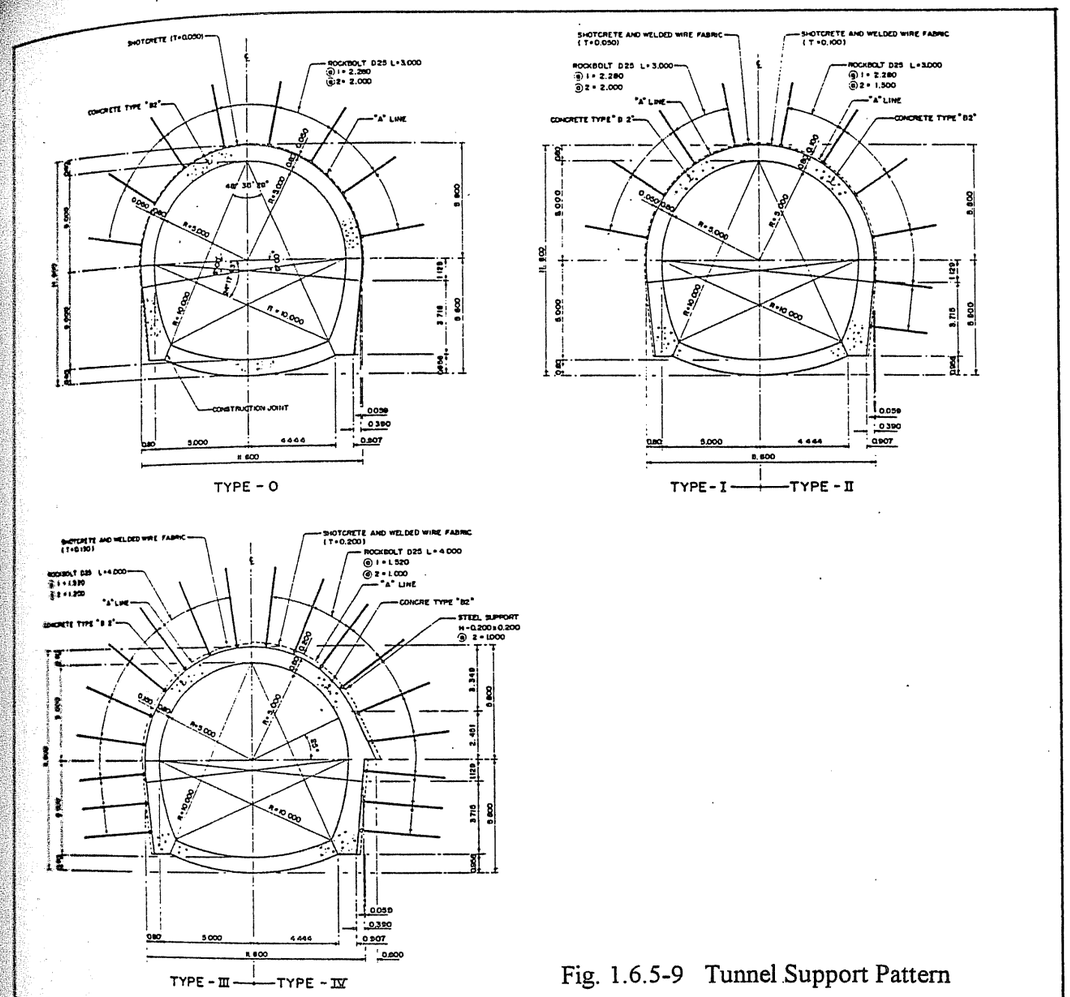


Fig. 1.6.5-9 Tunnel Support Pattern

Table 1.6.5-3 : Geomechanics Classification guide for excavation and support in rock tunnels
Shape : Horseshoe : Width 10m : Vertical stress : below 25 Mpa : Construction : drilling and blasting.

Rock Mass Class	Excavation	Support		
		Rockbolts (20mm dia, fully bonded	Shotcrete	Steel sets
1. Very good rock RMR : 81-100	Full Face : 3 m advance	Generally no support required except for occasional spot bolting		
2. Good rock RMR : 61-80	Full Face : 1.0 - 1.5m advance Complete support 20m from face	Locally bolts in crown 3m long, spaced 2.5 m with occasional weld wire fabric	50 mm in crown where required	None
3. Fair rock RMR : 41-60	Top heading and bench : 1.5 - 3m advance in top heading ; Commence support after each blast ; Complete support 10m from face	Systematic bolts 4m long, spaced 1.5m - 2m in crown and walls with welded wire fabric in crown	50-100 mm in crown and 30mm in sides	None
4. Poor rock RMR : 21-40	Top heading and bench 1.0 - 15m advance in top heading ; Install support concurrently with excavation - 10m from face	Systematic bolts 4 - 5m long, spaced 1 - 1.5m in crown and walls with welded wire fabric	100-150 mm in crown and 100 mm in sides	Light ribs spaced 1.5m where required
5. Very poor rock RMR : < 20	Multiple drifts : 0.5 - 1.5m advance in top heading ; Install support concurrently with excavation ; shotcrete as soon as possible after blasting	Systematic bolts 5-6m long spaced 1 - 1.5m in crown and walls with welded wire fabric Bolt invert	150 - 200 mm in crown 150 mm in sides and 50 mm on face	Medium to heavy ribs spaced 0.75m with steel lagging and fore polling if required. Close invert.

6.5.5 Plan of Diversion Plug

(1) General

After the reservoir started impounding by closing the closure gate at the a inlet, a concrete plug was constructed in the diversion tunnel in order to cut off flow and water leakage through the tunnel. Consolidation grouting and curtain grouting also was carried out for foundation rocks around the tunnel in order to consolidate rocks around the tunnel damaged from blasting during the tunnel excavation and cut off the water flow around the tunnel. The grout curtain of the plug concrete was generally planned to be on the same plane as the dam grout curtain.

(2) Original Plan of Diversion Plug

The plug concrete was designed to be located just downstream of the dam axis. The plug in the of diversion tunnels should resist against shearing and sliding forces caused by horizontal thrust from the water pressure acting on the front of the plug concrete. From the results of the above calculation and other project experiences, the design plug length was determined 40m, (temporary plug 10m and main plug 30m). Consolidation grouting and curtain grouting were designed to carry out within the range of 3m from the rock surface around the rock of main plug concrete and 10m from the rock surface around the rock on the same plane as the dam grout curtain.

Then the gallery was planned in the plug concrete for the purpose of carrying out cooling and curtain grouting works.

(3) Modified Plan During Construction

In the original construction program, the diversion plug works would be carried out in the dry season, so consolidation grouting was carried out in the tunnel before placing main plug concrete. The diversion plug works, however, could not be commenced in the rainy season based on the actual construction schedule. Therefore, it was necessary to complete the plugging works earlier because the reservoir water level would rise up rapidly and reach HWL about 5 months later after the impounding started in the rainy season at the Kotapanjang Dam.

As mentioned above, the construction procedure was revised for the purpose to improve the safety and reliability of consolidation grouting as shown in the following table.

Construction Procedure of Plugging works

Original : Closing Gate → Temporary Plug → Consolidation Grouting *1 → Main Plug → Curtain Grouting *2

Modified : Closing Gate → Temporary Plug → Main Plug → Consolidation Grouting *2 → Curtain Grouting *2

Note : *1 From Diversion Tunnel
*2 From Gallery

And the shape, size and length of the gallery were modified as shown in the table below and Fig 1.6.5-10 during construction in order to increase the efficiency of grouting works and complete the plugging works as fast as possible.

Dimension of Plug Works

	Original Design	Modified Design	Remarks
Plug length	40.0 m	40.0 m	
Consolidation Grouting			
Drilling length	3.0 m	3.0 m	From rock surface
Spacing of ring direction.	3.9 m	3.9 m	
Spacing of progress direction.	3.0 m	Basically 3.0 m	
Curtain Grouting			
Drilling length	10.0 m	10.0 m	From rock surface
Spacing of ring direction	0.9 m	0.9 m	
Gallery			
Size	(H)2.5mx(W)2.0m	(H)3.0mx(W)2.5m	
Length	27.0 m	28.0 m	Fig. 1.6.5-10
		Type 1 : 3.0 m	(a)
		Type 2 : 25.0 m	(b)

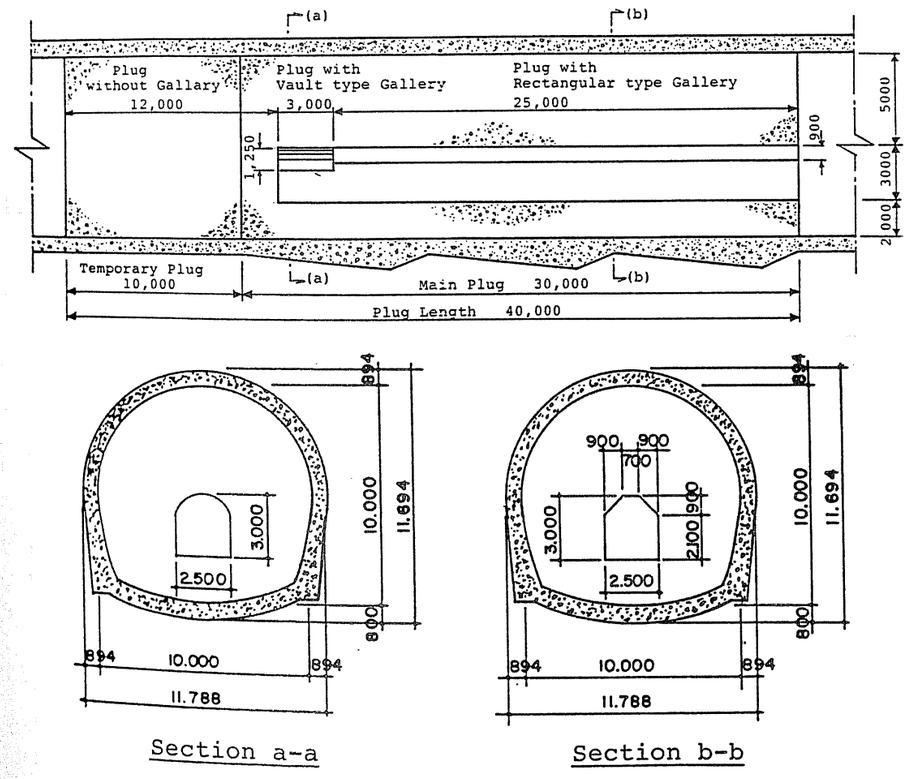


Fig. 1.6.5-10 Shape and Size of Gallery

(4) Stability Analysis for Plug Concrete

(a) Required plug length against shearing stress.

The plug length against shearing stress is calculated by the following equation.

$$L = \alpha \cdot P \cdot A_o / (\tau \cdot l) \quad = 7.513\ m$$

where, L : Plug length (m)
α : Safety factor (=4.0)
P : Water pressure (=44.0 t/m^2)
A_o : Area acted water pressure (=115.238m^2)
τ : Shear strength for contact face concrete and rock (=70 t/m^2)
l : Bond length (=38.566m)

(b) Required plug length against sliding

The plug length against sliding is calculated by the following equation.

$$n = (F.V + \tau.B.L) / H = [F (A_1 l_1 + A_2 l_2) (\gamma_c - 1.0) + \tau.B.L (l_1 + l_2)] / P.A_o$$

$$l_2 = \frac{n.P.A_o - [F.A_1.l_1 (\gamma_c - 1.0) + \tau.B.l_1]}{F.A_2.(\gamma_c - 1/0) + \tau.B} = 6.604\ m$$

$$L = l_1 + l_2 = 21.604\ m$$

where, n : Safety factor of shearing friction (=4.0)
F : Coefficient of internal friction (=0.75)
V : Vertical force (=Σ $A_i l_i$ (γ_c-1.0))
A_1 : Area of plug concrete without gallery or gallery which length was fixed.
l_1 : Length of plug concrete above mentioned (=15.0m)
A_2 : Area of plug concrete with gallery which length is required.
l_2 : Length of plug concrete above mentioned
γ_c : Unit weight of plug concrete (=2.35 t/m^3)
τ : Shear strength for contact face concrete and rock (=70 t/m^2)
B : Plug width (=11.788 m)
P : Water pressure (=44.0 t/m^2)
A_o : Area acted water pressure (= 115.238 m^2)

The dimension of each part of the plug concrete is shown in the table below.

Dimension of each part of plug concrete

		Unit	Dimension	Remarks
A1	Plug area without gallery	m^2	115.238	
	Plug length without gallery	m	12.0	
	Plug area with gallery	m^2	108.409	Fig.1.6.5-10 (a)
	Plug length with gallery	m	3.0	
ℓ_1	Total (A1 x l_1 x 1.35)	t	2305.912	
A2	Plug area with gallery	m^2	108.548	Fig.1.6.5-10 (b)

The plug length was determined at 40 m taking account of the above calculated length and the other project experiences as shown in Fig. 1.6.5-11. From the data of the existing dams in Japan (39 concrete gravity dams), the plug length is made larger than the calculated length against sliding. Several dams with a large discharge, large cross section of tunnel and also with a high design head (40 - 100 m) which could be considered similar to the Kotapanjang dam have shown L = 0.8 H (L : Plug length, H : Design water depth, L = 0.8 H = 0.8 x 44.0 = 35.2m in the case of Kotapanjang). Therefore the plug length of 40m was reasonably selected considering practical condition.

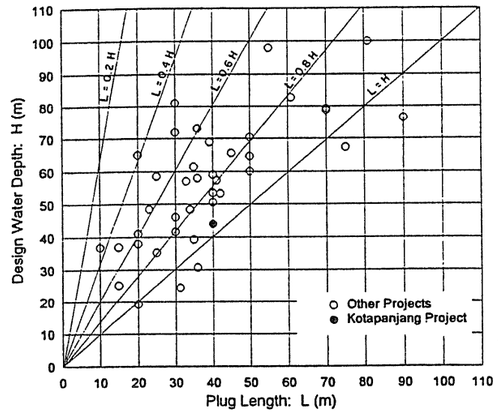


Fig. 1.6.5-11 Relation between Plug Length and Design Water Depth for Concrete Dam

6.5.6 Design of Cofferdam

(1) Original Design

In the original design, primary cofferdams and secondary cofferdams at the upstream and downstream were planned as a fill type and concrete gravity type respectively. And their principal dimensions are shown in Table 1.6.5-4.

(2) Modified Design during Construction

According to the practical construction schedule of starting the diversion in October 1993, the cofferdam construction would take 3.5 months and completion of this works would be in the middle of the rainy season and there was a possibility that the design flood could occur and overtop the cofferdams that were just under construction.

Therefore, it was necessary to make their construction period as short as possible and keep the damage of cofferdams at a minimum during construction in the rainy season. So, the upstream and downstream main cofferdam were modified to be capable of rapid construction as follows ;

(i) The dam body of upstream secondary cofferdam was modified from conventional concrete to roller compacted concrete (RCC). A RCC dam can be built with lean cement concrete which is batched, transported to the site by dump trucks and compacted by vibration roller same as the rockfill dam, resulting in a reduction of construction period greatly.

(ii) The type of downstream secondary cofferdam was modified from a concrete gravity dam to a fill type dam as an extension of the primary cofferdam because the fill type secondary cofferdam could be built more rapidly than the original designed dam and it would also be easier to remove and save construction cost.

(iii) The crest elevation of the downstream primary cofferdam was modified from EL. 44.00 in the design to EL. 46.00 because a flood occurred on October 1993 and had a possibility to overtop the cofferdam just under construction.

Table 1.6.5-4 Dimension of Cofferdams

	Original Design	Modified Design
Upstream primary cofferdam - Type - Crest Level - Dam Height - Gradient Upstream Downstream - Dam Crest Length - Dam Crest Width	Earth - Rock fill EL. 45.50 10.5 m 1:2.0 1:2.0 80.0 m 6.0 m	Earth - Rock fill EL. 46.50 12.5 m 1:2.0 1:2.0 84.8 m 6.0 m
Downstream primary cofferdam - Type - Crest Level - Dam Height - Gradient Upstream Downstream - Dam Crest Length - Dam Crest Width	Earth - Rock fill EL. 43.60 8.1 m 1:2.0 1:2.0 103.0 m 6.0 m	Earth - Rock fill EL. 46.00 11.2 m 1:1.05 1:1.73 106.8 m 7.0 m
Upstream secondary cofferdam - Type - Crest Level - Dam Height - Gradient Upstream Downstream - Dam Crest Length - Dam Crest Width	Concrete gravity (by conventional concrete) EL. 51.50 17.5 m Vertical (With 1:0.5 fillet) 1:0.5 125.0 m 1.0 m	Concrete Gravity (by RCC) EL. 51.50 19.5 m Vertical 1:0.88 132.0 m 3.0 m
Downstream secondary cofferdam - Type - Crest Level - Dam Height - Gradient Upstream Downstream - Dam Crest Length - Dam Crest Width	Concrete Gravity EL. 45.30 10.8 m 1:0.5 Vertical 105.0 m 1.0 m	Earth - Rock fill EL. 45.30 10.5 m 1:2.0 100.3 m 8.0 m

(3) Stability of Cofferdams

(a) Fill Type Dams

The slip circle analysis was applied to calculate the safety factor against sliding under the condition shown in the table below. From the calculation results a minimum safety factor was larger than the required value of 1.2 and therefore the stability was adequate assured against sliding.

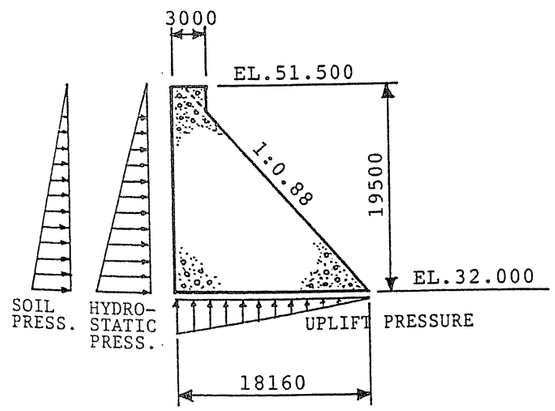
	Upstream Primary	Dowsntream Primary	Secondary
Water level (m)	EL. 45.00	EL. 42.20	EL. 44.30
Embankment material			
(Main part)			
- Cohesion (t/m^2)	5.0	5.0	5.0
- Internal friction Angle (^{0})	22	22	22
- Unit weight (t/m^3)	1.75	1.75	1.75
(Primary part)			
- Cohesion (t/m^2)	5.0	-	-
- Internal friction Angle (^{0})	15	-	-
- Unit weight (t/m^3)	1.60	-	-

(b) Concrete gravity dam

(i) Design Condition

Water level		EL.49.90
Material Property		
- Concrete	Unit weight	2.3 t/m^3
- Soil	Unit weight (in water)	1.2 t/m^3
	(in air)	1.8 t/m^3
	Coefficient of earth pressure	0.5
- Foundation	Shearing strength	70 t/m^2
	Internal friction angle	30^0
	Load bearing capacity	60 t/m^2

(ii) Shape and Calculation Model



(iii) Calculation Result

Total of external force and dead weight

	Vertical force (t/m) (V)	Horizontal force (t/m) (H)	X (m)	M x (tm/m)	Y (m)	My (tm/m)
Dam own weight	355.402	-	5.440	1933.271	-	-
Upstream water pressure	-	126.405	-	-	5.300	669.947
Silt pressure	-	92.259	-	-	5.877	542.206
Uplift pressure	-130.380	-	5.467	-712.787	-	-
Total	225.022	218.664		1220.484		1212.153

Stability against overturning

M (t/m)	V (t)	x (m)	e (m)	B/6
2432.637	225.022	10.811	2.611	< 2.733

Stability against compressive stress

V (tm)	V/B (t/m²)	e (m)	6. e/B	Pu (t/m²)	Pd (t/m²)	Bearing capacity
218.664	13.333	2.611	0.955	26.066	0.600	< 60 t/m²

Stability against sliding

f.V(t)	τ.B (t)	H (t)	n
130.513	1148.000	218.664	5.84 > 4

From the above calculations, the stability against overturning, compressive stress and sliding were assured.

6.5.7 Design of Diversion Closure Gate

(1) General

The closure gate was used firstly for closing the diversion tunnel for reservoir impounding and secondly served as a bulkhead for tunnel plug construction works. Four (4) pieces of sluice gates 5.800 m wide and 10.160 m high, were installed in the inlets of the diversion tunnels. Each gate consisted of skin plate, main girders, intermediate auxiliary beams, seals and hanging brackets. The gates operated in slots in the inlet structures. The slots extended from tunnel invert to the top of the inlet structure.

In the original plan, lowering of the gate was planned to be carried out by using a 80 ton class crawler crane and gantry cranes in the rainy season under the condition that the upstream water level was less than EL. 41.500. Actual lowering of the gates was carried out successfully on 12th March 1997 without externally applied weights under the condition that the upstream water level was EL. 39.70 which was almost equivalent to the dry season's water level.

(2) Gate leaf design

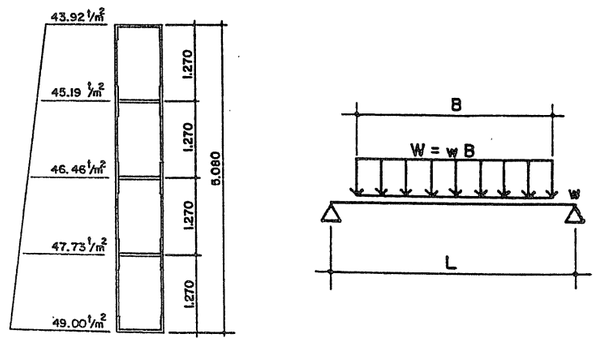
(a) Design condition

(i)	Type of gate	: Sluice gate
(ii)	Quantity	: 2 sets (2 Block/1 set)
(iii)	Gate leaf width	: 5.800 m
(iv)	Clear span	: 5.000 m
(v)	Gate leaf height	: 10.160 m
(vi)	Clear height	: 10.000 m
(vii)	Design head (Front)	: 49.000 m (EL. 85.000)
	(Rear)	: 0.000 m (EL. 36.000)
(viii)	Operation head (Front)	: 5.500 m (EL. 41.500)
(ix)	Sealing system	: Rubber seal (4 edges)
(x)	Allowable corrosion	: 0.5 mm for surface attached water
(xi)	Allowable stress	: In accordance with "Technical Standard for Gate and Penstock" in Japan. Shown in each section *.
(xii)	Allowable deflection	: 1/500 *.

* Give for temporary members acting against temporary load.

(b) Design of main girder

(i) Shape and calculation model



Maximum bending moment : $M\,max = 1/8.W.(2L-B)$
Maximum shearing force : $S\,max = W/2$
Bending stress : $\sigma = M\,max/Z$
Shearing stress : $\tau = S\,max/A$
Deflection : $\delta = W(L^3-LB^2/2 + B^3/8) / (48EI)$

where, W : Charging hydraulic load (kg)
L : Distance of supports (550 cm)
B : Distance of seals (535 cm)

Z : Modulus of section (cm^3)
A : Sectional area of web plate (cm^2)
I : Moment of inertia of section (cm^4)
E : Young's modulus (2.1 x 10^6 kg/cm^2)

(ii) Calculation result

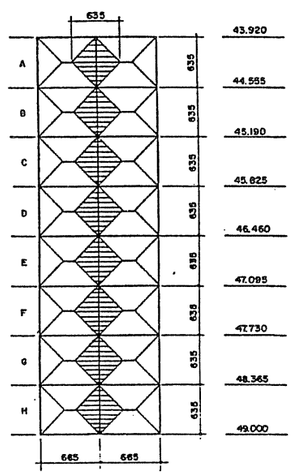
From the calculation result with corrosion allowance of 1.0 mm shown in the table below, the bending stress, shearing force and deflection at each main girder satisfied the following allowable values respectively.

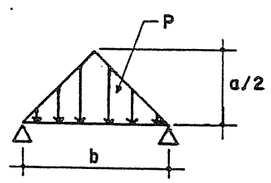
Allowable bending stress σsa = 2400 kg/cm^2
Allowable shearing force τsa = 1350 kg/cm^2
Allowable deflection δa = 1/500

	Main Girder Member				
	G 1	G 2	G 3	G 4	G 5
Used member	C700x200 x 22/36	H700x400 x 22/36	H700x400 x 22/36	H700x400 x 22/36	C700x200 x 22/36
W (t)	149.207	307.043	315.672	324.301	166.465
Mmax (t.m)	105.377	216.849	222.943	229.038	117.566
Smax (t)	74.604	153.522	157.836	162.151	83.233
Z (cm^3)	5031	10062	10062	10062	5031
A (cm^2)	66.05	135.45	135.45	135.45	66.05
I (cm^4)	176085	351691	351691	351691	176085
σ (kg/cm^2)	2095	2155	2215	2276	2337
τ (kg/cm^2)	1129	1133	1165	1197	1260
δ (cm)	0.898	0.925	0.951	0.977	1.002
δ/L	1/557	1/595	1/578	1/563	1/549

(c) Design of Intermediate Auxiliary Beam

(i) Shape and calculation model





Maximum bending moment : $Mmax = 1/12.P.a.b^2$
Maximum shearing force : $Smax = 1/4.P.a.b$
Bending stress : $\sigma = Mmax/Z$
Shearing stress : $\tau = Smax/A$

where, P : Hydraulic pressure (kg/cm^2)
a : Distance of intermediate auxiliary beam (63.5 cm)
b : Distance of horizontal auxiliary (63.5 cm)
Z : Modulus of section (427.2 cm^3)
A : Sectional area of section (15.51 cm^2)
Used member : H 300 x 150 x 6.5/9

(ii) Calculation result

From the calculation result with corrosion allowance of 1.0mm as shown in the table below, the bending stress and shearing stress at each beam satisfied the following allowable values respectively.

Allowable bending stress σsa = 1200 kg/cm^2
Allowable shearing stress τsa = 700 gk/cm^2

	P (kg/cm^2)	Mmax (kg.cm)	σ (kg/cm^2)	Smax (kg)	τ (kg/cm^2)
S1	4.392	93713	219	4427	285
S2	4.456	95079	222	4491	289
S3	4.519	96423	226	4555	294
S4	4.583	97789	229	4619	298
S5	4.646	99133	232	4683	302
S6	4.710	100499	235	4749	306
S7	4.773	101843	238	4811	310
S8	4.837	103208	241	4875	314
S9	4.900	104553	244	4939	318

(d) Design of skin plate

Stress in skin plate was calculated by S. Timoshenkos equation.

$$\sigma = k.p.a^2/[100\ (t-\epsilon)^2]$$

Where, σ : Stress of skin plate (kg/cm^2)
k : Coefficient by b/a (1.40)
a,b : Length of minor or major edges of rectangular plate (a = 36.0 cm, b = 50.5 cm)
p : Mean hydraulic pressure (kg/cm^2)
t : Thickness of skin plate (1.6 cm)
ϵ : Corrosion allowance (0.05 cm)

Calculation result was shown in the following table. From the calculation result, maximum stress of 1167 kg/cm^2 generated at plate "H" satisfied the allowable stress of 2400 kg/cm^2.

	P (kg/cm²)	t - ϵ (cm)	σ (kg/cm²)
A	4.430	1.55	1063
B	4.481	1.55	1076
C	4.557	1.55	1094
D	4.608	1.55	1106
E	4.684	1.55	1124
F	4.775	1.55	1146
G	4.811	1.55	1155
H	4.862	1.55	1167

6.5.8 Temporary Work Adit

(1) General

The progress of the diversion tunnel works could not be accelerated for the following reasons.

(i) The excavation works of the diversion tunnels was not commenced until the open excavation of the inlet and outlet was completed.

(ii) The construction of concrete structures of the inlet and outlet were not able to be commenced while the excavation work of the diversion tunnels was preceding, even after the completion of their open excavation.

(iii) Great damage would be inflicted in the case of the over-topping of the temporary coffering due to the increase in the river water level in the rainy season of early March 1993 when the tunnels were expected to be ready.

In consideration of the above three problems and in an effort to reduce the construction period of the diversion tunnels, an additional temporary work adit was planned during the construction period.

This adit entrance was arranged at the existing old National Road about 60 meters downstream of the No. 2 diversion tunnel, because this area would not interfere with any construction works of other structures, especially dam, power house, tailrace. The alignment of the adit was determined to be satisfied on the following conditions.

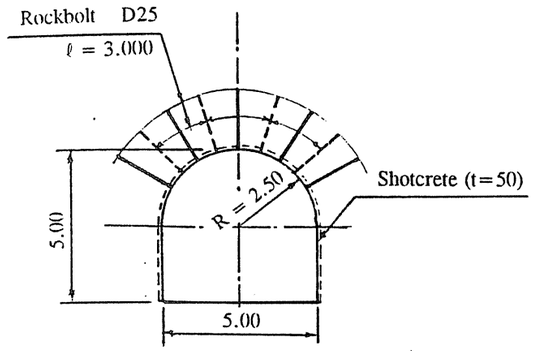
(i) The tunnel length should be become as short as possible.

(ii) The crossing angle between the adit and both diversion tunnels would become almost 90^0.

(2) Design of Temporary Adit

(a) Typical section

The size of the temporary adit was determined to be a vault type of 5.0m (width) x 5.0m (height) in consideration of the size of construction machines for making the diversion tunnel works. Typical section and tunnel support patterns are shown in the following figures.

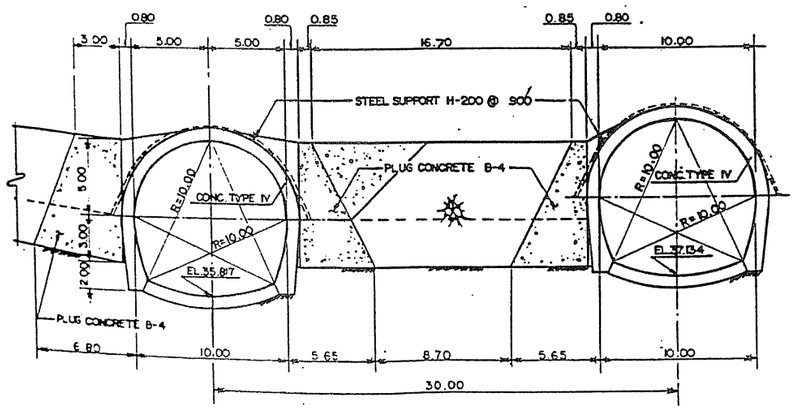


(b) Plug concrete and reinforcement at the junction of the adit and diversion tunnel.

Plug concrete in the temporary work adit located at the junction of the diversion tunnels was determined to provide safety against overturning and sliding caused by external hydraulic pressure when the upstream water level rose above EL.67.50.

And the reinforcement of the diversion tunnel lining concrete located at the junction was determined as shown in the following table in order to provide safety of lining concrete under the condition that external hydraulic pressure acted on the lining ununiformally when upstream water level was EL. 67.50.

Intrados of arch	D 25 @ 300
Extrados of arch	H-200 x 200 x 8 x 12 @ 900



6.6 Intake

6.6.1 Arrangement of Intake.

The function of the intake is to ensure the required water reaches the turbines, to check trash and debris entering with the water and to secure the entry of water with a minimum disturbance so that head loss is a minimum. Therefore, the design of the intake has to satisfy for structural stability, limitation of entrance velocity and hydraulic efficiency.

The intake structure was provided on the upstream of the non-overflow section of the dam, block No.5 to No. 7. It houses the trashrack, stoplog guides and intake gate of the penstock feeding the generating unit of the powerhouse. The stoplog and intake gate guides extend from the bottom of the intake opening to the top of the dam so that the equipment should be approachable for operation under all conditions. In front of the intake, the trash rack is provided with the slope of 1 (V) to 0.3 (H) so that the trash rake is able to be operated smoothly. The general arrangement of intake structure is shown in Fig. 1.6.6-1.

The intake structure consists of (i) bottom platform which is cantilevered from the dam upstream face and supports two side walls and piers , (ii) the two side walls and the two side piers which carry the grooves for the trashrack from EL. 64.0 to EL. 87.5 and the guide for the intake gate and the stoplogs , (iii) a platform at EL.87.5 where the travelling gantry is provided for trash rake operation, (iv) upper portion of the piers supporting the bridge for hoist equipment of the intake gate and (v) the operation bridge.

The data for the intake design

Gradient of upstream face of dam	1 : 0.15
Diameter of penstock	5.000 m
Angle of penstock with vertical	90 °
EL.of center line of penstock at intake	EL.63.500
EL.of dam crest	EL.87.500
High water level	EL.85.000
Normal water level	EL.80.600
Low water level	EL.73.500
Maximum discharge through each turbine	116 m^3/s.

6.6.2 Hydraulic Design

(1) Hydraulic condition

Intake velocity through trashrack

It is known that the permissive velocity through the trashrack is commonly desirable in the range of 0.6 to 1.5 m/s. based on USBR'S criteria. The intake velocity at the position of the trashrack was designed to be under 1.0 m/s. in order to avoid the trashrack operation in high velocity flow. The mean velocity at the trashrack position is as follows

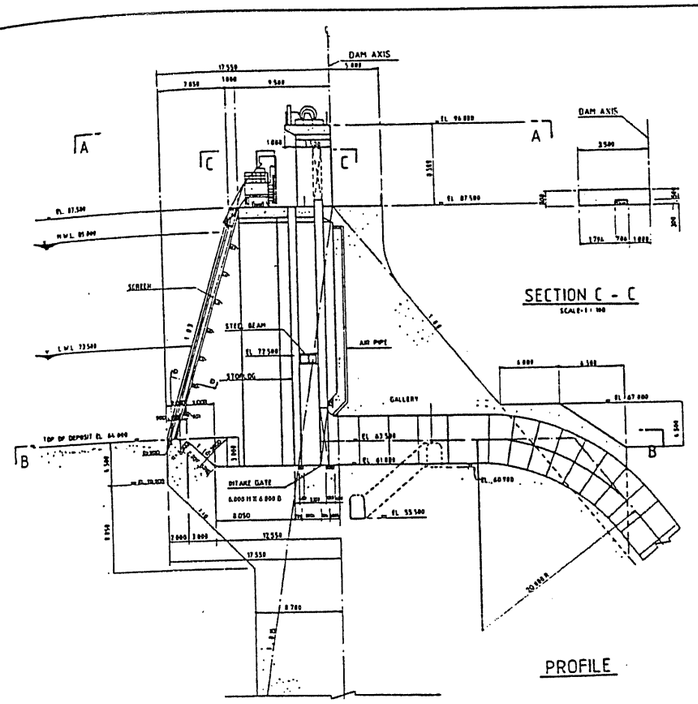


Fig. 1.6.6-1 General Arrangement of Intake Structure

	Maximum Discharge (m^3/s)	Mean Velocity (m/s)
at HWL	102.698	0.489
at NWL	116.000	0.699
at LWL	97.439	1.026

Intake base level

The top of the silt deposit will be at EL. 64.000 based on the possible total silt deposit in one hundred years. Therefore, the intake base was decided at EL.64.000 at the inlet trashrack position.

Height of the intake

The relation between the width B and the height H of the intake is normally selected in the range of B/H = 1.0 to 3.0 . For this site, B = 6.0 m, H = 6.0 m, that is B/H = 1.00 was selected.

The velocity of flow inside the penstock connected with the intake on normally in the range of 4 to 6 m/s. Therefore, on the basis that the velocity at the trashrack was assumed to be 1 m/s, the area of section must reduce to approximately 1/4 to 1/6 between the trashrack and the inlet of the penstock. When the cross section changes abruptly, the streamlines become discontinuous, thus forming retardation areas mainly caused by vortexes which increases head loss at the inlet. Therefore, the gradual change of the cross section was taken into consideration with sufficient care both in plan and in longitudinal section.

The depth of the inlet

The entry to the penstock take off from the reservoir was designed to be placed for having adequate water cover depth in order to avoid the generation of vortices (air entrainment into penstock) occurring in ordinary intake. The possibility of vortex formation depends on the water cover depth and the shape of the inlet structure. Therefore , the final design of the intake was determined after the hydraulic model test was done.

(2) Design by Hydraulic Model test.

In general, it is said that the water depth over the penstock entrance has to be more than 2.5 times the entrance diameter for avoiding the vortex formation. In this case, however, water depth over the penstock entrance is 1.5 times entrance diameter at low water level, and therefore there is possibility of vortex formation. If vortices form at the penstock entrance and air is sucked into the penstock, they may cause a variation in water pressure and vibration of machinery.

The hydraulic model test was carried out at the Hydraulic Laboratory of PLN-LMK for checking the vortex formation, observing flow patterns and velocity distribution, and proposing countermeasure if necessary.

The model was prepared on the basis of the Froudian similarity law, and the scale of the model was fixed at 1/25. In this test, only one intake among three was modeled. Tests were carried out mainly under the following discharge and water level.

Discharge (m^3/s)	Water level (m)
102	EL.85.00 (HWL)
116	EL.80.60 (NWL)
98	EL.73.50 (LWL)

(a) Observation of Vortex

Some of the tests to observe vortex formation in the intake model were carried out under the condition that the reservoir water level and discharge varied by about ten elevations including three main elevations mentioned above and three discharges respectively without trashrack.

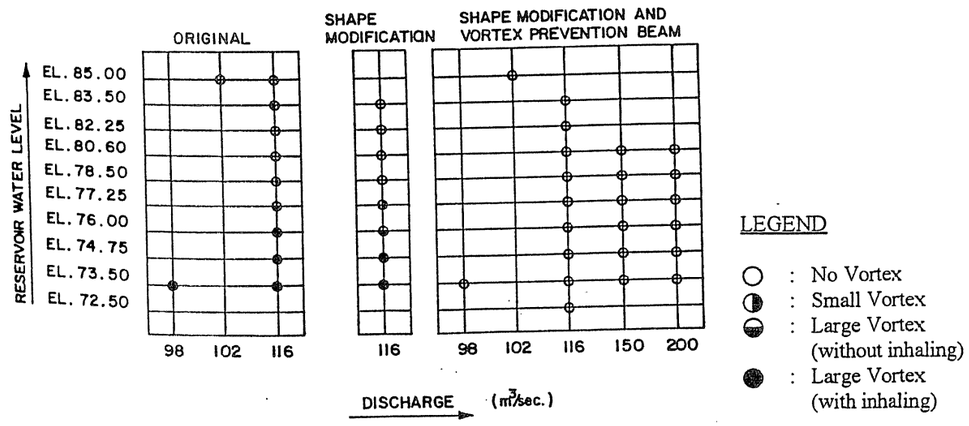
From the test result for the original design model, the vortex was formed at the reservoir water level lower than EL.77.25, that was 3.75 m higher than L.W.L 73.50. And there was observed the swirling flow from the entrance of the intake to gradually constricted transitional part. The results of tests are shown in the figure below.

For preventing vortex formation, the model was modified as followings ;

(i) A vortex preventing beam was added between the gate slot and stoplog slot. Its upper elevation was EL. 72.50 and its height was 1.0 m.

(ii) The guide wall shape inside the intake structure was modified to be straight from the inflow part to the end of the transition part.

From the test results for the modified design model, no vortex occurred at all reservoir water levels. The modified design is shown in Fig.1.6.6-2.



Vortex Formation

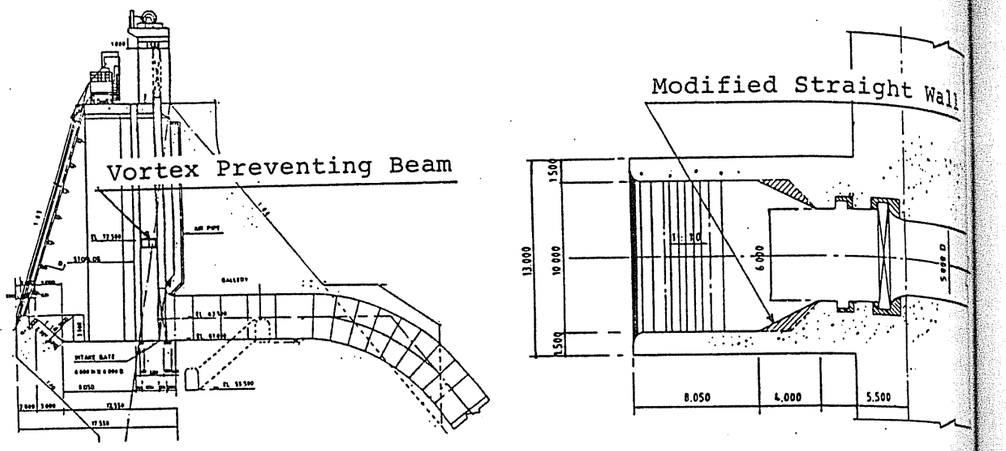


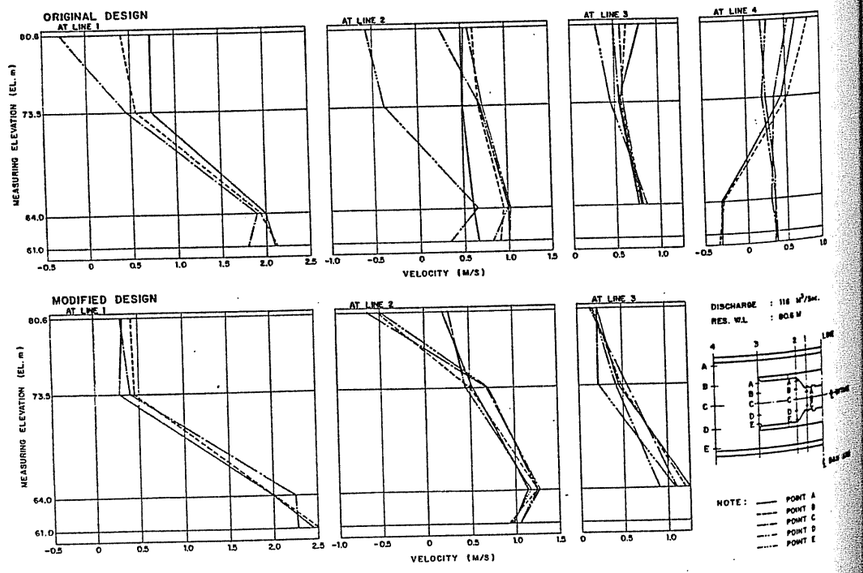
Fig. 1.6.6-2 Modified Intake Design

(b) Velocity Distribution

From the results of these measurements, this modification also improved to make the flow smooth and velocity distribution uniform. Velocity distribution of the original design model and modified design model was shown in the figure below.

(c) Test result for modified intake with Trashrack

At the test for modified intake with trashrack, the flow pattern was improved because the trashrack decreased the size of swirling flow in the intake structure.

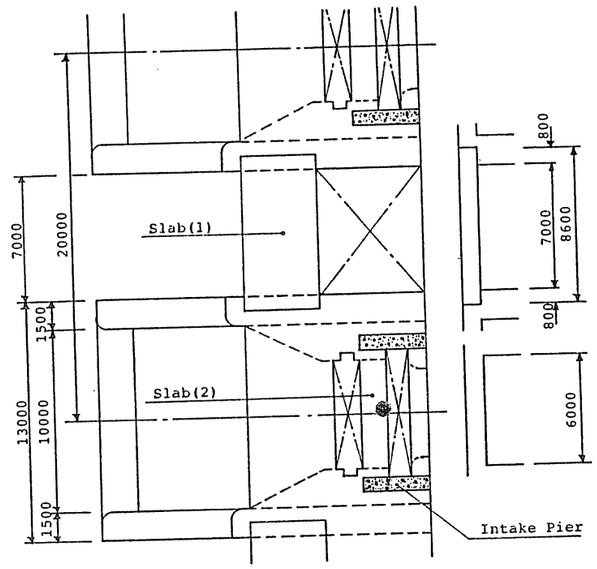


Velocity Distribution

6.6.3 Structural Design

(1) Design Condition

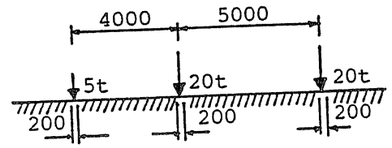
Structural general and Design Component



Design Load

Design Load

Dead Load	Unit weight of Reinforced concrete	2.40 t/m³
Load for Intake Pier	Hoisting machine	10 t/unit
	Motor	3 t/set
	Gate	26 t/unit
	Earthquake force : Seismic intensity	0.10
Load for Slab (1)	Trailer load as shown below	
	Distribution load	300 kg/m²
	Trashrake machine (upstream)	7 t/wheel
	(downstream)	4 t/wheel
	Impact load = Trailer load x Coefficient of impact	
Load for Slab (2)	Gate (temporary kept)	26 t/unit
	Distribution load	300 kg/m²



Allowable stresses

	Item		Long period	Short period
Concrete	Allowable compressive stress	σ ca	60 kg/cm²	90 kg/cm²
	Allowable shear stress	τ a	8 kg/cm²	12 kg/cm²
	Allowable bond stress intensity	τ oa	14 kg/cm²	21 kg/cm²
Reinforcement	Allowable tensile stress intensity	σ sa	2000 kg/cm²	3000 kg/cm²

(2) Structural Calculation

The calculation model, load condition and calculation results of each structure is shown in Table 1.6.6-1.

(3) Calculation Result

The result of reinforcement distribution for pier

Member	Required reinforcement (cm²/m)		Reinforcement Distribution	Stress Intensity (kg/cm²)
	Normal	Earthquake		
1	36.64	2.73	D 32 @ 150 (As = 53.0 cm²)	
2	31.97	28.45	D 25 @ 150 (As = 33.8 cm²)	σ s = 1896 σ c = 55.0 τ = 0, τ o = 0
3	36.72	44.73	D 25 @ 150 (As = 53.0 cm²)	σ s = 1307 σ c = 83.8 τ = 3.6, τ o = 5.5

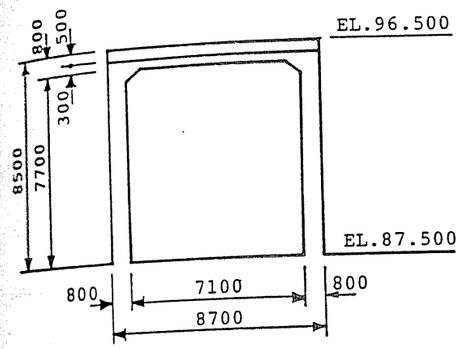
The result of reinforcement distribution for slab

		Cross sectional force (t.m or t)	Required reinforcement (cm²/m)	Reinforcement distribution	Stress Intensity (kg/cm²)
Slab (1)	During Construction	M = 364.87 N=0, Q=0	15.25	D 22 @ 200 (As = 19.4 cm²)	
	Normal Condition	M = 154.88 N=0, Q=0	18.57		σ s = 1917 σ c = 36.9
Slab (2)	During Construction	M = 59.46 N=0, Q=0	19.85	D 25 @ 200 (As = 25.3 cm²)	σ s = 2375 σ c = 53.1
	Normal Condition	M = 19.44 N=0, Q=0	9.52		

Table 1.6.6-1 (1) Structural Calculation Result

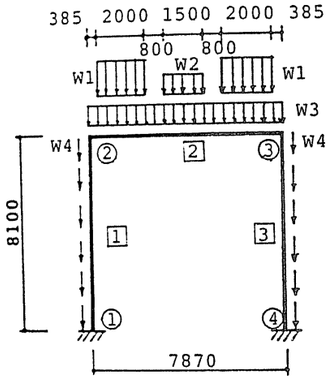
INTAKE PIER

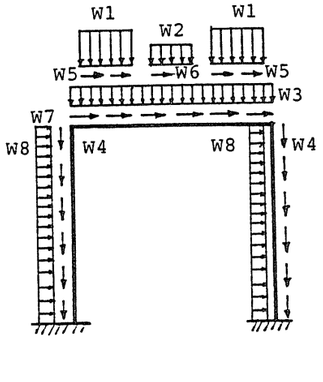
1. Calculation Model and Load Condition



		B	H
1 3	Member	3.50m	0.77m
2	Member	2.15m	0.80m

Gate and Hoisting machine	W1 = 9.0 t/m
Motor	W2 = 2.0 t/m
Own weight of member 2	W3 = 5.69 t/m
Own weight of member 1.3	W4 = 6.72 t/m
Earthquake force of W1	W5 = 0.90 t/m
Earthquake force of W2	W6 = 0.20 t/m
Earthquake force of W3	W7 = 0.57 t/m
Earthquake force of W4	W8 = 0.67 t/m



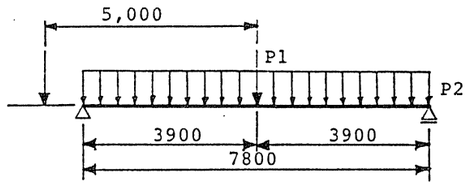
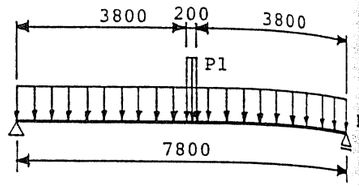


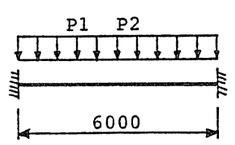
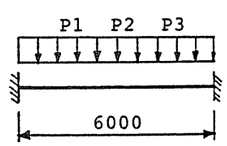
2. Calculation for stresses of reinforced concrete

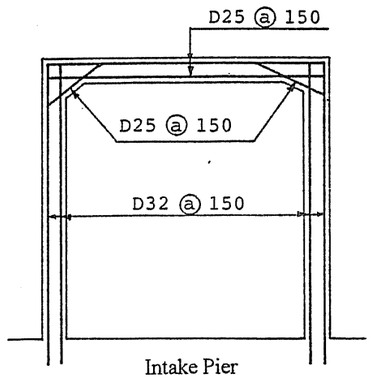
		Normal Condition							Earthquake Condition							
		L (m)	M (txm)	N (t)	Q (t)	B (cm)	T (cm)	D=D' (cm)	L (m)	M (txm)	N (t)	Q (t)	B (cm)	T (cm)	D=D' (cm)
1	2	0.000	40.108	53.890	8.894	100.0	93.7	10.0	0.000	23.357	47.600	3.501	100.0	93.7	10.0
1	Hunch	0.500	43.661	57.250	8.894	100.0	77.0	10.0	0.500	21.690	50.960	3.166	100.0	77.0	10.0
1	1	8.100	23.930	100.322	8.894	100.0	77.0	10.0	4.087	15.298	75.739	0.000	100.0	77.0	10.0
2	2	0.000	48.108	8.894	53.890	100.0	96.7	10.0		21.138	95.984	3.022	100.0	77.0	10.0
2	Hunch	0.885	21.973	8.894	41.355	100.0	80.0	10.0	8.100	22.975	102.032	3.924	100.0	93.7	10.0
2	Max.	3.935	42.835	8.894	0.000		100.0	80.0							
2	Hunch	6.985	21.973	8.894	53.890	100.0	80.0	10.0	0.000	23.357	3.501	47.600	100.0	96.7	10.0
2	3	7.870	48.108	8.894	53.890	100.0	96.7	10.0	0.885	0.367	4.755	35.065	100.0	80.0	10.0
3	3	0.000	48.108	53.890	8.894	100.0	93.7	10.0	3.093	45.413	8.264	0.000	100.0	80.0	10.0
3	Hunch	0.500	43.661	59.938	8.894	100.0	77.0	10.0	6.985	43.579	13.032	47.644	100.0	80.0	10.0
3	4	8.100	23.930	108.322	8.894	100.0	96.7	10.0	7.870	72.859	14.287	60.180	100.0	96.7	10.0
									0.000	72.859	60.180	14.287	100.0	93.7	10.0
									0.500	65.632	66.229	14.890	100.0	77.0	10.0
									8.100	70.835	114.612	21.712	100.0	77.0	10.0

		Normal Condition						Earthquake Condition					
		AS (cm²)	ASD (cm²)	P	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)	AS (cm²)	ASD (cm²)	P	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)
1	2	19.37	19.37	.00232	60.00	1918	1.13	1.73	1.73	.00021	56.52	2999	0.34
1	Hunch	36.64	36.64	.00547	60.00	1279	1.47	2.73	2.73	.00041	68.12	2999	0.42
1	1	1.00	1.00	.00015	43.53	228	0.05	1.00	1.00	.00015	27.36	88	0.00
2	2	28.30	28.30	.00327	47.55	2000	6.83	1.00	1.00	.00012	27.79	60	0.01
2	Hunch	14.80	14.80	.00211	41.71	2000	6.38	8.92	8.92	.00103	39.82	3000	5.80
2	Max.	31.97	31.97	.00457	56.45	2000	0.00	1.00	1.00	.00014	.93	0	0.46
2	Hunch	14.80	14.80	.00211	41.71	2000	6.38	22.45	22.45	.00321	70.81	3000	0.00
2	3	28.30	28.30	.00327	47.55	2000	6.83	20.63	20.63	.00295	70.91	3000	7.44
3	3	19.37	19.37	.00232	60.00	1918	1.13	28.45	28.45	.00328	71.88	3000	7.63
3	Hunch	36.72	36.72	.00548	60.00	1245	1.47	22.16	22.16	.00265	86.51	3000	1.84
3	4	1.00	1.00	.00015	43.53	228	0.05	36.33	33.33	.00542	90.00	2177	2.48
								44.73	44.73	.00668	90.00	1524	3.59

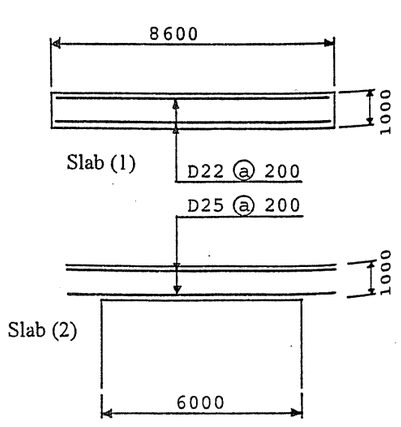
Table 1.6.6-1 (2) Structural Calculation Result

SLAB (1)	
Normal Condition	**During Construction**
1. Calculation Model and Load Condition	1. Calculation Model and Load Condition
	
Trashrake load P1 = 11 t	Trailer load (included impact load) P1 = 268 t/m
Own weight of slab and distribution load P2 = 13.5 t/m	Own weight of slab and distribution load P2 = 25.65 t/m
2. Bending Moment Maximum Bending Moment	2. Bending Moment Maximum Bending Moment
Mmax = 154.88 tm/9.5m	Mmax = 364.89 tm/9.5m

SLAB (2)	
Normal Condition	**During Construction**
1. Calculation Model and Load Condition	1. Calculation Model and Load Condition
	
Own weight of slab P1 = 2.88 t/m Distribution load P2 = 0.36 t/m	Own weight of slab P1 = 2.88 t/m Distribution load P2 = 0.36 t/m Gate (temporary kept) P3 = 6.67 t/m
2. Bending Moment Maximum Bending Moment	2. Bending Moment Maximum Bending Moment
Mmax = 19.44 t.m	Mmax = 59.46 t/m



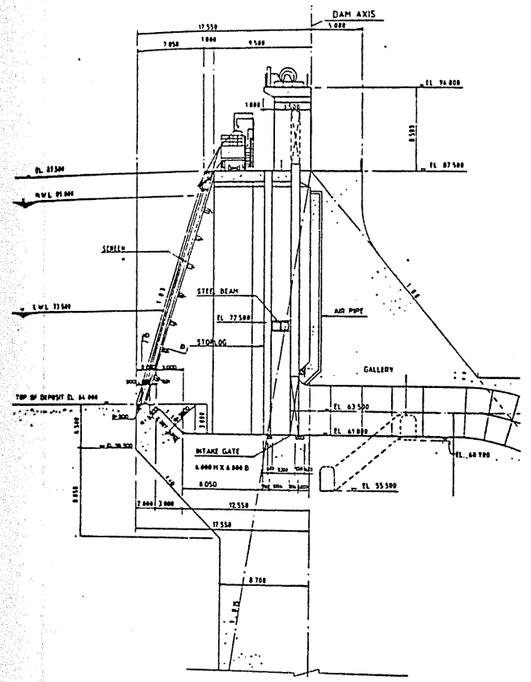
Reinforcement Arrangement



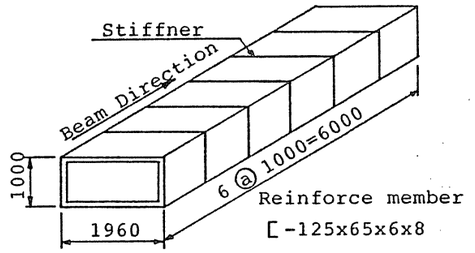
6.6.4 Design of Vortex Prevention Beam.

(1) Design Conditions

At the entrance of the inlet, a beam which prevents the generation of vortex was planned at EL. 72.500 (L = 6.0 m), as shown in the figure below. This beam was finally designed based on the hydraulic model test.



General structure (Inlet)



Shape of the beam

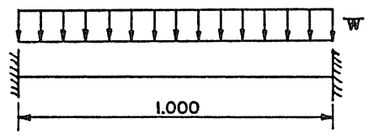
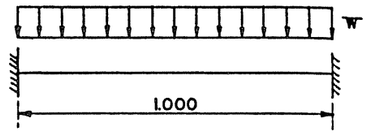
Design Load

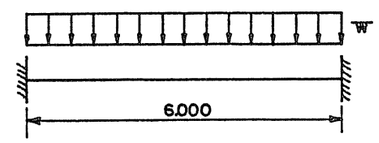
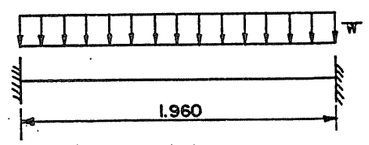
Unit weight	Steel (Ws)	7.85 t/m³
	Water (Ww)	1.00 t/m³
Acting load	Dynamic water pressure (=Ww.V²/2q ; V= 5m/s)	1.276 t/m²
Allowable stress	Steel type : SS41	
	Allowable tensile stress (σsa)	1400 kg/cm²
	Allowable shearing stress (τa)	800 kg/cm²

(2) Structural Calculation.

Calculation was done for (i) Ceiling at direction of cross section, (ii) Side at direction of cross section, (iii) Beam at beam direction and (iv) Reinforce member.

Structural Calculation of Vortex Prevention Beam

Direction of Cross Section	
Ceiling	**Side**
1. Calculation Model.  Weight of ceiling : W1 = 0.071 t/m² Dynamic water pressure : W2 = 1.276 t/m² Total : W = 1.347 t/m²	1. Calculation Model.  Dynamic water pressure : W2 = 1.276 t/m²
2. Bending Moment and Shearing force. Mmax = 0.112 tm Smax = 0.674 t	2. Bending Moment and Shearing force. Mmax = 0.106 tm Smax = 0.638 t
3. Stress intensity (kg/cm²) σ s = 830 < σ sa = 1400 τ = 7.5 < τ a = 800 Where ; I = 6.075 cm⁴, A = 90 cm2	3. Stress intensity (kg/cm²) σ s = 785 < σ sa = 1400 τ = 7.1 < τ a = 800 Where ; I = 6.075 cm⁴, A = 90 cm²

Beam Direction	Reinforce Member
1. Calculation Model.  Own weight : W3 = 0.418 t/m Dynamic water pressure : W2 = 2.501 t/m Total : W = 2.919 t/m	1. Calculation Model.  Own weight : W4 = 0.013 t/m Weight of ceiling : W1 = 0.071 t/m² Dynamic water pressure : W2 = 1.276 t/m² Total W = 1.360 t/m
2. Bending Moment and Shearing force. Mmax = 8.757 tm Smax = 8.757 t	2. Bending Moment and Shearing force. Mmax = 0.871 tm Smax = 1.333 t
3. Stress intensity (kg/cm²). σ s = 42.8 < σ sa = 1400 τ = 16.4 < τ a = 800 Where ; I = 1.032 x 10⁶ cm⁴, A = 532.8 cm²	3. Stress intensity (kg/cm²) σ s = 1281 < σ sa = 1400 τ = 78 < τ a = 800 Where ; I = 425 cm⁴, A = 17.11 cm²

6.7 Penstock

6.7.1 General

In general, when a powerhouse has two or more turbines, there are two types of arrangement of the penstock as (i) An individual penstock for each turbine, and (ii) A single penstock with header system which requires complicated branch connection and valve to isolate each turbine.

In the Kotapanjang dam, three individual lines of penstock with 5.0m diameter were designed for the following reasons.

- There is not sufficient space between the dam and powerhouse to arrange a branch connection which is required for a single penstock system.
- In the dam body, it is not desirable for its stability to have a large opening as a single penstock that has approximate 9m diameter.
- With a single penstock system, the inspection or repair of the penstock will require shutting down the entire plant.

Each penstock consists of (i) an upper horizontal portion where a penstock is embedded in the dam concrete, (ii) an upper bending portion with fixed block located at the downstream face of the dam (iii) an inclined portion installed on downstream face of the dam being supported by a ring girder, and (iv) a lower bending and horizontal portion where the penstock pipe with concrete lining is embedded in the backfill soil.

The inclined portion of the penstock was determined to be an exposed pipe because the construction schedule of the penstock would not affect on the schedule of placing dam concrete.

The diameter of 5.0m was determined from the design considerations of (i) the minimum diameter of the inlet of the turbine casing is 5.0m and (ii) the head loss due to friction is not significant, since the length of the penstock is only 86.905m.

6.7.2 Hydraulic Design

The internal pressure consists of hydrostatic pressure and the pressure rise due to water hammering, which are calculated as follows.

(1) Maximum hydrostatic pressure is the water head difference between the center line elevation of the penstock and design water level at normal state (non-earthquake state) D.W.L 86.400. Then the maximum hydrostatic pressure is 53.10m, as the spiral casing center is at EL. 33.300 that is equal to the penstock center line elevation.

(2) The pressure rise due to water hammering was calculated as follows.

(a) Propagation velocity of pressure wave

The propagation velocity of pressure wave was given by following formula.

$$\alpha = 1/\sqrt{W/g \left(\frac{1}{K} + \frac{1}{E} \times \frac{D}{t} \right)} = 692 \ m/\text{sec}$$

Where,
- α : Propagation velocity of pressure wave
- W : Unit weight of water (1 t/m^3)
- g : Acceleration of gravity (9.8m/s^2)
- K : Bulk modulus of water (2.07 x 10^5 t/m^2)
- E : Modulus of liner elasticity (2.1 x 10^7 t/m^2)
- t : Thickness of pipe (15.2 x 10^{-3}m)
- D : Diameter of penstock (5.00m)

(b) Maximum pressure rise due to water hammer

Maximum pressure rise due to the water hammering without pressure regulators was calculated as follows.

$$\rho = \frac{\alpha . Vo}{2gHo} = 3.928, \quad \theta = \frac{\alpha T}{2Lo} = 21.875, \quad = \rho / \theta = 0.18$$

Where,
- ρ : Allievi's pipeline constant
- θ : Closing time constant of a closing equipment
- ho : Pressure rise due to water hammer at a closing equipment (m)
- Ho : Hydrostatic pressure after entirely shutting off a closing equipment at the turbine end (53.1m)
- Lo : Length of pipeline (94.905m)
- Vo : Average velocity in penstock (5.908 m/s at max. discharge 116 m^3/s)
- T : Closing time of a closing equipment (6 sec)
- g : Acceleration of gravity (9.8 m/s)
- α : Propagation velocity of pressure wave (692 m/s)

$\rho > 1.0$, therefore Allievi's formula was applicable for the pressure rise due to the water hammering.

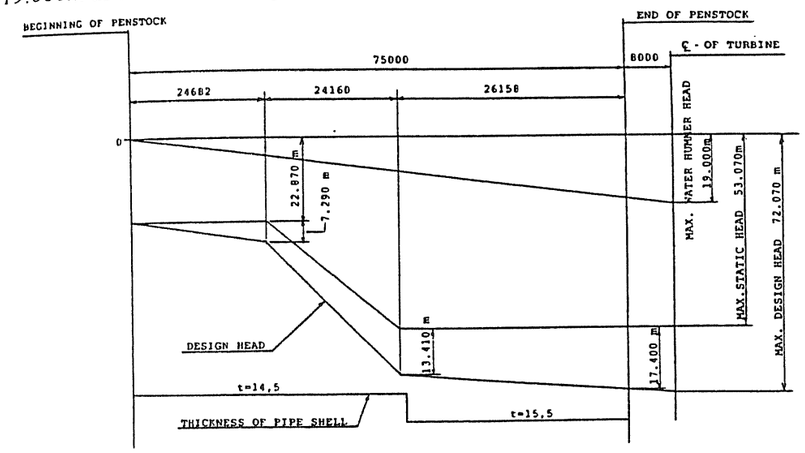
$$ho/Ho = \frac{n}{2} (n + \sqrt{n^2 + 4}) = 0.197$$

In general, it may be mentioned that the actual measurements of the pressure rise due to water hammering may differ somewhat from the calculated value, so it is desirable

to take some allowance. Accordingly to some latest design examples on pressure rise due to water hammering in Japan, values in the range 0.300 ~ 0.350 for ho/Ho have been adopted although calculated values were only around 0.170. Accordingly, in the interest of ensuring certain safety, it was recommended to adopt 0.350 as the value of ho/Ho for this penstock,

$$ho = 0.350 \times Ho = 18.585 \fallingdotseq 19.000m$$

Accordingly, the maximum pressure rise due to water hammer is assumed to be 19.000m as shown in the figure below.



Design Head Diagram

6.7.3 Structural Design

(1) Design components

The penstocks divided four portions, the upper horizontal portion, the upper bending portion, the inclined portion and lower horizontal portion as shown in Fig. 1.6.7-1. A steel penstock pipe in the inclined portion does not require concrete lining because no extra external force acts on the steel penstock pipe except design external pressures and the pressure difference during drainage assumed generally. The concrete portion around the steel penstock pipe at the other three portions was designed as a reinforced concrete structure for the following reasons.

(a) Upper horizontal portion

A steel penstock pipe in the upper horizontal portion is embedded in the dam concrete, and forms the opening of 5.0m in diameter in the dam body. When the stresses generated in the dam body due to dam self weight and all forces acting on the dam body act around the opening formed by the embedded pipe, stress concentration occurs and tensile stresses generate in the concrete around the pipe. Therefore, the dam concrete around the steel penstock pipe requires reinforcement to resist the concentrated stresses.

(b) Upper bending portion

An anchor block generally is installed at the bending point of the penstock in order to resist the forces which tend to exceed the resisting force due to weight of pipe and water in the pipe and cause displacement. In this case, the resultant force acting normally to the downstream face determined as a tension force of about 61 ton depending on the acting direction of the seismic force. Therefore, the anchor block at the upper bending portion required reinforcement to resist this tension force.

(c) Lower horizontal portion

A steel penstock pipe in the lower horizontal portion is installed in the backfill part between the dam and powerhouse, below where the main transformer was build. It is not an economical design that the steel penstock pipe should be designed to resist extra external forces caused by superimposed load, backfill soil and main transformer. Therefore the steel penstock pipe was designed as a box culvert to resist these external forces in reinforced concrete.

(2) General Design Condition

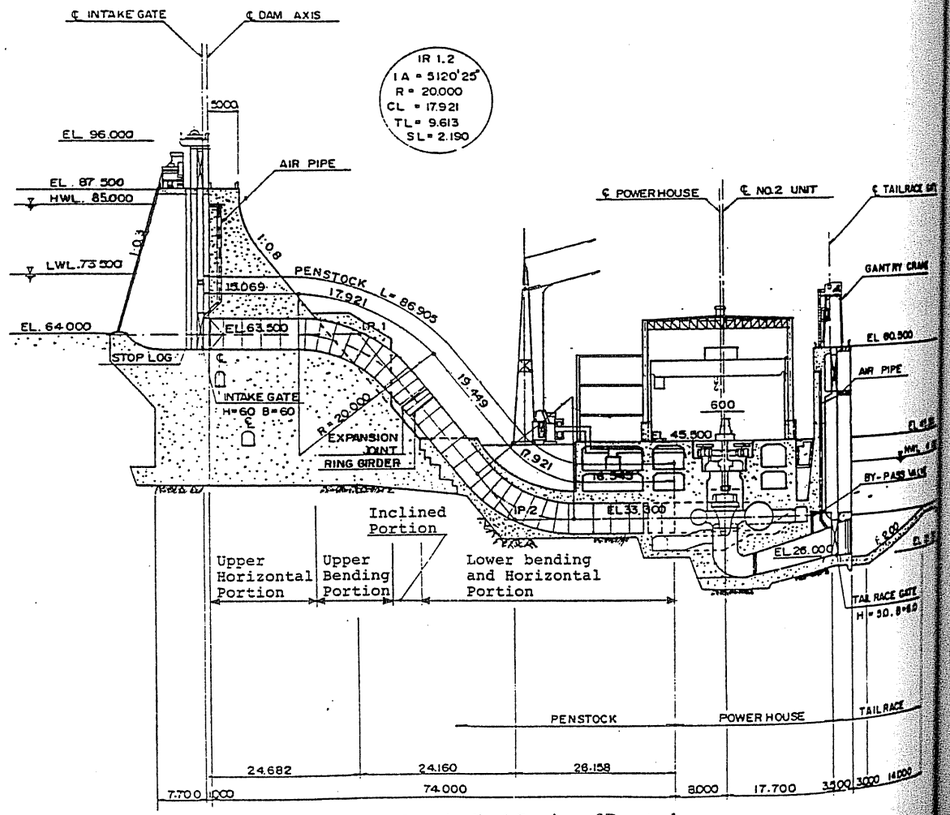


Fig. 1.6.7-1 Longitudinal Section of Penstock

Design loads

Unit weight	Concrete	2.30 t/m3
	Water	1.00 t/m3
Earthquake	Coefficient	0.10
Water level	Normal state	86.400 m
	Earthquake state	86.800 m
	Hydrodynamic pressure	85.000 m

Allowable stress

	Item	Long period	Short period
Concrete	Allowable compressive stress intensity due to bending σca	60	90
	Allowable shear stress intensity τca	8	12
	Allowable bond stress intensity τoa	14	21
Reinforcement	Allowable tensile stress intensity σsa	2000	3000

(3) Structural analysis of dam concrete around penstock pipe

(a) Calculation method

(i) Stress concentration at opening.

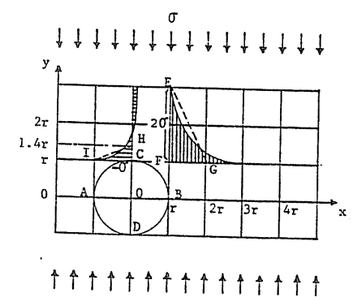
Openings in thin plates theory (Stress concentration at holes) are applied to the reinforcement design of the penstock tunnel.

If a thin plate subject to a uniform unidirectional primary stress is pierced by a hole, concentrations of stresses will occur at the sides of the hole parallel to the primary stress and a transverse stress of opposite sign, will occur at the opposite side. For a circular hole laid flat to the direction of primary stress, as indicated in the figure below, Timoshenko gives the following equations.

$$\sigma y = 1/2\ \sigma\ (2 + r^2/x^2 + 3r^4/x^4), \quad \sigma x = 1/2\ \sigma\ (r^2/y^2 - 3r^4/y^4)$$

Where; x, y : Distance from the center of the hole
r : Radius of the hole

Stress Variation near an opening



The stresses at the sides of the hole are

$$\sigma y = 3\sigma \quad at\ x = r,\ y = r$$

$$\sigma x = -\sigma \quad at\ x = 0,\ y = r \quad \text{(Transverse stress)}$$

Hence reinforcement will be made for the value of σ x.

The stress reverses at point J about y = 1.7 r. If the point H represented as 1.4 r, the line I-H is a good practical approximation of the stress diagram. Therefore, the total tension force was calculated as follows.

$$Ft = 0.4\,r \times (-\sigma) \times 1/2 = -0.2\,r\sigma$$

For this case the stress at the surface of the hole is of opposite sign from the primary stress.

(ii) Stress in the dam concrete

The gravity analysis was applied for the calculation of the stress in the dam concrete. The dam is considered to be made up of a number of vertical cantilevers which act independently of each other. The design criteria are given in section 6.2.2, Main Dam. However, those assumptions peculiar to a gravity analysis are listed below ;

- The concrete in the dam is a homogeneous, isotropic, and uniformly elastic material
- There are no differential movements which occur at the dam site due to water loads on the reservoir wall and floors.
- All loads are carried by the gravity action of vertical, parallel-side cantilevers which receive no support from the adjacent elements on either side.
- Unit vertical pressure, or normal stresses on horizontal planes, vary uniformly as a straight line from the upstream face to the downstream face.
- Horizontal shear stresses have a parabolic variation across horizontal planes from the upstream face to the downstream face of the dam.

A summary of the equations of stresses computed by the gravity method based on the following figure. Symbols and definitions related to the figure and following equations are given below.

Properties and Dimensions

O	:	Origin of coordinates at downstream edge of section.
ϕ	:	Angle between face of element and vertical (upstream and downstream).
T	:	Horizontal distance from upstream edge to downstream edge of section.
wc	:	Unit weight of concrete (t/m^3).
w	:	Unit weight of water (t/m^3).
h or h'	:	Vertical distance from reservoir or tailrace surface, respectively to section (m).
p or p'	:	Reservoir water or tailwater pressure respectively. It is equal to wh or wh' (t/m^2)
pd	:	Dynamic water pressure (t/m^2)
pe	:	Silt pressure (t/m^2)
Ce	:	Coefficient of silt pressure
we	:	Unit weight of silt deposit (t/m^3)
λ	:	Seismic intensity

Forces and Moments

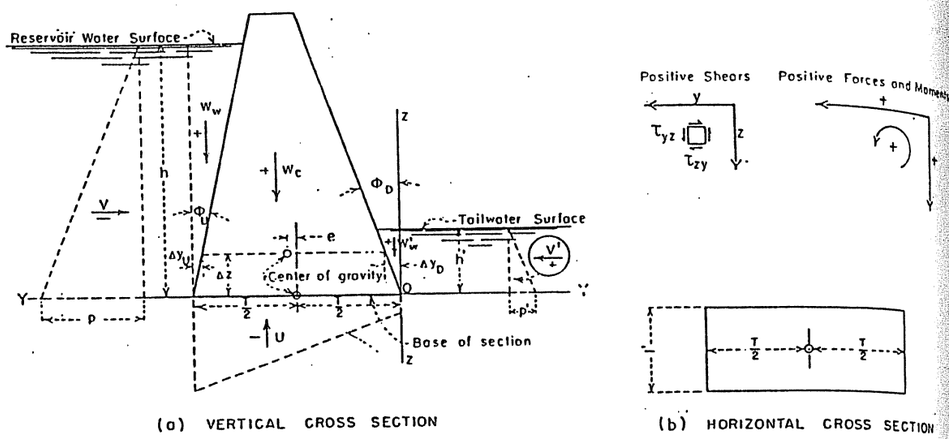
Wc	; Dead load weight above base of section under consideration (t).
Mc	; Moment of Wc about center of gravity of section.
Ww or Ww'	; Vertical component of reservoir or tailwater load, respectively on face above section (t).
Ww or Mw'	; Moment of Ww or Ww' about centre of gravity of section (tm).
V or V'	; Horizontal component of reservoir or tailwater load respectively, on face above section. This is equal to 1/2 wh^2 or 1/2 wh'^{2} for normal condition (t)
Mp or Mp'	; Moment of V or V' about centre of gravity of section, equal to Wh^3/6 or Wh'^{3}/6 (t.m)
We	; Vertical component of sedimentation (t)
Me	; Moment of We about center of gravity of section (tm)
Md or Mep	; Moment of Pd or Pe about center of gravity of section (tm)
ΣW	; Resultant vertical force above section, equal to Wc + Ww + Ww' + We (t)
ΣV	; Resultant horizontal force above section, equal to V + V' + Pd + Pe (t)
ΣM	; Resultant moment of forces above section about centre of gravity. It is equal to Mc + Mw + Mw' + Mp + Mp' + Me + Md + Mep (tm)
U	; Total uplift force on horizontal section (t)

Stresses

σ z	; Normal stress on horizontal plane (t/m^2)
σ y	; Normal stress on vertical plane (t/m^2)
τzy.τyz	; Shear stress on vertical or horizontal plane (t/m^2)
a, a1, a2, b, b1, b2, c1, c2, d2	: constant
σ p1	; First principal stress (t/m^2)
σ p2	; Second principal stress (t/m^2)

Subscripts

U	: Upstream face
D	: Downstream face



(a) VERTICAL CROSS SECTION (b) HORIZONTAL CROSS SECTION

Cross section of a parallel-side cantilever showing usual loading combination.

① Normal stress on Horizontal plate : σ z

$$\sigma z = a + by, \quad a = \sigma_{ZD} = \frac{\Sigma W}{T} - \frac{6\Sigma M}{T^2}, \qquad b = \frac{12\Sigma M}{T^3}$$

② Shear stress on Horizontal and Vertical plate : τyz, τzy

$$\tau zy = \tau yz = a1 + b1y + c1\, y^2$$

$$a1 = \tau zy_D = (\sigma z_D - p')\tan\phi_D, \quad b1 = \frac{1}{T}\left(\frac{6\Sigma V}{T} + 2\,\tau zyu + 4\,\tau ztD\right)$$

$$c1 = \frac{1}{T^2}\left(\frac{6\Sigma V}{T} + 3\,\tau zyu + 3\,\tau ztD\right), \quad \tau zyu = -(\sigma zu - P - Pd - Pe)\tan\phi u$$

③ Normal stress on vertical plane : σy

$$\sigma y = a_2 + b_2 y + c_2 y^2 + d_2 y^3, \quad a2 = \sigma yD = a1.\tan\phi D + P'$$

$$b2 = b1.\tan\phi D + \frac{\partial a1}{\partial z} \pm \lambda wc, \quad C2 = C1.\tan\phi D + \frac{1}{2}\frac{\partial a1}{\partial z}, \quad d2 = \frac{1}{3}\frac{\partial c1}{\partial z}$$

$$\frac{\partial a1}{\partial z} = \tan\phi_D\left(\frac{\partial \sigma_{ZD}}{\partial z} - w\right) + \frac{\partial \tan\phi_D}{\partial z}(\sigma_{ZD} - p')$$

$$\frac{\partial \sigma_{ZD}}{\partial z} = wc + \tan\phi u\left(\frac{12\Sigma M}{T^3} + \frac{2\Sigma W}{T^2} - \frac{2P}{T} - \frac{2Pd}{T} - \frac{2pe}{T}\right) + \tan\phi_D \left(\frac{12\Sigma M}{T^3} - \frac{4\Sigma W}{T^2} + \frac{4P'}{T}\right) - \frac{6\Sigma V}{T^2}$$

$$\frac{\partial \Sigma V}{\partial z} = -(P + Pd + Pe - P' + \lambda\, wcT), \quad \frac{\partial T}{\partial z} = \tan\phi u + \tan\phi_D$$

$$\frac{\partial \tau zyu}{\partial z} = \tan\phi u\left(w - \frac{\partial \sigma zu}{\partial z} + \frac{\partial pd}{\partial z} + we.ce\right) + \frac{\partial \tan\phi u}{\partial z}(p + pd + pe - \sigma zu)$$

$$\frac{\partial \sigma zu}{\partial z} = wc + \tan\phi u\left(\frac{4p}{T} + \frac{4pd'}{T} + \frac{4pe}{T} - \frac{4\Sigma w}{T^2} - \frac{12\Sigma M}{T^3}\right) + \tan\phi_D\left(\frac{2\Sigma w}{T} - \frac{2p'}{T} - \frac{12\Sigma M}{T^3}\right) + \frac{6\Sigma v}{T^2}$$

$$\frac{\partial \tau_{zyD}}{\partial z} = \frac{\partial a1}{\partial z}$$

$$\frac{\partial C1}{\partial z} = \frac{1}{T^3}\left[6\left(\frac{\partial \Sigma v}{\partial z}\right) - \frac{\partial T}{\partial z}\left(\frac{18\Sigma v}{T} + 6\tau zyu + 6\tau zyD\right)\right] + \frac{1}{T^2}\left[3\left(\frac{\partial \tau zyu}{\partial z}\right) + 3\left(\frac{\partial \tau zyD}{\partial z}\right)\right]$$

④ Magnitude of principal stress : σp1 - σp2

$$\sigma p1 = \frac{\sigma z + \sigma y}{2} \pm \sqrt{\left(\frac{\sigma z - \sigma y}{2}\right)^2 + (\tau zy)^2}$$

if $(\sigma z - \sigma y) > 0$, use (+)
if $(\sigma z - \sigma y) < 0$, use (-)
Alternative sign gives σp2 which is perpendicular to σp1

⑤ Direction of first principal stress σp1 : φp1

$$\phi p1 = \frac{1}{2}\tan^{-1}\left(-\frac{2\tau zy}{\sigma z - \sigma y}\right)$$

if $\tan 2\phi p1 = (+)$, then $0 < \phi p1 < (+45^0)$
if $\tan 2\phi p1 = (-)$, then $(-45^0) < \phi p1 < 0$
Measured from vertical clockwise positive

⑥ Magnitude of maximum shear stress : τmax

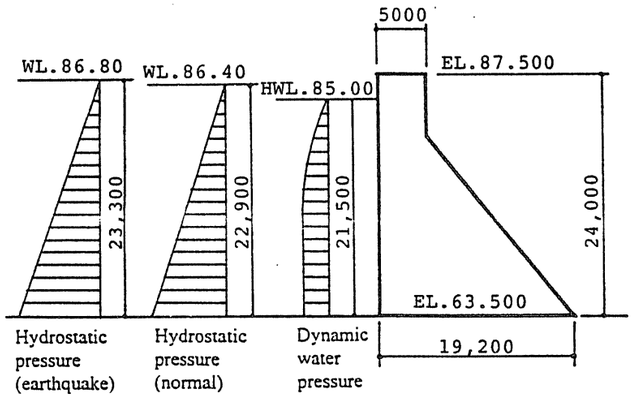
$$\tau\max = \sqrt{\left(\frac{\sigma z - \sigma y}{2}\right)^2 + (\tau zy)^2}$$

(b) Calculation case

Case 1 : Normal condition with H.W.L
Case 2 : Earthquake condition with H.W.L

(c) Calculation of loads

The calculation was made at the axial line of penstock at EL. 63.500. The uplift pressure was not taken into consideration in this calculation from a safety point of view.



Dam own weight		565.84 t
Hydrostatic pressure	(normal)	22.90 t/m²
	(earthquake)	23.30 t/m²
Hydrodynamic pressure		3.02 t/m²

(d) Calculation of stress and reinforcement

The values and figures given in the design condition introduce the following stress equation.

For normal state,

$\sigma z = 30.78 - 0.136y$, $\sigma y = 19.70 - 0.18y + 0.05y^2$, $\tau zy = 24.62 - 0.86y - 0.02y^2$

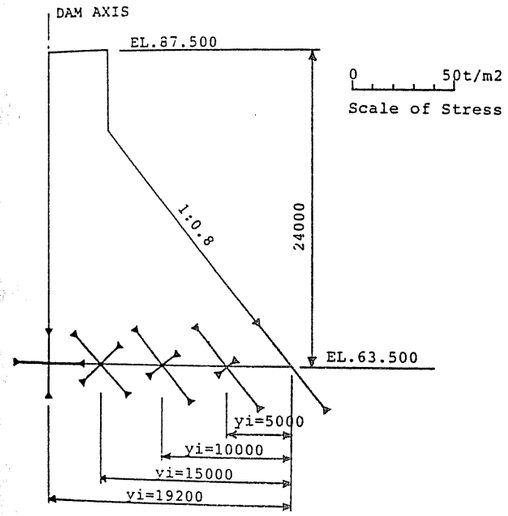
For earthquake state,

$\sigma z = 56.35 - 2.80y$, $\sigma y = 36.06 - 0.96y + 0.03y^2$, $\tau zy = 45.08 - 2.23y - 0.01y^2$

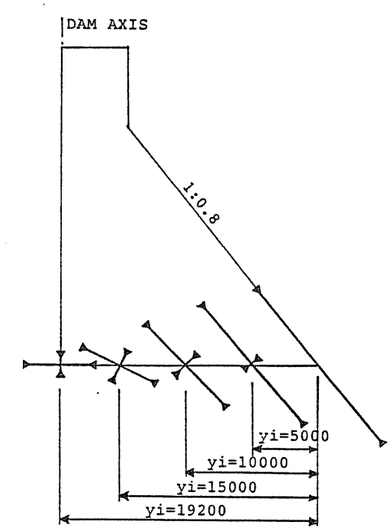
(i) Calculation result of stresses

(Unit t/m²)

	yi	0	5.00	10.00	15.00	19.20
Normal	σz	30.78	30.10	29.42	28.74	28.17
	σy	19.70	20.05	22.90	28.25	34.68
	τzy	24.62	19.82	14.02	7.22	0
	σ1	50.48	45.52	40.55	35.72	28.17
	σ2	0	4.63	11.77	21.27	34.68
	ϕp1	38.66	37.89	38.45	44.03	0
Earthquake	σz	56.35	42.35	28.35	14.35	2.59
	σy	36.06	32.01	29.46	28.41	28.69
	τzy	45.08	33.68	21.78	9.38	0
	σ1	92.42	72.41	7.12	9.66	2.59
	σ2	0	1.95	50.70	33.10	28.69
	ϕp1	38.66	40.64	-44.27	-26.57	0



Inner stresses on normal state



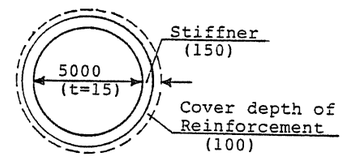
Inner stresses on earthquake state

(ii) Required reinforcement

The required reinforcement is given below.

$As = Ft / \sigma sa$

where, As : Required reinforcement (cm²/1m)
Ft : Tension force (kg) (-0.2 rσ)
r : Radius (2.765 m)
σ : Internal stress (principal stress)
σsa : Allowable tensile stress



Required reinforcement for the embedded part

Calculation Yi Point	0 m		5.000 m		10.000 m		15.000 m		19.200 m	
	Normal	Earthquake	Normal	Earthquake	Normal	Earthquake	Normal	Earthquake	Normal	Earthquake
σ	50.48	92.42	45.52	72.41	40.55	50.70	35.72	33.10	28.17	28.69
Ft (t)	27.72	51.11	35.17	40.04	22.42	28.04	19.75	18.30	15.58	15.87
σsa	2000	3000	2000	3000	2000	3000	2000	3000	2000	3000
Required reinforcement.	13.06	17.04	12.59	13.35	11.21	9.35	9.88	6.10	7.79	5.29

(4) Structural analysis of anchor block

Structural calculations for the anchor block at the upper bending part were carried out under the condition with earthquake during operation.

(a) Design loads

Self weight of anchor block is W = 883.08 t, and load P acting on the bending part of the penstock is given as follows,

$$P = Pc + Pr = 573.649\ t$$

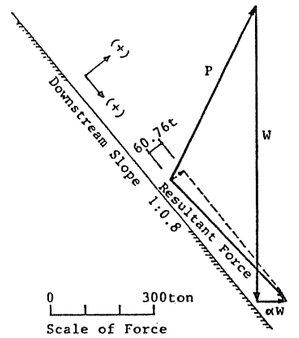
where, Pc : Centrifugal force (= $2V^2/g$. A.sin (θ/2) = 60.589 t)
Pr : Unbalanced force due to inner pressure (= 2HA sin (θ/2) = 513.060 t)
V : Mean velocity in pipe (V = Q/A = 5.908 m/s)
A : Inner cross section of pipe (A = $\pi D^2/4$ = 19.635 m^2)
g : Gravity acceleration (= 9.8 m/s^2)
H : Design inner pressure of the center of bending part (= 30.16 t/m^2)

(b) Calculation of forces and reinforcement

(i) Calculation result of required forces

The self weight of the anchor block W, its earthquake force αW and the force acting on the bending part P are shown in the force diagram below. And the resultant forces of the parallel and normal components to the downstream face of the dam are shown below. The tensile force of about 61t acted normally to the downstream face of the dam.

Load Items	Component of Force	
	Parallel to Slope	Normal to Slope
Own weight (w)	689.57	- 551.66
Earthquake force (αw)	55.17	68.96
Centrifugal & unbalanced force (P)	- 183.64	543.46
Total	561.10	60.76

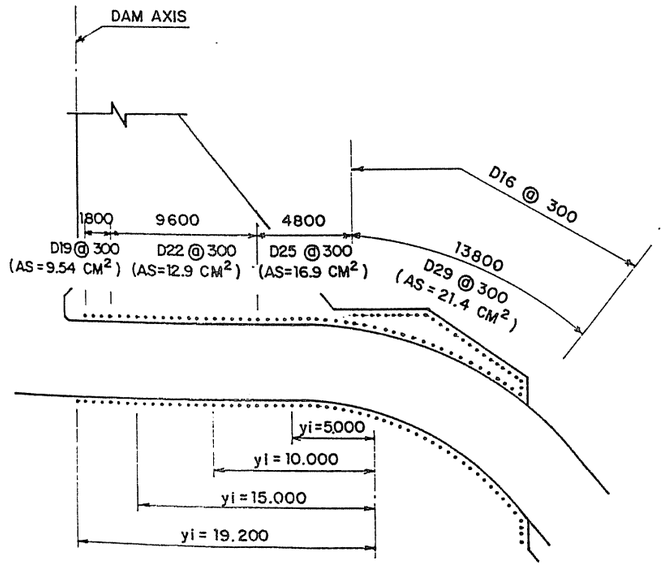


(ii) Reinforcement

For the tensile force Ft = 60.76t, the required reinforcement was as follows.

$$As = Ft \times 1000 / \sigma sa = 60.76 \times 1000/3000 = 20.26\ cm^2$$

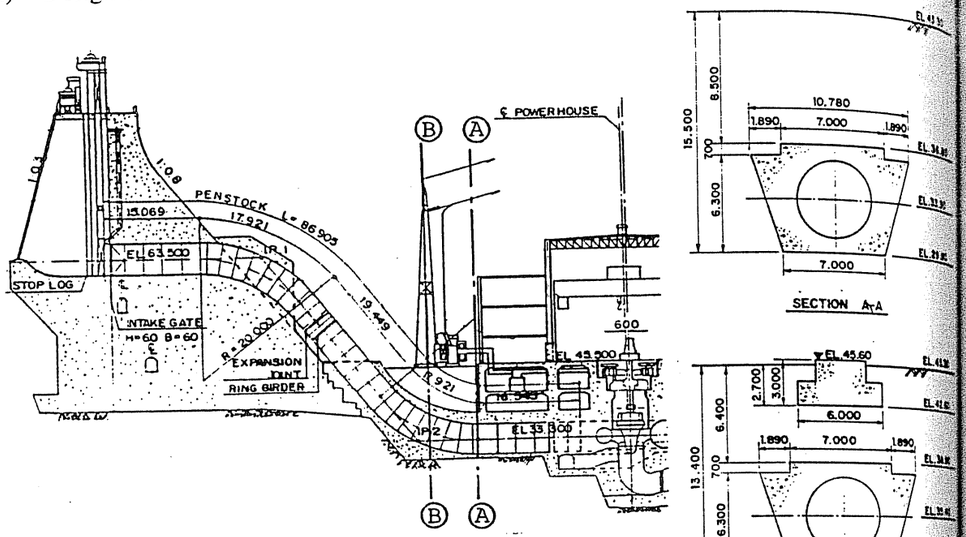
Based on the calculation of required reinforcement, the arrangement of reinforcement is shown in the figure below.



Arrangement of reinforcement at upper horizontal portion

(5) Reinforcement design around penstock embedded in backfill part.

(a) Design condition



Design Loads

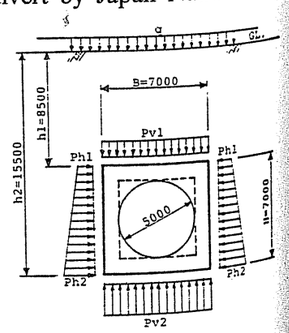
Unit weight	Reinforced concrete	2.40 t/m³
	Soil	1.80 t/m³
Surcharged load	Gantry pylon	6.1 t
	Weight of foundation	193.7 t
	Surcharged load	1.00 t/m²

(i) External pressure

A-A Section

The following equations are applied to the backfill part assuming a box culvert on which earth pressure and surcharge load act as external pressures (Source : The Design Guide Line for Retaining Wall and Culvert by Japan National Road Association).

$Pv1 = Pvd1 + q + D1/B = 21.76\ t/m^2$
$Pvd1 = \alpha.\gamma.h1 = 18.36\ t/m^2$
$Ph1 = ko\ (\gamma h1 + 1) = 8.15\ t/m^2$
$Ph2 = ko\ (\gamma h2 + 1) = 14.45\ t/m^2$
$Pv2 = Pv1 + D2/B = 29.43\ t/m^2$



where, α : Coefficient of vertical earth pressure (=1.2, given for h1/B=1.2)
γ : Unit weight of backfill soil (= 1.8 t/m³)
h1 : Depth of backfill soil (= 8.5 m)
ko : Coefficient of earth pressure at rest (assume 0.5)
h2 : Depth from ground surface to the base of wall (= 15.50 m)
D1 : Weight of top slab (= 16.80 t/m)
D2 : Weight of the side walls and base (= 53.68 t/m)
q : Surcharge load (= 1.0 t/m²)

B-B Section

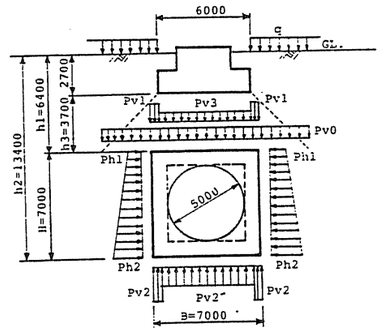
This encased concrete section of the steel penstock pipe is acted on by earth pressure and the weight of the gantry pylon and its foundation. Therefore, the following equations are applied to the backfill part assuming a box culvert on which earth pressure and surface load act as external pressures.

$Pvo = D3/A + D1/B = 3.75\ t/m^2$

$Pv1 = Pvd1 + q = 12.52\ t/m^2$
$Pvd1 = \alpha.\gamma.h1 = 1.0.\gamma.h1 = 11.52\ t/m^2$

$Ph1 = ko\ (\gamma h1 + 1.0 + D3/A) = 6.94\ t/m^2$
$Ph2 = ko\ (\gamma h2 + 1.0 + D3/A) = 13.24\ t/m^2$

$Pv2 = Pvo + Pv1 + D2/B = 23.94\ t/m^2$
$Pv2' = Pvo + Pv3 + D2/B = 18.08\ t/m^2$
$Pv3 = \alpha.\gamma.h3 = 1.0.\gamma h3 = 6.66\ t/m^2$



where, h1, h3 : Depth of backfill (=6.40m, 3.70m)
h2 : Depth from ground surface to the base of wall (=13.40m)
ko : Coefficient of earth pressure at rest (=0.5m)
q : Uniform distributed live load (1.0 t/m²)
D1 : Weight of top slab (=16.80 t/m)
D2 : Weight of side wall and base (=53.68 t/m)
D3 : Weight of gantry pylon and its foundation (= 242.8 t)
A : Acting area of D3 (= 179.56 m²)

(ii) Internal pressure

A-A Section		B-B Section	
Hydrostatic pressure	P = 53.10 m	Hydrostatic pressure	P= 51.70 m
Water hammer	P' = 15.70 m	Water hammer	P'=12.60 m

Total water hammer pressure is proportionally distributed by the channel length. The internal pressure acting on the concrete around the penstock will be shared with the penstock itself as shown in the following calculation. The share of the

internal pressure is determined by assuming the penstock and the concrete around the penstock are thin-walled and a thick walled circular cylinders respectively.

The pressure shared by the penstock (P2) and by the concrete (P1) are given below.

$$P2 = \Sigma\Delta.t.Es/r^2 = 0.03x1.9x2.1x10^6/250^2 = 1{,}915\ Kg/cm^2 = 19.2\ t/m^2$$

$$P1 = P\text{-}P2$$

Where, $\Sigma\Delta$: Disjoint of around the penstock (0.03)
t : Thickness of penstock (1.9 cm)
Es : Modulus of elasticity of steel (2.1×10^6 kg/cm^2)
r : Inner radius of penstock (250 cm)

Share of internal pressure of cross section (A-A section)

Inner radius of penstock	Outer radius of concrete	Thickness penstock	Disjoint	Hydrostatic pressure		Include water hammer	
				P = 53.10 t/m²		P = 67.3 t/m²	
				P1(t/m²)	P2(t/m²)	P1(t/m²)	P2(t/m²)
250	350	1.9	0.03	33.9	19.2	48.1	19.2

From the above table, the maximum internal pressure is P1 = 48.1 t/m^2, and this value was taken at section A-A as the design internal pressure.

Share of internal pressure of cross section (B-B Section)

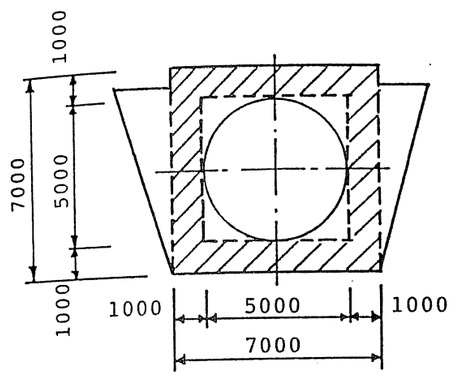
Inner radius of penstock	Outer radius of concrete	Thickness penstock	Disjoint	Hydrostatic pressure		Include water hammer	
				P = 51.70 t/m²		P = 64.3 t/m²	
				P1(t/m²)	P2(t/m²)	P1(t/m²)	P2(t/m²)
250	350	1.9	0.03	32.5	19.2	45.1	19.2

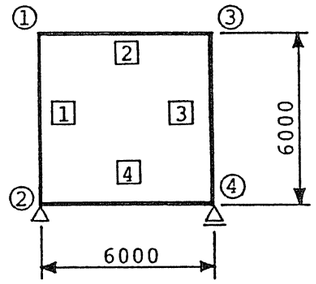
From the above, the maximum internal pressure is P1 = 45.1 t/m^2, and this value was taken at section B-B as the design internal pressure.

(b) Structural calculation

The frame structure analysis was applied to evaluate the stress produced by the internal and external pressures. The external force does not act on the penstock.

(i) Calculation model





○ : Nodal number

□ : Member number

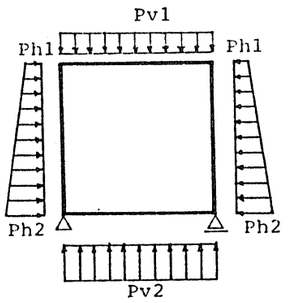
(ii) Calculation cases

Case 1 : Normal state, penstock empty
Case 2 : Normal state, penstock full at H.W.L.

The loading condition and calculation result are shown in the following tables.

(Section A-A) Penstock Empty

1. Loading Condition



Pv_1 = 21.76 t/m²
Ph_1 = 8.15 t/m²
Ph_2 = 14.45 t/m²
Pv_2 = 29.43 t/m²

2. Calculation for Stress of Reinforced Concrete

		L (m)	M (t.m)	N (t)	Q (t)	B (cm)	T (cm)	D=D' (cm)
1	1	0.000	46.369	65.280	27.759	100.0	146.7	10.0
1	Hunch	1.760	15.974	69.840	10.379	100.0	100.0	10.0
1	Max.	2.874	4.403	72.178	0.000	100.0	100.0	10.0
1	Hunch	4.100	21.939	75.120	14.481	100.0	100.0	10.0
1	2	6.000	64.316	79.680	40.041	100.0	146.7	10.0
2	1	0.000	46.369	27.759	65.280	100.0	146.7	10.0
2	Hunch	1.900	23.698	27.759	23.936	100.0	100.0	10.0
2	Max.	3.000	51.551	27.759	0.000	100.0	100.0	10.0
2	Hunch	4.100	23.698	27.759	23.936	100.0	100.0	10.0
2	3	6.000	46.369	27.759	65.280	100.0	146.7	10.0
3	3	0.000	46.369	65.280	27.759	100.0	146.7	10.0
3	Hunch	1.900	15.974	69.840	10.379	100.0	100.0	10.0
3	Max.	2.874	4.403	72.178	0.000	100.0	100.0	10.0
3	Hunch	4.100	21.939	75.120	14.481	100.0	100.0	10.0
3	4	6.000	64.316	79.680	40.041	100.0	146.7	10.0
4	2	0.000	64.316	40.041	88.290	100.0	146.7	10.0
4	Hunch	1.900	30.449	40.041	32.373	100.0	100.0	10.0
4	Max.	2.874	68.119	40.041	0.000	100.0	100.0	10.0
4	Hunch	4.100	30.449	40.041	32.373	100.0	100.0	10.0
4	3	6.000	64.316	40.041	88.290	100.0	146.7	10.0

3 Required Reinforcement

		AS (cm²)	ASD (cm²)	P	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)
1	1	2.23	2.23	.00016	42.00	2000	1.43
1	Hunch	1.00	1.00	.00011	17.06	26	.02
1	Max.	1.00	1.00	.00011	9.82	0	0.00
1	Hunch	1.00	1.00	.00011	23.82	154	.06
1	2	6.23	6.23	.00046	48.64	2000	2.57
2	1	11.22	11.22	.00082	33.74	2000	4.95
2	Hunch	7.07	7.07	.00079	39.04	2000	2.70
2	Max.	24.63	24.63	.00274	51.90	2000	0.00
2	Hunch	7.07	7.07	.00079	39.04	2000	2.70
2	3	11.22	11.22	.00082	33.74	2000	4.95
3	3	2.23	2.23	.00016	42.00	2000	1.43
3	Hunch	1.00	1.00	.00011	17.06	26	.02
3	Max.	1.00	1.00	.00011	9.82	0	0.00
3	Hunch	1.00	1.00	.00011	23.82	154	.06
3	4	6.23	6.23	.00046	48.64	2000	2.57
4	2	15.54	15.54	.00014	40.55	2000	6.74
4	Hunch	8.26	8.26	.00092	46.11	2000	3.61
4	Max.	33.13	33.13	.00068	60.00	1949	0.00
4	Hunch	8.26	8.26	.00092	46.11	2000	3.61
4	4	15.54	15.54	.00014	40.55	2000	6.74

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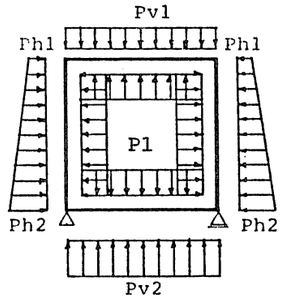
1. Loading Condition



Pv_1 = 21.76 t/m²
Ph_1 = 8.15 t/m²
Ph_2 = 14.45 t/m²
Pv_2 = 29.43 t/m²
Pl = 48.1 t/m²

2. Calculation for Stress of Reinforced Concrete

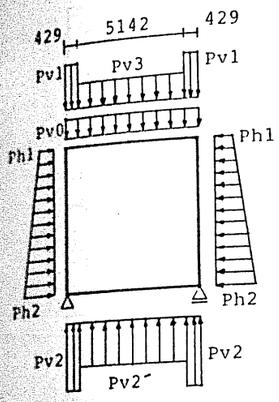
		L (m)	M (t.m)	N (t)	Q (t)	B (cm)	T (cm)	D=D' (cm)
1	1	0.000	92.252	-54.970	92.491	100.0	146.7	10.0
1	Hunch	1.900	26.222	-50.410	42.531	100.0	100.0	10.0
1	Max.	3.039	76.336	-47.678	0.000	100.0	100.0	10.0
1	Hunch	4.100	32.187	-45.130	38.429	100.0	100.0	10.0
1	2	6.000	74.306	-40.570	80.209	100.0	146.7	10.0
2	1	0.000	92.252	-92.491	54.970	100.0	146.7	10.0
2	Hunch	1.900	13.450	-92.491	28.974	100.0	100.0	10.0
2	Max.	3.000	20.265	-92.491	0.000	100.0	100.0	10.0
2	Hunch	4.100	13.450	-92.491	28.974	100.0	100.0	10.0
2	3	6.000	92.252	-92.491	54.970	100.0	146.7	10.0
3	3	0.000	92.252	-54.970	92.491	100.0	146.7	10.0
3	Hunch	1.900	26.222	-50.410	42.531	100.0	100.0	10.0
3	Max.	2.874	76.336	-47.678	0.000	100.0	100.0	10.0
3	Hunch	4.100	32.187	-45.130	38.429	100.0	100.0	10.0
3	4	6.000	74.306	-40.580	80.209	100.0	146.7	10.0
4	2	0.000	74.306	-80.209	31.960	100.0	146.7	10.0
4	Hunch	1.900	20.201	-80.209	20.537	100.0	100.0	10.0
4	Max.	3.000	3.697	-80.209	0.000	100.0	100.0	10.0
4	Hunch	4.100	20.201	-80.209	20.537	100.0	100.0	10.0
4	3	6.000	74.306	-80.209	31.960	100.0	146.7	10.0

3. Required Reinforcement

		AS (cm²)	ASD (cm²)	P	σc (kg/cm²)	σs (kg/cm²)	
1	1	49.88	49.88	.00365	27.01	2000	
1	Hunch	28.68	28.68	.00319	15.60	2000	
1	Max.	58.84	58.84	.00654	43.79	2000	
1	Hunch	30.89	30.89	.00343	22.27	2000	
1	2	39.18	39.18	.00287	26.14	2000	
2	1	59.25	59.25	.00434	20.15	2000	
2	Hunch	54.59	54.59	.00607	8.96	2000	
2	Max.	12.01	12.01	.00133	28.28	2000	
2	Hunch	54.59	54.59	.00607	8.96	2000	
2	3	59.25	59.25	.00434	20.15	2000	
3	3	49.88	49.88	.00365	27.81	2000	
3	Hunch	28.68	28.68	.00319	15.60	2000	
3	Max.	58.84	58.64	.00654	43.79	2000	
3	Hunch	30.89	30.89	.00343	22.27	2000	
3	4	39.18	39.18	.00287	26.14	2000	
4	2	49.13	49.13	.00359	17.23	2000	
4	Hunch	11.97	11.97	.00133	28.24	2000	
4	Max.	42.38	42.38	.00471	2.80	2000	
4	Hunch	11.97	11.97	.00133	28.24	2000	
4	4	49.13	49.13	.00359	17.23	2000	

(Section B-B) Penstock Empty

1. Loading Condition



Pv0 = 3.75 t/m²
Pv1 = 12.52 t/m²
Pv3 = 6.66 t/m²
Ph1 = 6.94 t/m²
Ph2 = 13.24 t/m²
Pv2 = 23.94 t/m²
Pv2' = 18.08 t/m²

2 Calculation for Stress of Reinforced Concrete

		L (m)	M (t.m)	N (t)	Q (t)	B (cm)	T (cm)	D= D' (cm)
1	1	0.000	27.297	32.741	23.859	100.0	146.7	10.0
1	Hunch	1.900	1.087	37.301	83.949	100.0	100.0	10.0
1	Max.	2.857	8.831	39.599	0.000	100.0	100.0	10.0
1	Hunch	4.100	7.053	42.581	13.051	100.0	100.0	10.0
1	2	6.000	45.243	47.141	36.141	100.0	146.7	10.0
2	1	0.000	27.297	23.859	32.741	100.0	146.7	10.0
2	Hunch	1.900	6.003	23.859	11.253	100.0	100.0	10.0
2	Max.	3.000	19.097	23.859	0.000	100.0	100.0	10.0
2	Hunch	4.100	6.003	23.859	11.253	100.0	100.0	10.0
2	3	6.000	45.243	23.859	32.741	100.0	146.7	10.0
3	3	0.000	27.297	32.741	23.859	100.0	146.7	10.0
3	Hunch	1.900	1.087	31.301	8.949	100.0	100.0	10.0
3	Max.	2.874	8.831	39.599	0.000	100.0	100.0	10.0
3	Hunch	4.100	7.053	42.581	13.051	100.0	100.0	10.0
3	4	6.000	45.243	47.141	36.141	100.0	146.7	10.0
4	2	0.000	45.243	36.141	55.751	100.0	146.7	10.0
4	Hunch	1.900	12.754	36.141	19.690	100.0	100.0	10.0
4	Max.	3.000	35.666	36.141	0.000	100.0	100.0	10.0
4	Hunch	4.100	12.754	36.141	19.690	100.0	100.0	10.0
4	3	6.000	45.243	36.141	55.751	100.0	146.7	10.0

3 Required Reinforcement

		AS (cm²)	ASD (cm²)	P	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)
1	1	2.34	2.34	.00017	29.62	2000	1.55
1	Hunch	1.00	1.00	.00011	4.37	26	.08
1	Max.	1.00	1.00	.00011	9.48	11	0.00
1	Hunch	1.00	1.00	.00011	8.45	0	.11
1	2	6.09	6.09	.00045	37.78	2000	2.53
2	1	4.53	5.53	.00033	27.37	2000	2.38
2	Hunch	1.00	1.00	.00011	6.36	19	.03
2	Max.	5.20	5.20	.00258	35.14	2000	0.00
2	Hunch	1.00	1.00	.00011	6.36	19	.03
2	3	4.53	4.53	.00033	27.37	2000	2.38
3	3	2.34	2.34	.00017	29.62	2000	1.55
3	Hunch	1.00	1.00	.00011	4.37	0	.08
3	Max.	1.00	1.00	.00011	9.48	11	0.00
3	Hunch	1.00	1.00	.00011	8.45	0	.11
3	4	6.09	6.09	.00045	37.78	2000	2.53
4	2	8.73	8.73	.00064	35.34	2000	4.13
4	Hunch	1.00	1.00	.00011	15.85	231	.23
4	Max.	12.51	12.51	.00139	47.44	2000	0.00
4	Hunch	1.00	1.00	.00011	15.85	231	.23
4	4	8.73	8.73	.00064	35.34	2000	4.13

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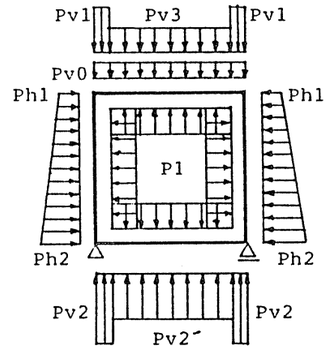
1. Loading Condition



Pv0 = 3.75 t/m²
Pv1 = 12.52 t/m²
Pv3 = 6.66 t/m²
Ph1 = 6.94 t/m²
Ph2 = 13.24 t/m²
Pv2 = 23.94 t/m²
Pv2' = 18.08 t/m²
Pl = 45.1 t/m²

2. Calculation for Stress of Reinforced Concrete

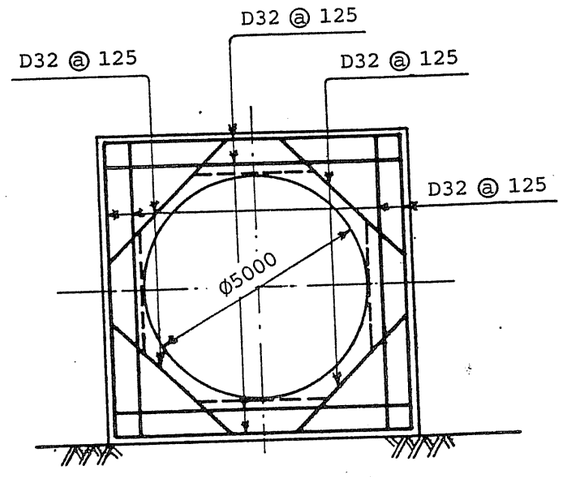
		L (m)	M (t.m)	N (t)	Q (t)	B (cm)	T (cm)	D=D' (cm)
1	1	0.000	102.679	-80.009	88.891	100.0	146.7	10.0
1	Hunch	1.900	10.696	-75.449	40.661	100.0	100.0	10.0
1	Max.	3.040	58.635	-72.712	0.000	100.0	100.0	10.0
1	Hunch	4.100	16.661	-70.169	36.559	100.0	100.0	10.0
1	2	6.000	84.732	-65.609	76.609	100.0	146.7	10.0
2	1	0.000	102.679	-88.891	80.009	100.0	146.7	10.0
2	Hunch	1.900	3.606	-88.891	38.357	100.0	100.0	10.0
2	Max.	3.000	48.240	-88.891	0.000	100.0	100.0	10.0
2	Hunch	4.100	3.606	-88.891	38.357	100.0	100.0	10.0
2	3	6.000	102.679	-88.891	80.009	100.0	146.7	10.0
3	3	0.000	102.679	-80.009	88.891	100.0	146.7	10.0
3	Hunch	1.900	10.696	-75.449	40.661	100.0	100.0	10.0
3	Max.	3.040	58.635	-72.712	0.000	100.0	100.0	10.0
3	Hunch	4.100	16.661	-70.167	36.559	100.0	100.0	10.0
3	4	6.000	84.732	-65.609	76.609	100.0	146.7	10.0
4	2	0.000	84.732	-76.609	56.999	100.0	146.7	10.0
4	Hunch	1.900	3.145	-76.609	29.920	100.0	100.0	10.0
4	Max.	3.000	31.671	-76.609	0.000	100.0	100.0	10.0
4	Hunch	4.100	3.145	-76.609	29.920	100.0	100.0	10.0
4	3	6.000	84.732	-76.609	56.999	100.0	146.7	10.0

3. Required Reinforcement

		AS (cm²)	ASD (cm²)	P	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)
1	1	60.24	60.24	.00441	25.48	2000	7.01
1	Hunch	44.32	44.32	.00492	7.93	2000	5.04
1	Max.	54.12	54.12	.00601	30.50	2000	0.00
1	Hunch	9.82	9.82	.00109	25.59	2000	4.30
1	2	49.55	49.55	.00363	23.69	2000	6.01
2	1	62.45	62.45	.00157	23.75	2000	6.29
2	Hunch	46.67	46.67	.00519	2.60	2000	4.77
2	Max.	51.91	51.91	.00577	20.49	2000	0.00
2	Hunch	46.67	46.67	.00519	2.60	2000	4.77
2	3	62.45	62.45	.00457	23.75	2000	6.29
3	3	60.24	60.24	.00441	25.48	2000	7.01
3	Hunch	44.32	44.32	.00492	7.93	2000	5.04
3	Max.	54.12	54.12	.00601	30.50	2000	0.00
3	Hunch	9.82	9.82	.00109	25.59	2000	4.30
3	4	49.55	49.55	.00363	23.69	2000	6.01
4	2	52.31	52.31	.00383	21.35	2000	4.45
4	Hunch	40.24	40.24	.00447	2.45	2000	3.70
4	Max.	38.76	38.76	.00431	11.06	2000	0.00
4	Hunch	40.24	40.24	.00447	2.45	2000	3.70
4	4	52.31	52.31	.00383	21.35	2000	4.45

(c) Arrangement of reinforcement

Cross Section	Member	Maximum required reinforcement (cm^2/m)		Distribution	Checking of stress intensity. (Maximum value)
		Empty	Full		
A - A Section	1	6.23	58.84	D 32 @ 125 ($As=63.5cm^2$)	
	2	24.63	59.25	"	σs =1866 kg/cm² σc = 19.3 " τ = 4.3 " τo = 5.4 "
	3	6.23	58.84	"	
	4	33.13	49.13	"	
B - B Section	1	6.09	60.24	D 32 @ 125 ($As=63.5cm^2$)	
	2	5.20	62.45	"	σs =1967 kg/cm² σc = 23.5 " τ = 6.3 " τo = 7.9 "
	3	6.09	60.24	"	
	4	12.51	52.31	"	



Arrangement of reinforcement

6.8 Powerhouse

6.8.1 General Layout

The location of the powerhouse was selected at just downstream of the dam on the terrace area of the left bank considering the topography and the river condition of downstream area so that the length of penstock can become as short as possible. In order to reduce the excavation volume of the powerhouse, the powerhouse is also adjacent to the spillway's retaining wall as close as possible.

The location between dam body and powerhouse was determined referring to existing concrete gravity dams with similar layout, in additional to the aforementioned considerations. The distance from downstream toe of dam body to the center of the powerhouse is about 30m.

For the powerhouse building, the long axis of the building was selected to be parallel with the dam axis, taking account of the spillway's retaining wall and the location of the dam block and intake and also from excavation volume.

The turbine center was set at EL. 33.300 considering the tail water level at EL. 41.000 (Normal Water Level) and the draft head of 7.7m. The foundation of the powerhouse was set at EL. 21.000 considering the setting of draft tube line.

The proposed powerhouse is a ground type with a conventional mass concrete and reinforced concrete sub-structure, and the superstructure is a steel structure.

The powerhouse was designed to house three 38 MW generating units. Its dimensions are 35.6 m in width, 44.0m in height and 80.3m in length. The dimensions were determined essentially by the sizes of turbine and generator of current manufacture, the size and shape of turbine discharge pits, the area and the height required for erecting and installing the generating equipment.

The width of 35.6m was dictated by the width of the turbine which allows dismounting by the overhead crane. The height of 43.2m was obtained by adding the heights of elbow-type draft tube, turbine, generator, overhead crane lift and the crane supporting beam. The length is 80.3m above EL. 45.5 main floor containing three main machines, erection bay, access for machine transportation and control room. Below EL. 40.7 the length is reduced to 58.5m as the room for the erection bay is excluded. The main units are spaced at 20.0m centers considering spaces for installation of turbine and for arrangement of turbine auxiliaries and electrical equipment.

The powerhouse is composed of five main floors EL. 37.10 turbine room, cooling water cooled system room EL. 40.7 auxiliary equipment room, cubicle room, EL. 45.5 generator room, EL. 50.0/54.40 control room. Below the turbine room are provided three sump pits for draining building leakage water out to the tailrace. The erection bay is located at the main entrance side of the generator room, where erection of spiral case, turbine and generator, etc. are carried out by two 85 ton overhead traveling cranes. Major generating equipment installed on each floor are listed in the Table below.

Powerhouse Floor Division and Equipment Installed by Floor

Floor name	Floor elevation	Type of equipment installation and room
3rd floor	EL. 54.50	Stock and storage office and conference rooms
2nd floor	EL. 50.00	Central control equipment, computer and relay room, stock and meeting rooms
Ground floor	EL. 45.50	Generator, cable treatment, air conditioning machine, communication equipment, cable treatment storage
Basement 1st floor	EL. 40.70	Auxiliary equipment, cubicle, inverter, battery
Basement 2nd floor	EL. 37.10	Turbine, cooling water cooled system, tools work shop and storage
Basement	EL. 21.0-37.10	Casing, draft tube, gallery and sumpit

The inside diameter of the cylindrical portion was selected as 9.4m at EL. 45.500 in the generator room, so that one turbine unit of 39.4 MW, could be installed. The barrel thickness of the cylindrical portion was designed as 1.5m and 3.0m for the upper bracket and the lower bracket respectively.

The ground floor of the powerhouse is EL. 45.500 and this corresponds to the same elevation of the access roads to the powerhouse. It is still 4.500m higher than the normal water level of EL. 41.000 at the tailrace.

The elevation of turbines center was decided at EL. 33.300 considering the draft head. Main power transformers will be located external to the station at ground-level, on the upstream side.

A drainage pump pit will be provided on the upstream side of the machines, located at the lowest part at EL. 26.300, to collect station drainage water and also to permit water draining from the turbines and draft tubes. Pumps were installed to pump the water to the tailbay.

Access to each floor will be provided by an elevator and stairs. Stairs between floors will also be provided in each machine bay.

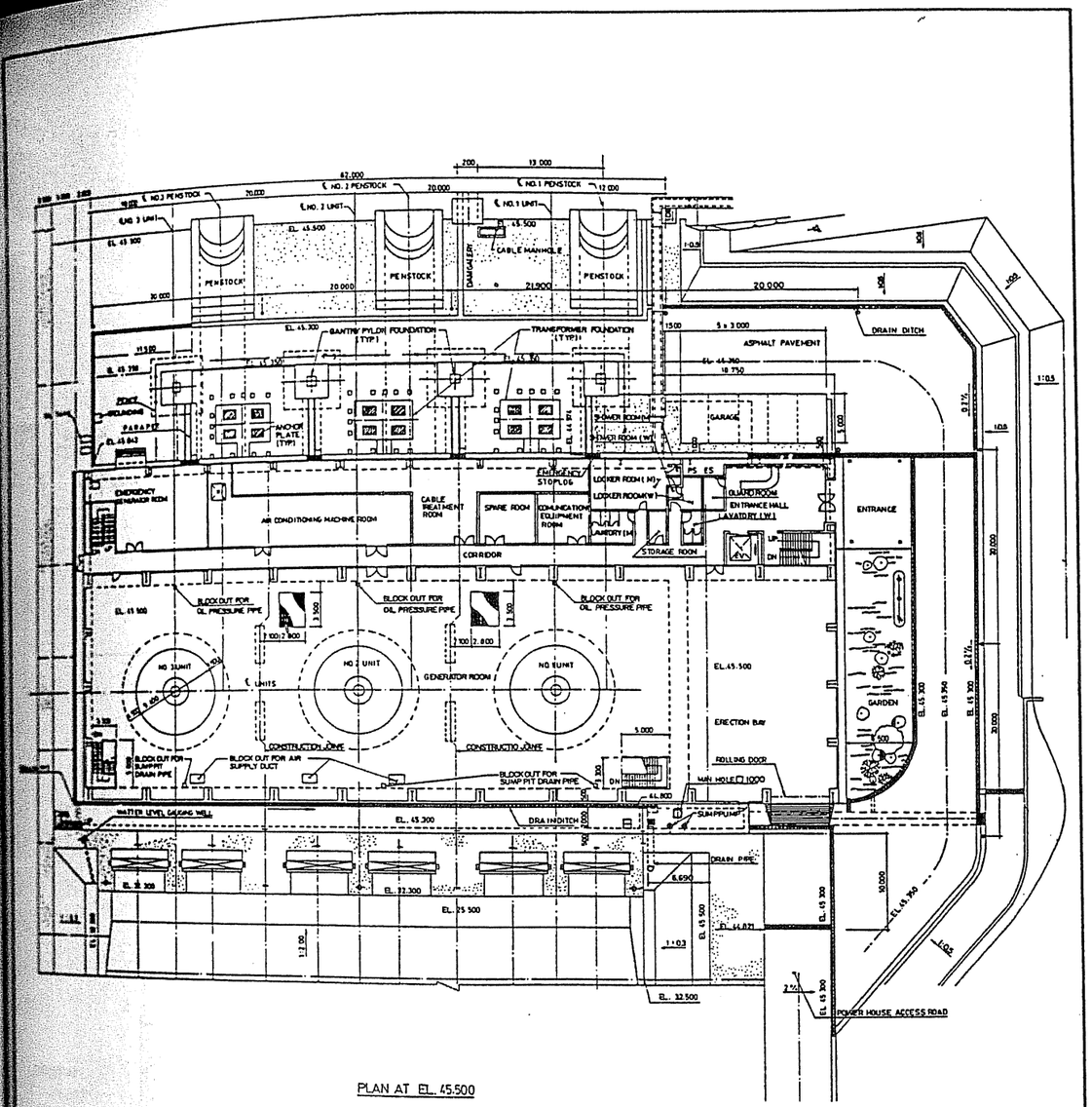


Fig. 1.6.8-1 Powerhouse Ground Floor Plan (EL. 45.50)

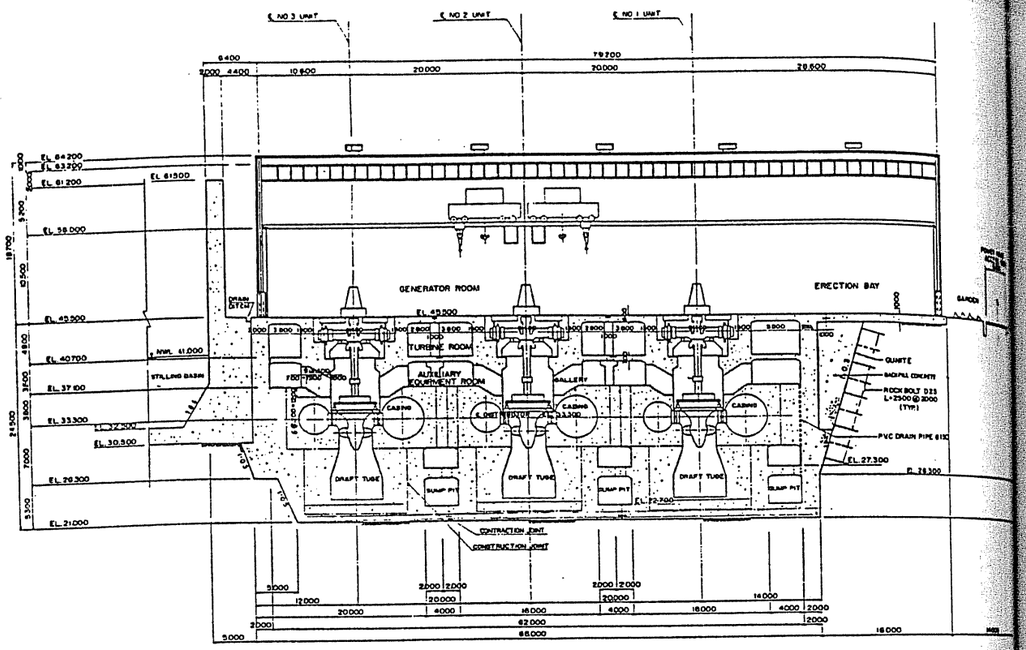


Fig. 1.6.8-2 Powerhouse Section A - A (Main Building)

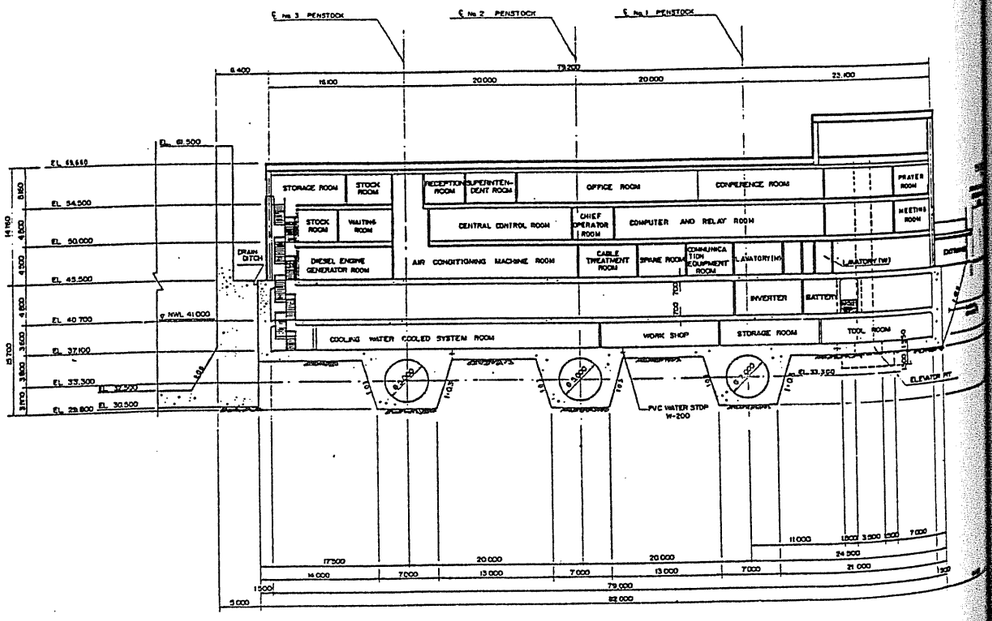


Fig. 1.6.8-3 Powerhouse Section B - B (Control Building)

6.8.2 Basic Principles of Powerhouse Structural Design

(1) Structure Components and Loading Assumptions

The powerhouse was designed as a reinforced concrete structure. For structural design purposes, the whole structure was divided into nine parts : 1) floor slab and floor beam, 2) side wall, 3) generator support (barrel), 4) spiral case encasement, 5) draft tube encasement, 6) powerhouse main building and sub-building (control room), 7) structure under over head traveling crane, 8) tailrace structure, and 9) powerhouse foundation.

a). Floor slab and floor beams

The floor slabs and beams support the surcharged floor load of generator equipment. Each generator unit is separated by contraction joints which control cracks occurring due to the drying and temperature shrinkage and the transmission of vibrations of each generator. Therefore, in consideration of the boundary conditions, the floor slabs were assumed to be beams with both ends fixed or with one end fixed and the other end simply supported or with one end fixed and the other end free. The floor beams were assumed to be beams with one end fixed and the other end free.

b). Side wall

Rock bolts are installed on the excavated rock behind the wall to ensure stability of rock walls. Underground water in flowing to the powerhouse will be drained by a drain system installed between the excavated rock and wall concrete. Therefore earth load was assumed to act on the side wall but water pressure was assumed not to act on the side wall. Seismic load was considered to act on the side wall and the water pressure during flood was also taken into account for the side wall of the stilling basin. The loads transmitted from the floor slabs were not taken into account.

c). Generator support (Barrel)

The generator support was assumed as a cylindrical shell structure with octagonal sections supported by mass concrete. This type of a structure is called a "barrel". The loads acting on the barrel are as follows.

- Load due to generator equipment
- Dead load of barrel
- Load due to floor
- Load due to earthquake

Structural analysis of the barrel as a whole is divided into a vibration analysis and a stress analysis. The results of this analysis are as follows.

i). Vibration analysis

The most serious forced vibration occurs at the single phase short-circuit of the generator. The frequency of this torsional vibration is equal to that of the electric current, i.e., 50 Hz. Generally the resonance of the barrel occurs if the ratio (Ø/f) of natural frequency (f) and frequency of forced vibration (Ø) is between 0.8 and 1.4. Since the natural frequency of the barrel was calculated as 65 Hz, and the frequency of forced vibration was 50 Hz, the ratio was obtained as 1.3. Therefore this barrel is slightly within the resonance but it does not coincide with the resonance vibration point.

ii). Stress analysis

The barrel body was considered to be a rigid body and was designed to resist the vertical load and the torsional moment due to the single phase short-circuit. The maximum shearing stress due to compressive stress and shearing stress was calculated 6.1 kg/cm^2, which was less than the allowable shearing stress of 12 kg/cm^2. The calculated maximum principal stress was 16.9 kg/cm^2, which was less than the allowable stress of 60 kg/cm^2.

d). Spiral case encasement

The concrete encasing the spiral case protects the spiral case from the external loads of the generator and internal loads of water pressure, 10.4 kg/cm^2. The casing concrete was designed to resist the external load by the rigid frame structure supported on the encasement concrete of draft tube and the internal load by the encasing mass concrete. The analysis was also made of the stress which occurs in the body of the barrels and casings to ensure that unusual stress will not appear.

e). Draft tube encasement

The concrete encasing the draft tube was designed to resist the external load of the generator and spiral casing by the rigid frame structure of a gate type.

f). Powerhouse main building and sub building (See Part 3 Architectural Works)

The erection bay was designed as the rigid frame structure having two layers and two continuities in the longitudinal section, and two layers and three continuities in the direction of transverse section. Loads acting on the erection bay are the dead load, the floor dead and live loads, the loads of generator equipment, rotor and stator and the load due to temperature change.

The control building was also designed as the rigid frame structure having layers in the direction of longitudinal section, and four layers and two continuities in the direction of transversa section. Loads acting on the control building are the dead load, the floor line and dead load, the lateral load and the load due to temperature charge.

g). Structure under OHT crane (See Part 3 Architectural Works)

Loads due to the OHT crane are the static load (without hoisting operation), the maximum static load during hoisting the generator equipment and the dynamic load in running acting on this structure as the continuous wheel load. This wheel load acts on the structure eccentrically, and the eccentric load acts on the cantilever of this structure right angle to the direction of crane running.

h). Tailrace structure

The tailrace structure adjacent to the powerhouse consists of slabs and columns which support the surcharged load of the gantry crane, gate and water pressure in the tailrace bay. The slabs were assumed to be beams with both ends fixed but the projection slab of setting water level gauge was analyzed as fixed and free two sides of two way slab. Columns were assumed to be cantilever beams with the other end fixed.

i). Powerhouse foundation

Sliding calculations were made under normal and earthquake conditions and proved that the safety factor for the vertical wall was very large therefore there was no problem on safety of the powerhouse foundation. For practical considerations for the vertical wall, rock bolts were arranged assuming the angle of the sliding surface.

(2) Common Design Criteria and Parameters

Structural design of the powerhouse was based on the following parameters and allowable stresses.

(a) Allowable Stress Intensity (kg/cm^2)

	Item	Long term	Short term	Note
Concrete	1. Allowable compressive stress intensity due to bending (σ_{ca})	60	90	
	2. Allowable bearing stress intensity (σ_{ca})	54	81	
	3. Allowable shearing stress intensity (τ_{ca})	4	6	Beam
		8	12	Slab
	4. Allowable bond stress intensity (σ_{ca})	14	21	Deformed reinforcement steel bar
Reinforcement steel bar	5. Allowable tensile stress intensity (σ_{sa})	2,000	3,000	
	6. Allowable shearing stress intensity (τ_{sa})	800	1,200	

Note : * Standard design strength of concrete σ_{ck} = 180 kg/cm^2
* The quality of reinforcement steel bars = SD 35

(b) Material Property Coefficient of Seismic

Concrete (τck = 180 kg/cm²)	Modulus elasticity Ec = 2.4×10^5 kg/cm² Poisson's ration n = 0.16	Unit weight rc = 2.4 t/m³
Reinforcement steel bar	Modulus elasticity Er = 2.1×10^6 kg/cm²	Unit weight rs = 7.8t/m³
Coefficient of seismic	Horizontal direction : kh = 0.10	Vertical direction : kv = 0.0

Note : The allowable stress intensity and material property mentioned above is based on the standard design guideline by the Japanese Civil Engineers Society.

6.8.3 Design of Slabs and Beams

(1) General Design Conditions

The superimposed loads were determined based on the main loads at each floor. For EL. 45.50 slab of generator room, main load consists of rotor 150t, stator 70t, runner 70t and the total is 290t. The area occupied for the loads is 89.21 m². Then 300t/89.21 m² = 3.25 t/m². Considering current practice in Japan and also with some additional allowance, 5t/m² was adopted for the slab design at this floor. For EL. 40.70 slab of auxiliary equipment room and EL. 37.100 of turbine room, main loads are Governor 6t, oil tank 11t and the area occupied for the load is 11.5 m² . Then 17t/11.5 m² = 1.48 t/m², then, 2 t/m² was adopted for these floors. For the floor of power station sub-building. The main load is the diesel engine generator of 10t for area 7 m², then 10t/7m² = 1.43t/m². Therefore, 2 t/m² was adopted with some additional allowance.

Superimposed Loads

	Sub building	Powerhouse main building
EL 45.50 Slab	2 t/m²	5 t/m²
EL. 40.70 Slab	2 t/m²	2 t/m²
EL 37.10 Slab	2 t/m²	2 t/m²
EL 27.30 - EL 31.80 Slab	---	---

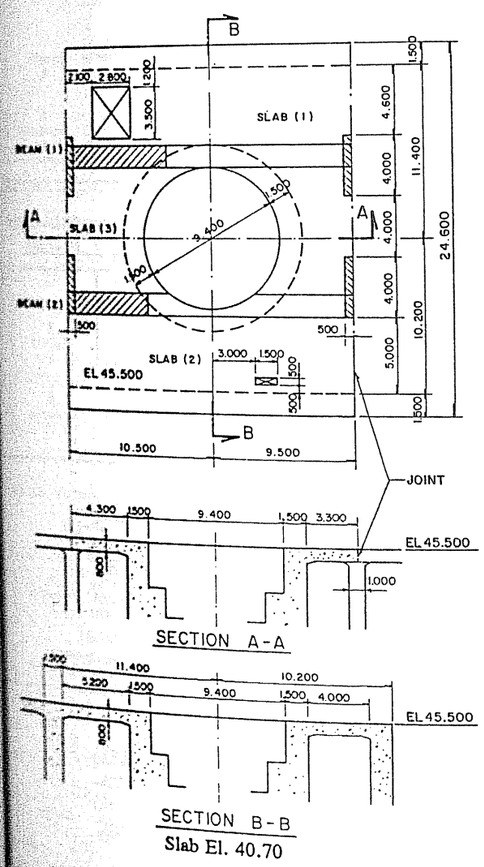
Effective thickness of slabs

	Sub building	Powerhouse main building
EL. 45.50 Slab	Space of cable duct = 20 cm Concrete = 50 cm	Thickness of finishing = 5 cm Concrete = 75cm
EL. 40.70 Slab	Space of cable duct = 20 cm Concrete = 50 cm	Space of cable duct = 20 cm Concrete = 50 cm
EL. 37.10 Slab	Space of cable duct = 30 cm Concrete = 120 cm	Space of cable duct = 30 cm Concrete = ---
EL. 27.30 - EL. 31.80	---	Concrete = 100cm

(2) Structural Analysis

Structural calculations using frame analysis were made on the basis of the following conditions.

a) Slab (1) EL. 45.5 and slab (4) EL. 40.7 are modeled as beam which is fixed and supported by the side wall and beam (1) respectively.
b) Slab (2) EL. 45.5 and slab (5) EL. 40.7 are modeled as beam which is fixed and supported by the side wall and beam (2) respectively.
c) Slab (3) EL. 45.5 and slab (6) EL. 40.7 are modeled as beam which is fixed and supported by the wind tunnel's side wall and column respectively.
d) Beam (1) EL. 45.5 and Beam (3) EL. 40.7 are modeled as beam which is fixed and supported by the wind tunnel's side wall and column respectively.
e) Beam (2) EL. 45.5 and Beam (4) EL. 40.7 are modeled as beam which is fixed and supported by the wind tunnel's side wall and column respectively.
f) Slab (S1) EL. 45.5 and slab (S2) are modeled as beam which is fixed by the side wall.
g) Slab (p1) of sumpit room at EL. 22.7 is modeled as beam which is fixed by one side wall (cantilever).



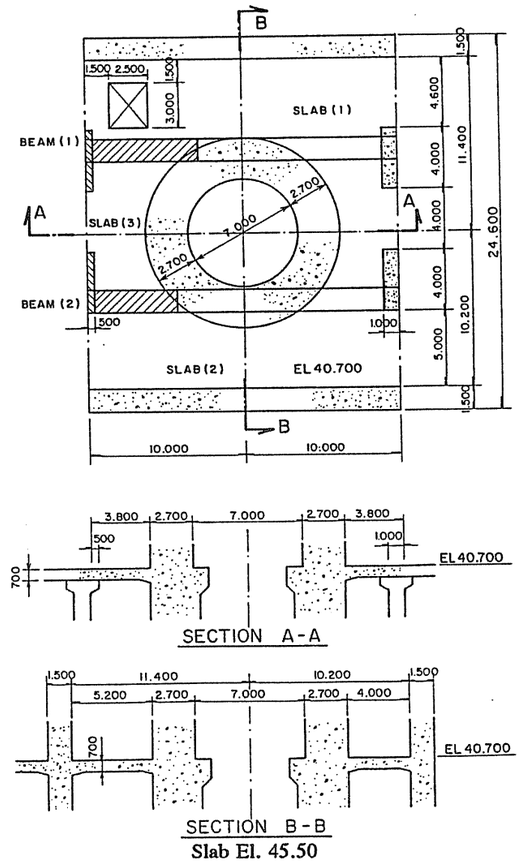


Fig. 1.6.8-4 Calculation Model of Powerhouse Slabs

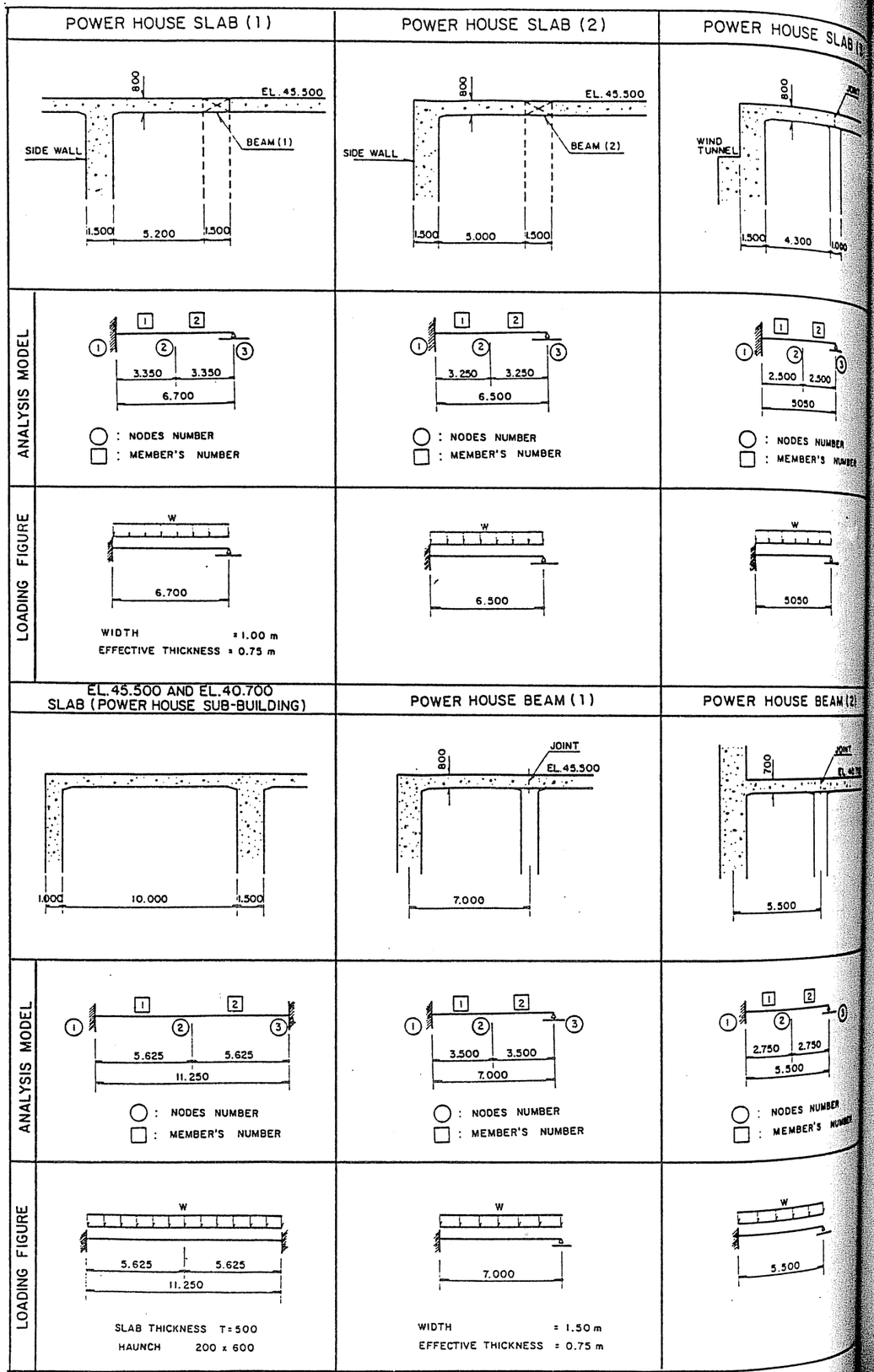


Table 1.6.8-1(1) Calculation of Reinforce Concrete for Powerhouse Slabs and Beams

POWERHOUSE SLAB (1)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	- 38.830	0.000	28.978
1	Haunch	1.650	- 15.553	0.000	17.560
1	2	3.350	- 19.415	0.000	5.796
2	2	0.000	19.415	0.000	5.795
2	Maximum	0.838	21.842	0.000	0.000
2	3	3.350	- 0.000	0.000	17.387

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	38.830	0.000	28.978	100.0	105.0	7.5	7.5
1	Haunch	15.553	0.000	17.560	100.0	75.0	7.5	7.5
1	2	19.415	0.000	5.796	100.0	75.0	7.5	7.5
2	2	19.415	0.000	5.795	100.0	75.0	7.5	7.5
2	Maximum	21.842	0.000	0.000	100.0	75.0	7.5	7.5
2	3	0.000	0.000	17.387	100.0	75.0	7.5	7.5

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	21.44	21.44	00220	35.37	2000	3.21
1	Haunch	12.40	12.40	00184	33.13	2000	2.80
1	2	15.59	15.59	00231	37.07	2000	0.93
2	2	15.59	15.59	00231	37.07	2000	0.93
2	Maximum	17.60	17.60	00261	39.33	2000	0.00
2	3	1.00	1.00	00015	.00	0	2.63

POWERHOUSE SLAB (2)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	- 36.546	0.000	28.113
1	Haunch	1.650	- 14.048	0.000	16.695
1	2	3.250	18.273	0.000	5.623
2	2	0.000	18.273	0.000	5.622
2	Maximum	0.813	20.557	0.000	0.000
2	3	3.250	0.000	0.000	16.868

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	36.546	0.000	28.113	100.0	105.0	7.5	7.5
1	Haunch	14.048	0.000	16.695	100.0	75.0	7.5	7.5
1	2	18.273	0.000	5.623	100.0	75.0	7.5	7.5
2	2	18.273	0.000	5.622	100.0	75.0	7.5	7.5
2	Maximum	20.557	0.000	0.000	100.0	75.0	7.5	7.5
2	3	0.000	0.000	16.868	100.0	75.0	7.5	7.5

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	20.14	20.14	00207	34.32	2000	3.11
1	Haunch	11.17	11.17	00165	31.46	2000	2.65
1	2	14.64	14.64	00217	35.95	2000	0.90
2	2	14.64	14.64	00217	35.95	2000	0.90
2	Maximum	16.54	16.54	00245	38.15	2000	0.00
2	3	1.00	1.00	00015	0.00	0	2.55

POWERHOUSE SLAB (3)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	- 22.060	0.000	21.841
1	Haunch	1.650	- 5.205	0.000	10.423
1	2	2.500	10.918	0.000	4.541
2	2	0.000	10.918	0.000	4.541
2	Maximum	0.856	12.409	0.000	0.000
2	3	2.550	- 0.000	0.000	13.105

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	22.060	0.000	21.841	100.0	105.0	7.5	7.5
1	Haunch	5.205	0.000	10.423	100.0	75.0	7.5	7.5
1	2	10.918	0.000	4.541	100.0	75.0	7.5	7.5
2	2	10.918	0.000	4.541	100.0	75.0	7.5	7.5
2	Maximum	12.409	0.000	0.000	100.0	75.0	7.5	7.5
2	3	0.000	0.000	13.105	100.0	75.0	7.5	7.5

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	12.01	12.01	00123	26.62	2000	2.36
1	Haunch	4.03	4.03	00060	18.92	2000	1.61
1	2	8.81	8.81	00129	27.67	2000	0.72
2	2	8.81	8.81	00129	27.67	2000	0.72
2	Maximum	9.83	9.83	00146	29.53	2000	0.00
2	3	1.00	1.00	00015	0.00	0	1.98

SUB - BUILDING (CONTROL ROOM)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	- 38.813	0.000	20.700
1	Haunch	1.400	- 21.673	0.000	15.548
1	2	5.625	19.406	0.000	0.000
2	2	0.000	19.406	0.000	0.000
2	Haunch	3.975	- 21.673	0.000	14.628
2	3	5.625	- 38.813	0.000	20.700

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	38.813	0.000	20.700	100.0	80.0	7.5	7.5
1	Haunch	21.673	0.000	15.548	100.0	50.0	7.5	7.5
1	2	19.406	0.000	0.000	100.0	50.0	7.5	7.5
2	2	19.406	0.000	0.000	100.0	50.0	7.5	7.5
2	Haunch	21.673	0.000	14.628	100.0	50.0	7.5	7.5
2	3	38.813	0.000	20.700	100.0	80.0	7.5	7.5

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	29.47	29.47	00407	48.40	2000	3.16
1	Haunch	34.72	34.72	00817	60.00	1690	4.19
1	2	27.77	27.77	00653	60.00	1873	0.00
2	2	27.77	27.77	00653	60.00	1873	0.00
2	Haunch	34.72	34.72	00817	60.00	1690	3.94
2	3	29.47	29.47	00407	48.40	2000	3.16

Table 1.6.8-1(2) Calculation of Reinforce Concrete for Powerhouse Slabs and Beams

POWERHOUSE BEAM (1)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	-138.284	0.000	98.774
1	Haunch	1.650	- 58.531	0.000	61.522
1	2	3.500	69.142	0.000	19.755
2	2	0.000	69.142	0.000	19.755
2	Maximum	0.875	77.785	0.000	0.000
2	Haunch	2.100	44.194	0.000	27.657
2	3	3.500	0.000	0.000	59.265

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	138.284	0.000	98.774	150.0	105.0	7.5	7.5
1	Haunch	58.531	0.000	61.522	150.0	75.0	7.5	7.5
1	2	69.142	0.000	19.755	150.0	75.0	7.5	7.5
2	2	69.142	0.000	19.755	150.0	75.0	7.5	7.5
2	Maximum	77.785	0.000	0.000	150.0	75.0	7.5	7.5
2	Haunch	44.194	0.000	27.657	150.0	75.0	7.5	7.5
2	3	0.000	0.000	59.265	150.0	105.0	7.5	7.5

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	77.98	77.98	00533	53.76	2000	7.56
1	Haunch	48.06	48.06	00475	52.37	2000	6.77
1	2	57.06	57.06	00564	56.71	2000	2.19
2	2	57.06	57.06	00564	56.71	2000	2.19
2	Maximum	64.41	64.41	00636	59.95	2000	0.00
2	Haunch	35.96	35.96	00355	45.65	2000	3.01
2	3	1.00	1.00	00007	.00	0	4.11

POWERHOUSE BEAM (2)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)
1	1	0.000	- 83.407	0.000
1	Haunch	1.650	- 24.098	0.000
1	2	2.750	41.703	0.000
2	2	0.000	41.703	0.000
2	Maximum	0.688	46.916	0.000
2	Haunch	1.350	32.012	0.000
2	3	2.750	0.000	0.000

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)
1	1	83.407	0.000	75.824	150.0	105.0	7.5
1	Haunch	24.098	0.000	39.429	150.0	75.0	7.5
1	2	41.703	0.000	15.165	150.0	75.0	7.5
2	2	41.703	0.000	5.165	150.0	75.0	7.5
2	Maximum	46.916	0.000	0.000	150.0	75.0	7.5
2	Haunch	32.012	0.000	14.613	150.0	75.0	7.5
2	3	0.000	0.000	45.495	150.0	105.0	7.5

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)
1	1	46.47	46.47	00318	42.22	2000
1	Haunch	19.23	19.23	00190	33.88	2000
1	2	33.87	33.87	00335	44.36	2000
2	2	33.87	33.87	00335	44.36	2000
2	Maximum	38.25	38.25	00378	47.02	2000
2	Haunch	25.78	25.78	00255	38.88	2000
3		1.00	1.00	00007	.00	0

(3) Calculation result

a) Required quantities of reinforcement steel bars at each sections

EL 45.50	Upper surface	Lower surface
Sub building slab (Control building)	D29 @150 = 42.9>34.72 cm²	D25 @150 = 33.8 > 27.77 cm²
Powerhouse slab (1)	D22 @150 = 25.8>21.44 cm²	D19 @150 = 19.1 > 17.60 cm²
Powerhouse slab (2)	D22 @150 = 25.8>20.14 cm²	D19 @150 = 19.1 > 16.54 cm²
Powerhouse slab (3)	D22 @ 300 = 12.9>12.01 cm²	D22 @ 300 = 12.9 > 9.03 cm²
Powerhouse beam (1)	D29 x 10 pcs = 64.2>51.99 cm² (@ 100)	D25 x 10 pcs = 50.7>42.94 cm² (@ 100)
Powerhouse beam (2)	D22 x 10 pcs = 38.7>30.98 cm² (@ 100)	D19 x 10 pcs = 28.7>25.50 cm² (@ 100)

EL. 40.70	Upper surface	Lower surface
Sub building slab (Control building)	D29@150 = 42.9>34.72 cm²	D25@150 = 33.8>27.77 cm²
Powerhouse slab (1)	D19@150 = 19.1>15.38 cm²	D19@150 = 19.1>15.15 cm²
Powerhouse slab (2)	D25@300 = 16.9>14.45 cm²	D25@300 = 16.9>14.22 cm²
Powerhouse slab (3)	D19@300 = 9.54>8.60 cm²	D19@300 = 9.54>8.39 cm²
Powerhouse beam (1)	D22x10 pcs = 38.7>37.49 cm² (@ 100)	D29x10 pcs = 64.2>55.21 cm² (@ 100)
Powerhouse beam (2)	D19x10 pcs = 28.7>22.27 cm² (@ 100)	D19x10 pcs = 28.7>22.06 cm² (@ 100)

b) Stress intensity of each cross section

EL 45.50		Upper surface	Lower surface
Sub-building slab	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D29 @ 150 1380 < 2000 kg/cm² 53.9 < 60 4.24 < 8 7.10 < 14	D25 @ 150 1552 < 2000 kg/cm² 54.4 < 60 0 < 8 0 < 14
Power house slab (1)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D22 @ 150 1669 < 2000 kg/cm² 32.3 < 60 3.23 < 8 6.90 < 14	D19 @ 150 1848 < 2000 kg/cm² 37.8 < 60 0 < 8 0 < 14
Power house slab (2)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D22 @ 150 1571 < 2000 kg/cm² 30.4 < 60 3.14 < 8 6.70 < 14	D19 @ 150 1739 < 2000 kg/cm² 35.6 < 60 0 < 8 0 < 14
Power house slab (3)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D22 @ 300 1864 < 2000 kg/cm² 25.7 < 60 2.38 < 8 10.2 < 14	D22 @ 300 1536 < 2000 kg/cm² 25.9 < 60 0 < 8 0 < 14
Power house beam (1)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D29 x 10 pcs 1627 < 2000 kg/cm² 48.1 < 60 7.63 < 8 8.50 < 14	D29 x 10 pcs 1702 < 2000 kg/cm² 55.0 < 60 0 < 8 0 < 14
Power house beam (2)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D22 x 10 pcs 1610 < 2000 kg/cm² 37.7 < 60 5.73 < 8 8.20 < 14	D19 x 10 pcs 1784 < 2000 kg/cm² 44.3 < 60 0 < 8 0 < 14

EL 40.70		Upper surface	Lower surface
Sub-building slab	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D29 @ 150 1380 < 2000 kg/cm² 53.9 < 60 4.24 < 8 7.10 < 14	D25 @ 150 1552 < 2000 kg/cm² 54.4 < 60 0 < 8 0 < 14
Power house slab (1)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D19 @ 150 1621 < 2000 kg/cm² 31.9 < 60 2.31 < 8 5.80 < 14	D19 @ 150 1603 < 2000 kg/cm² 43.0 < 60 0 < 8 0 < 14
Power house slab (2)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D25 @ 300 1718 < 2000 kg/cm² 31.8 < 60 2.23 < 8 8.40 < 14	D25 @ 300 1696 < 2000 kg/cm² 42.8 < 60 0 < 8 0 < 14
Power house slab (3)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D19 @ 300 1807 < 2000 kg/cm² 25.3 < 60 1.70 < 8 8.50 < 14	D19 @ 300 1769 < 2000 kg/cm² 33.7 < 60 0 < 8 0 < 14
Power house beam (1)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D22 x 10 pcs 1939 < 2000 kg/cm² 53.3 < 60 5.41 < 8 7.70 < 14	D25 x 10 pcs 1191 < 2000 kg/cm² 55.2 < 60 0 < 8 0 < 14
Power house beam (2)	Reinforcement Tensile stress Compressive stress Shear stress Bond stress	D19 x 10 pcs 1564 < 2000 kg/cm² 37.4 < 60 4.10 < 8 6.80 < 14	D19 x 10 pcs 1556 < 2000 kg/cm² 50.6 < 60 0 < 8 0 < 14

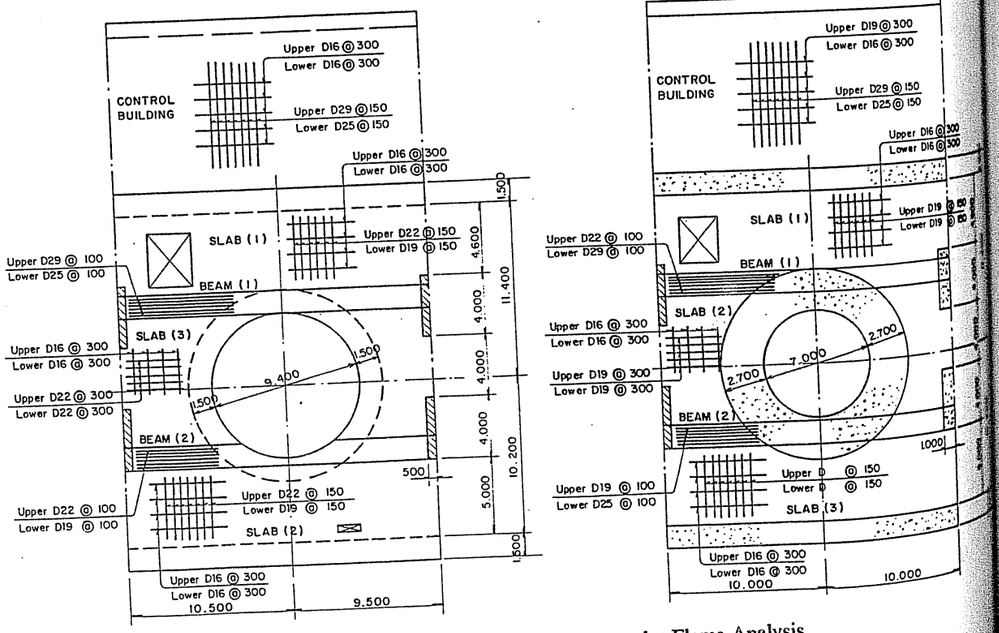
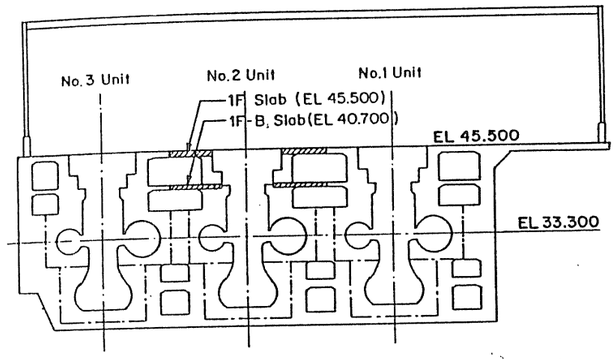


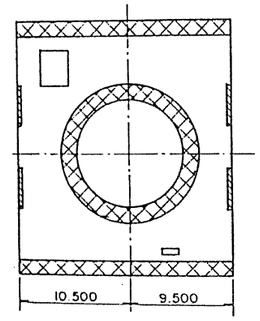
Fig. 1.6.8-5 Reinforce Bar Arrangement by Flame Analysis

(4) Stress Analysis of Powerhouse Slabs

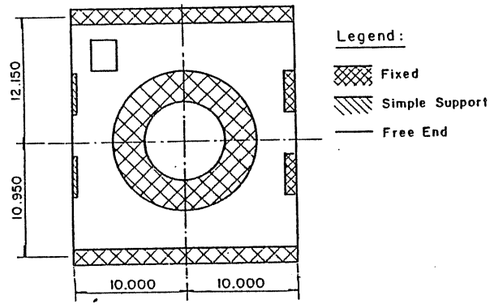
The stresses occurring in the slabs were analyzed by the finite element method which is useful to determine the final bar arrangement for slabs. Analysis was made for 2 cases for 1F (EL. 45.500) slab and 1F-B (EL. 40.700) slab in the center of the power station. The following figures show the analytical model.



Location of Analysis



I F El. 45.00 slab



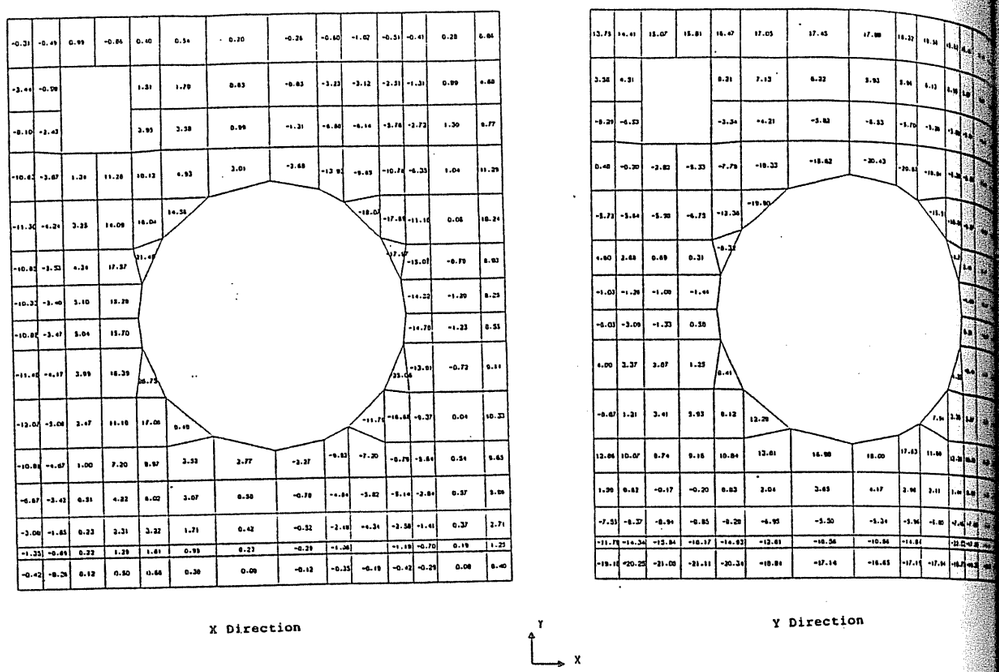
I F-B El. 45.70 slab

Figures 1.6.8-6 and 1.6.8-7 show the bending moments and the shearing forces for each element of 1F (EL. 45.500) slab and 1F-B (EL. 40.700) slab, respectively.

From the bending moment and shearing force diagram, the quantity of reinforcement can be obtained. The (Reinforce steel bar arrangement for the slabs obtained by FEM Analysis) is shown in the Figure 1.6.8-8. For convenience of reinforcement arrangement, the floor slab are divided into five block, i.e. block A and B (control room side with an opening part), bock C and E (tailrace side) and block D (control room side with no opening part). The slab of block A, B and C in this analysis can be compared with the slab (1), slab (3) and slab (2) which was carried out in the preceding section (2) Structural Analysis.

Fig. 1.6.8-6 Powerhouse Slabs Strength by FEM Analysis (I F El. 45.50)

Bending Moment



Shearing Force

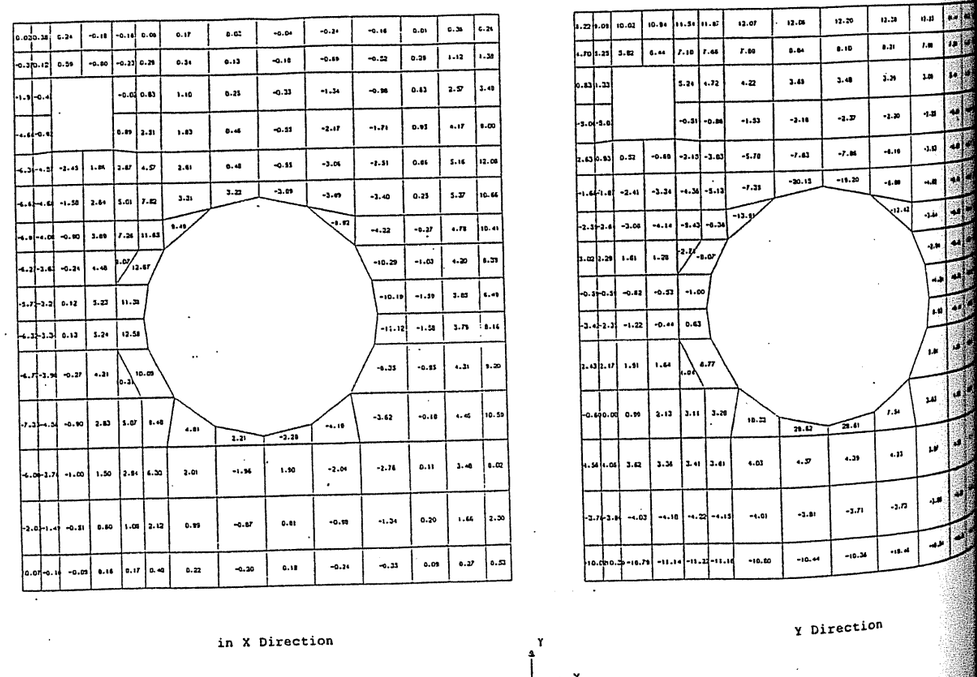
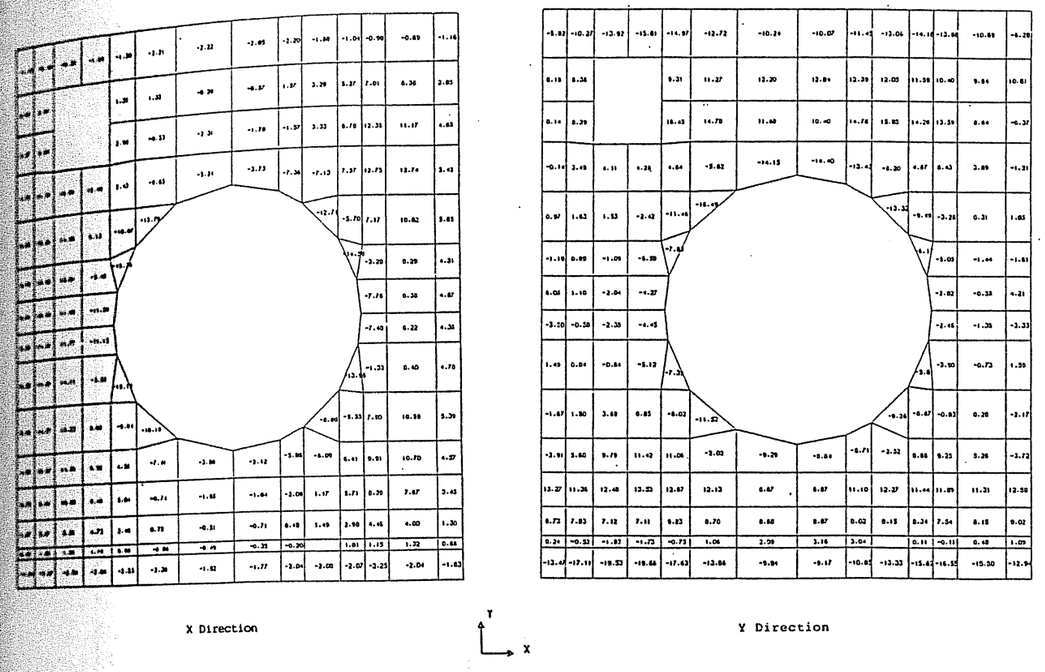
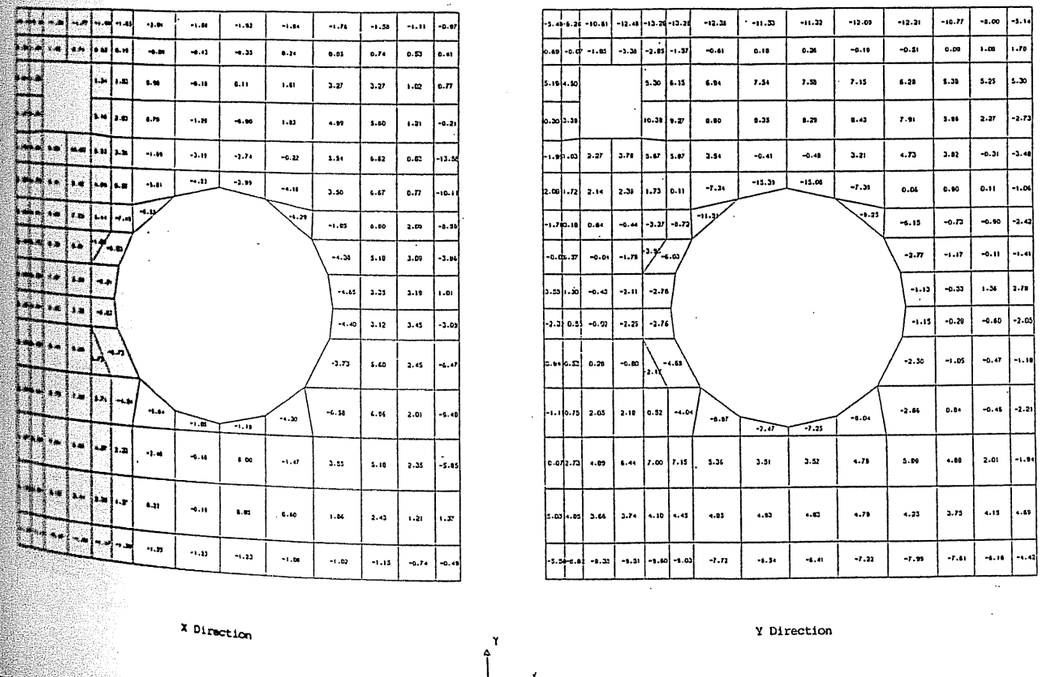


Fig. 1.6.8-7 Powerhouse Slabs Strength by FEM Analysis (I F-B El. 40.70)

Bending Moment



Shearing Force



Reinforce steel bar arrangement for slabs obtained by FEM Analysis

Floor	Block	Direction	Surface	Bar Arrangement
1F Slab (EL. 45.5)	Block-A	X	Upper	D16 @ 300
			Lower	D16 @ 300
		Y	Upper	D19 @ 150
			Lower	D19 @ 150
	Block-B	X	Upper	D19 @ 150
			Lower	D19 @ 150
		Y	Upper	D19 @ 150
			Lower	D16 @ 150
	Block-C	X	Upper	D16 @ 300
			Lower	D22 @ 300
		Y	Upper	D19 @ 150
			Lower	D22 @ 300
	Block-D	X	Upper	D16 @ 300
			Lower	D16 @ 150
		Y	Upper	D19 @ 150
			Lower	D16 @ 150
	Block-E	X	Upper	D16 @ 300
			Lower	D19 @ 300
		Y	Upper	D19 @ 150
			Lower	D22 @ 300
1F - B Slab (EL. 40.7)	Block-A	X	Upper	D16 @ 300
			Lower	D16 @ 300
		Y	Upper	D19 @ 150
			Lower	D19 @ 150
	Block-B	X	Upper	D19 @ 150
			Lower	D19 @ 150
		Y	Upper	D19 @ 150
			Lower	D16 @ 150
	Block-C	X	Upper	D19 @ 300
			Lower	D22 @ 300
		Y	Upper	D19 @ 300
			Lower	D22 @ 300
	Block-D	X	Upper	D22 @ 150
			Lower	D19 @ 300
		Y	Upper	D19 @ 300
			Lower	D22 @ 300
	Block-E	X	Upper	D19 @ 150
			Lower	D19 @ 300
		Y	Upper	D19 @ 150
			Lower	D19 @ 300

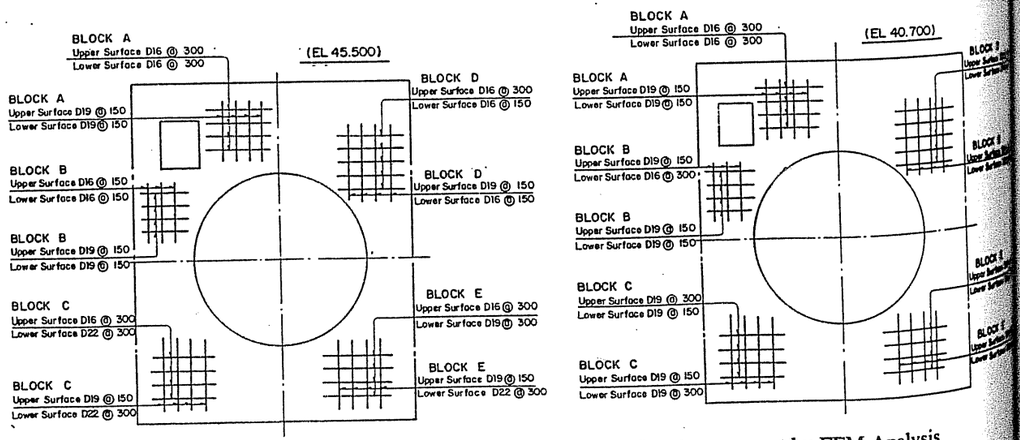


Fig. 1.6.8-8 Powerhouse Slabs Reinforce Bar Arrangement by FEM Analysis

(5) Consideration

Bar arrangement for slabs obtained by FEM analysis was compared with that separately obtained by the frame analysis calculation. The following table shows this comparison. The slabs of block A, B and C correspond to the slab (1), slab (3) and slab (2) in the frame structure analysis respectively.

Comparison of Quantities of Bar Arrangement

Floor	Block	Direction	Surface	FEM analysis	Frame analysis
1F Slab EL.45.5	A Slab (1)	X	Upper	D16 @ 300	D16 @ 300
			Lower	D16 @ 300	D16 @ 300
		Y	Upper	D19 @ 150	D22 @ 150
			Lower	D19 @ 150	D19 @ 150
	B Slab (3)	X	Upper	D19 @ 150	D22 @ 300
			Lower	D19 @ 150	D22 @ 300
		Y	Upper	D19 @ 150	D16 @ 300
			Lower	D16 @ 150	D16 @ 300
	C Slab (2)	X	Upper	D16 @ 300	D16 @ 300
			Lower	D22 @ 300	D16 @ 300
		Y	Upper	D19 @ 150	D22 @ 150
			Lower	D22 @ 300	D19 @ 150
1F-B Slab EL.40.7	A Slab (2)	X	Upper	D16 @ 300	D16 @ 300
			Lower	D16 @ 300	D16 @ 300
		Y	Upper	D19 @ 150	D19 @ 150
			Lower	D19 @ 150	D19 @ 150
	B Slab (3)	X	Upper	D19 @ 150	D19 @ 300
			Lower	D19 @ 150	D19 @ 300
		Y	Upper	D19 @ 150	D16 @ 300
			Lower	D16 @ 150	D16 @ 300
	C Slab (2)	X	Upper	D19 @ 300	D16 @ 300
			Lower	D22 @ 300	D16 @ 300
		Y	Upper	D19 @ 300	D25 @ 300
			Lower	D22 @ 300	D25 @ 300

From the above table comparing reinforcement arrangements, the FEM analysis has showed a somewhat more severe condition than the frame analysis at the block B of 1F and 1F-B slabs because of the influence of the opening part (See Fig. 1.6.8-7 for Bar Arrangement of 1F and 1F-B respectively), hence, the FEM analysis may be applied to the reinforcement of block B.

However, the difference between the quantities of bar arrangement based on the both calculation method is small as a whole, therefore the reinforcement by the frame analysis was basically adopted. The reinforcement arrangement also practically provide additional safety for the powerhouse slabs which are very important to support the main heavy loads such as Rotor, Stator and Runner.

6.8.4 Design of Side Walls

(1) Load Conditions

(i) Earth pressure : Coulomb's theory of earth pressure was applied

$$P_A = \frac{1}{2} K_A.\gamma.H^2, \quad K_A = \frac{\cos(\varnothing-\alpha)}{\cos^2\alpha.\cos(\alpha+\delta)\left[1+\sqrt{\frac{\sin(\varnothing+\delta).\sin(\varnothing-\beta)}{\cos(\alpha+\delta).\cos(\alpha-\beta)}}\right]} = 0.24$$

Where, P_A : Coulomb's active earth pressure acting at wall per unit width
K_A : Coefficient of active earth pressure = 0.31
γ : Unit weight of backfilling soil = 1.8 t/m^3
H : Wall's height to be adapted for calculation of earth pressure
α : The degree between wall's back and vertical = 0
β : The degree between backfilling surface ground and horizontal = 0
$\varnothing$: Internal friction angle of soil = 35^0
δ : Friction angle of wall face $\varnothing$ = 23^0 (δ=⅔$\varnothing$ is standardized)

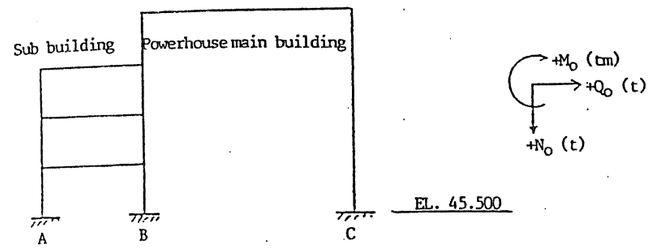
$$P_{AE} = \frac{1}{2}.K_{AE}.\gamma.H^2, \quad K_{AE} = \frac{\cos^2(\varnothing-\alpha-\theta)}{\cos\theta.\cos^2\alpha.\cos(\alpha+\delta+\theta)\left[1+\frac{\sin(\varnothing+\delta).\sin(\varnothing-\beta-\theta)^2}{\cos(\alpha+\delta+\theta).\cos(\alpha-\beta)}\right]} = 0.31$$

Where, P_{AE} : active earth pressure by earthquake
K_{AE} : Coefficient of active earth pressure by earthquake
θ : Compounded degree by earthquake $\theta = \tan^{-1} k_h = \tan^{-1}(0.1) = 5.7^0$
K_h : Designed horizontal seismic coefficient = 0.1
δ : $\varnothing/2 = 35/2 = 17.5^0$

(ii) Superimposed loads around side wall and seismic coefficient : q = 1.0 t/m^2 and $K_n = 0.10$

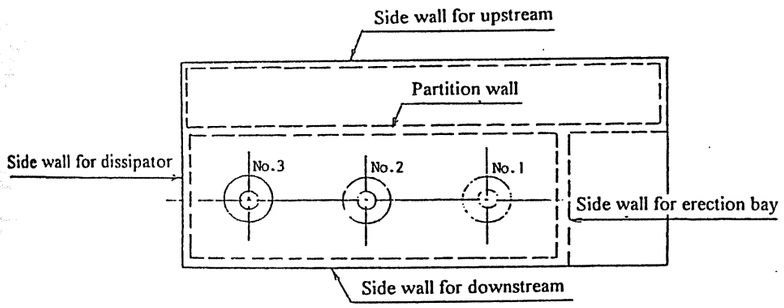
(iii) Superstructure load : The load indicated below is given by the superstructure of Architectural Work.

	Long Term			Short Term		
	N0 (+)	Q0 (+)	M0 (+)	N0 (+)	Q0 (+)	M0 (+)
A	68.8	15.0	0	86.5	49.5	0
B	248.8	8.5	13.5	211.2	93.6	245
C	180.1	6.5	38.5	121.0	57.1	136



(2) Calculation Case

- Side wall for upstream : normal and during earthquake
- Side wall for downstream : normal and during earthquake
- Partition wall : during earthquake
- Side wall for dissipator : during flood time
- Side wall for erection bay : this case is not calculated (no external pressure act on the wall because of rock foundation)

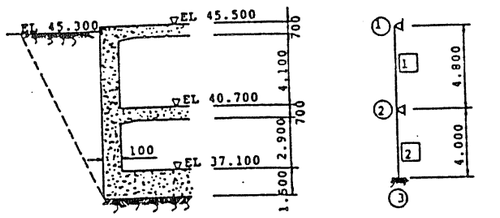


(3) Calculation models and loads are shown in the following figure 1.6.8-9.

Fig. 1.6.8-9 Calculation Model of Powerhouse Side Wall

Side Wall for Upstream

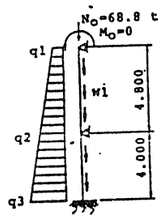
(a) Calculation model



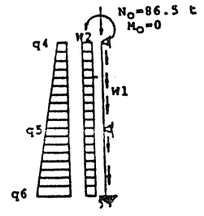
Properties member T = 1.00 m
B = 1.00 m

(b) Calculation of loads

Normal



During earthquake



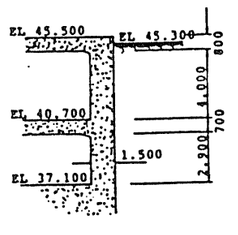
q1 = 0.28 t/m
q2 = 2.19
q3 = 3.78

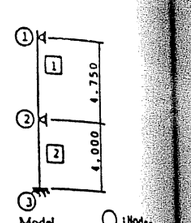
q4 = 0.08 t/m
q5 = 2.63
q6 = 4.76

Note : Dead weight W_1 = 2.4 t/m
Inertia force of side wall W_2 = 0.24 t/m

Side Wall for Downstream

(a) Calculation model

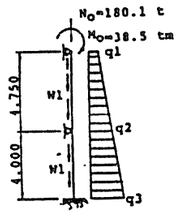




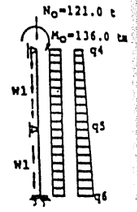
Properties of member T = 1.00 m
B = 1.00 m

(b) Calculation of loads

Common



During earthquake



q1 = 0.30 t/m
q2 = 2.19
q3 = 3.78

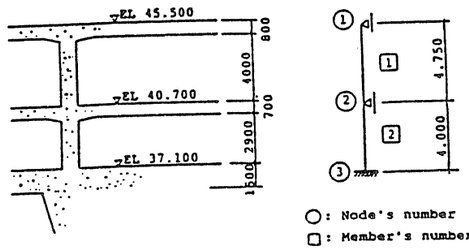
q4 = 0.11 t/m
q5 = 2.63
q6 = 4.76

Note : Dead weight W_1 = 5.4 t/m
Inertia force of side wall W_2 = 0.54 t/m

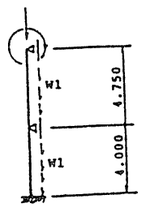
Partition Wall

(a) Calculation model

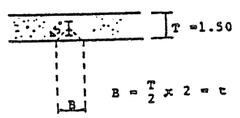
It should be continuous beam shown as figure of right.



(b) Calculation of loads (During earthquake)



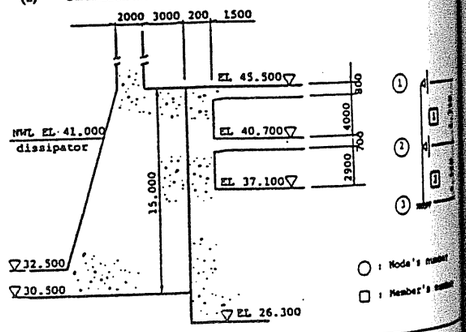
Calculation for effective width



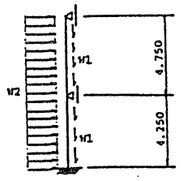
Dead weigth W_1 = 5.40 t/m

Side Wall for Energy Dissipator

(a) Calculation model



(b) Calculation of loads (During flood)



W1 = 4.80 t/m
W2 = 16.61 t/m

Load figure

Table 1.6.8-2(1) Calculation of Reinforce Concrete for Powerhouse Walls

Side Wall Upstream (Normal)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	-0.000	66.800	1.419
1	maximum	2.053	-1.748	73.739	0.000
1	2	4.800	3.751	80.320	4.509
2	2	0.000	3.751	80.320	5.328
2	maximum	2.051	-1.998	85.242	0.000
2	3	4.000	4.201	89.920	6.612

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	0.000	66.800	1.149	100.0	100.0	10.0	10.0
1	maximum	1.748	73.739	0.000	100.0	100.0	10.0	10.0
1	2	3.751	80.320	4.509	100.0	100.0	10.0	10.0
2	2	3.751	80.320	5.329	100.0	100.0	10.0	10.0
2	maximum	1.998	85.242	0.000	100.0	100.0	10.0	10.0
2	3	4.201	89.920	6.612	100.0	100.0	10.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	1.00	1.00	00011	8.86	0	0.01
1	maximum	1.00	1.00	00011	8.39	0	0.00
1	2	1.00	1.00	00011	10.25	0	0.04
2	2	1.00	1.00	00011	10.25	0	0.05
2	maximum	1.00	1.00	00011	9.69	0	0.00
2	3	1.00	1.00	00011	11.47	0	0.06

Side Wall Upstream (Earthquake)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	0.000	68.500	1.790
1	maximum	2.063	-2.235	91.450	0.000
1	2	4.800	4.686	98.020	5.866
2	2	0.000	4.886	98.020	6.989
2	maximum	2.047	-2.647	105.932	0.000
2	3	4.000	5.569	107.620	8.751

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	0.000	86.500	1.790	100.0	100.0	10.0	10.0
1	maximum	2.235	91.450	0.000	100.0	100.0	10.0	10.0
1	2	4.686	98.020	5.866	100.0	100.0	10.0	10.0
2	2	4.886	98.020	6.989	100.0	100.0	10.0	10.0
2	maximum	2.647	102.932	0.000	100.0	100.0	10.0	10.0
2	3	4.201	107.620	8.751	100.0	100.0	10.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	1.00	1.00	00011	8.62	0	0.02
1	maximum	1.00	1.00	00011	10.45	0	0.00
1	2	1.00	1.00	00011	12.69	0	0.05
2	2	1.00	1.00	00011	10.69	0	0.06
2	maximum	1.00	1.00	00011	11.84	0	0.00
2	3	1.00	1.00	00011	14.05	0	0.08

Side Wall Downstream (Normal)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	38.500	160.100	9.163
1	2	4.750	-15.518	205.750	15.077
2	2	0.000	-15.518	205.750	9.740
2	maximum	3.399	- 2.337	224.105	0.000
2	3	4.000	1.683	227.350	2.200

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1		38.500	160.000	9.163	150.0	150.0	10.0	10.0
2		15.518	205.750	15.077	150.0	150.0	10.0	10.0
2		15.518	205.750	9.740	150.0	150.0	10.0	10.0
2	maximum	2.337	224.105	0.000	150.0	150.0	10.0	10.0
3		1.683	222.350	2.200	150.0	150.0	10.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1		1.00	1.00	00005	14.82	0	0.02
2		1.00	1.00	00005	11.88	0	0.03
2		1.00	1.00	00005	11.88	0	0.02
2	maximum	1.00	1.00	00005	10.36	0	0.00
3		1.00	1.00	00005	10.39	0	0.00

Side Wall Downstream (Earthquake)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	136.000	121.000	35.030
1	2	4.750	- 47.200	146.650	44.102
2	2	0.000	- 47.200	146.650	23.307
2	3	4.000	14.988	168.250	6.367

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1		136.000	121.000	35.030	150.0	150.0	10.0	10.0
2		47.200	146.650	44.102	150.0	150.0	10.0	10.0
2		47.200	146.650	23.307	150.0	150.0	10.0	10.0
3		14.988	168.250	6.367	150.0	150.0	10.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1		15.15	15.15	00072	62.56	3000	1.66
2		1.00	1.00	00005	15.18	20	0.01
2		1.00	1.00	00005	15.18	20	0.01
3		1.00	1.00	00005	10.12	0	0.01

Table 1.6.8-2(2) Calculation of Reinforce Concrete for Powerhouse Walls

Side Wall for Stilling Basin Side (P/H Building)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	0.000	0.000	31.433
1	maximum	1.892	-29.743	9.084	0.000
1	2	4.750	38.073	22.800	47.464
2	2	0.000	38.073	22.800	39.910
2	maximum	2.403	- 9.874	34.333	0.000
2	3	4.250	18.466	43.200	30.683

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	0.000	0.000	31.433	100.0	100.0	10.0	10.0
1	maximum	29.743	9.084	0.000	100.0	100.0	10.0	10.0
1	2	38.075	22.800	47.464	100.0	100.0	10.0	10.0
2	2	38.073	22.800	39.910	100.0	200.0	10.0	10.0
2	maximum	9.874	34.333	0.000	100.0	200.0	10.0	10.0
2	3	18.466	43.200	30.683	100.0	200.0	10.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	1.00	1.00	00005	0.00	0	1.68
1	maximum	3.85	3.85	00020	21.64	3000	0.00
1	2	3.11	3.11	00016	27.42	2999	2.49
2	2	3.11	3.11	00016	27.42	2999	2.09
2	maximum	1.00	1.00	00005	3.19	0	0.00
2	3	1.00	1.00	00005	5.01	8	0.01

Side Wall for Stilling Basin Side (Sub - Building)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)
1	1	0.000	- 0.000	- 0.000
1	maximum	1.892	-29.743	6.813
1	2	4.750	38.073	17.100
2	2	0.000	38.073	17.100
2	maximum	2.403	- 9.874	25.750
2	3	4.250	18.466	32.400

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)
1	1	0.000	- 0.000	31.433	100.0	150.0	10.0
1	maximum	29.743	6.813	0.000	100.0	150.0	10.0
1	2	38.073	17.100	47.464	100.0	150.0	10.0
2	2	38.073	17.100	39.910	100.0	150.0	10.0
2	maximum	9.874	25.750	0.000	100.0	150.0	10.0
2	3	18.466	32.400	30.683	100.0	150.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)
1	1	1.00	1.00	00007	0.00	0
1	maximum	6.23	6.23	00044	28.19	3000
1	2	6.63	6.63	00047	34.34	3000
2	2	6.63	6.63	00047	34.34	3000
2	maximum	1.00	1.00	00007	4.66	11
2	3	1.00	1.00	00007	11.42	258

Partition Wall (Earthquake)

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	245.000	211.200	67.385
1	2	4.750	- 75.081	236.850	67.385
2	2	0.000	- 75.081	236.850	28.155
2	3	4.000	37.540	158.450	28.155

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

	M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)
1	245.000	211.200	67.385	150.0	150.0	10.0
2	75.081	236.850	67.385	150.0	150.0	10.0
2	75.081	236.850	28.155	150.0	150.0	10.0
3	37.540	258.450	28.155	150.0	150.0	10.0

	AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)
1	30.45	30.45	00145	86.19	3000
2	1.00	1.00	00005	24.25	29
2	1.00	1.00	00005	24.25	29
3	1.00	1.00	00005	18.13	9

(4) Calculation Results

Required Quantities of Reinforcement Steel Bars at Each Section

		Outside	Inside wall
Side wall of upstream side	Common During earthquake Determination	1.0 cm² 1.0 cm² D16 @ 300 = 6.61 cm²	1.0 cm² 1.0 cm² D16 @ 300 = 6.61 cm²
Side wall of downstream side	Common During earthquake Determination	1.0 cm² 15.15 cm² D19 @ 150 = 16.1 cm²	1.0 cm² 15.15 cm² D19 @ 150 = 19.1 cm²
Partition wall	During earthquake Determination	30.45 cm² D25 @ 150 = 33.8 cm²	D25 @ 150 = 33.8 cm²
Side wall of Energy dissipator side	During - Flooding Determination	6.63 cm² D19 @ 300 = 9.54 cm²	6.23 cm² D19 @ 300 = 6.61 cm²
Side wall for erection bay	Determination	D16 @ 300 = 6.61 cm²	D16 @ 300 = 6.61 cm²

Stress Intensity of Each Cross Section

		Unit	Outside wall	Inside wall
Sidewall of upstream side	Reinforcement	cm²	D16 @ 300 = 6.61 > 1.0	D16 @ 300= 6.61 > 1.0
	Tensile stress	kg/cm²	0 < 3000	0 < 3000
	Compressive stress		13.8 < 90	11.6 < 90
	Shear stress		0.38 < 12	0 < 12
	Bond stress		2.3 < 21	0 < 21
Sidewall of downstream side	Reinforcement	cm²	D19 @150= 19.1 > 15.15	D19 @ 150= 19.1>15.15
	Tensile stress	kg/cm²	2350 < 3000	2350 < 3000
	Compressive stress		56.2 < 90	56.2 < 90
	Shear stress		1.67 < 12	1.67 < 12
	Bond stress		5.0 < 21	5.0 < 21
Partition wall	Reinforcement	cm²	D25 @150= 33.8 > 30.45	D25 @150=33.8 > 30.45
	Tensile stress	kg/cm²	2423 < 3000	2423 < 3000
	Compressive stress		78.5 < 90	78.5 < 90
	Shear stress		3.30 < 12	3.30 < 12
	Bond stress		7.1 < 21	7.1 < 21
Sidewall of stilling basin side	Reinforcement	cm²	D19 @ 300 = 9.54 > 6.63	D16 @ 300 = 6.61> 6.23
	Tensile stress	kg/cm²	1839 < 3000	2662 <3000
	Compressive stress		28.6 < 90	27.3 < 90
	Shear stress		3.50 < 12	0 < 12
	Bond stress		17.5 < 21	0 < 21

Around Barrel

ditions

The shape of section is shown in Figure 1.6.8-9

Designed loads of generators foundation (ton)

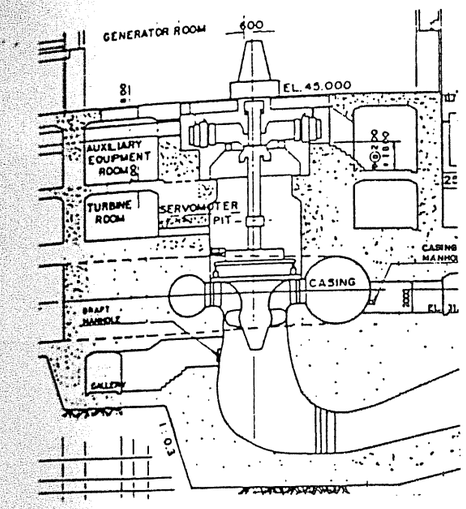
Foundation		Stator foundation			Lower bracket foundation			Note
		Vertical loads	Horizontal tangential direction on loads	Horizontal radius direction on loads	Vertical loads	Horizontal tangential direction on loads	Horizontal radius direction on loads	Long term
During stop operation		100	0	0	220	0	0	*
During braking operation		100	0	0	300	5	0	*
During jack up operation		100	0	0	220	0	0	*
During regular operation		100	100	15	1000	5	25	*
During short-out of single phase		100	750	15	1000	5	25	short term
During earthquake		100	100	30	1000	5	45	
During burnt thrust		-	-	-	-	70	-	
Design load	long term	100	100	15	1000	5	25	
	Short term	100	750	30	1000	70	45	

The loads of generating equipment are basically given by Part 6 Turbine and Generator. Breakdown of the above loads are given as follows ;

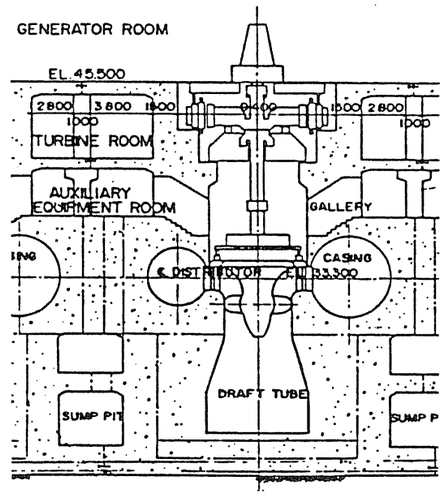
(a) Vertical loads of stator foundation

Wv = Stator + Upper bracket = 70 + 10t = 80t

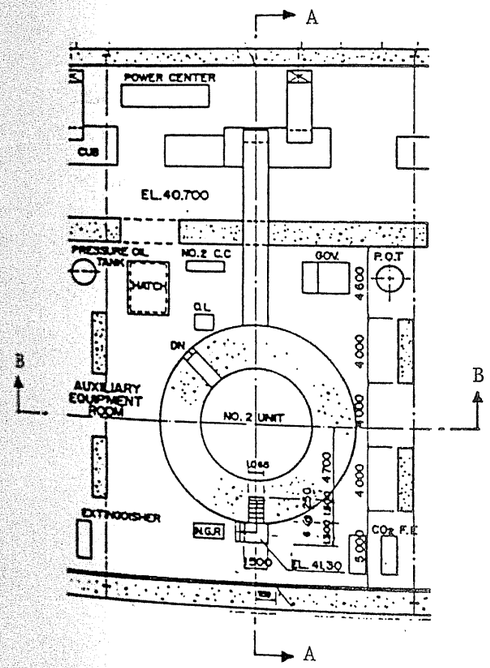
Weight of Stator consists of Stator itself, Cooler, Foundation base, Installed materials and Pull out bus etc., and the total weight was estimated to be about 70t. Taking some additional allowances 100t was adopted.



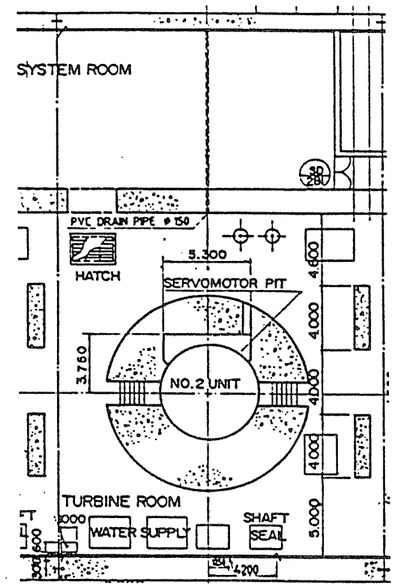
Cross Sectional View (A - A)



Longitudinal Section (B - B)



EL. 40.700 PLAN



EL. 37.10 PLAN

Fig. 1.6.8-10 Powerhouse Structure Around Barrel

(b) Horizontal tangential direction loads of stator foundation

(i) During regular operation

The horizontal tangential direction loads,

$Fu = Tn/RSB = (0.974 \times KVA \times Pf/N)/RSB = 62.1t$

where Tn : Rated torque (Tn = 0.974 x KVA xPf/N)
RSB : Center average radius of fixed base bolt is 3.0m
KVA : Generator capacity (45000 kVA)
Pf : Power factor of generator (0.85)
N : Rated speed (200 rpm)

Thus, taking account of the coefficient of loads, the horizontal tangential direction loads FuLF = Fu x 1.5 = 93.2t. Taking some additional allowances 100t was adopted for design.

(ii) During short circuit of single phase

The horizontal tangential direction load,

$FN1 = Fu \times 5.67 = 352.1t$ *(Fu : the horizontal tangential direction loads)*

Considering the load coefficient, the horizontal tangential direction load, FN = FN1 x 2 = 352.1 x 2 = 704.2 t. Taking some additional allowance 750t was adopted.

Note : During short circuit of single phase, 5.67 times load will act compared with during the rated torque. The value of 5.67 is derived from the following equation.

$Te\text{-}e = 2.6\, E^2 (X'' + Xd'') \cdot [1 + 17 (X_2 - Xd'')^2 / (X_2 + Xd'')^2] = 5.67$

where Te-e : Torque coefficient during sudden short circute of single phase.
E : Generator terminal voltage before short circute normally 1.1 P.U
X_2 : Negative phase sequence 0.276 P.U
Xd" : Initial subtransient time reactance 0.279 P.U

(c) Horizontal radius direction loads of stator foundation

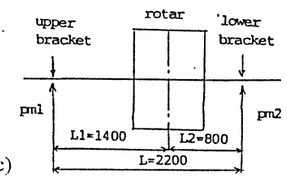
(i) During regular operation

Total horizontal radial direction load is,

$P1 = Pf + Pm = (8/3 \times \pi\, DLB^2 \Delta G \times 10^4) + (WRT/g - \Delta e.W^2) = 20.95t$

Where Pf : Radius direction load due to magnetic unbalanced force
Pm : Radial direction load due to mechanical unbalanced force
B : Space magnetic flux density in rating voltage (0.95 Tesla)
G : Equivalent space length (0.0275 m)
Δ : Distance of eccentricity for standard space length (0.00138 m)

D : Inner diameter of stator (5.8 m)
L : Effective iron core length of stator (0.88 m)
WRT : Weight of generator rotary part (118000 kg)
e : Eccentric distance of rotary body (0.03 cm)
g : Gravity acceration (980 cm/sec²)
w : Rotary angular velocity
(W = 2πN/60 = 2π x 200/60 = 20.94 rad/sec)



Horizontal radial direction load Pm1 acting on the upper bracket part is, Pm1 = P1 x L2/L = 20.95 x 800/2200 = 7.6 t, Considering the load coefficient, F1' = Pm1 x 1.5 = 11.4 t. 15 ton was adopted with some additional allowance.

(ii) During short circuit of single phase : Same as during regular operation.

(iii) During earthquake

The horizontal radial direction load PSG at the generator fixed part,

$PSG = (Wstt + Woh)\, \alpha h = 16t$

Where, Wstt + Woh : Vertical load of stator
αn : Seismic coefficient (0.2)

Considering the load coefficient PSG' = PSG x 1.5 = 24t, 30 ton was adopted with some additional allowances.

(d) Vertical load of lower bracket foundation

(i) During stop operation

Wvst = Lower bracket + Generator rotary part + Turbine rotary part
= WBT + WRT + WWT = 200 t

220 ton was adopted with some additional allowances.

(ii) During regular operation

Wvro = Lower bracket oil part + Generator rotary part + Turbine rotary part + Turbine driving force
= WBT + WRT + WWT + WTH = 992 t

1000 ton was adopted with some additional allowances.

(iii) During braking operation

Wvb = Lower bracket oil tank + Generator rotary part + Turbine rotary part
= WBT + WRT + WWT = 285 t

300 ton was adopted with some additional allowances.

(iv) During jack up operation : Same as during stop operation
(v) During earthquake : Same as during regular operation
(vi) During short circuit of single phase : Same as during regular operation

(e) Horizontal tangential direction load of lower bracket foundation

(i) During braking operation

The horizontal tangential direction load is,

$FB = TB/RB = (\mu PB.Rb.h') / RB = 1.21\ t$

where TB : Braking torque (TB = μRB.Rb.h')
RB : Installed center radius of lower bracket foundation 2.9m
μ : Coefficient of braking friction (0.45)
PB : Damper pushing up force (1700 kg)
Rb : Installed damper radius (2.3 m)
h' : Number of damper (2)

Then, considering the load coefficient, FB = FB x 1.5 = 1.82 t, 5 ton was adopted with some additional allowances.

(ii) During burnt thrust bearing

Horizontal tangential load acting on the lower bracket,

$FSB = TSB/RB = (\mu SB\ .\ WSB)/RB = 33.4t$

where, TSB : Torque during burned thrust bearing
μ SB : Coefficient of friction during burnt thrust bearing 0.15
WSB : Thrust bearing load during local loading (Generator rotary part + Turbine rotary part + Water thrust = 645 t)

Considering the load coefficient FSB1' = FSB x 1.5 = 50.1 t, 70 ton was adopted with some additional allowances.

(iii) During regular operation

The horizontal tangential direction force acting on the lower bracket

$FSB2 = TSB'.RB = (TSB\ .\ \mu SB' / \mu SB) / RB = 1.1\ t$

Where, TSB : Torque during burned thrust bearing
TSB' : Torque for burned thrust bearing during braking operation
μSB' : Coefficient of friction of thrust bearing during regular operation = 0.005
μSB : Coefficient of friction during burnt thrust bearing 0.15

Considering the load coefficient FSB2' = FSB2 x 1.5 = 1.7 t, 5 ton was adopted with some additional allowances.

(f) Horizontal radial direction load of lower bracket foundation

(i) During regular operation

The total horizontal radial direction P1 = 20.95t is given in the proceeding explanation c) (i). The horizontal radial direction load acting on the lower bracket Pm2 = P1 x L1/L = 20.95 x 1400/2200 = 13.3 t Considering the load coefficient, F2' = Pm2 x 1.5 = 20.0t, 25 ton was adopted with some additional allowances.

(ii) During earthquake

The horizontal radial direction load of generator rotary part,

$PRG = (WRT + WBT)\ x\ dh = 26\ ton$

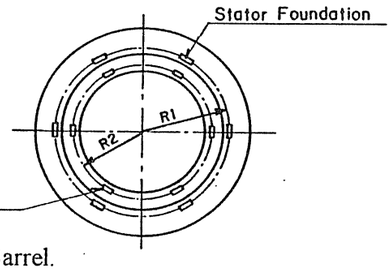
where, WRT : Generator rotary part (100 ton)
WBL : Lower bracket part (30 ton)
dh : Lateral force during earthquake (0.2)

The horizontal radial direction load acting on the lower bracket is, therefore PRG = PRG x 1.5 = 39 ton, 45 ton was adopted with some additional allowances.

(iii) During short circuit of single phase : Same as during regular operation

The number and the size of stator foundation and lower bracket foundation are as follows.

- Stator foundation : 6 pieces (300x700mmx6)
- Lower bracket : 6 pieces (300x500mmx6)
- Radius of loading point : R_1 = 3.70m, R_2 = 2.70m



(2) Calculation for designed torque moment acting on Barrel.

The calculation is done for during short-circuit of single phase.

(a) In case of considering for shock

$Ti = H_k \times R_1 - H_b \times R_2 = 2788\ tm$

Where, T_i : Torque moment of shock during short-cut of single phase.
H_k : Horizontal tangential direction loads of the stator = 750 T
H_b : Horizontal tangential direction loads of the stator = 5 T
$R_{1,2}$: Radius of loading point (R_1 = 3.7 m, R_2 = 2.6 m)

(b) In case of considering for resonance vibration of Barrel.

(i) Free frequency of vibration for Barrel.

$$f = \frac{1}{4\ell}\sqrt{G.\frac{\sigma}{\gamma}} \quad , \quad \ell = \frac{Ro^4}{6}\,\frac{-Ri^4}{Ro^3} + L + \frac{N}{1.5} \quad , \quad G = \frac{m}{2\,(m+1)}\,.E$$

where G : Shearing modulas of elasticity of concrete (t/m³)
g : Acceleration of gravity (9.8 m/s²)
γ : Unit weight of concrete (2.4 t/m³)
Ro : The outer diameter of Barrel (6.2 m)
Ri : The inner diameter of Barrel (3.24 m)
L : Height from floor of Barrel (5.40 m)
N : Height of stator (2.30 m)
m : Poisons ratio (6)
E : Young's modulus (2.40 x 10^6 t/m² for σck = 180 kg/cm²)

Thus,

G = 1.03 x 10^6 t/m², ℓ = 7.89 m, f = 65 Hz

Compulsory vibration frequency caused by accident or so is assumed w = 50 Hz, the dangerous ranges normally are,

$$0.8 < \frac{f}{w} < 1.4 \quad , \qquad 40 < f < 70$$

Therefore, this barrel is slightly within the resonance but it does no coincide with the resonance vibration point.

(ii) Torque moment by resonance vibration

Terminated magnification of by compulsory vibration is given as following expression.

$$\alpha = \frac{1}{\sqrt{\{1-(\frac{w}{f})^2\}^2 + (\frac{2\varepsilon w}{f})^2}} = 2.29$$

where ε/f : Coefficient of decrease for Barrel = 0.1
f : Free vibration frequency of Barrel = 65 Hz
w : Compulsory vibration frequency = 50 Hz

Therefore, the moment of torque Tr caused resonance vibration during short-cut of single phase is,

$T_r = T_n \times \alpha = 3650\ tm$

where Tn : Standard torque (Ti/k = 1594 tm)
Ti : Torque moment of shock = 3188 tm
K : Coefficient of impact load = 2

(iii) Torque moment applied for design

As mentioned above, the torque moment to be applied for design should be the larger value of T_i or T_r, therefore To = 3650 tm is adopted during short-cut of simple phase.

(3) Study for stress of Barrel by common formula

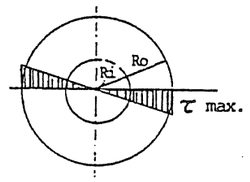
(a) The stress by horizontal tangential direction loads moment of torque.

(i) In the case for a perfect hollow cylinder

The torque at normal operation is smaller than at short-circuit single phase therefore the largest torsion moment of M (T_o = 3192 tm) was studied.

The shearing stress at position r is given as the following expression.

$$\tau = \frac{2M}{\pi(Ro^4 - Ri^4)}.r$$



where, Ri : Internal radius = 3.24 m
Ro : External radius = 6.20 m
M : Moment of torque = 3650 tm

The shearing stress reaches the greatest value at the outside. Thus, τ max = 10.5 t/m²
Therefore, total of vertical force W = (Barrel and wind tunnel + slab + Vertical load of generator and the like) = 3256t

Vertical compressive stress intensity of concrete σc is

$$\sigma c = \frac{w}{Ac} = 39.0\ t/m^2$$

where W : Vertical force (3256 t)
Ac : Section area of barrel (82.89 m²)

Thus, the composed stress intensity

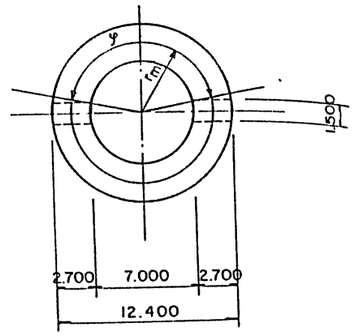
$$\tau\ max = \sqrt{\frac{\sigma^2 c}{4} + \tau max^2} = 2.23\ kg/cm^2 < \tau ca = 12.0\ kg/cm^2\ (\tau a)$$

$$\sigma max = \frac{\sigma c}{2} + \tau max = 4.20\ kg/cm^2 < \sigma ca = 90\ kg/cm^2$$

Where, σc : 3.9 kg/m²
τmax : 0.95 kg/m²

(ii) In the case for fan shape section

There are two inspection corridors which are 1.5m wide and 2.5m height from EL 37.10 till EL 39.60. The barrel base changes to be two fan shape in calculation model as shown the figure right, and half the torque moment is shared at each part, the largest shearing stress intensity is given as the following expression (Mechanical Engineering Handbook, Mr. Tsumur's formula).



$$\tau max = \frac{\left\{1.336\left(\frac{t}{Ro}\right)^3 - 0.586\left(\frac{t}{Ro}\right)^2 + 1.121\left(\frac{t}{Ro}\right)\right\}}{\left\{\left(0.329 + 0.0266\frac{t}{Ro}\right)A - \left(0.197 + 0.060\frac{t}{Ro}\right)t^2\right\}} \cdot \frac{Ro}{t^2} \cdot \frac{M}{2} = 6.18\ kg/cm^2 < \tau ca = 12.0\ kg/cm^2$$

where
- Ri : Internal radius (3.24 m)
- Ro : External radius (6.20 m)
- t : Thickness (Ro - Ri = 2.96 m)
- r_m : Average radius {(Ro+Ri)/2 = 4.72 m}
- φ : Open angles for fan shape $(\pi.r_m\text{-}1.50)/r_m = 2.824$ rad
- A : Section area $(\varphi/2.(R0^2\text{-}Ri^2) = 39.45\ m^2$

$\frac{t}{Ro} = 0.4774$, $(\frac{t}{Ro})^2 = 0.2279$, $(\frac{t}{Ro})^3 = 0.1088$

Accordingly, for the barrel changes fan shape section, the largest shearing stress intensity of concrete is less than allowable stress intensity. then the barrel is safe against torque moment.

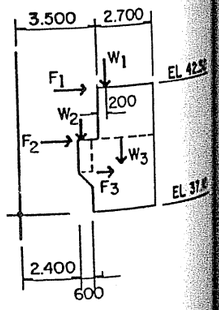
(b) Stress by horizontal radius direction loads

The barrel is considered a hollow cylinder of fixing EL 37.10.

The calculation is made in the case of seismic load

W1 = 100 t (stator foundation)
W2 = 1,000 t (bracket)
W3 = 1182.71 t (Weight of barrel concrete)

F1 = 30 t (stator)
F2 = 45 t (bracket)



F3 = W3 x Kh = 118.87 t (kh = 0.1), Y3 = M i (tm), / W3 = 2.630m

Moment at EL 37.10 is Σ M = 1122.63 tm, ΣW = W1 + W2 + W3 = 2389.1t, then $x = \Sigma M/W = 0.470m$

The core of the hollow cylinder at elevation of EL 37.10 is,

$$e = \frac{D^2 + d^2}{8D} = 1.187\ m > \quad x = 0.470\ m$$

where D : Diameter of external perimeter (12.40 m)
d : Diameter of internal perimeter (6.00 m)

Therefore acting point of resultant stress is within a core, tensile stress does not arise at EL 37.10. Accordingly, it is safe against horizontal direction loads.

(4) Calculation for reinforcement steel bars and quantities.

Each quantity of reinforcement steel bars were calculated by dividing into of general part and around opening area at barrel.

(a) Calculation for general part

The barrel is assumed a long thick cylinder, following "Shizuo Saka's" common formula.

$$A_{sj} = \frac{Rj}{Ro} \cdot A_{sa}, \qquad A_{sa} = \frac{M.P}{2\sigma_{sa}\left\{F_a + \left(\frac{R_j}{R_a}\right) F_j\right\}}$$

Where,
- Fa : Half area of fan shape surrounded by external reinforcement steel bars = ½ Ra^2 φ
- Fj : Half area of fan shape surrounded by internal reinforcement steel bars = ½ Rj φ
- Ra : Radius for external reinforcement steel bars position of barrel.
- Rj : Radius for internal reinforcement steel bars position of barrel.
- Asa : Required area for a piece of external reinforcement steel bar.
- Asj : Required area for a piece of internal reinforcement steel bar.
- P : An internal of reinforcement steel bar (m) (0.20 m)
- M : The largest torsion moment (3650/2 = 1825 tm)
- φ : Open angles for fan shape section (2.824 rad)
- sa : Allowable tension stress intensity of reinforcement steel bar (2700 kg/cm^2 = 2.7 t/cm^2)

The concrete cover to the reinforcement steel bars shall taken 10 cm both internal and external, and the bar spacing shall be taken at 20 cm. This 20 cm was determined by referring to the concrete standard specification of Japan and also taking account of the structure's importance since in the case the spacing of main reinforcing bars is taken two large it will not act as one body structure.

$Rj = 3.10\ m$, $Ra = 6.10\ m$, $Fa = 52.54\ m^2$, $Fj = 13.57\ m^2$

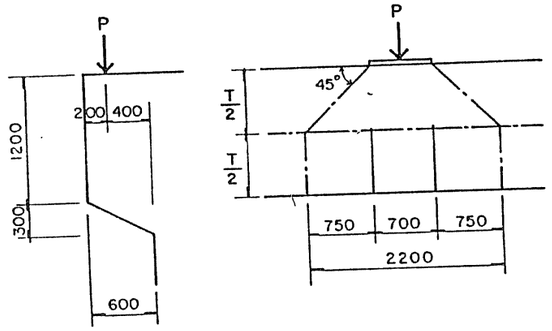
Thus, $A_{sa} = 1.137\ cm^2$, $A_{sj} = \frac{Rj}{Ra} \cdot A_{sa} = 0.58\ cm^2$

Therefore, reinforcement bars shall be arranged basically D22 @ 200 (As = 3.87 cm^2/piece) for both internal and external faces. The horizontal direction for reinforcement bars shall be adapted the same as vertical ones, since these reinforcement bar are resisting against torsion moment.

(5) Design for lower bracket foundation.

(a) Design load and foundation shape

Vertical load P = 1051 t (long period = 1000t, short period = 1051t)
Vertical load per one P = 1051/6 = 175 t



(b) Stress

Calculation was made by cantilever ℓ = 0.60m. Own weight W = 9.216 t/m^3

Calculation for stress reinforcement concrete is as follows

M = 54.16 (tm)	b = 240.0 (cm)	d = 10.0 (cm)
N = 0.000 (t)	t = 150.0 (cm)	d' = 10.0 (cm)
Q = 1180.53 (t)		

Reinforcement : D 29 t 200 = 70.62 (cm^2)

Compressive stress	σC = 11.2 (kg/cm^2)	< σCa = 60	(kg/cm^2)
Tensile stress	σS = 591.5 (kg/cm^2)	< σSa = 2000	(kg/cm^2)
Shear stress	τC = 5.80 (kg/cm^2)	< τCa = 8.0	(kg/cm^2)
Bolt stress	τO = 13.4 (kg/cm^2)	< τOa = 14.0	(kg/cm^2)

6.8.6 Design Around the Casing

(1) Design internal pressure

The internal pressure acting on the concrete around the steel pipe casing is considered internal share of casing steel pipe according to mentioned below's calculation. The steel pipe casing and concrete around the casing are modeled as a thin plate cylinder and a thick concrete cylinder respectively. The total deformation of the concrete due to internal water pressure shall be considered to equal the deformation of the steel pipe, then the share of internal water pressure is decided.

The section of the casing is assumed a real circle, water pressure P2 shared by the casing is given as following expression.

$$P2 = \frac{\Sigma\Delta \cdot t \cdot Es}{r^2}$$

Where ΣΔ : Clearance for around the casing (0.03 cm)
t : The thickness of casing steel pipe (2.0 cm)
Es : The modulus of elasticity of steel (2.1×10^6 kg/cm^2)
r : Internal diameter of casing steel (cm)

The static water pressure and water hammer are given as follows.

P = hd - hr = 53.10 m (static water pressure)
P' = 19.00 m (this value of water hammer is given by Part 5 Metal Works)

Where, hd : Design water level (HWL + wave height by wind = EL 86.40)
hr : Riverbed water level (EL 33.30)

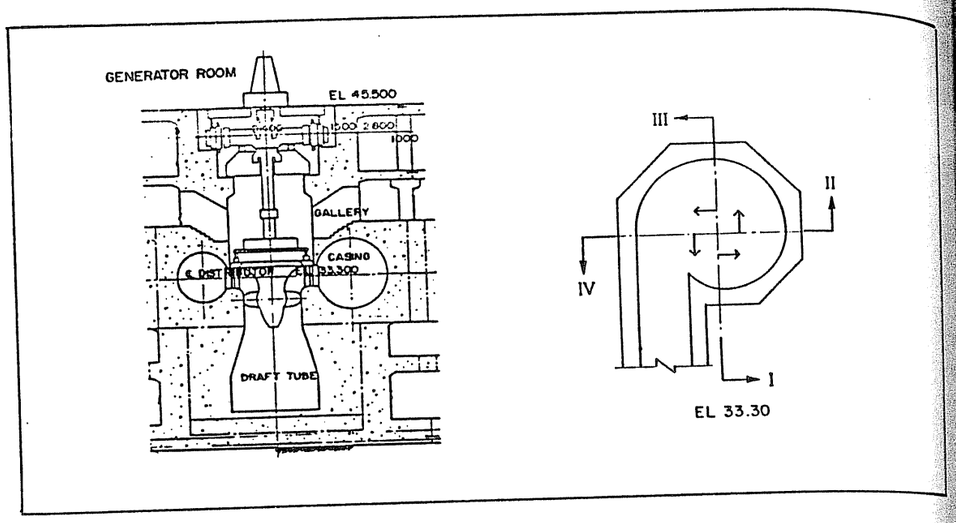
The calculations are made at four sections from I to IV shown on the following table and figure.

The share of internal pressure at each section

	Int.dia.of concrete r (cm)	Ext.dia.of concrete R (cm)	Thickness of casing t (cm)	Initial clearance ΣΔ (cm)	Int.pressure shared by casing P2 (t/m^2)	Static water pressure P = 53.10 t/m P1 (t/m^2)	Static water pressure P = 53.10 t/m P2 (t/m^2)	Including water hammer P = 72.10 t/m P1 (t/m^2)	Including water hammer P = 72.10 t/m P2 (t/m^2)
I	125	275	2.0	0.03	80.6	---	53.10	---	72.10
II	175	325	"	"	41.1	12.0	41.10	31.00	41.10
III	220	370	"	"	26.0	27.10	26.00	46.10	26.00
IV	250	400	"	"	20.2	32.90	20.20	51.90	20.20

P1 : Internal pressure shared by concrete
P2 : External pressure shared by steel pipe casing

For section I - I, from the equation given above, the pressure shared by casing P2 = 80.6 t/m^2. As the static water pressure is 53.1 t/m^2 < 80.6 t/m^2 and 72.1 t/m^2 < 80.6 t/m^2, the pressure shared by concrete P1 at section I-I is not taken into account. However, for section II-II, III-III, and IV-IV, the pressure P1 is respectively 12 t/m^2, 27.1 t/m^2, 32.9 t/m^2 and 31.0 t/m^2, 46.1 t/m^2, 51.9 t/m^2 when including the pressure of water hammer. Therefore the largest internal pressure of P1 = 51.9 t/m^2 ≒ 52 t/m^2 was taken as the design internal pressure.



(2) The stress due to the internal pressure.

A study of stress due to the internal pressure was made assuming the concrete around the casing being a thick plate cylinder. For the stress due to the differential of temperature, the temperature difference between outer side and inner side of cylinder is very small and neglected as it will be safety design.

The hoop stress σ_{op} occurring within the thick plate cylinder caused by the internal pressure P1 is given as follows.

$$\sigma_{op} = P_1 \frac{a^2}{b^2 - a^2} \left(1 + \frac{b^2}{r^2}\right)$$

Where, a : Internal diameter of cylinder (1.25 m)
b : External diameter of cylinder (2.75 m)
r : The distance up to calculation point of stress
P_1 : Internal water pressure (52 t/m² = 5.2 kg/cm²)

Stress due to internal pressure

Section	r (m)	Δr (m)	$\frac{b}{r}$	$1+\frac{b^2}{r^2}$	σ_{op} (kg/cm²)
I-I	1.25	0	2.200	5.840	7.9
	1.55	0.30	1.774	4.147	5.6
	1.25	0.60	1.486	3.208	4.3
	2.15	0.90	1.279	2.636	3.6
	2.45	1.20	1.122	2.259	3.1
	2.75	1.50	1.0	2.0	2.7
II-II	1.75	0	1.857	4.448	9.4
	2.05	0.30	1.585	3.512	7.5
	2.35	0.60	1.383	2.913	6.2
	2.65	0.90	1.226	2.503	5.3
	2.95	1.20	1.102	2.214	4.7
	3.25	1.50	1.0	2.0	4.2
III-III	2.20	0	1.682	3.829	10.9
	2.50	0.30	1.480	3.190	9.1
	2.80	0.60	1.321	2.745	7.8
	3.10	0.90	1.194	2.426	6.9
	3.40	1.20	1.088	2.184	6.2
	3.70	1.50	1.0	2.0	5.7
IV-IV	2.50	0	1.600	3.560	11.9
	2.80	0.30	1.429	3.042	10.1
	3.10	0.60	1.290	2.664	8.9
	3.40	0.90	1.176	2.383	7.9
	3.70	1.20	1.081	2.169	7.2
	4.00	1.50	1.0	2.0	6.7

Calculation for volume of steel bars

Since there is no bond stress exceeding σ_a = 14 kg/cm² between the casing and the concrete shown in the previous tables, no reinforcement bars against stress are required. But for design safety the reinforcement steel bars arrangement D 22 @ 250 cm with A_s = 15.5cm² are provided.

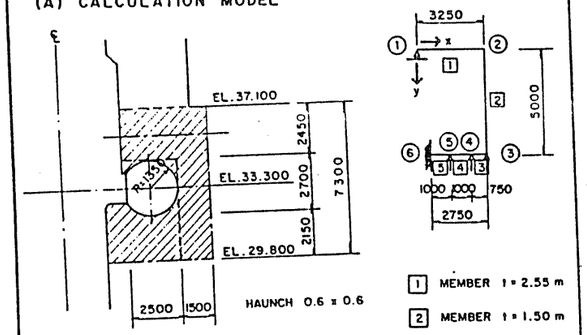
(3) Study of stress due to vertical loads and internal pressures

The stress due to the vertical loads (concrete own weight and other loads acting on the around of casing structure) and the internal pressures obtained in the preceding section (2) are applied to a framed structure analysis. The frame analysis is done for each section I-I, II-II, III-III and IV-IV. The analysis models and design loads are shown in the Fig. 1.6.8-11 .

Fig. 1.6.8-11 Calculation Model of Around Casing

Section I-I

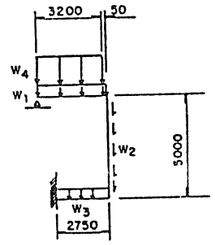
(A) CALCULATION MODEL



1 MEMBER t = 2.55 m
2 MEMBER t = 1.50 m
3~5 MEMBER t = 2.15 m

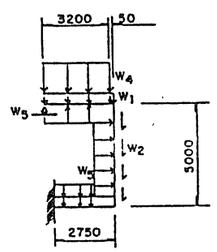
(B) DESIGN LOAD

VERTICAL



W_1 = 5.88 t/m²
W_2 = 3.60 t/m²
W_3 = 5.16 t/m²
W_4 = 39.0 t/m²

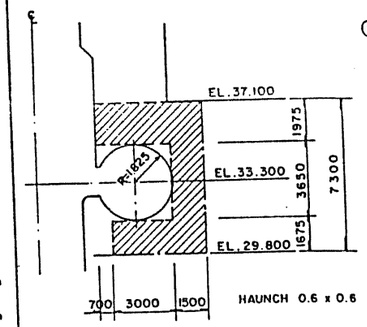
INNER PRESSURE

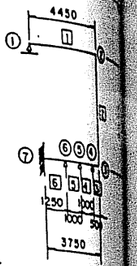


W_5 = 22.38 t/m²

Section II-II

(A) CALCULATION MODEL

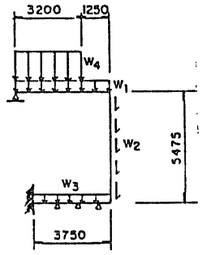




1 MEMBER
2 MEMBER
3~8 MEMBER

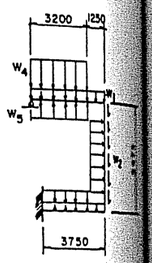
(B) DESIGN LOAD

VERTICAL



W_1 = 4.74 t/m²
W_2 = 3.60 t/m²
W_3 = 4.02 t/m²
W_4 = 39.0 t/m²

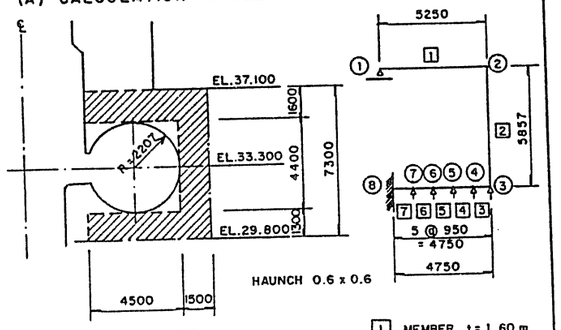
INNER PRESSURE



W_5 = 22.8 t/m²

Section III-III

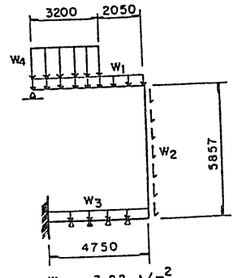
(A) CALCULATION MODEL



1 MEMBER t = 1.60 m
2 MEMBER t = 1.50 m
3~7 MEMBER t = 1.30 m

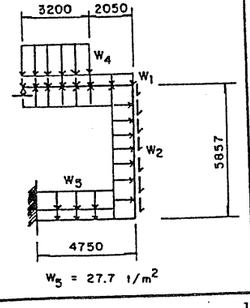
(B) DESIGN LOAD

VERTICAL



W_1 = 3.82 t/m²
W_2 = 3.60 t/m²
W_3 = 3.10 t/m²
W_4 = 39.0 t/m²

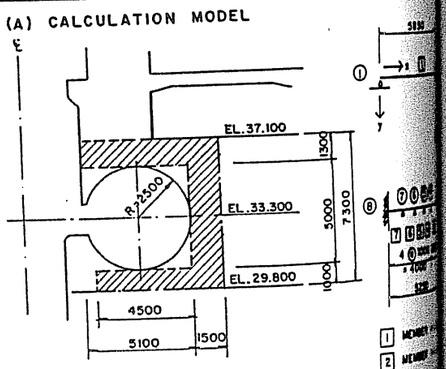
INNER PRESSURE



W_5 = 27.7 t/m²

Section IV-IV

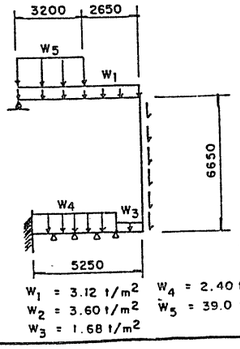
(A) CALCULATION MODEL



1 MEMBER
2 MEMBER
3 MEMBER
4~7 MEMBER

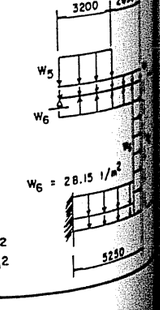
(B) DESIGN LOAD

VERTICAL



W_1 = 3.12 t/m²
W_2 = 3.60 t/m²
W_3 = 1.68 t/m²
W_4 = 2.40 t/m²
W_5 = 39.0 t/m²

INNER PRESSURE



W_6 = 28.15 t/m²

Table 1.6.8-3(1) Calculation of Reinforce Concrete for Around Barrel

Section I-I

部材 No	L(m)	Case 1 CASING3/VERTICAL M (tm)	S (t)	N (t)
1- 2	0.000	0.000	89.228	-9.354
* 1	1.313	80.244	33.002	-9.354
* 2	2.625	86.685	-23.182	-9.354
* 3	3.938	29.955	-50.627	-9.354
2- 1	5.250	-39.758	-55.643	-9.354
2- 3	0.000	-39.758	9.354	-55.643
* 1	1.464	-26.063	9.354	-60.913
* 2	2.929	-12.359	9.354	-66.187
* 3	4.393	1.336	9.354	-71.457
3- 2	5.857	15.030	9.354	-76.728
3- 4	0.000	15.030	-21.223	9.354
4- 3	0.950	-3.732	-18.275	9.354
4- 5	0.000	-3.732	3.818	9.354
5- 4	0.950	1.296	6.766	9.354
5- 6	0.000	1.296	-2.894	9.354
6- 5	0.950	-0.053	0.054	9.354
6- 7	0.000	-0.053	-1.087	9.354
7- 6	0.950	0.315	1.861	9.354
7- 8	0.000	0.315	-1.603	9.354
8- 7	0.950	0.192	1.345	9.354
2 Max	2.084	92.960	0.000	-9.354
Min	5.250	-39.758	-55.643	-9.354
3 Max	5.857	15.030	9.354	-76.728
Min	0.000	-39.758	9.354	-55.643
4 Max	0.000	15.030	-21.223	9.354
Min	0.950	-3.732	-18.275	9.354
5 Max	0.950	1.296	6.766	9.354
Min	0.000	-3.732	3.818	9.354
6 Max	0.000	1.296	-2.894	9.354
Min	0.933	-0.053	0.000	9.354
7 Max	0.950	0.315	1.861	9.354
Min	0.350	-0.243	0.000	9.354
8 Max	0.000	0.315	-1.603	9.354
Min	0.517	-0.099	0.000	9.354

部材 No	L(m)	Case 2 CASING3/INNER M (tm)	S (t)	N (t)
1- 2	0.000	0.000	35.096	72.188
* 1	1.313	32.071	13.756	72.188
* 2	2.625	36.130	-7.568	72.188
* 3	3.938	22.804	-0.126	72.188
2- 1	5.250	42.215	29.718	72.188
2- 3	0.000	42.215	-72.188	29.718
* 1	1.464	-34.994	-33.289	24.447
* 2	2.929	-55.249	5.636	19.173
* 3	4.393	-18.525	44.534	13.903
3- 2	5.857	75.148	83.433	8.632
3- 4	0.000	75.148	-111.415	83.433
4- 3	0.950	-17.307	-83.225	83.433
4- 5	0.000	-17.307	11.985	83.433
5- 4	0.950	7.469	40.174	83.433
5- 6	0.000	7.469	-21.092	83.433
6- 5	0.950	0.822	7.098	83.433
6- 7	0.000	0.822	-12.186	83.433
7- 6	0.950	2.635	16.003	83.433
7- 8	0.000	2.635	-14.731	83.433
8- 7	0.950	2.030	13.459	83.433
2 Max	5.250	42.215	29.718	72.188
Min	0.000	0.000	35.096	72.188
3 Max	5.857	75.148	83.433	8.632
Min	2.717	-55.847	0.000	19.937
4 Max	0.000	75.148	-111.415	83.433
Min	0.950	-17.307	-83.225	83.433
5 Max	0.950	7.469	40.174	83.433
Min	0.000	-17.307	11.985	83.433
6 Max	0.000	7.469	-21.092	83.433
Min	0.711	-0.027	0.000	83.433
7 Max	0.950	2.635	16.003	83.433
Min	0.411	-1.681	0.000	83.433
8 Max	0.000	2.635	-14.731	83.433
Min	0.496	-1.022	0.000	83.433

Section II-II

部材 No	L(m)	Case 1 CASING4/ M (tm)	S (t)	N (t)
1- 2	0.000	0.000	92.788	-0.222
* 1	1.463	90.673	31.167	-0.222
* 2	2.925	91.224	-30.413	-0.222
* 3	4.388	29.175	-45.702	-0.222
2- 1	5.850	-40.977	-50.264	-0.222
2- 3	0.000	-40.977	0.222	-50.264
* 1	1.500	-40.644	0.222	-55.664
* 2	3.000	-40.311	0.222	-61.064
* 3	4.500	-39.978	0.222	-66.464
3- 2	6.000	-39.645	0.222	-71.864
3- 4	0.000	-39.645	71.864	0.222
4- 3	1.250	51.498	73.964	0.222
4- 5	0.000	51.498	-66.248	0.222
5- 4	1.000	-13.550	-63.848	0.222
5- 6	0.000	-13.550	16.252	0.222
6- 5	1.000	3.902	18.652	0.222
6- 7	0.000	3.902	-5.960	0.222
7- 6	1.000	-0.858	-3.560	0.222
7- 8	0.000	-0.858	0.387	0.222
8- 7	1.000	0.729	2.787	0.222
2 Max	2.203	102.204	0.000	-0.222
Min	5.850	-40.977	-50.264	-0.222
3 Max	6.000	-39.645	0.222	-71.864
Min	0.000	-40.977	0.222	-50.264
4 Max	1.250	51.498	73.964	0.222
Min	0.000	-39.645	71.864	0.222
5 Max	1.000	51.498	-66.248	0.222
Min	0.000	-13.550	-63.848	0.222
6 Max	1.000	3.902	18.652	0.222
Min	0.000	-13.550	16.252	0.222
7 Max	1.000	3.902	-5.960	0.222
Min	0.000	-0.858	-3.560	0.222
8 Max	1.000	0.729	2.787	0.222
Min	0.000	-0.858	0.387	0.222

部材 No	L(m)	Case 2 CASING4/INNER M (tm)	S (t)	N (t)
1- 2	0.000	0.000	27.956	91.159
* 1	1.463	25.949	7.518	91.159
* 2	2.925	22.011	-12.906	91.159
* 3	4.388	15.700	12.988	91.159
2- 1	5.850	61.438	49.582	91.159
2- 3	0.000	61.438	-91.159	49.582
* 1	1.500	-43.633	-48.934	44.182
* 2	3.000	-85.365	-6.709	38.782
* 3	4.500	-63.761	35.516	33.382
3- 2	6.000	21.181	77.741	27.982
3- 4	0.000	21.181	-27.982	77.741
4- 3	1.250	9.509	9.306	77.741
4- 5	0.000	9.509	-24.105	77.741
5- 4	1.000	0.679	6.445	77.741
5- 6	0.000	0.679	-12.906	77.741
6- 5	1.000	3.048	17.644	77.741
6- 7	0.000	3.048	-15.921	77.741
7- 6	1.000	2.402	14.629	77.741
7- 8	0.000	2.402	-15.060	77.741
8- 7	1.000	2.618	15.490	77.741
2 Max	5.850	61.438	49.582	91.159
Min	0.000	0.000	27.956	91.159
3 Max	0.000	61.438	-91.159	49.582
Min	3.238	-86.165	0.000	37.924
4 Max	0.000	21.181	-27.982	77.741
Min	0.938	8.057	0.000	77.741
5 Max	0.000	9.509	-24.105	77.741
Min	0.789	-0.001	0.000	77.741
6 Max	1.000	3.048	17.644	77.741
Min	0.422	-2.047	0.000	77.741
7 Max	0.000	3.048	-15.921	77.741
Min	0.521	-1.100	0.000	77.741
8 Max	1.000	2.618	15.490	77.741
Min	0.493	-1.310	0.000	77.741

Table 1.6.8-3(2) Calculation of Reinforce Concrete for Around Barrel

Section III-III

Case 1 CASING1/VERTICAL

No	L(m)	M (tm)	S (t)	N (t)
1-# 2	0.000	0.000	74.455	1.454
* 1	0.813	45.700	37.967	1.454
* 2	1.625	61.733	1.525	1.454
* 3	2.500	45.887	-37.745	1.454
2-# 1	3.250	5.004	-69.455	1.454
2-# 3	0.000	5.004	-1.454	-69.455
* 1	1.225	3.224	-1.454	-73.865
* 2	2.500	1.370	-1.454	-78.455
* 3	3.925	-0.701	-1.454	-83.585
3-# 2	5.000	-2.264	-1.454	-87.455
3-# 4	0.000	-2.264	2.320	-1.454
4-# 3	0.750	0.927	6.190	-1.454
4-# 5	0.000	0.927	-3.219	-1.454
5-# 4	1.000	0.288	1.941	-1.454
5-# 6	0.000	0.288	-2.367	-1.454
6-# 5	1.000	0.501	2.793	-1.454

lo		L(m)	M (tm)	S (t)	N (t)
1-2	Max	1.659	61.759	0.000	1.454
	Min	0.000	0.000	74.455	1.454
2-3	Max	0.000	5.004	-1.454	-69.455
	Min	5.000	-2.264	-1.454	-87.455
3-4	Max	0.750	0.927	6.190	-1.454
	Min	0.000	-2.264	2.320	-1.454
4-5	Max	0.000	0.927	-3.219	-1.454
	Min	0.624	-0.077	0.000	-1.454
5-6	Max	1.000	0.501	2.793	-1.454
	Min	0.459	-0.255	0.000	-1.454

Case 2 CASING1/INTERNAL

No	L(m)	M (tm)	S (t)
1-# 2	0.000	0.000	49.200
* 1	0.813	32.564	30.908
* 2	1.625	50.243	12.638
* 3	2.500	52.688	-7.050
2-# 1	3.250	41.121	-21.975
2-# 3	0.000	41.121	-54.719
* 1	1.225	-9.118	-27.304
* 2	2.500	-25.739	1.231
* 3	3.925	-1.262	33.122
3-# 2	5.000	47.276	57.181
3-# 4	0.000	47.276	-85.233
4-# 3	0.750	-8.903	-64.578
4-# 5	0.000	-8.903	0.628
5-# 4	1.000	5.495	28.168
5-# 6	0.000	5.495	-18.569
6-# 5	1.000	0.695	8.971

lo		L(m)	M (tm)	S (t)
1-2	Max	2.187	53.792	0.000
	Min	0.000	0.000	49.200
2-3	Max	5.000	47.276	57.181
	Min	2.445	-25.773	0.000
3-4	Max	0.000	47.276	-85.233
	Min	0.750	-8.903	-64.578
4-5	Max	1.000	5.495	28.168
	Min	0.000	-8.903	0.628
5-6	Max	0.000	5.495	-18.569
	Min	0.674	-0.766	0.000

Sect

Case 1 CASING/VERTICAL

No	L(m)	M (tm)	S (t)	N (t)
1-# 2	0.000	0.000	87.110	-2.566
* 1	0.000	0.000	87.110	-2.566
* 2	1.200	73.040	34.622	-2.566
* 3	2.225	85.550	-10.211	-2.566
* 4	3.700	27.783	-55.228	-2.566
2-# 1	4.450	-14.971	-58.783	-2.566
2-# 3	0.000	-14.971	2.566	-58.783
*. 1	0.750	-13.047	2.566	-61.483
* 2	1.500	-11.122	2.566	-64.183
* 3	2.738	-7.946	2.566	-68.639
* 4	4.725	-2.848	2.566	-75.793
3-# 2	5.475	-0.924	2.566	-78.493
3-# 4	0.000	-0.924	78.493	2.566
4-# 3	0.500	38.825	80.503	2.566
4-# 5	0.000	38.825	-50.810	2.566
* 1	0.000	38.825	-50.810	2.566
* 2	0.250	26.248	-49.805	2.566
5-# 4	1.000	-9.975	-46.790	2.566
5-# 6	0.000	-9.975	11.052	2.566
6-# 5	1.000	3.087	15.072	2.566
6-# 7	0.000	3.087	-5.589	2.566
7-# 6	1.250	-0.758	-0.564	2.566

2	Max	1.992	86.742	0.000	-2.566
	Min	4.450	-14.971	-58.783	-2.566
3	Max	5.475	-0.924	2.566	-78.493
	Min	0.000	-14.971	2.566	-58.783
4	Max	0.500	38.825	80.503	2.566
	Min	0.000	-0.924	78.493	2.566
5	Max	0.000	38.825	-50.810	2.566
	Min	1.000	-9.975	-46.790	2.566
6	Max	1.000	3.087	15.072	2.566
	Min	0.000	-9.975	11.052	2.566
7	Max	0.000	3.087	-5.589	2.566
	Min	1.250	-0.758	-0.564	2.566

No	L(m)	M (tm)	
1- 2	0.000	0.000	44.355
* 1	0.000	0.000	44.355
* 2	1.200	39.921	22.179
* 3	2.225	52.946	3.237
* 4	3.700	42.493	-4.521
2- 1	4.450	44.874	10.869
2- 3	0.000	44.874	-67.488
* 1	0.750	1.363	-48.543
* 2	1.500	-27.940	-29.598
* 3	2.738	-45.224	1.674
* 4	4.725	7.968	51.866
3- 2	5.475	53.971	70.811
3- 4	0.000	53.971	8.841
4- 3	0.500	62.052	23.481
4- 5	0.000	62.052	-90.360
* 1	0.000	62.052	-90.360
* 2	0.250	40.377	-83.040
5- 4	1.000	-13.668	-61.080
5- 6	0.000	-13.668	6.289
6- 5	1.000	7.261	35.569
6- 7	0.000	7.261	-22.438
7- 6	1.250	2.088	14.162

2	Max	2.400	53.230	0.000
	Min	0.000	0.000	44.355
3	Max	5.475	53.971	70.811
	Min	2.672	-45.280	0.000
4	Max	0.500	62.052	23.481
	Min	0.000	53.971	8.841
5	Max	0.000	62.052	-90.360
	Min	1.000	-13.668	-61.080
6	Max	1.000	7.261	35.569
	Min	0.000	-13.668	6.289
7	Max	0.000	7.261	-22.438
	Min	0.766	-1.337	0.000

(4) Calculation result

The required quantities of reinforcement steel bars at each section are shown below :

Section	Load type	Member 1 (Inside)	Member 1 (Outside)	Member 2 (Inside)
I-I	Vertical load	D22 @200 = 19.4	-	-
	Internal water pressure	D22 @200 = 19.4	-	
	Determination	D22 @200 = 19.4	D22 @200 = 19.4	D16 @200 = 9.93

Section	Load type	Member 2 (Outside)	Member 3 to 5 (Inside)	Member 3 to 5 (Outside)
I-I	Vertical load	-	-	-
	Internal water pressure	D16 @200 = 9.9	D22 @200 = 19.4	D22 @200 = 19.4
	Determination	D16 @200 = 9.93	D22 @200 = 19.4	D22 @200 = 19.4

Section	Load type	Member 1 (Inside)	Member 1 (Outside)	Member 2 (Inside)
II-II	Vertical load	D25 @200 = 25.3	D22 @200 = 19.4	-
	Internal water pressure	D25 @200 = 25.3	-	D19 @200 = 14.3
	Determination	D25 @200 = 25.3	D22 @200 = 19.4	D19 @200 = 14.3

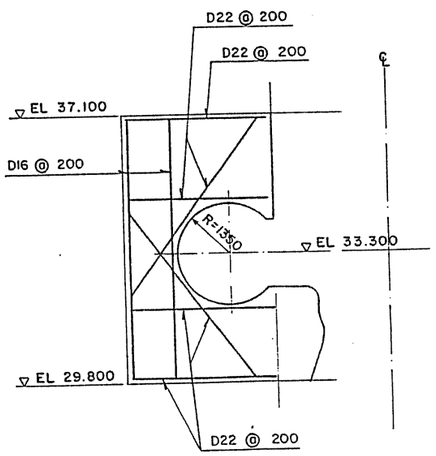
Section	Load type	Member 2 (Outside)	Member 3 to 6 (Inside)	Member 3 to 5 (Outside)
II-II	Vertical load	-	D29 @200 = 32.1	-
	Internal water pressure	D19 @200 = 14.3	D29 @200 = 32.1	D19 @200 = 14.3
	Determination	D19 @200 = 14.3	D29 @200 = 32.1	D19 @200 = 14.3

Section	Load type	Member 1 (Inside)	Member 1 (Outside)	Member 2 (Inside)
III-III	Vertical load	2xD22 @200 = 2x19.4	D22 @200 = 19.4	D25 @200 = 25.3
	Internal water pressure	2xD22 @200 = 2x19.4	2xD22 @200 = 2x19.4	D25 @200 = 25.3
	Determination	2xD22 @200 = 2x19.4	2xD22 @200 = 2x19.4	D25 @200 = 25.3

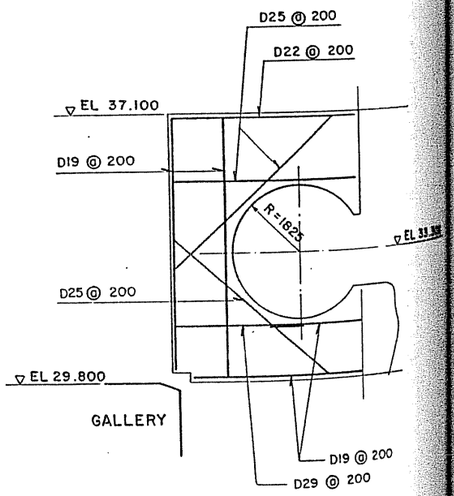
Section	Load type	Member 2 (Outside)	Member 3 to 7 (Inside)	Member 3 to 7 (Outside)
III-III	Vertical load	D25 @200 = 25.3	D25 @200 = 25.3	D22 @200 = 19.4
	Internal water pressure	D25 @200 = 25.3	D25 @200x2 = 50.68	D22 @200 = 19.4
	Determination	D25 @200 = 25.3	D25 @200x2 = 50.68	D22 @200 = 19.4

Section	Load type	Member 1 (Inside)	Member 1 (Outside)	Member 2 (Inside)	Member 2 (Outside)
IV-IV	Vertical load	D25 @200x2 = 50.7	D22 @200 = 19.4	-	D29 @200 = 32.1
	Internal water pressure	D25 @200x2 = 50.7	-	D29 @200 = 32.1	D29 @200 = 32.1
	Determination	2xD25 @200 = 50.7	D22 @200 = 19.4	D29 @200 = 32.1	D29 @200 = 32.1

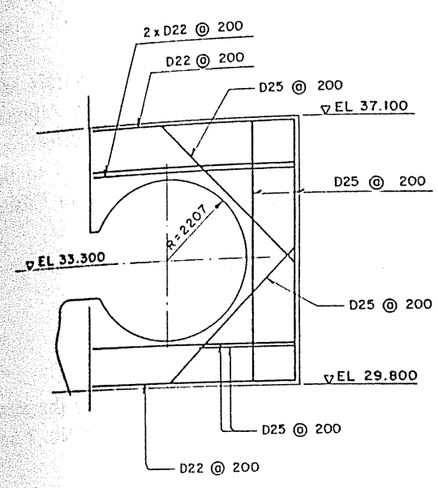
Section	Load type	Member 3 (Inside)	Member 3 (Outside)	Member 4 to 7 (Inside)	Member 3 to 5 (Outside)
IV-IV	Vertical load	D32 @200 = 39.7	D29 @200 = 32.1	D32 @200 = 39.7	-
	Internal water pressure	D32 @200 = 39.7	-	D32 @200 = 39.7	D22 @200 = 19.4
	Determination	D32 @200 = 29.7	D29 @200 = 32.1	D32 @200 = 39.7	D22 @200 = 19.4



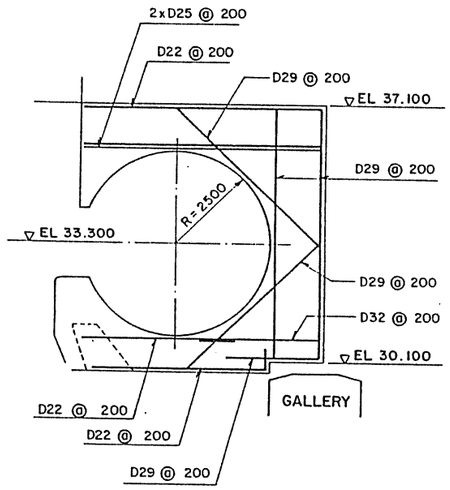
SECTION I-I



SECTION II-II



SECTION III-III



SECTION IV-IV

Stress intensity of each cross section

Section		Unit	1 member	2 member outside	3 (to 7) member
I-I	Reinforcement	cm²	D22 @200 = 19.4	D16 @200 =9.4	D16 @200 = 9.93
	Tensile stress	kg/cm²	1495 < 1800	563 < 2700	1719 < 2700
	Compressive stress	"	17 < 60	17 < 90	all tensition
	Shear stress	"	1.6 < 8	2.1 < 12	3.4 < 12
II-II	Reinforcement	cm²	D25 @ 25.3 = 25.3	D19 @200 = 14.3	D29 @200 = 32.1
	Tensile stress	kg/cm²	374 < 1800	2642 < 2700	894 < 1800
	Compressive stress	"	6 < 60	39 < 60	16 < 60
	Shear stress	"	3.2 < 8	2.8 < 12	2.6 < 8
III-III	Reinforcement	cm²	D22 @200 = 19.4	D25 @200 = 25.3	D25 @200x2 = 50.68
	Tensile stress	kg/cm²	1236 < 1800	2524 < 2700	1501 < 2700
	Compressive stress	"	22 < 60	38 < 90	32 < 90
	Shear stress	"	3.2 < 8	4.1 < 12	6.3 < 12
IV-IV	Reinforcement	cm²	D25 @200 = 19.4	D29 @200 = 32.1	D22 @ 200 = 39.7
	Tensile stress	kg/cm²	2294 < 2700	2635 < 2700	1699 < 1800
	Compressive stress	"	23 < 90	36 < 90	51 < 60
	Shear stress	"	2.2 < 12	4.3 < 12	5.0 < 8

Note : Typical cross section of the member 1 to 3 (~7) are shown in this table.
All the cross section of the members is checked and ensured that the stress intensity is within the allowable stress.

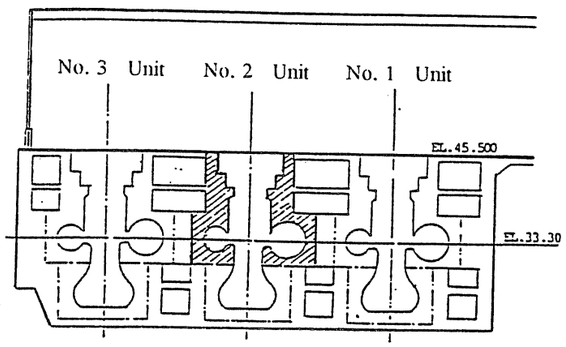
6.8.7 Stress Analysis of Around Barrel and Casing

Analysis was made of the stress which occurs in the body of the wind tunnels, barrels, and casings, as well as the stress which occurs locally in vicinities of the openings. A finite-element method was used for this analysis and the analysis was made in two directions, that is, circumferential direction and radial direction. The analysis referred to the arrangement of reinforcement given in section 6.8.5 for Barrel and 6.8.6 for Casing Designs.

(1) Conditions of Analysis

(i) Location of Analysis

The three units of the generator to be installed at this power station are the same configuration. Consequently, the portion marked in hatched line below at No. 2 unit was made the subject for the analysis.



(ii) Cases of Analysis

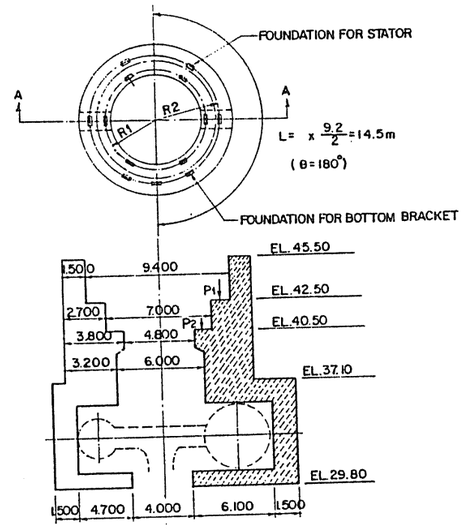
Case		Generator in opening condition
Cases-1 Circumferential direction	Cases 1-1 (Long period)	Stopped, Braked, Jacked up and in Normal Run
	Cases 1-2 (Short period)	Single phase shorted, at Earthquake and Thrust Burned
Cases -2 Radial direction	Cases 2-1 (Long period)	Stopped, Braked, Jacked up and in Normal Run
	Cases 2-2 (Long period)	Stopped, Braked, Jacked up, In Normal Run
	Cases 2-3 (Short period)	Single phase shorted, at Earthquake and Thrust Burned

Note : Internal hydraulic pressure considered for case-2 radial direction

(iii) Analytical Model

Circumferential Direction

To study the stress in the barrel body and the vicinities of the opening, a two dimensional model was obtained by expanding the portion of θ = 180 shown in the drawing below. The following figures shows the analytical model and an element breakdown chart.

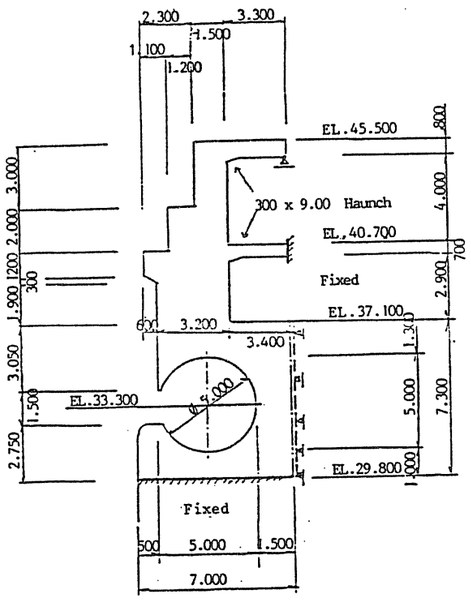


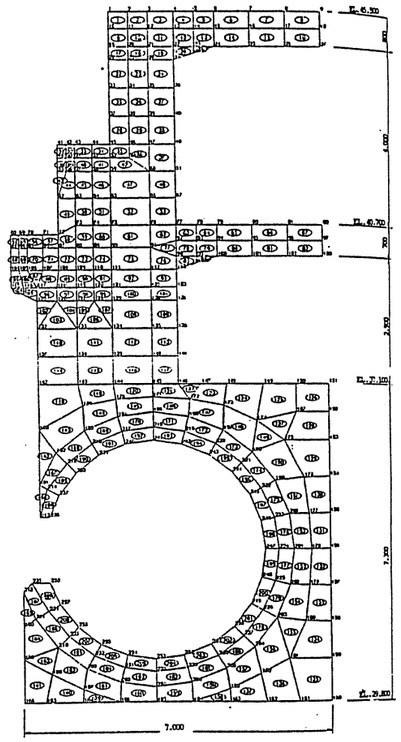
Element Chart of Circumference Direction

Radial Direction

To study stress near the generator foundations (the foundations for the stator and the bottom bracket) and that around the casing, an analytical model shown below was used.

Element Chart of Radial Direction

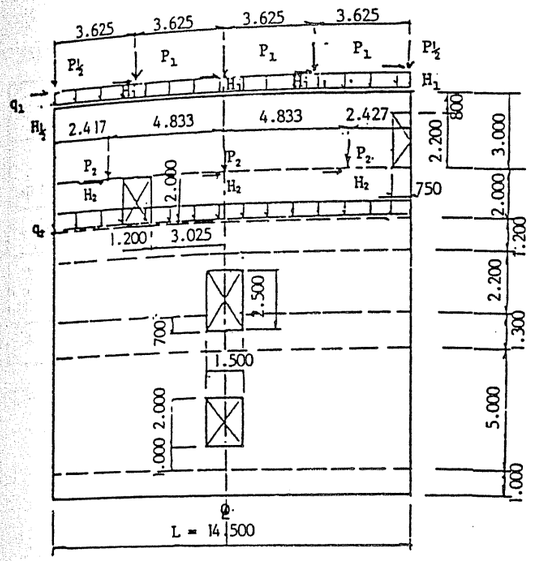




(iv) Conditions of Load

Circumferential direction

The following is a load diagram for case 1 (long period) and case 2 (short period). The load on generator foundations and dead weight of floor slabs are also shown as follows. The load on generator foundation is referred to the preceding section 6.8.5 Design of Around Barrel (2) Design conditions.



P_1, P_2, H_1, H_2..... : Load of generator foundation

q_1, q_2..... : Dead weight of floor slabs superimposed load

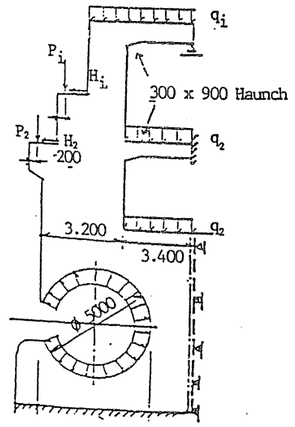
Floor slab EL 45.50 q_1 = 10.27 t/m
Floor slab EL 40.70 q_2 = 8.99 t/m

Design Load on Generator Foundation (Unit : t)

Case 1 long period	Total load Load per place	100 P_1=16.8	100 H_1= 12.5	15 H_3= 1.9	1000 P_2= 166.7	5 H_2= 0.8	25 H_4= 4.2
Case 2 short period	Total load Load per place	100 P_1=16.8	750 H_1= 93.8	55 H_3= 6.9	1000 P_2= 166.7	70 H_2= 11.7	45 H_4= 7.5

Radial direction

The following is a load diagram for cases 2-1 and 2-2 (long period) and case 2-3 (short period). The load on generator foundation is referred to 6.8.5 Design of Around Barrel (2) Design condition.



P_1, P_2, H_1, H_2..... : load of generator
q_1, q_2..... : Super imposed load
w ..... : internal hydraulic pressure

Design Load of Generator Foundation (unit : t)

Case 2-1 Case 2-2 (long period)	Total load	100	100	15	1000	5	25
	Load per meter	P_1 = 4.4 t/m	H_1= -	H_3= 0.65 t/m	P_2= 61.2 t/m	H_2= -	H_4= 1.53 t/m
Case 2-3 (short period)	Total load	100	750	30	1000	70	45
	Load per place	P_1 = 4.4 t/m	H_1= -	H_3= 1.3 t/m	P_2= 61.2 t/m	H_2= -	H_4= 2.75
Length of load (circumference)		23.248m			16.336m		

Superimposed load and internal hydraulic pressure

EL. 45.500 slab EL. 40.700 - EL. 37.100 slab	q1 = 5.0 t/m^2 q2 = 2.0 t/m^2

For internal hydraulic pressure, the following values are used from Section 6.8.6. Design of Areas Around Casing.

Internal hydraulic pressure

Case 2-1 long period Case 2-3 short period	w = 33 t/m^2 (hydrostatic pressure) w = 52 t/m2 (including water hammer)

(2) Results of Analysis

a) Circumferential direction

Figures 1.6.8-10 and 1.6.8-11 show charts of main stress and charts of displacement. The maximum values of displacement and main stress for each case are shown as follows.

Maximum Values of Displacement and Main Stress

	Case 1-1 (long period)	Case 1-2 (short period)
Displacement δx	0.013 mm (855 joint)	0.107 mm (855 joint)
Displacement δy	0.183 mm (867 joint)	0.200 mm (867 joint)
Compressed stress σt	6.67 kg/cm^2 (645 element)	11.61 kg/cm^2 (825 element)
Tensile stress σt	4.22 kg/cm^2 (675 element)	10.23 kg/cm^2 (816 element)

Note : long period σca = 60 kg/cm^2 and short period σca = 90 kg/cm^2

b) Radial direction

Fig. 1.6.8-12 and 1.6.8-13 shows charts of displacement and Figures charts of main stress. The maximum values of displacement and main stress for each case are shown below.

Maximum Values of Displacement and Main Stress

	Case 2-1 long period	Case 2-2 short period	Case 2-2 short period
Displacement (δx)	0.17 mm (213 joint)	0.09 mm (9 joint)	0.2 mm (213 joint)
Displacement (δy)	0.20 mm (69 element)	0.50 mm (69 element)	0.33 mm (213 element)
Compressed stress (σc)	16.98 kg/cm^2 (55 element)	16.97 kg/cm^2 (55 element)	16.79 kg/cm^2 (55 element)
Tensile stress (σt)	6.05 kg/cm^2 (57 element)	13.00 kg/cm^2 (84 element)	7.46 kg/cm^2 (196 element)

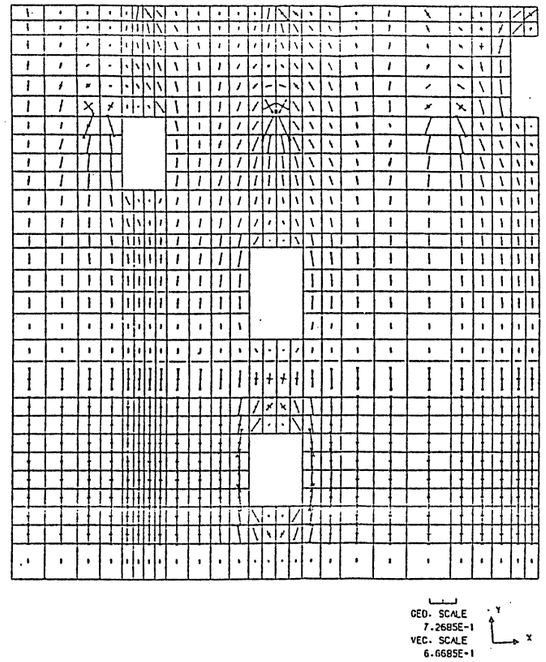
Note : long period σca = 60 kg/cm^2 and short period σca = 90 kg/cm^2

c) Consideration

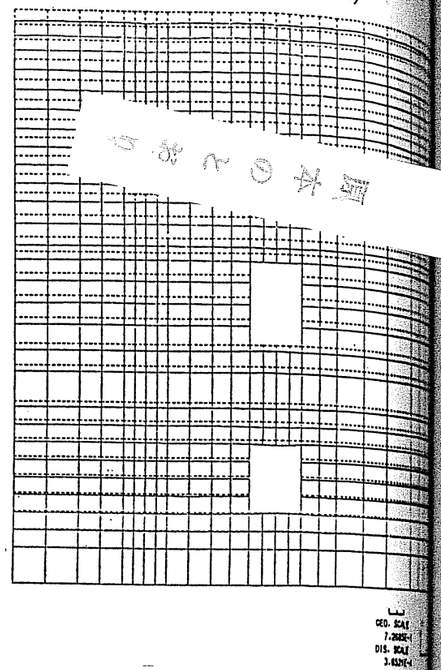
From the result of analysis, following points are found : (i) The amount of displacement presents no problem, being less than 1.0 mm for both long and short periods, (ii) Compressive stress is below permissible stress for both cases and (iii) Tensile stress occurs in circumferential direction in areas near the points where the generator load acts and near the load point, and in radial direction around the casing, near the load point and in the area where slabs are installed. To cope with this tensile stress, reinforcement is necessary.

Fig. 1.6.8-12 Stress and Displacement of Circumference Direction

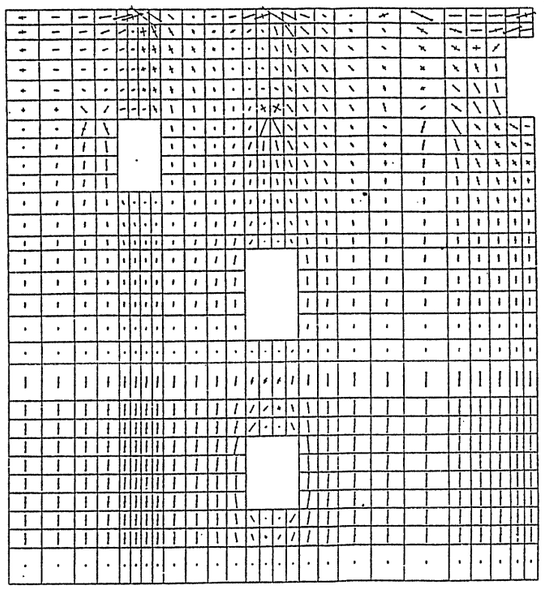
Principal Stress (Case 1)



Displacement (Case 1)



Principal Stress (Case 2)



Displacement (Case 2)

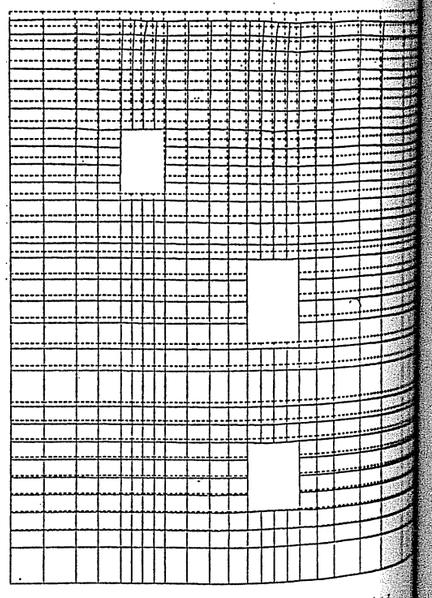
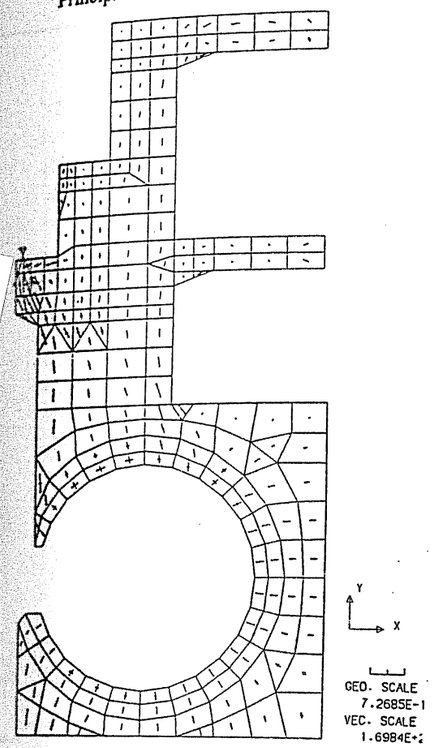
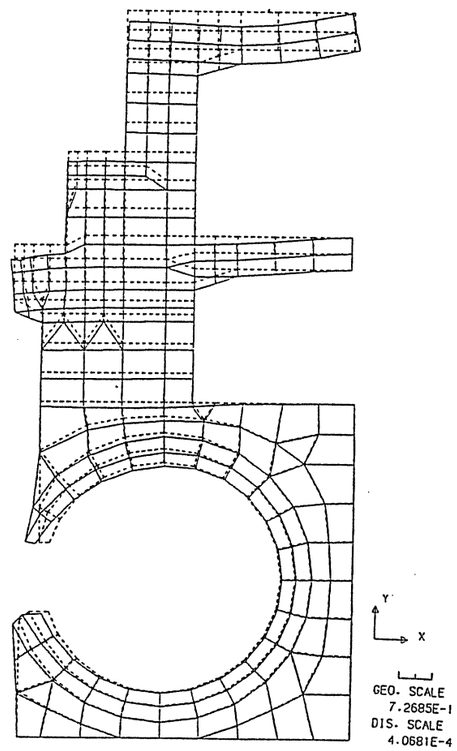


Fig. 1.6.8-13(1) Stress and Displacement of Radial Direction

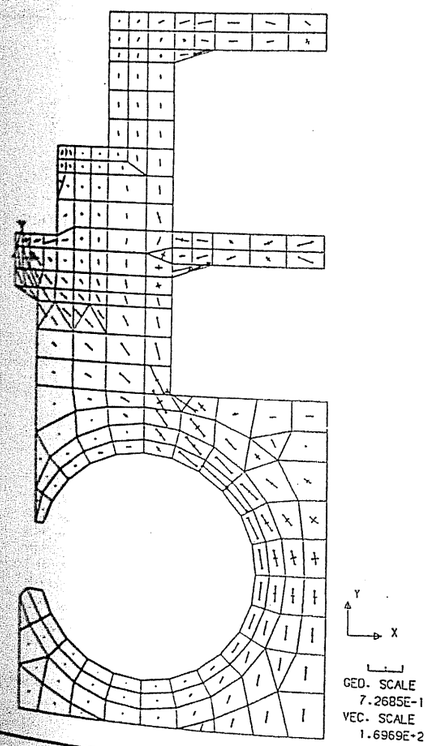
Principal Stress (Case 2-1)



Displacement (Case 2-1)



Principal Stress (Case 2-2)



Displacement (Case -2-2)

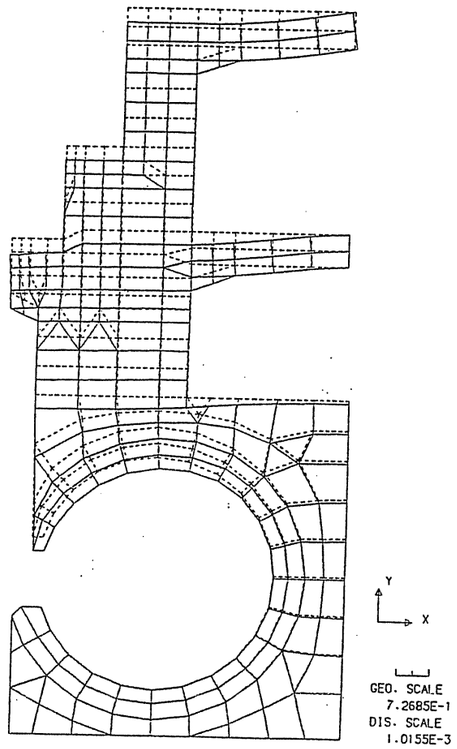
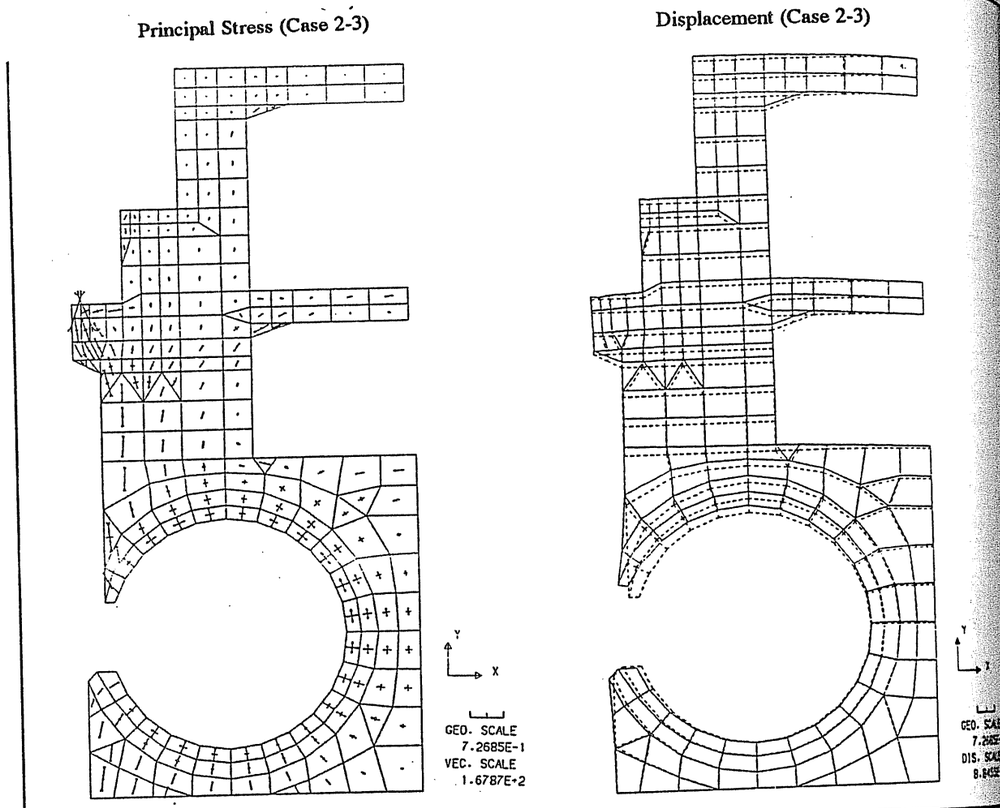


Fig. 1.6.8-13(2) Stress and Displacement of Radial Direction



6.8.8 Design of Draft Tube

(1) Design Condition

a) Conditions of load

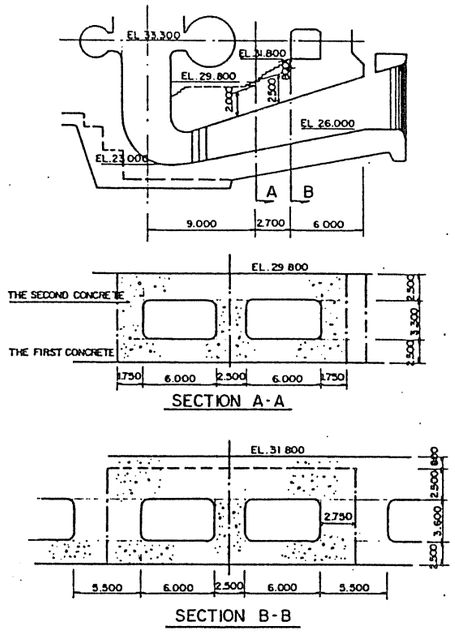
External pressure : The external pressure shall be considered a superimposed loads acting on the top side of draft section and uplift pressure acting on the base. The uplift shall be considered water head at NWL 41.00.

Internal pressure : The internal pressure shall be considered due to a static water pressure. The static water pressure shall be considered at NWL 41.00 for common operation of powerstation and at FWL 54.82 for maximum flooding water level.

b) The case study

During common operation	:	Superimposed load + Internal Pressure (NWL 41.00) + Uplift (NWL 41.00)
During flooding	:	Superimposed load + Internal Pressure (FWL 54.82) + Uplift (NWL 41.00)

The studied section shall be according to two sections of A and B shown on the following figure.

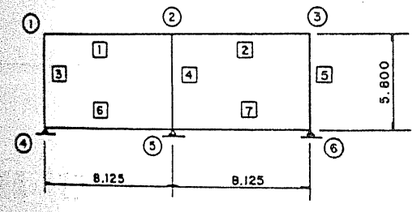


(2) Calculation for Reinforced Concrete

The calculation model and design loads for the draft A-A section and B-B section, and the calculation for stress of reinforced concrete are shown on the following figures and table.

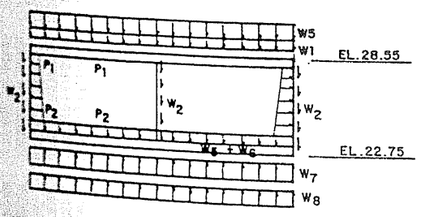
Fig. 1.6.8-14 Calculation Model of Powerhouse Draft Tube

(a) Calculation Model A-A SECTION



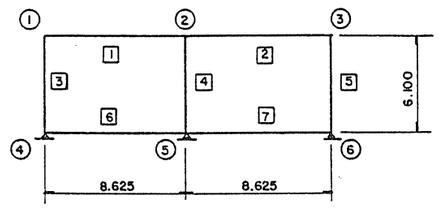
1 2	MEMBER	t = 2.50 m
3 5	MEMBER	t = 1.75 m
4	MEMBER	t = 2.50 m
6 7	MEMBER	t = 2.50 m

(b) Design Loads



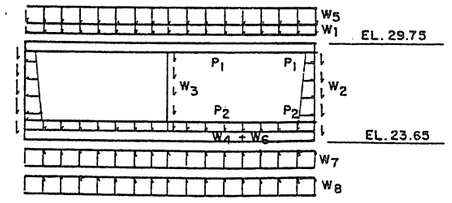
SLAB	W_1 = 6.00 t/m
SIDE WALL	W_2 = 4.20 t/m
BULK HEAD	W_3 = 6.00 t/m
BASE SLAB	W_4 = 6.00 t/m
VERTICAL LOAD AND INTERNAL PRESSURE	W_5 = 28.7 t/m
WEIGHT OF WATER	W_6 = 3.30 t/m
REACTION FORCE	W_7 = 48.52 t/m
UPLIFT PRESSURE	W_8 = 18.25 t/m
INTERNAL WATER PRESSURE	P_1 = 12.05 t/m
INTERNAL WATER PRESSURE	P_2 = 18.25 t/m

(a) Calculation Model B-B SECTION



1 2	MEMBER	t = 2.50 m
3 5	MEMBER	t = 2.75 m
4	MEMBER	t = 2.50 m
6 7	MEMBER	t = 2.50 m.

(b) Design Loads



SLAB	W_1 = 6.00 t/m
SIDE WALL	W_2 = 6.60 t/m
BULK HEAD	W_3 = 6.00 t/m
BASE SLAB	W_4 = 6.00 t/m
VERTICAL LOAD AND INTERNAL PRESSURE	W_5 = 28.0 t/m
WEIGHT OF WATER	W_6 = 3.60 t/m
REACTION FORCE	W_7 = 49.32 t/m
UPLIFT PRESSURE	W_8 = 17.35 t/m
INTERNAL WATER PRESSURE	P_1 = 11.25 t/m
INTERNAL WATER PRESSURE	P_2 = 17.35 t/m

Table 1.6.8-4(1) Calculation for Stress of Reinforcement Concrete

Draft A-A Section for Normal Condition

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	59.108	-86.400	16.152	100.0	250.0	15.0	10.0
1	Maximum	74.581	-86.400	0.000	100.0	250.0	15.0	10.0
1	2	87.915	-86.400	52.342	100.0	250.0	10.0	15.0
2	2	87.915	-86.400	52.342	100.0	250.0	10.0	15.0
2	Hunch	74.581	-86.400	0.000	100.0	250.0	15.0	10.0
2	3	59.108	-86.400	16.152	100.0	250.0	15.0	10.0
3	1	59.108	16.152	86.400	100.0	175.0	15.0	10.0
3	Maximum	77.541	29.195	0.000	100.0	175.0	10.0	15.0
3	4	32.367	40.512	82.786	100.0	175.0	15.0	10.0
4	2	.000	104.684	.000	100.0	250.0	15.0	15.0
4	5	.000	139.484	.000	100.0	250.0	15.0	15.0
5	3	59.108	16.152	86.400	100.0	175.0	15.0	10.0
5	Maximum	77.541	29.152	0.000	100.0	175.0	10.0	15.0
5	6	32.367	40.152	82.786	100.0	175.0	15.0	10.0
6	4	32.367	-82.786	71.145	100.0	250.0	15.0	10.0
6	Maximum	132.763	-82.786	0.000	100.0	250.0	15.0	10.0
6	5	225.783	-82.786	134.960	100.0	250.0	10.0	15.0
7	5	225.783	-82.786	134.960	100.0	250.0	10.0	15.0
7	Maximum	132.763	-82.786	0.000	100.0	250.0	15.0	10.0
7	6	32.367	-82.786	71.415	100.0	250.0	15.0	10.0

		AS (cm^2)	ASD (cm^2)	P	SIGMA C (Kg/cm^2)	SIGMA S (Kg/cm^2)	TAU (Kg/cm^2)
1	1	9.65	9.65	00041	20.42	2700	.71
1	Maximum	12.22	12.22	00052	22.97	2700	0.00
1	2	14.16	14.16	00059	24.73	2700	2.28
2	2	14.16	14.16	00059	24.73	2700	2.28
2	Maximum	12.22	12.22	00052	22.97	2700	0.00
2	3	9.65	9.65	00041	20.42	2700	.71
3	1	11.34	11.34	00071	34.02	2700	5.64
3	Maximum	13.18	13.18	00080	40.28	2700	0.00
3	4	1.00	1.00	00006	23.06	1194	4.55
4	2	1.00	1.00	00004	4.18	0	.00
4	5	1.00	1.00	00004	5.57	0	.00
5	3	11.34	11.34	00071	34.02	2700	5.64
5	Maximum	13.18	13.18	00080	40.28	2700	0.00
5	6	1.00	1.00	00006	23.06	1194	2.55
6	4	36.07	36.07	00153	5.88	2700	3.25
6	Maximum	[illegible]	[illegible]	[illegible]	[illegible]	2700	[illegible]
6	5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	Maximum	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Draft B-B Section for Normal Condition

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	57.301	-47.931	85.675	100.0	250.0	10.0	15.0
1	Maximum	104.021	-47.931	0.000	100.0	250.0	15.0	10.0
1	2	164.550	-47.931	110.544	100.0	250.0	10.0	15.0
2	2	164.550	-47.931	110.544	100.0	250.0	10.0	15.0
2	Hunch	104.021	-47.931	0.000	100.0	250.0	15.0	10.0
2	3	57.301	-47.931	85.675	100.0	250.0	10.0	15.0
3	1	57.301	85.675	47.931	100.0	275.0	10.0	15.0
3	Maximum	149.206	109.856	0.000	100.0	275.0	10.0	15.0
3	4	102.543	125.935	39.299	100.0	275.0	10.0	15.0
4	2	.000	221.088	.000	100.0	250.0	15.0	15.0
4	5	.000	257.688	.000	100.0	250.0	15.0	15.0
5	3	57.301	85.675	47.931	100.0	275.0	10.0	15.0
5	Maximum	149.206	109.856	0.000	100.0	275.0	10.0	15.0
5	6	102.543	125.935	39.299	100.0	275.0	10.0	15.0
6	4	102.543	-39.299	146.303	100.0	250.0	10.0	15.0
6	Maximum	166.900	-39.299	0.000	100.0	250.0	15.0	10.0
6	5	318.078	-39.299	196.282	100.0	250.0	10.0	15.0
7	5	318.078	-39.299	196.282	100.0	250.0	10.0	15.0
7	Maximum	166.900	-39.299	0.000	100.0	250.0	15.0	10.0
7	6	102.543	-39.299	146.303	100.0	250.0	10.0	15.0

		AS (cm^2)	ASD (cm^2)	P	SIGMA C (Kg/cm^2)	SIGMA S (Kg/cm^2)	TAU (Kg/cm^2)
1	1	[illegible]	27.10	00113	4.69	1800	3.64
1	Maximum	[illegible]	39.22	00167	14.21	1800	0.00
1	2	[illegible]	53.37	00222	21.21	1800	4.92
2	2	[illegible]	53.37	00222	21.21	1800	4.92
2	Maximum	[illegible]	39.22	00167	14.21	1800	0.00
2	3	[illegible]	27.10	00113	4.69	1800	3.64
3	1	[illegible]	1.00	00004	8.06	30	.01
3	Maximum	[illegible]	4.78	00018	37.16	1800	0.00
3	4	[illegible]	1.00	00004	14.89	127	.03
4	2	[illegible]	1.00	00004	8.83	0	.00
4	5	[illegible]	1.00	00004	10.30	0	.00
5	3	[illegible]	1.00	00004	8.06	30	.01
5	Maximum	[illegible]	4.78	00018	37.16	1800	0.00
5	6	[illegible]	1.00	00004	14.89	127	.03
6	4	[illegible]	35.69	00149	15.47	1800	6.42
6	Maximum	[illegible]	52.63	00224	22.67	1800	0.00
6	5	[illegible]	89.79	00374	35.90	1800	8.97
7	5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	Maximum	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
7	6	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Draft A-A Section for Flood Condition

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	9.512	-45.493	68.195	100.0	250.0	15.0	10.0
1	Maximum	114.020	-45.493	0.000	100.0	250.0	15.0	10.0
1	2	170.825	-45.493	122.586	100.0	250.0	10.0	15.0
2	2	170.825	-45.493	112.586	100.0	250.0	10.0	15.0
2	Hunch	114.020	-45.493	0.000	100.0	250.0	15.0	10.0
2	3	9.512	-45.493	68.195	100.0	250.0	15.0	10.0
3	1	9.512	68.195	45.493	100.0	175.0	15.0	10.0
3	Maximum	66.870	81.778	0.000	100.0	175.0	10.0	15.0
3	4	12.421	92.555	43.537	100.0	175.0	10.0	15.0
4	2	.000	-225.172	00	100.0	250.0	15.0	15.0
4	5	.000	-259.972	00	100.0	250.0	15.0	15.0
5	3	9.512	68.195	45.493	100.0	175.0	15.0	10.0
5	Maximum	66.870	81.778	0.000	100.0	175.0	10.0	15.0
5	6	12.421	92.555	43.537	100.0	175.0	10.0	15.0
6	4	12.412	-43.537	121.792	100.0	250.0	10.0	15.0
6	Maximum	176.681	-43.537	0.000	100.0	250.0	15.0	10.0
6	5	317.431	-43.537	196.871	100.0	250.0	10.0	15.0
7	5	317.431	-43.537	196.871	100.0	250.0	10.0	15.0
7	Maximum	176.681	-43.537	0.000	100.0	250.0	15.0	10.0
7	6	12.421	-43.537	121.792	100.0	250.0	10.0	15.0

		AS (cm^2)	ASD (cm^2)	P	SIGMA C (Kg/cm^2)	SIGMA S (Kg/cm^2)	TAU (Kg/cm^2)
1	1	27.65	27.65	00118	1.97	1800	3.08
1	Maximum	41.03	41.03	00175	15.99	1800	0.00
1	2	54.28	54.28	00226	22.15	1800	5.02
2	2	54.28	54.28	00226	22.15	1800	5.02
2	Maximum	41.03	41.03	00175	15.99	1800	0.00
2	3	27.65	27.65	00118	1.97	1800	3.08
3	1	1.00	1.00	00006	5.75	0	.14
3	Maximum	2.48	2.48	00015	40.76	1800	0.00
3	4	1.00	1.00	00006	7.71	0	.14
4	2	1.00	1.00	00004	9.00	0	.00
4	5	1.00	1.00	00004	10.39	0	.00
5	3	1.00	1.00	00006	5.75	0	.14
5	Maximum	2.48	2.48	00015	40.76	1800	0.00
5	6	1.00	1.00	00006	7.71	0	.14
6	4	27.23	27.23	00113	2.55	1800	5.37
6	Maximum	56.27	56.27	00239	23.00	1800	0.00
6	5	90.73	90.73	00378	33.42	1800	8.99
7	5	90.23	90.73	00378	33.42	1800	8.99
7	Maximum	56.27	56.27	00239	23.00	1800	0.00
7	6	27.23	27.23	00113	2.55	1800	5.37

Draft B-B Section for Flood Condition

		M ([illegible])	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	16.096	-91.899	27.197	100.0	250.0	15.0	10.0
1	Maximum	57.510	-91.899	0.000	100.0	250.0	15.0	10.0
1	2	81.487	-91.899	49.825	100.0	250.0	10.0	15.0
2	2	81.487	-91.899	49.825	100.0	250.0	10.0	15.0
2	Hunch	57.510	-91.899	0.000	100.0	250.0	15.0	10.0
2	3	16.096	-91.899	27.197	100.0	250.0	15.0	10.0
3	1	16.096	27.197	91.899	100.0	275.0	15.0	10.0
3	Maximum	144.920	49.841	0.000	100.0	275.0	10.0	15.0
3	4	40.233	67.457	79.633	100.0	275.0	10.0	15.0
4	2	.000	99.649	.000	100.0	250.0	15.0	15.0
4	5	.000	136.249	.000	100.0	250.0	15.0	15.0
5	3	16.096	27.197	91.899	100.0	275.0	15.0	10.0
5	Maximum	144.920	49.841	0.000	100.0	275.0	10.0	15.0
5	6	40.233	67.457	76.633	100.0	275.0	10.0	15.0
6	4	40.233	-79.633	90.767	100.0	250.0	10.0	15.0
6	Maximum	118.816	-79.633	0.000	100.0	250.0	15.0	10.0
6	5	220.733	-79.633	132.620	100.0	250.0	10.0	15.0
7	5	220.723	-79.633	132.620	100.0	250.0	10.0	15.0
7	Maximum	118.816	-79.633	0.000	100.0	250.0	15.0	10.0
7	6	40.233	-79.633	90.676	100.0	250.0	10.0	15.0

		AS (cm^2)	ASD (cm^2)	P	SIGMA C (Kg/cm^2)	SIGMA S (Kg/cm^2)	TAU (Kg/cm^2)
1	1	36.73	36.73	00156	2.90	2700	1.24
1	Maximum	9.38	9.38	00040	20.14	2700	0.00
1	2	13.10	13.10	00055	23.80	2700	2.16
2	2	13.10	13.10	00055	23.80	2700	2.16
2	Maximum	9.38	9.38	00040	20.14	2700	0.00
2	3	36.73	36.73	00156	2.90	2700	1.24
3	1	1.00	1.00	00004	2.31	4	.02
3	Maximum	12.39	12.39	00047	35.85	2700	0.00
3	4	1.00	1.00	00004	5.76	11	.02
4	2	1.00	1.00	00004	3.98	0	.00
4	5	1.00	1.00	00004	5.44	0	.00
5	3	1.00	1.00	00004	2.31	4	.02
5	Maximum	12.39	12.39	00047	35.85	2700	0.00
5	6	1.00	1.00	00004	5.76	11	.02
6	4	6.39	6.39	00027	16.63	2700	3.89
6	Maximum	34.55	34.55	00147	13.43	2700	0.00
6	5	50.44	50.44	00210	27.99	2700	5.87
7	5	50.44	50.44	00210	27.99	2700	5.87
7	Maximum	34.55	34.55	00147	13.43	2700	0.00
7	6	6.39	6.39	00027	16.63	2700	3.89

(3) Calculation Result

Required quantities of reinforcement bar at each sections is shown as follows.

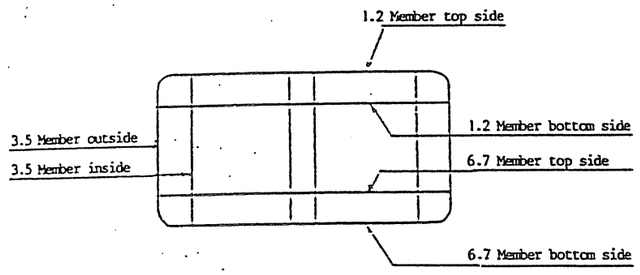
Calculation section		1,2Member top side slab	1,2Member bottom side slab	3,5Member outside side wall	3,5Member inside side wall	6,7Member top side base slab	6,7Member bottom base slab
A-A	Common	54.28	41.03	2.48	-	56.27	90.73
	Flooding	14.16	12.22	13.13	-	37.44	51.84
	Determination	D25 @ 150 = 67.6 cm²	D29 @ 150 = 42.9 cm²	D16 @ 150 = 13.3 cm²	D16 @150 =13.3 cm²	D25 @ 150 = 67.6 cm²	D32 @ 150 = 106 cm²
B-B	Common	53.37	39.22	4.78	-	52.63	89.79
	Flooding	13.10	36.73	12.39	-	34.55	50.44
	Determination	Double D25 @ 150 = 67.6 cm²	D29 @ 150 = 42.9 cm²	D16 @ 150 = 13.3 cm²	D16 @ 150 = 13.3 cm²	Double D25 @ 150 = 67.6 cm²	Double D32 @ 150 = 106 cm²

Stress intensity of each cross section

		1, 2 member top side	1, 2 member bottom side
A - A	Reinforcement	D25 @ 150 = 67.6 cm²	D29 @ 150 = 42.9 cm²
	Tensile stress	1481 < 2000 kg/cm²	1722 < 2000 kg/cm²
	Compressive stress	20.5 < 60 kg/cm²	15.6 < 60 kg/cm²
	Shear stress	5.17 < 8 kg/cm²	0 < 8 kg/cm²
	Bond stress	4.8 < 14 kg/cm²	0 < 14 kg/cm²
B - B	Reinforcement	D25 @ 150 = 67.6 cm²	D29 @ 150 = 42.9 cm²
	Tensile stress	1456 < 2000 kg/cm²	1646 < 2000 kg/cm²
	Compressive stress	19.5 < 60 kg/cm²	13.5 < 60 kg/cm²
	Shear stress	5.06 < 8 kg/cm²	0 < 8 kg/cm²
	Bond stress	4.7 < 14 kg/cm²	0 < 14 kg/cm²

		3, 5 member outside	3, 5 member inside
A - A	Reinforcement	D16 @ 150 = 13.3 cm²	D16 @ 150 = 13.3 cm²
	Tensile stress	2676 < 3000 kg/cm²	2676 < 3000 kg/cm²
	Compressive stress	40.1 < 90 kg/cm²	40.1 < 90 kg/cm²
	Shear stress	0 < 12 kg/cm²	0 < 12 kg/cm²
	Bond stress	0 < 21 kg/cm²	0 < 21 kg/cm²
B - B	Reinforcement	D16 @ 150 = 13.3 cm²	D16 @ 150 = 13.3 cm²
	Tensile stress	2525 < 3000 kg/cm²	2525 < 3000 kg/cm²
	Compressive stress	34.8 < 90 kg/cm²	34.8 < 90 kg/cm²
	Shear stress	0 < 12 kg/cm²	0 < 12 kg/cm²
	Bond stress	0 < 21 kg/cm²	0 < 21 kg/cm²

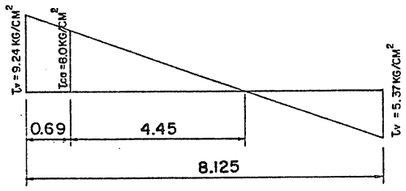
		6, 7 member topside	6, 7 member bottom side
A - A	Reinforcement	D25 @ 150 = 67.6 cm²	D32 @ 150 = 106.0 cm²
	Tensile stress	1535 < 2000 kg/cm²	1579 < 2000 kg/cm²
	Compressive stress	21.7 < 60 kg/cm²	31.9 < 60 kg/cm²
	Shear stress	0 < 8 kg/cm²	9.24 < 8 kg/cm²
	Bond stress	0 < 14 kg/cm²	6.9 < 14 kg/cm²
B - B	Reinforcement	D25 @ 150 = 67.6 cm²	D32 @ 150 =106.0 cm²
	Tensile stress	1436 < 2000 kg/cm²	1563 < 2000 kg/cm²
	Compressive stress	20.6 < 60 kg/cm²	32.1 < 60 kg/cm²
	Shear stress	0 < 8 kg/cm²	9.22 < 8 kg/cm²
	Bond stress	0 < 14 kg/cm²	6.9 < 14 kg/cm²



(4) Study of Shear Stress

(i) Reinforcement by stirrup

For 6 and 7 members the concrete cannot resist the shear stresses and therefore, reinforcement by stirrup is necessary. Calculation was made by assuming the stirrup should resist all shear forces. Stress intensity diagram of maximum shear force is as follows.



Shear stress intensity taken by stirrup is,

$$\tau = Av / S.bo.\sigma sa$$

Where

τ : Stress intensity stirrup (kg/cm²)
S : Spacing of stirrup = 30 cm
Av : Area of cross section of stirrup = 4 - D22 = 15.5 cm²
bo : Width of beam = 100 cm
sa : Allowable tensile stress of reinforcement = 2000 kg/cm²

τ = 15.5 / (30 x 100) x 2000 = 10.33 kg/cm² > v = 9.24 kg/cm²

Therefore = D22 @ 300 (2 set) was adopted.

6.8.9 Structural Design of Tailrace

(1) Design condition

a) Design loads

The design water level of the tailrace is taken for flood water level (FWL = 54.82) and normal water level (NWL = 41.00). Gantry crane and gate weight is given in Part 5 Metal Works. For the tailrace structural design, some additional allowance is also taken into consideration.

b) Backfilling soil

Unit weight d = 1.8 t/m^3, internal friction angle Ø = 35^0 are taken respectively

c) Allowable stress intensity and modulus of elasticity of concrete are shown in the section 6.8.2 (2) Common Design Criteria and Parameter

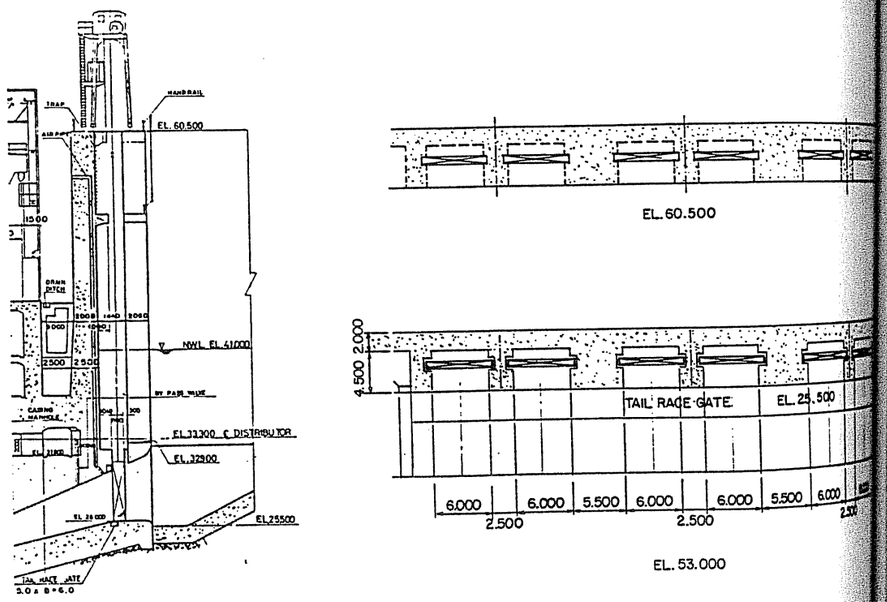


Fig. 1.6.8-15 Tailrace General Structure

(2) Calculation model and load

The calculation model and the load acting on the model is shown as follows.

Fig. 1.6.8-16(1) Calculation Model of Tailrace Structure

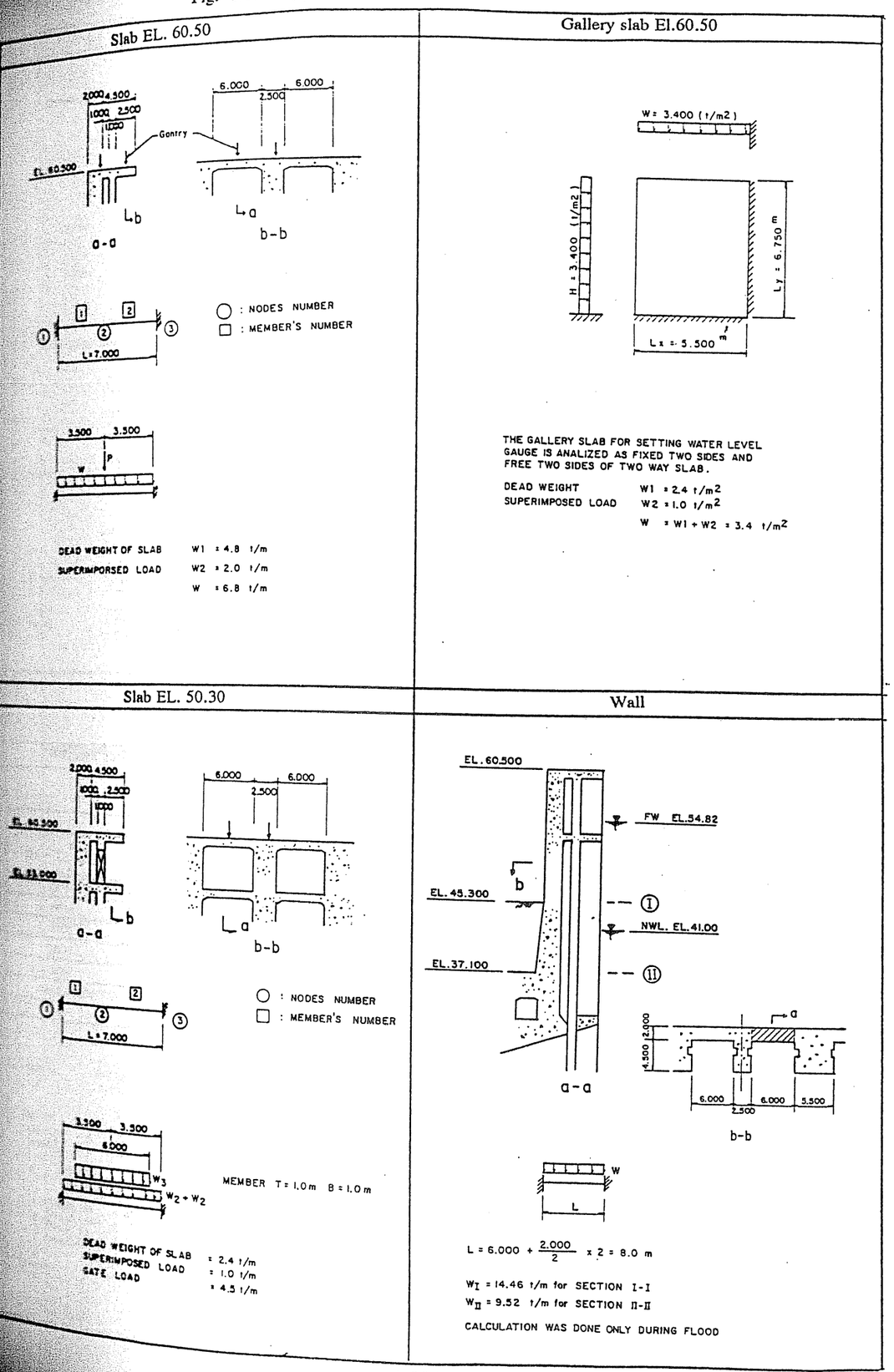


Fig. 1.6.8-16(2) Calculation Model of Tailrace Structure

Column

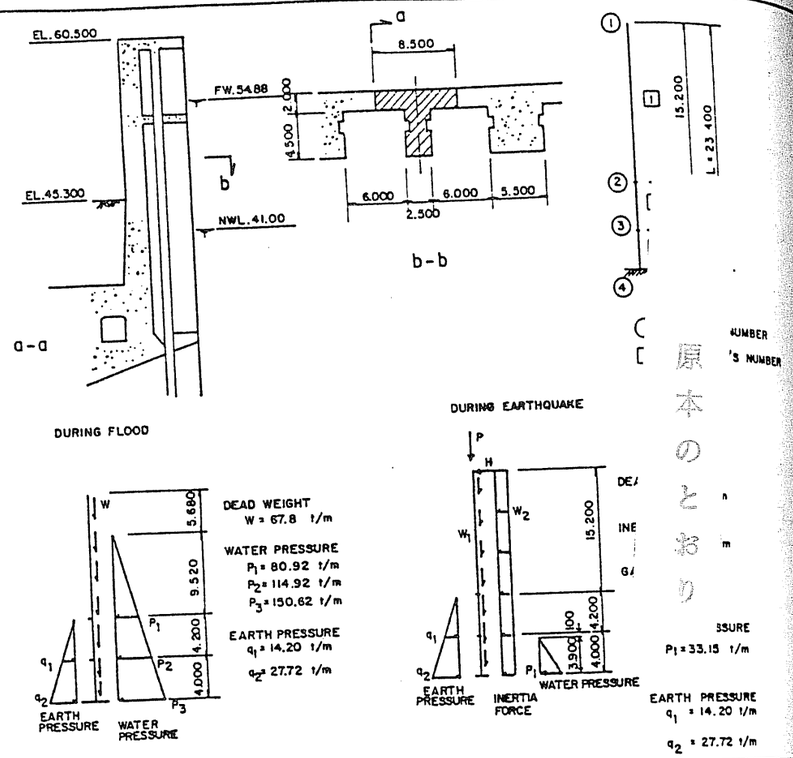


Table 1.6.8-5(1) Calculation for Stresses of Reinforce Concrete

EL. 60.50 Slab for Tailrace

BENDING MOMENT AND SHEARING FORCE

			Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1		0.000	- 37.217	0.000	29.200
1		Hunch	1.400	- 13.691	0.000	19.680
1	2		3.500	23.333	0.000	5.400
2	2		0.000	23.333	0.000	5.400
2		Hunch	2.100	- 13.691	0.000	19.680
2	3		3.500	- 37.217	0.000	29.200

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

			M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1		37.217	0.000	29.200	200.0	130.0	10.0	10.0
1		Hunch	13.691	0.000	19.680	200.0	100.0	10.0	10.0
1	2		23.333	0.000	5.400	200.0	100.0	10.0	10.0
2	2		23.333	0.000	5.400	200.0	100.0	10.0	10.0
2		Hunch	13.891	0.000	19.680	200.0	100.0	10.0	10.0
2	3		37.217	0.000	29.200	200.0	130.0	10.0	10.0

			AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1		16.24	16.24	00068	19.85	2000	1.27
1		Hunch	7.89	7.89	00044	16.21	2000	1.13
1	2		13.62	13.62	00076	21.31	2000	0.31
2	2		13.62	13.62	00076	21.31	2000	0.31
2		Hunch	7.89	7.89	00044	16.21	2000	1.13
2	3		16.24	16.24	00068	19.85	2000	1.27

EL. 60.50 Slab of Gallery Part For Tailrace

BENDING MOMENT

W = 3.400 (t/m²)

Lx = 5.500 m Ly = 6.750 m

		W*Lx²	M (tm)
Mx 1	- .3415	102.850	- 35.12
Mx 2	.0322	102.850	3.31
My 1	- .3415	102.850	- 35.12
My 2	.0491	102.850	5.05
		W*Lx²/E/T²	Delta (m)
Delta	.5319	.00120	0.64

CALCULATION FOR STRESSES OF REINFORCED CONCRETE

——Slab——

M = 35.123 (tm) b = 100.0 (cm) d = 90.0 (cm)
N = 0.000 (t) t = 100.0 (cm) d' = 10.0 (cm)
Q = 0.000 (t)

D 22 @ 150 = 25.80 (cm²) [p = .287 (%)]

Sig C = 34.0 (kg/cm²) (Sig Ca = 60.0 (kg/cm²))
Sig S = 1650.6 (kg/cm²) (Sig Sa = 2000.0 (kg/cm²))
Tau C = 0.0 (kg/cm²) (Tau Ca = 6.0 (kg/cm²))
Tau O = 0.0 (kg/cm²) (Tau Oa = 12.0 (kg/cm²))

Table 1.6.8-5(2) Calculation for Stresses of Reinforce Concrete

EL. 53.00 Slab for Tailrace

BENDING MOMENT AND SHEARING FORCE

			Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1		0.000	-30.223	0.000	23.150
1		Hunch	1.400	-10.765	0.000	16.590
1	2		3.500	14.915	0.000	0.000
2	2		0.000	15.915	0.000	0.000
2		Hunch	2.100	-10.765	0.000	16.590
2	3		3.500	-30.223	0.000	23.150

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

			M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1		30.223	0.000	23.150	100.0	130.0	10.0	10.0
1		Hunch	10.765	0.000	16.590	100.0	100.0	10.0	10.0
1	2		15.915	0.000	0.000	100.0	100.0	10.0	10.0
2	2		15.915	0.000	0.000	100.0	100.0	10.0	10.0
2		Hunch	10.765	0.000	16.590	100.0	100.0	10.0	10.0
2	3		30.223	0.000	23.150	100.0	130.0	10.0	10.0

			AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1		13.34	13.34	00111	25.41	2000	2.04
1		Hunch	8.27	8.27	00070	20.45	2000	1.93
1	2		9.36	9.36	00104	24.99	2000	0.00
2	2		9.36	9.36	00104	24.99	2000	0.00
2		Hunch	8.27	8.27	00070	20.45	2000	1.93
2	3		13.34	13.34	00111	25.41	2000	2.04

Wall I-I Section

BENDING MOMENT AND SHEARING FORCE

		Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1	0.000	-50.773	0.000	38.080
1	2	4.000	25.387	0.000	0.000
2	2	0.000	25.387	0.000	0.000
2	3	4.000	-50.773	0.000	38.080

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

		M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1	50.773	0.000	38.080	100.0	200.0	10.0	10.0
1	2	25.387	0.000	0.000	100.0	200.0	10.0	10.0
2	2	25.387	0.000	0.000	100.0	200.0	10.0	10.0
2	3	50.773	0.000	38.080	100.0	200.0	10.0	10.0

		AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1	9.26	9.26	00049	24.85	3000	2.08
1	2	4.58	4.58	00024	17.48	3000	0.00
2	2	4.58	4.58	00024	17.48	3000	0.00
2	3	9.26	9.26	00049	24.85	3000	2.08

Wall II-II Section

BENDING MOMENT AND SHEARING FORCE

			Length (m)	Bending moment (t x m)	Axial force (t)	Shearing force (t)
1	1		0.000	-77.115	0.000	57.836
1		Maximum	4.000	38.557	0.000	0.000
1	2		4.000	38.557	0.000	0.000
2	2		0.000	38.557	0.000	0.000
2		Maximum	0.000	38.557	0.000	0.000
2	3		4.000	-77.115	0.000	57.836

CALCULATION FOR STRESSES OF REINFORCE CONCRETE

			M (t x m)	N (t)	Q (t)	B (cm)	T (cm)	D (cm)	D' (cm)
1	1		77.115	0.000	57.836	100.0	250.0	10.0	10.0
1		Maximum	38.557	0.000	0.000	100.0	250.0	10.0	10.0
1	2		38.557	0.000	0.000	100.0	250.0	10.0	10.0
2	2		38.557	0.000	0.000	100.0	250.0	10.0	10.0
2		Maximum	38.557	0.000	0.000	100.0	250.0	10.0	10.0
2	3		77.115	0.000	57.836	100.0	250.0	10.0	10.0

			AS (cm²)	ASD (cm²)	P	SIGMA C (Kg/cm²)	SIGMA S (Kg/cm²)	TAU (Kg/cm²)
1	1		11.11	11.11	00048	24.08	3000	2.50
1		Maximum	5.50	5.50	00023	16.95	3000	0.00
1	2		5.50	5.50	00023	16.95	3000	0.00
2	2		5.50	5.50	00023	16.95	3000	0.00
2		Maximum	5.50	5.50	00023	16.95	3000	0.00
2	3		11.11	11.11	00048	24.08	3000	2.50

Column for Tailrace

Required quantities of reinforcement steel bars for T shaped section

	NO.	POST 1	POST 2	POST 3
b	(cm)	850.0	850.0	850.0
bO	(cm)	250.0	250.0	250.0
h	(cm)	700.0	676.0	650.0
t	(cm)	250.0	226.0	200.0
d	(cm)	685.0	661.0	635.0
d'	(cm)	10.0	10.0	10.0
As	(cm^2)	384.263	188.198	65.432
As'	(cm^2	192.131	94.099	32.716
M	(t.m)	7571.76	3616.99	1222.30
S	(t)	1220.84	770.20	385.18
P = As / (b*d) (%)		0.066	0.033	0.012
d' / d		0.015	0.015	0.015
As' / As		0.500	0.500	0.500
t / d		0.365	0.342	0.315
bO / b		0.294	0.294	0.294
X	(cm)	87.098	61.716	36.752
C		15.338	21.140	35.446
S		105.290	205.275	560.710
σc	(kg/cm^2)	29.1	20.6	12.3
σs	(kg/cm^2)	2998	2999	2999
σs'	(kg/cm^2)	387	259	134
τm	(kg/cm^2)	7.1	4.7	2.4
σca	(kg/cm^2)	90	90	90
σsa	(kg/cm^2)	3000	3000	3000
τa	(kg/cm^2)	12.0	12.0	12.0

(3) Calculation results

Required quantities of reinforcement bars at each section are shown as follows.

Slab	EL. 60.500	Upper side D19 @ 300 = 9.54 > 8.12 cm^2	Lower side D19 @ 300 = 9.54 > 6.81 cm^2
Slab	EL. 53.000	Upper side D25 @ 300 = 16.9 > 13.34 cm^2	Lower side D19 @ 300 = 9.54 > 9.36 cm^2
Wall	EL. 45.30	Outside D19 @ 300 = 9.54 > 9.26 cm^2	Inside D16 @ 300 = 6.61 > 4.58 cm^2
	EL. 37.10	Outside D22 @ 300 = 12.9 > 11.11 cm^2	Inside D16 @ 300 = 6.61 > 5.50 cm^2
Column	EL. 45.30	D32 @ 200 = 39.7 > 26.17 cm^2	D16 @ 300 = 6.61 > 3.83 cm^2
	EL. 41.10	Outside Double D32 @ 200 = 79.4 > 75.28 cm^2	D22 @ 300 = 12.9 > 11.07 cm^2
	EL. 41.10	Double D32 @ 200 =158.8 > 153.71 cm^2	D22 @ 150 = 25.8 > 22.60 cm^2

Stress intensity of each cross section

		Upper side or outside	Lower side or inside
Slab EL. 60.500	Reinforcement	D19 @ 300 cm^2	D19 @ 300 cm^2
	Tensile stress	1708 < 2000 kg/cm^2	1440 < 2000 kg/cm^2
	Compressive stress	18.4 < 60 kg/cm^2	18.2 < 60 kg/cm^2
	Shear stress	1.28 < 8 kg/cm^2	0.32 < 8 kg/cm^2
	Bond stress	6.4 < 14 kg/cm^2	1.6 < 14 kg/cm^2
Slab EL. 53.000	Reinforcement	D25 @ 300 cm^2	D19 @ 300 cm^2
	Tensile stress	1588 < 2000 kg/cm^2	1964 < 2000 kg/cm^2
	Compressive stress	22.7 < 60 kg/cm^2	24.8 < 60 kg/cm^2
	Shear stress	2.06 < 8 kg/cm^2	0 < 8 kg/cm^2
	Bond stress	7.7 < 14 kg/cm^2	0 < 14 kg/cm^2
Wall EL. 45.300	Reinforcement	D19 @ 300 cm^2	D16 @ 300 cm^2
	Tensile stress	2913 < 3000 kg/cm^2	2090 < 2000 kg/cm^2
	Compressive stress	24.5 < 90 kg/cm^2	14.6 < 90 kg/cm^2
	Shear stress	2.08 < 12 kg/cm^2	0 < 12 kg/cm^2
	Bond stress	10.4 < 21 kg/cm^2	0 < 21 kg/cm^2
Wall EL. 37.100	Reinforcement	D22 @ 300 cm^2	D16 @ 300 cm^2
	Tensile stress	2591 < 3000 kg/cm^2	2503 < 3000 kg/cm^2
	Compressive stress	25.1 < 90 kg/cm^2	15.5 < 90 kg/cm^2
	Shear stress	2.51 < 12 kg/cm^2	0 < 12 kg/cm^2
	Bond stress	10.8 < 21 kg/cm^2	0 < 21 kg/cm^2

Column EL. 45.300	Reinforcement	D32 @ 200 cm^2	D16 @ 300 cm^2
	Tensile stress	1986 < 3000 kg/cm^2	1424 < 3000 kg/cm^2
	Compressive stress	12.3 < 90	7.3 < 90 kg/cm^2
	Shear stress	2.4 < 12	0 < 12 kg/cm^2
	Bond stress	-	-
Column (2) EL. 41.100	Reinforcement	D32 @ 200 cm^2	D22 x @300 = 12.9 >11.0 cm^2
	Tensile stress	2845 < 3000 kg/cm^2	2948 < 3000 kg/cm^2
	Compressive stress	20.5 < 90 kg/cm^2	12.6 < 90 kg/cm^2
	Shear stress	4.7 < 12 kg/cm^2	0 < 12 kg/cm^2
	Bond stress	-	-
Column (2) EL. 37.100	Reinforcement	D32 @ 100 cm^2	D22 @ 150 cm^2
	Tensile stress	2904 < 3000 kg/cm^2	2804 < 3000 kg/cm^2
	Compressive stress	29.1 < 90 kg/cm^2	18.0 < 90 kg/cm^2
	Shear stress	7.1 < 12 kg/cm^2	0 < 12 kg/cm^2
	Bond stress	-	-

6.8.10 Tailrace Retaining Wall.

(a) General layout.

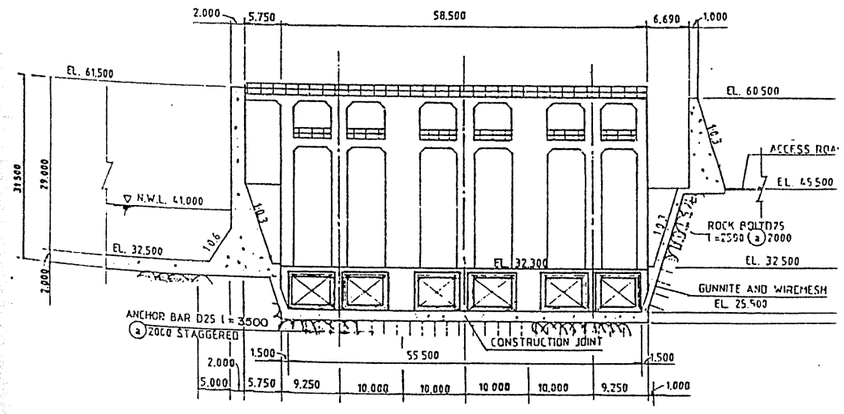
The retaining wall of the tailrace left abutment was designed for the purpose of discharging water from the tailrace to downstream in hydraulically smooth way, protecting the left abutment of the tailrace and the foundation of the access road to the powerhouse from scouring by the tailrace discharge water, and preventing the powerhouse area from submergence in flood time.

In consideration of the above condition, the retaining wall consists of the following two portions, as shown in the figure below.

(i) The leaning type wall 100 cm thick was designed from the tailbay level up to the access road level.

(ii) The gravity type wall was designed from the access road level up to the crest level of EL.60.50 m in consideration of the necessary free board above flood water level for the 200 year return period.

Fig 1.6.8-17 Typical Section of Tailrace Retaining Wall



(b) Stability calculation of leaning wall.

Design Condition

In situ - rock		Others	
Unit weight	: 2.3 t/m³	Rock bolt : D25 @ 2.000 ℓ	= 2.500 m
Internal friction angle	: 38°	Load due to gravity wall	: 22.243 t/m
Coefficient of earth pressure	: 0.083		

Total of external force and dead load

Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Y (m)	Mx (tm/m)	My (tm/m)
Wall weight	36.829	-	3.522	-	129.725	-
Earth pressure	2.774	18.341	3.200	4.667	8.877	85.597
Superimposed	3.865	25.556	3.900	7.000	15.038	178.892
Rock bolting	-9.173	-30.576	3.900	7.000	-35.775	-214.032
Total	34.295	13.321			117.865	50.457

Stability against overturning : The safety factor against overturning is larger than 1.5, so the stability against overturning is assured.

$$Fs = Mx/My = 2.33 > 1.5$$

Stability against compressive stress : Maximum compressive stress acting on the ground surface , q max was less than the allowable bearing capacity of the ground. So the stability against compressive stress was assured.

$$q \text{ max} = 4/3 \left(\frac{\sum V}{B-2|e|} \right) = 138\ t/m^2 < 150\ t/m^2$$

Stability against sliding : The safety factor against sliding was larger than 1.5 so that stability against sliding was assured.

$$Fs = \sum V \cdot \tan\varnothing \ / \ \sum H = 2.00 > 1.5$$

(c) Stability calculation of gravity wall.

Design Condition

Water Level		Backfilling Soil		Others	
FWL	: 54.82 m	Unit weight	: 1.9 t/m³	Superimposed load	: 1.0 t/m²
NWL	: 41.00 m	Internal friction angle	: 30°	Seismic intensity	: 0.10
		Coefficient of earth pressure			
		Normal	: 0.442		
		Earthquake	: 0.508		

Total external force and dead load

	Load Items	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Y (m)	Mx (tm/m)	My (tm/m)
Normal	Wall weight	125.120	-	1.982	-	248.032	-
	Earth press	10.369	13.912	5.157	2.142	53.473	29.800
	Total	135.489	13.912	-	-	301.505	29.800
Flood	Wall weight	125.120	-	1.982	-	248.032	-
	Earth press	10.369	13.912	5.157	2.142	53.473	29.800
	Hydropress	-	- 53.251	-	3.440	-	- 183.183
	Total	135.489	- 39.339	-	-	301.505	- 153.383
Earth quake	Wall weight	125.120	12.512	1.982	6.117	248.032	76.536
	Earth press	7.597	18.423	5.157	2.142	39.178	39.462
	Total	132.717	30.935	-	-	287.210	115.998

Stability against overturning : The length of foundation B is 5.8 m, then the effective point of the resultant force, x, and the eccentricity, e, are calculated as follows. From the calculation results, the effective point of the resultant force is within the middle third so that stability against overturning is assured.

$$X = (Mx - My) / \sum V \ , \ e = |B/2 - x|$$

Cases	x (m)	e (m)	B/6
Normal state	2.005	0.895	< B/6 = 0.967 m
Flood state	3.357	0.457	< B/6 = 0.967 m
Earthquake state	1.290	0.390	< B/6 = 0.967 m

Stability against compressive stress : The maximum compressive stress is less than the allowable bearing capacity of the ground. So that stability against compressive stress is assured.

$$\left.\begin{matrix} q \text{ max} \\ q \text{ min} \end{matrix}\right\} = \frac{\sum V}{B} \left(1 \pm \frac{6.e}{B}\right)$$

Cases	q max (t/m²)	q min (t/m²)	qa (t/m²)
Normal state	44.99	1.73	< 150
Flood state	34.40	12.32	< 150
Earthquake state	32.11	13.65	< 150

Stability against sliding : The safety factor Fs , against sliding is larger than 1.5 (for normal) or 1.2 (for the case of flood or earthquake) so that stability against sliding is assured.

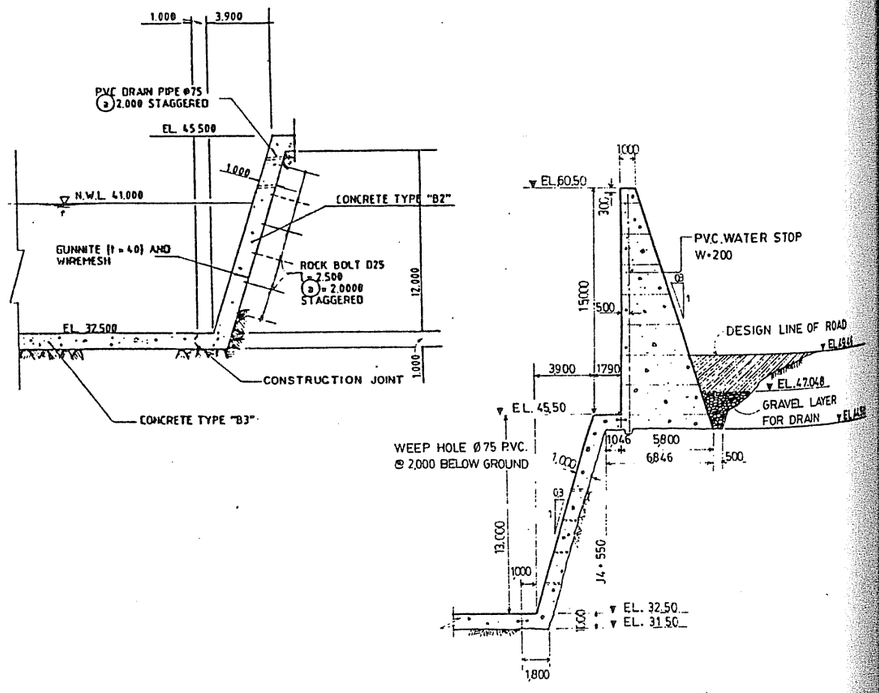
$$Fs = \sum V.tan\Phi \;/\; \sum H$$

Cases	∑V. Tan Ø	∑H	Fs	Fs min.
Normal state	105.681	13.912	7.59	> 1.5
Flood state	105.681	39.339	2.68	> 1.2
Earthquake state	103.519	30.935	3.34	> 1.2

(d) Summary of Calculation.

Stability Calculation	Leaning wall	Gravity wall		
	Normal	Normal	Flood	Earthquake
Overturning	Fs = 2.33 > 1.5	e = 0.895 m < B/6	e = 0.457 m < B/6	e = 0.390 m < B/6
Max stress (t/m²)	138 < 150 t/m²	44.99 < 150 t/m²	34.40 < 150 t/m²	32.11 < 150 t/m²
Sliding	Fs = 2.00 > 1.5	Fs = 7.59 > 1.5	Fs = 2.68 > 1.2	Fs = 3.34 > 1.2

(5) Typical section of gravity wall.



6.8.11 Excavation Slope of Erection Bay Side

In the Tender Design, the powerhouse excavation slopes of the erection bay side are 1:1 from EL 44.5 to EL 42.5 and 1:0 (vertical) from EL 42.5 to EL 21.0. The design is based on the geological investigation carried out in the detailed design stage in 1987 and the foundation rock was expected to be moderately hard (CH class to CM class).

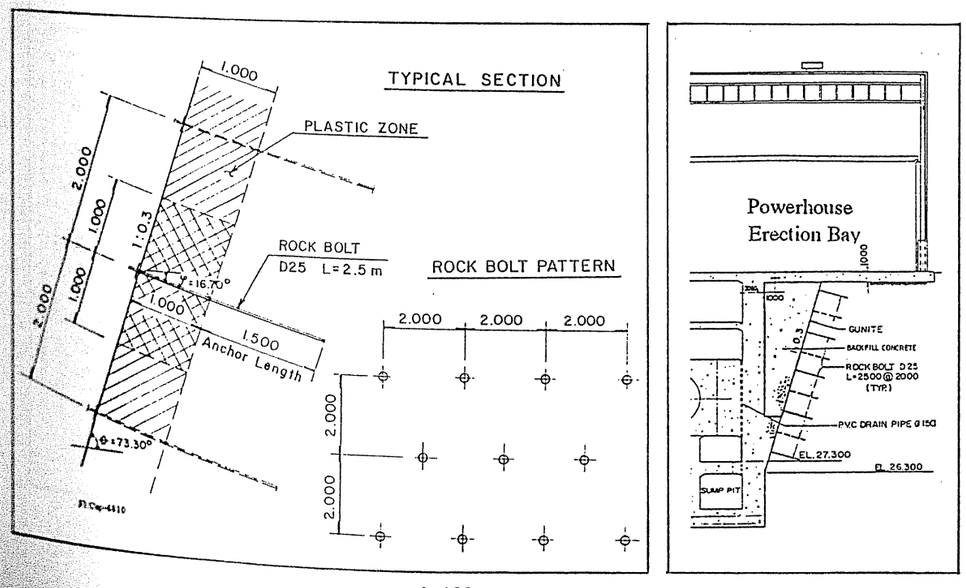
According to a geological observation during construction, however, the exposed rock was found to be partly highly weathered and jointed weak turf with abundant clay seams that is classified as CL class. Therefore, in order to avoid the surface slide of the slope during construction, the excavation slope line was modified to the slope of 1:0.3 from EL 44.5 to EL 26.3 and the rockbolt arrangement was changed to the spacing of 2.0m with 2.5m long rockbolt. In consideration of long slope excavation, the rock bolt and gunite with wire-mesh are used for the slope protection.

Rockbolt Arrangement

The open excavation of the powerhouse is carried out by blasting or bulldozer's ripper. For the slope surface is used the ripper of the bulldozer or hydraulic breaker. It is assumed that the loose zone caused by the shock of excavation would be small and the foundation area defined as plasticity zone would be approximately 1.0m from the slope surface.

The plastic zone referred to the study of the diversion tunnels which are adjacent to the powerstation (The Kastner's Elastic - Plastic Analysis in the Detailed Design Stage) shows that the zone is generally very small.

For one rockbolt (the unit weight of rock 2.3 t/m³), the shearing stress of rockbolt is $\tau = 1739\ kg/cm^2 < \tau a = 1800\ kg/cm^2$ (SD290), and the bond and pull out stresses are $\tau o = 2.25\ kg/cm^2 < \tau oa = 14\ kg/cm^2$, $\sigma s = 522\ kg/cm^2 < \sigma sa = 1800\ kg/cm^2$ respectively. Therefore, the rockbolt of $\phi 25$, $\ell = 2.5m$ was selected.



6.9 Main Power Transformer Foundation

(1) General Layout

In consideration of the available space around of dam and powerhouse area, the main power transformers and gantry pylons are located external to the powerhouse at ground level on the upstream side. The parapet wall for fire fighting and a hydrant system was installed around the transformers.

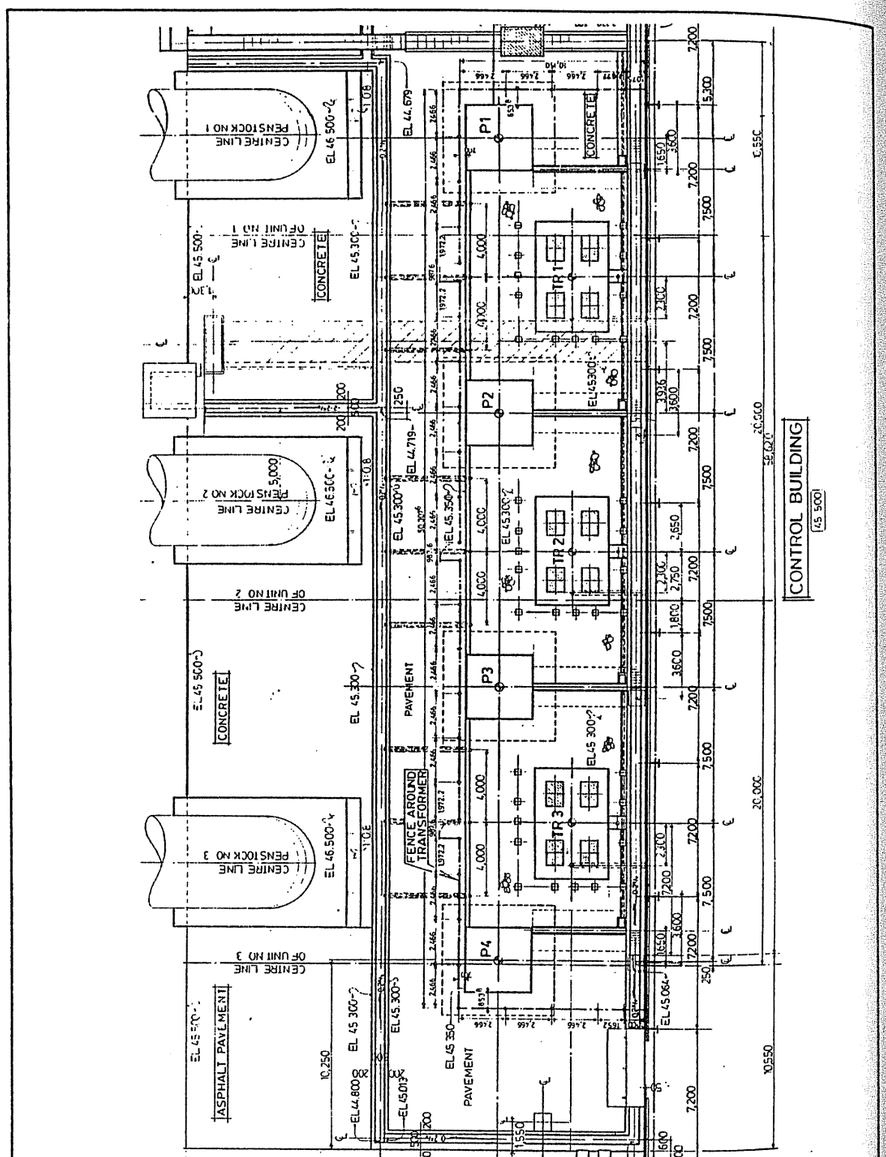


Fig. 1.6.9 General Layout of Transformer Site

(2) Design of transformer foundation

(a) Design condition

Soil condition		Other condition	
C =	0 (t/m²)	Unit weight of concrete	= 2.45 (t/m³)
γ =	1.9 (t/m³)	Unit weight of soil	= 1.9 (t/m³)
ϕ =	25	Seismic Load	= 0.1

Condition	Direction	Description	Vertical Load	Horizontal Load	Moment	
					Mx	My
Long Period	X	Foundation	35.280	0.000	0	0
		Structure	58.000	0.000	0	0
		Eccentric			0.638	6.438
		Total	93.280	0.000	0.638	6.438
	Y	Foundation	35.280	0.000	0	0
		Structure	58.000	0.000	0	0
		Eccentric			0.638	6.438
		Total	93.280	0.000	0.638	6.438
Short Period	X	Foundation	35.280	3.528	1.058	0
		Structure	58.000	5.800	13.63	0
		Eccentric			0.638	6.438
		Total	93.280	9.328	15.326	6.438
	Y	Foundation	35.280	3.528	0	1.058
		Structure	58.000	5.800	0	13.63
		Eccentric			0.638	6.438
		Total	93.280	9.328	0.638	21.126

(b) Stability analysis

Long Period (Eccentric Distance)		Short Period (Eccentric Distance)	
X Direction	ex = ΣMy / Σv = 0.069	X Direction	ex = ΣMy / Σv = 0.164
	ey = ΣMx / Σv = 0.007		ey = ΣMx / Σv = 0.069
Y Direction	ex = ΣMy / Σv = 0.069	Y Direction	ex = ΣMy / Σv = 0.007
	ey = ΣMx / Σv = 0.007		ey = ΣMx / Σv = 0.226

Judgement of stability

Long Period		Short Period	
X Direction	6.ex/L + 6.ey/B = 0.11 < 1	X Direction	3.ex/L + 3.ey/B = 0.16 < 1
Y Direction	6.ex/L + 6.ey/B = 0.11 < 1	Y Direction	3.ex/L + 3.ey/B = 0.12 < 1

(c) Bending moment and shearing stress

Condition	Direction	Bending Moment		Shearing Stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x Direction	-	1.406	-	2.629
Long Period	y Direction	-	2.631	-	3.759

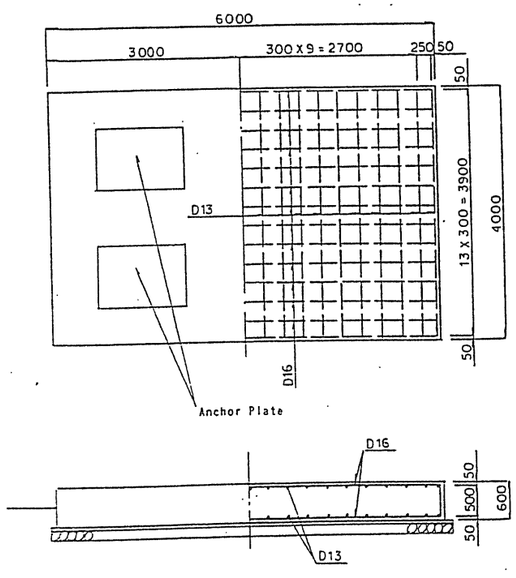
(d) Calculation of reinforcement bar

X Direction Bottom bar	Y Direction Bottom bar
As1=M/(δsa.j.d)=2.976 D13@300 = 4.223 D13@300 = 4.223 cm 2 >2.976 cm 2 (OK)	As2=M/(δsa.j.d)=5.568 D16@300 = 6.55 D16@300 = 6.554 cm 2 >5.568 cm 2 (OK)

(e) Calculation of unit stress of concrete and reinforcement

Concrete (δc)	Reinforcement Bar ((δs)	Shearing Stress (τ)	Bond Stres
X direction Bottom δc=18.005<70 OK Y Direction Bottom δc=27.968<70 OK	X Direction Bottom δs=1183.151<1800 OK Y Direction Bottom δs=1446.61<1800 OK	X Direction Bottom τ=0.934<4 OK Y Direction Bottom τ=1.355<4 OK	X Direction Bottom τ0=5.2 Y Direction Bottom τ0=7.5

(f) Arrangement of reinforce bar



(3) Design of Gantry Pylon

(a) Design condition (see (2) Design of transformer foundation on soil condition)

Condition	Direction	Description	Vertical Load	Horizontal Load		Moment	
				Hx	Hy	Mx	My
Long Period	X	Foundation	232.459	0	0	0	0
		Structure	10.343	4.213	3.57	108.63	101.98
		Total	242.802	4.213	3.57	108.63	101.98
	Y	Foundation	232.459	0	0	0	0
		Structure	10.343	1.785	6.293	155.856	54.264
		Total	242.802	1.785	6.293	155.856	54.264
Short Period	X	Foundation	232.459	23.246	0	0	31.671
		Structure	10.343	0	0	0	0
		Total	242.802	23.246	0	0	31.671
	Y	Foundation	232.459	0	23.246	31.671	0
		Structure	10.343	0	0	0	0
		Total	242.802	0	23.246	31.671	0

(b) Stability analysis

Long Period (Eccentric Distance)		Short Period (Eccentric Distance)	
X Direction	ex = ΣMy / Σv = 0.42 ey = ΣMx / Σv = 0.447	X Direction	ex = ΣMy / Σv = 0.13 ey = ΣMx / Σv = 0
Y Direction	ex = ΣMy / Σv = 0.223 ey = ΣMx / Σv = 0.642	Y Direction	ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.13

Judgement of stability

Long Period	Short Period
X Direction 6.ex/L+6.ey/B = 0.87<1 OK Y Direction 6.ex/L+6.ey/B = 0.87 OK	X Direction 3.ex/L + 3.ey/B = 0.07 < 1 OK Y Direction 3.ex/L + 3.ey/B = 0.07 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction		Bending Moment		Shearing Stress	
			Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x - x	x Direction	- 1.471	2.607	- 2.452	4.346
		y Direction	- 1.605	2.741	- 2.674	4.568
	y - y	x Direction	- 0.517	1.653	- 0.862	2.756
		y Direction	- 2.549	3.685	- 4.248	6.143

Because of unfavorable topography of the very steep slope near the dam site there was difficulty constructing the cable duct on the original route between the national road and dam site which was found after the start of excavation for dam access road near the dam site.

The left bank of the dam site generally consists of steep topography and there is no special good site, however, the alternative route is topographically rather easier than the other routes because the existing temporally access road to cable crane operation house was available for the upper part of the alignment. Also the length of cable duct is nearly the same as the original alignment. Therefore, the alternative was chosen as an optimum alignment. Water supply pipe from dam site to switchyard is also installed along the cable duct alignment.

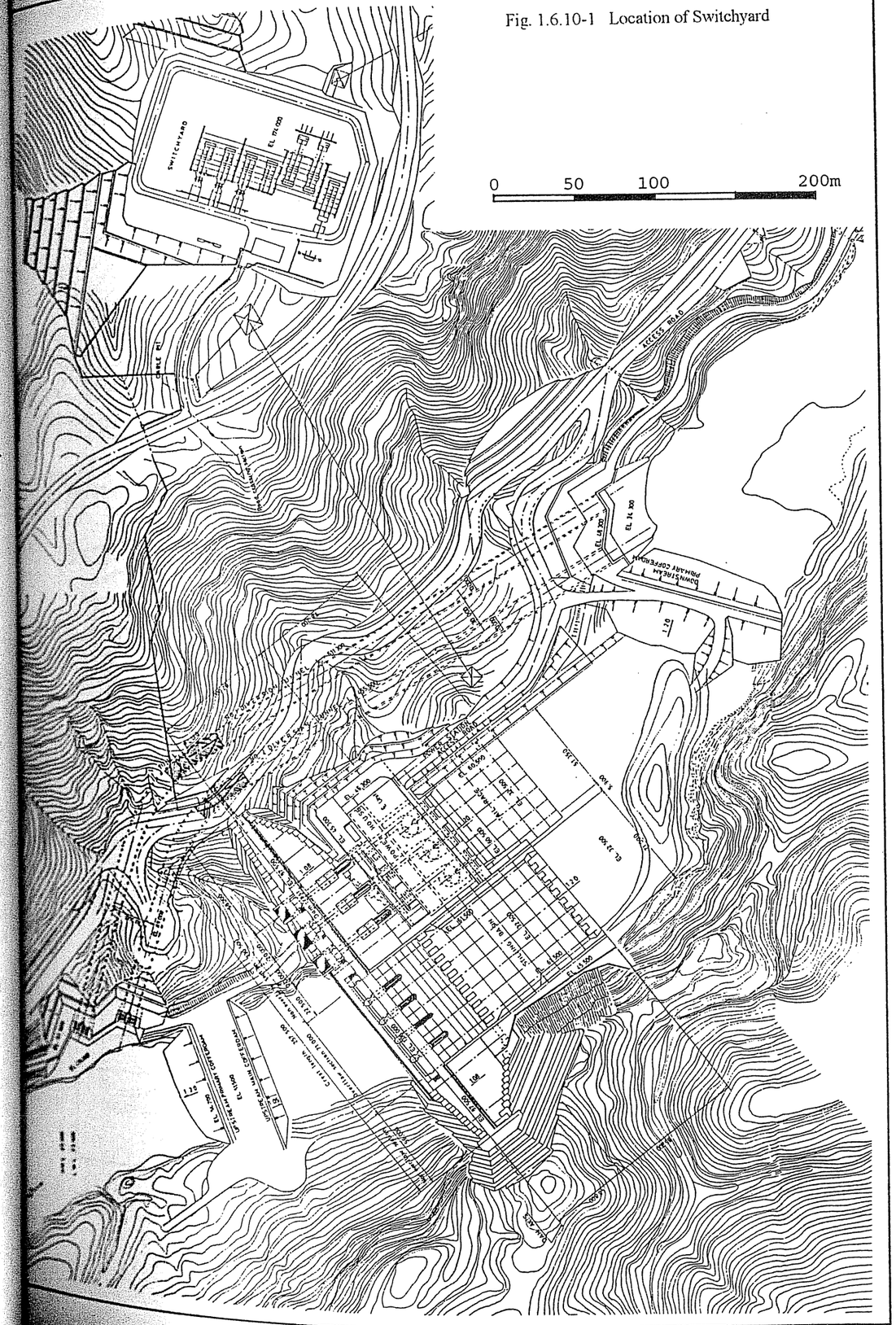
In the Tender Design, a steel plate cover was proposed to cover the whole duct except beside the switchyard access road and crossing of national road. It was decided, however, that especially considering risk of theft, a concrete cover was used for the entire length except at each 10m interval where a steel plate stopper was placed in consideration of maintenance and repairing. The thickness of steel plate cover originally 3.6 mm was changed to 6.0 mm in consideration of mans weight for maintenance and repairing.

For the switchyard design, the civil works were divided into the following structures:

		Position
-	Gantry pylon type I	: G2, G6
-	Gantry pylon type II	: G1, G3, G4
-	Gantry pylon type III	: G5
-	Main Transformer	: T1, T2, T3
-	Capacitive transformer	: T4, T5, T6, T16, T17, T25, T26, T27
-	Current transformer	
-	Aux. transformer	
-	Lighting aerator	: F1
-	Post insulator	: P1
-	Circuit breaker	: Q0
-	Disconnecting switch	: Q1, Q2, Q25, Q15, Q9/Q8
-	Neutral resister	: R1/T12
-	STR for 20 kV cable termination.	

Note : The position mark mentioned above is shown on the As-built drawings.

Fig. 1.6.10-1 Location of Switchyard



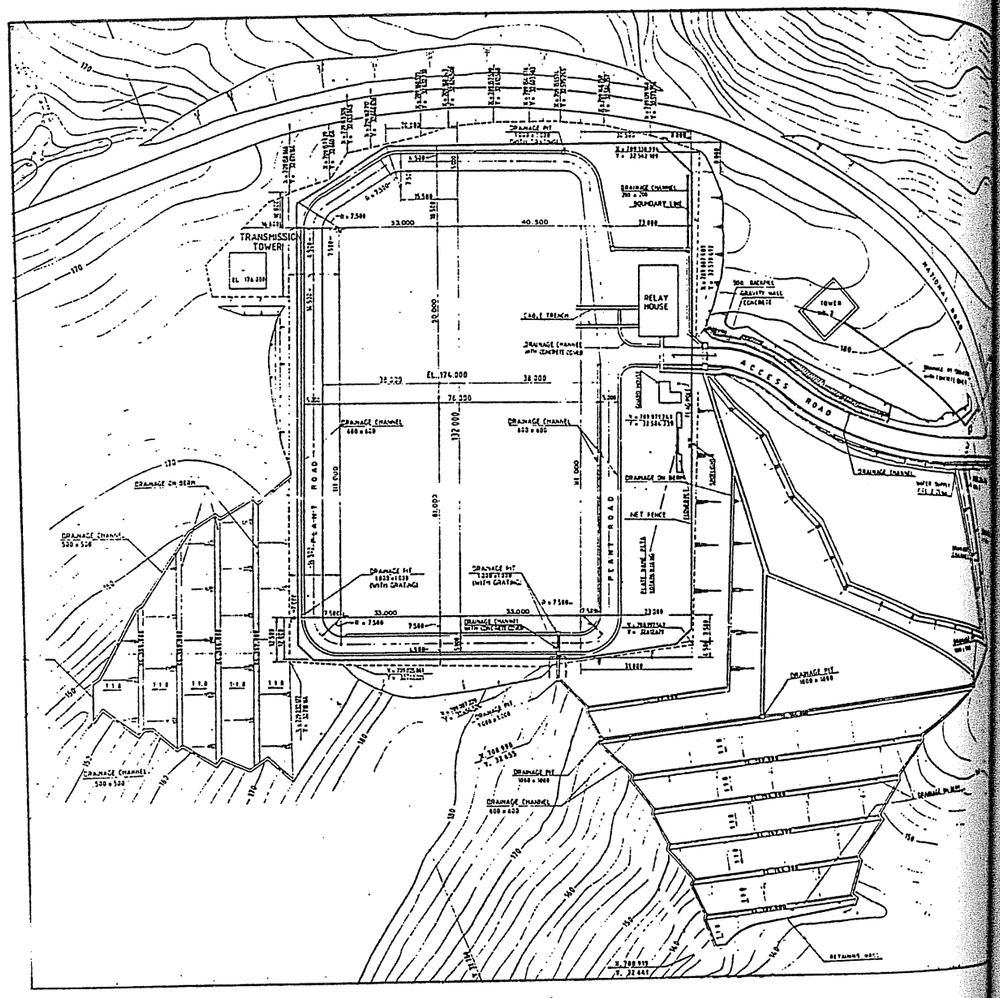


Fig. 1.6.10-2 General Layout of Switchyard

6.10.2 Foundation Design of Gantry Pylon

(1) Gantry Pylon Type 1

(a) Design condition

Soil condition (from Geological Report)		Other condition	
N =	10	Unit weight of concrete	= 2.45 (t/m³)
C =	6 (t/m²)	Unit weight of soil	= 1.65 (t/m³)
γ =	1.65 (t/m²)	Seismic Load	= 0.1
ϕ =	0		

Condition	Direction	Description	Vertical load	Horizontal load		Moment	
				Hx	Hy	Mx	My
Long Period	x	Foundation	43.366	0	0	0	0
		Structure	1.754	1.693	0	0	13.576
		Total	45.12	1.693	0	0	13.576
	y	Foundation	43.366	0	0	0	0
		Structure	1.754	1.224	0.949	6.926	10.649
		Total	45.12	1.224	0.949	6.926	10.649
Short Period	x	Foundation	43.366	4.337	0	0	3.108
		Structure	1.754	2.295	0	0	19.972
		Total	45.12	6.632	0	0	23.08
	y	Foundation	43.366	0	4.337	3.108	0
		Structure	1.754	2.295	0	0	19.972
		Total	45.12	2.295	4.337	3.108	19.972

(b) Stability analysis

Long period (Eccentric distance)		Short period (Eccentric distance)	
x Direction	ex = ΣMy / Σv = 0.301	x Direction	ex = ΣMy / Σv = 0.512
	ey = ΣMx / Σv = 0		ey = ΣMx / Σv = 0
y Direction	ex = ΣMy / Σv = 0.236	y Direction	ex = ΣMy / Σv = 0.443
	ey = ΣMx / Σv = 0.154		ey = ΣMx / Σv = 0.069

Judgement of stability

Long period		Short period	
x Direction	6.ex/L + 6.ey/B = 0.44 < 1 OK	x Direction	3.ex/L + 6.ey/B = 0.37 < 1 OK
y Direction	6.ex/L + 6.ey/B = 0.63 < 1 OK	y Direction	3.ex/L + 3.ey/B = 0.39 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction		Bending Moment		Shearing Stress	
			Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x - x	x Direction	- 0.473	0.794	- 1.182	1.984
		y Direction	-	0.161	-	0.402
	y - y	x Direction	- 0.336	0.657	- 0.84	1.642
		y Direction	- 0.084	0.406	- 0.211	1.014

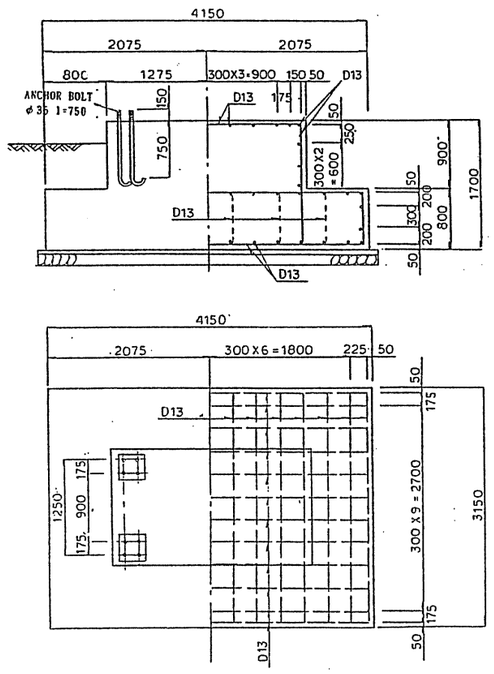
(d) Calculation of reinforcement content

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=1.201 D13@300 = 4.223 D13@300=4.223 cm² > 1.201 cm² (OK)	Top bar As1 = M/(σsa.j.d) = 0.213 D13@300 = 4.223 D13@300 = 4.223 cm² > 0.213 cm² (OK)
Bottom bar As2=M/(σsa.j.d)=2.017 D13@300 = 4.223 D13@300 = 4.223 cm² >2.017 cm² (OK)	Bottom bar As2 = M/(σsa.j.d) = 1.031 D13@300 = 4.223 D13@300 = 4.223 cm² >1.031 cm² (OK)

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement bar ((σs)	Shearing stress (τ)
x direction Top bar σc=12.64<70 OK Bottom bar σc=21.849< 70 OK y Direction Top bar σc=2.245<70 OK Bottom bar σc=10.849<70 OK	x Direction Top bar σs =155.888<1800 OK Bottom bar σs =261.681<1800 OK y Direction Top bar σs = 27.684 < 1800 OK Bottom bar σs =133.807<1800 OK	x Direction Top bar τ = 0.165<4 OK Bottom bar τ =0.276<4 OK y Direction Top bar τ = 0.029 < 4 OK Bottom bar τ =0.141<4 OK

(f) Arrangement of reinforce bar



(2) Gantry Pylon Type 2

(a) Design condition (See (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load		Moment	
				Hx	Hy	Mx	My
Long period	x	Foundation	118.821	0	0	0	0
		Structure	4.529	3.815	3.876	51.775	49.205
		Total	123.35	3.815	3.876	51.775	49.205
	y	Foundation	118.821	0	0	0	0
		Structure	4.529	1.938	4.57	55.039	25.888
		Total	123.35	1.938	4.57	55.039	25.888
Short period	x	Foundation	118.821	11.882	0	0	10.356
		Structure	4.529	0.255	0	0	4.519
		Total	123.35	12.137	0	0	14.875
	y	Foundation	118.821	0	11.882	10.356	0
		Structure	4.529	0.255	0	0	4.519
		Total	123.35	0.255	11.882	10.356	4.519

(b) Stability analysis

Long period (Eccentric distance)		Short period (Eccentric distance)	
x Direction	ex = ΣMy / Σv = 0.399 ey = ΣMx / Σv = 0.42	x Direction	ex = ΣMy / Σv = 0.121 ey = ΣMx / Σv = 0
y Direction	ex = ΣMy / Σv = 0.21 ey = ΣMx / Σv = 0.446	y Direction	ex = ΣMy / Σv = 0.037 ey = ΣMx / Σv = 0.084

Judgement of stability

Long period		Short period	
x Direction	6.ex/L + 6.ey/B = 0.92 < 1 OK	x Direction	3.ex/L + 6.ey/B = 0.08 < 1 OK
y Direction	6.ex/L + 6.ey/B = 0.71 < 1 OK	y Direction	3.ex/L + 3.ey/B = 0.07 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction		Bending moment		Shearing stress	
			Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x - x	x Direction	- 2.1	3.192	- 2.387	3.627
		y Direction	- 2.769	3.801	- 3.238	4.446
	y - y	x Direction	- 0.846	1.939	- 0.961	2.203
		y Direction	- 2.977	4.009	- 3.481	4.689

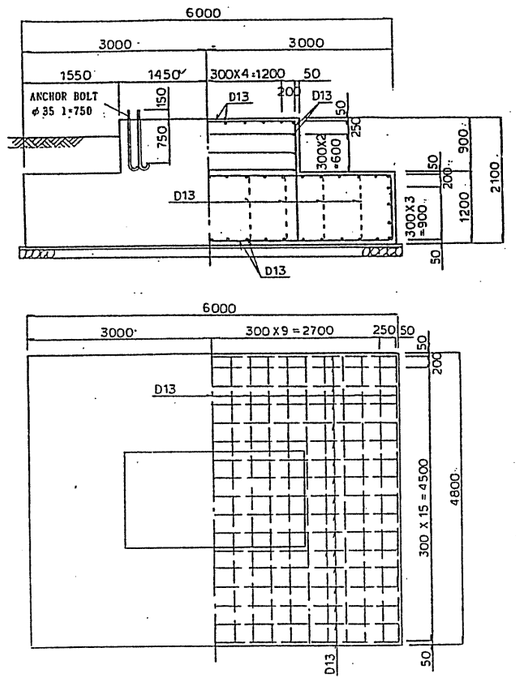
(d) Calculation of reinforcement

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=1.159 D13@300 = 4.223 D13@300=4.223 cm² > 1.159 cm² OK	Top bar As1 = M/(σsa.j.d) = 1.644 D13@300 = 4.223 D13@300 = 4.223 cm² > 1.644 cm² OK
Bottom bar As2=M/(σsa.j.d)=1.762 D13@300 = 4.223 D13@300 = 4.223 cm² >1.762 cm² OK	Bottom bar As2 = M/(σsa.j.d) = 2.213 D13@300 = 4.223 D13@300 = 4.223 cm² >2.213 cm² OK

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement bar (σs)	Shearing stress (τ)
x direction Top bar σc=3.164 < 70 OK Bottom bar σc=4.81 < 70 OK	x Direction Top bar σs=448.098 <1800 OK Bottom bar σs=681.109 <1800 OK	x Direction Top bar τ= 0.215 <4 OK Bottom bar τ=0.327<4 OK
y Direction Top bar σc=4.486 < 70 OK Bottom bar σc=6.041 < 70 OK	y Direction Top bar σs= 635.232 < 1800 OK Bottom bar σs=855.441<1800 OK	y Direction Top barτ= 0.314 <4 OK Bottom bar τ=0.423<4 OK

(f) Arrangement of reinforce bar



(3) Gantry Pylon Type 3

(a) Design condition (See (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load		Moment	
				Hx	Hy	Mx	My
Long Period	x	Foundation	87.666	0	0	0	0
		Structure	6.691	4.274	1.428	19.921	39.556
		Total	94.357	4.274	1.428	19.921	39.556
	y	Foundation	87.666	0	0	0	0
		Structure	6.691	2.448	3.825	41.626	21.298
		Total	94.357	2.448	3.825	41.626	21.298
Short Period	x	Foundation	87.666	8.767	0	0	7.386
		Structure	6.691	3.203	0	0	27.866
		Total	94.357	11.97	0	0	35.252
	y	Foundation	87.666	0	8.767	7.386	0
		Structure	6.691	0	2.887	38.444	0
		Total	94.357	0	11.654	45.83	0

(b) Stability analysis

Long period (Eccentric distance)		Short period (Eccentric distance)	
x Direction	ex = ΣMy / Σv = 0.419 ey = ΣMx / Σv = 0.211	x Direction	ex = ΣMy / Σv = 0.374 ey = ΣMx / Σv = 0
y Direction	ex = ΣMy / Σv = 0.226 ey = ΣMx / Σv = 0.441	y Direction	ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.486

Judgement of stability

Long period		Short period	
x Direction	6.ex/L + 6.ey/B = 0.8 < 1 OK	x Direction	3.ex/L + 6.ey/B = 0.24 <1 OK
y Direction	6.ex/L + 6.ey/B = 0.85< 1 OK	y Direction	3.ex/L + 3.ey/B = 0.31 < 1 OK

(c) Table of bending moment and shearing stress

Condition	Direction		Bending moment		Shearing stress	
			Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x - x	x Direction	- 0.465	0.997	- 1.163	2.494
		y Direction	- 0.103	0.634	- 0.256	1.586
	y - y	x Direction	- 0.128	0.66	- 0.319	1.65
		y Direction	- 0.504	1.036	- 1.259	2.59

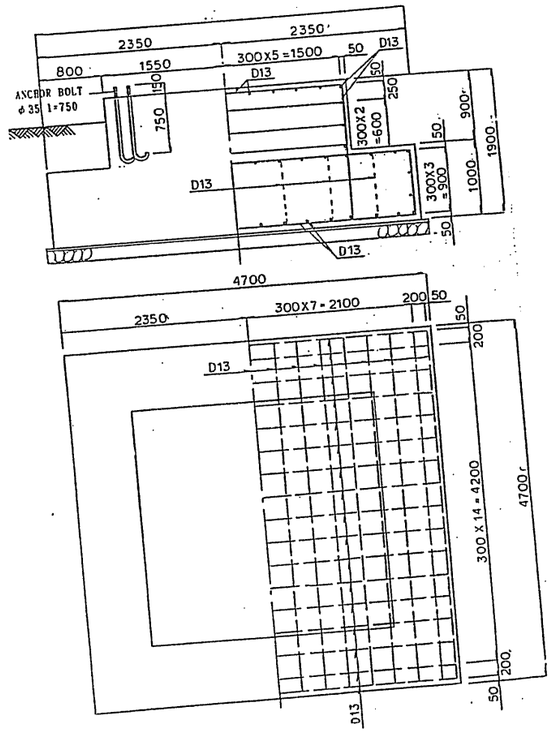
(d) Calculation of reinforcement content

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=0.311 D13@300 = 4.223 D13@300=4.223 cm² > 0.331 cm² OK Bottom bar As2=M/(σsa.j.d)=0.666 D13@300 = 4.223 D13@300 = 4.223 cm² >0.666 cm² OK	Top bar As1 = M/(σsa.j.d) = 0.337 D13@300 = 4.223 D13@300 = 4.223 cm² > 0.337 cm² OK Bottom bar As2 = M/(σsa.j.d) = 0.067 D13@300 = 4.223 D13@300 = 4.223 cm² >0.067 cm² OK

(e) Calculation of unit stresses of concrete and reinforcement

Concrete (σc)	Reinforcement bar (σs)	Shearing stress (τ)
x direction Top bar σc=1.027 < 70 OK Bottom bar σc=2.201 < 70 OK y Direction Top bar σc=1.113 < 70 OK Bottom bar σc=2.288 < 70 OK	x Direction Top bar σs=120.11 <1800 OK Bottom bar σs=257.527 <1800 OK y Direction Top bar σs= 130.184 < 1800 OK Bottom bar σs=267.601<1800 OK	x Direction Top bar τ= 0.127 <4 OK Bottom bar τ=0.272 <4 OK y Direction Top bar τ= 0.137 <4 OK Bottom bar τ=0.283<4 OK

(f) Arrangement of reinforce bar



6.10.3 Foundation Design of Main Transformer

(1) Main Transformer

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load	Moment	
					Mx	My
Long Period	x	Foundation	26.235	0	0	0
		Structure	31.000	0	0	0
		Eccentric		0	0	0
		Total	57.235	0	0	0
	y	Foundation	26.235	0	0	0
		Structure	31.000	0	0	0
		Eccentric		0	0	0
		Total	57.235	0	0	0
Short Period	x	Foundation	26.235	2.624	0.27	0
		Structure	31.000	3.1	0.513	0
		Eccentric			8.711	0
		Total	57.235	5.724	9.494	0
	y	Foundation	26.235	2.624	0	0.27
		Structure	31.000	3.1	0	0.513
		Eccentric			0	8.711
		Total	57.235	5.724	0	9.494

(b) Stability analysis

Check for overturning

Long period (Eccentric distance)	Short period (Eccentric distance)
x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0	x Direction ex = ΣMy / Σv = 0.166 ey = ΣMx / Σv = 0
y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0	y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.166

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0 < 1 OK y Direction 6.ex/L + 6.ey/B = 0 < 1 OK	x Direction 3.ex/L + 6.ey/B = 0.18 <1 OK y Direction 3.ex/L + 3.ey/B = 0.09 <1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x Direction	-	2.629	-	3.009
Long Period	y Direction	-	0.733	-	1.589
Short Period	x Direction	-	3.596	-	4.115
Exchange to long period		0	2.397	0	2.743
Short Period	y Direction	-	1.107	-	2.399
Exchange to long period		0	0.738	0	1.599

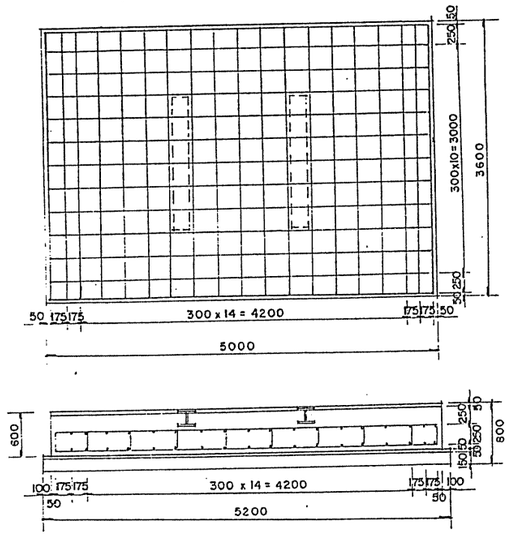
(d) Calculation of reinforcement content

x Direction	y Direction
Bottom bar As1=M/(σsa.j.d)=5.564 D16@300 = 5.564 D16@300 = 6.554 cm² >5.564 cm² OK	Bottom bar As2 = M/(σsa.j.d) = 1.562 D16@300 = 6.554 D16@300 = 6.554 cm² >1.562 cm² OK

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement bar (σs)	Shearing (τ)	Bond (τ0)
x direction Bottom bar σc=27.947 < 70 OK	x Direction Bottom bar σs=1445.51<1800 OK	x Direction Bottom bar τ=1.084 <4 OK	x Direction Bottom bar τ0=6.019 <14 OK
y Direction Bottom bar σc=7.845 < 70 OK	y Direction Bottom bar σs=405.777<1800 OK	y Direction Bottom bar τ=0.576<4 OK	y Direction Bottom bar τ0=3.198<14 OK

(f) Arrangement of reinforce bar



(2) Foundation Design of Capacitive Transformer

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load	Moment	
					Mx	My
Long Period	x	Foundation	5.308	-	-	-
		Structure	0.581	0.163	-	0.377
		Total	5.889	0.163	-	0.377
	y	Foundation	5.308	-	-	-
		Structure	0.581	-	-	-
		Total	5.889	-	-	-
Short Period	x	Foundation	5.308	0.531	-	0.223
		Structure	0.581	0.592	-	1.01
		Total	5.889	1.123	-	1.233
	y	Foundation	5.308	0.531	0.223	-
		Structure	0.581	0.173	0.643	-
		Total	5.889	0.704	0.866	-

(b) Stability analysis

Long period (Eccentric distance)	Short period (Eccentric distance)
x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.064 y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0	x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.209 y Direction ex = ΣMy / Σv = 0.147 ey = ΣMx / Σv = 0

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0.23 < 1 OK y Direction 6.ex/L + 6.ey/B = 0 < 1 OK	x Direction 3.ex/L + 6.ey/B = 0.37 < 1 OK y Direction 3.ex/L + 3.ey/B = 0.26 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long period	x Direction	-	0.049	0.054	0.33
Short period	x Direction	0.039	0.096	0.259	0.644
Exchange to long period		0.026	0.064	0.173	0.429

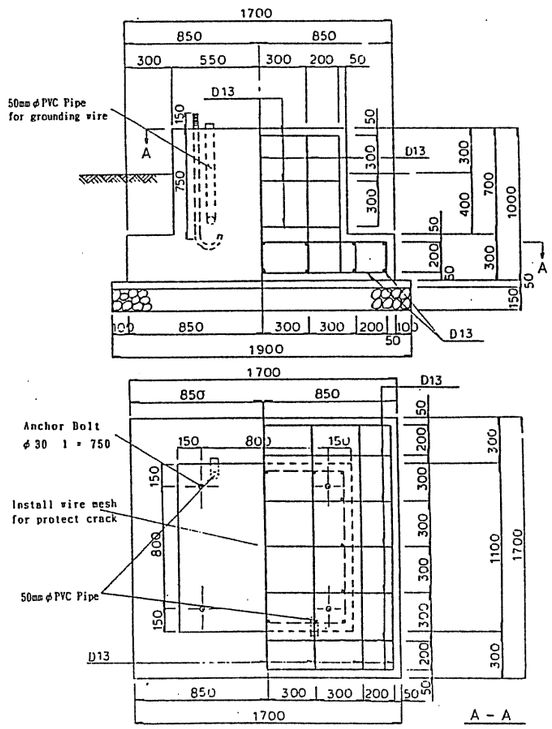
(d) Calculation of reinforcement bar

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=0.066 D13@300 = 4.223 D13@300 = 4.223 cm^2 >0.163 cm^2 OK	Bottom bar As2 = M/(σsa.j.d) = 0.163 D13@300 = 4.223 D16@300 = 4.223 cm^2 >0.163 cm^2 OK

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement bar (σs)	Shearing	Bond (τ0)
x direction Top bar σc=0.441 < 70 OK y Direction Bottom bar σc=1.887< 70 OK	x Direction Top bar σs=26.396<1800 OK y Direction Bottom bar σs=64.934<1800 OK	x Direction Top bar τ=0.074 <4 OK y Direction Bottom bar τ=0.184<4 OK	x Direction Top bar τ0=0.556 <14 OK y Direction Bottom bar τ0=1.38<14 OK

(f) Arrangement of reinforce bar



(3) Foundation Design of Current Transformer

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load	Moment	
					Mx	My
Long Period	x	Foundation Structure Total	5.308 0.938 6.246	- - -	- - -	- - -
	y	Foundation Structure Total	5.308 0.938 6.246	- 0.245 0.245	- 0.724 0.724	- - -
Short Period	x	Foundation Structure Total	5.308 0.938 6.246	0.531 0.173 0.704	- - -	0.223 0.663 0.886
	y	Foundation Structure Total	5.308 0.938 6.246	0.531 0.979 1.51	0.223 2.764 2.987	- - -

(b) Stability analysis

Long period (Eccentric distance)	Short period (Eccentric distance)
x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0 y Direction ex = ΣMy / Σv = 0.116 ey = ΣMx / Σv = 0	x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.142 y Direction ex = ΣMy / Σv = 0.478 ey = ΣMx / Σv = 0

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0 < 1 OK y Direction 6.ex/L + 6.ey/B = 0.41 < 1 OK	x Direction 3.ex/L + 6.ey/B = 0.25 < 1 OK y Direction 3.ex/L + 3.ey/B = 0.84 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long period	y Direction	0.006	0.074	0.036	0.495
Short period	y Direction	0.13	0.198	0.865	1.324
Exchange to long period		0.087	0.132	0.577	0.883

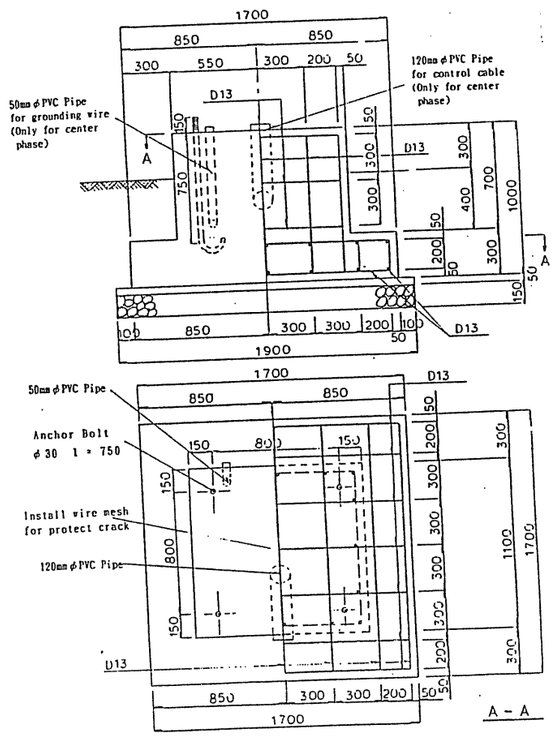
(d) Calculation of reinforcement content

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=0.221 D13@300 = 4.223 D13@300 = 4.223 cm² >0.221 cm² OK	Bottom bar As2 = M/(σsa.j.d) = 0.335 D13@300 = 4.223 D16@300 = 4.223 cm² >0.335 cm² OK

(e) Calculation of unit stresses of concrete and reinforcement

Concrete (σc)	Reinforcement bar (σs)	Shearing (τ)	Bond (τ0)
x direction Top bar σc=1.477 < 70 OK y Direction Bottom bar σc=2.241< 70 OK	x Direction Top bar σs=88.324<1800 OK y Direction Bottom bar σs=134.088<1800 OK	x Direction Top bar τ=0.247 <4 OK y Direction Bottom bar τ=0.379<4 OK	x Direction Top bar τ0=1.855 <14 OK y Direction Bottom bar τ0=2.84<14 OK

(f) Arrangement of reinforce bar



(4) Foundation Design of Auxiliary Transformer

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical Load	Horizontal Load	Moment	
					Mx	My
Long Period	X	Foundation	3.528	0	0	0
		Structure	1.500	0	0	0
		Eccentric		0	0	0
		Total	5.028	0	0	0
	Y	Foundation	3.528	0	0	0
		Structure	1.500	0	0	0
		Eccentric		0	0	0
		Total	5.028	0	0	0
Short Period	X	Foundation	3.528	0.353	0	0.106
		Structure	1.500	0.15	0	0.128
		Eccentric			0	0
		Total	5.028	0.503	0	0.234
	Y	Foundation	3.528	0.353	0.106	0
		Structure	1.500	0.15	0.128	0
		Eccentric			0	0
		Total	5.028	0.503	0.234	0

(b) Stability Analysis

Long Period (Eccentric Distance)	Short Period (Eccentric Distance)
X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0	X Direction ex = ΣMy / Σv = 0.047 ey = ΣMx / Σv = 0
Y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0	Y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.047

Judgement of Stability

Long Period	Short Period
X Direction 6.ex/L + 6.ey/B = 0 < 1 OK	X Direction 3.ex/L + 6.ey/B = 0.05 < 1 OK
Y Direction 6.ex/L + 6.ey/B = 0 < 1 OK	Y Direction 3.ex/L + 3.ey/B = 0.02 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long Period	x Direction y Direction	0.019 0.307	0.019 0.307	0.156 0.619	0.156 0.619
Short period Exchange to long period Short period Exchange to long period	x Direction y Direction	0.001 0.001 0.187 0.125	0.038 0.025 0.426 0.284	0.01 0.007 0.377 0.251	0.302 0.201 0.861 0.574

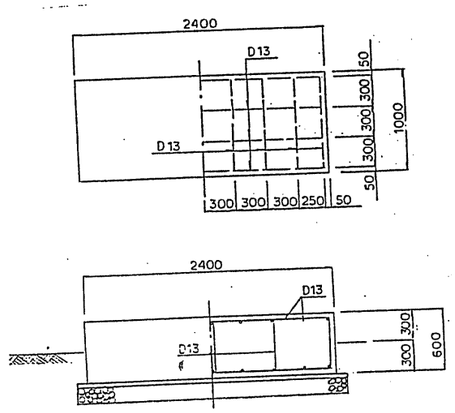
(d) Calculation of reinforcement content

x Direction	y Direction
Bottom bar As1=M/(σsa.j.d)=0 D13@300 = 4.223 D13@300 = 4.223 cm^2 >0 cm^2 OK	Bottom bar As1=M/(σsa.j.d)=0.029 D13@300 = 4.223 D13@300 = 4.223 cm^2 >0.029 cm^2 OK

(e) Check of unit stress of reinforcement bar

Concrete (σc)	Reinforcement bar (σs)	Shearing (τ)	Bond (τ0)
x direction Top bar σc=0< 70 OK Bottom bar σc=2.227 < 70 OK	x Direction Top bar σs=0<1800 OK Bottom bar σs=169.69<1800 OK	x Direction Top bar τ=0<4 OK Bottom bar τ=0.144 <4 OK	x Direction Top bar τ0=0<14 OK Bottom bar τ0=1.032<14 OK

(f) Arrangement of reinforce bar



6.10.4 Foundation Design of Other Civil Structure

(1) Foundation Design of Lightning Arrestor

(a) Design Condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical Load	Horizontal Load	Moment	
					Mx	My
Long Period	X	Foundation Structure Total	2.874 0.296 3.17	- - -	- - -	- - -
	Y	Foundation Structure Total	2.874 0.296 3.17	- 0.133 0.133	- 0.347 0.347	- - -
Short Period	X	Foundation Structure Total	2.874 0.296 3.17	0.287 0.173 0.46	- - -	0.114 0.653 0.767
	Y	Foundation Structure Total	2.874 0.296 3.17	0.287 0.326 0.613	0.114 0.734 0.848	- - -

(b) Stability analysis

Long Period (Eccentric Distance)	Short Period (Eccentric Distance)
X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0 Y Direction ex = ΣMy / Σv = 0.109 ey = ΣMx / Σv = 0	X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.242 Y Direction ex = ΣMy / Σv = 0.268 ey = ΣMx / Σv = 0

Judgement of stability

Long Period	Short Period
X Direction 6.ex/L + 6.ey/B = 0 < 1 OK	X Direction 3.ex/L + 6.ey/B = 0.56 < 1 OK
Y Direction 6.ex/L + 6.ey/B = 0.5 <1 OK	Y Direction 3.ex/L + 3.ey/B = 0.62 < 1 OK

(c) Table of bending moment and shearing stress

Condition	Direction	Bending Moment		Shearing Stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long Period Long Period *	x Direction y Direction	0.021 0.083 0.055	0.064 0.126 0.084	0.141 0.551 0.367	0.428 0.839 0.559

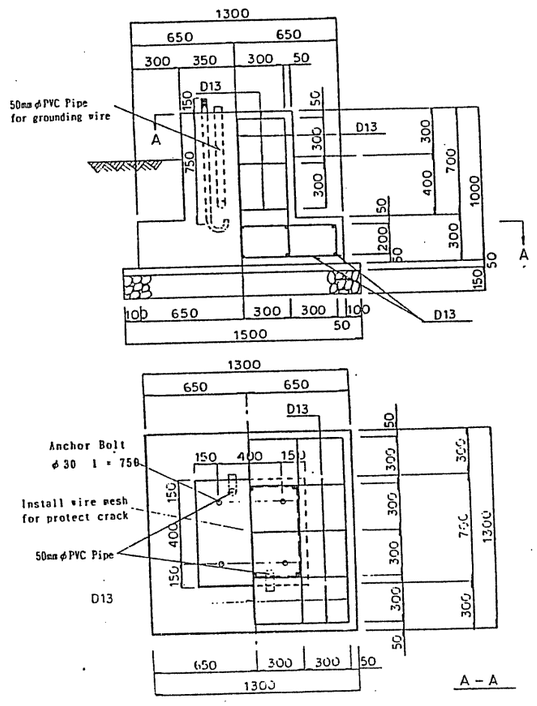
(d) Calculation of Reinforcement Content

X Direction	Y Direction
Top bar As1=M/(σsa.j.d)=0.14 D13@300 = 4.223 D13@300 = 4.223 cm^2 >0.140 cm^2 OK	Bottom bar As2 = M/(σsa.j.d) = 0.213 D13@300 = 4.223 D16@300 = 4.223 cm^2 >0.213 cm^2 OK

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement Bar (σs)	Shearing (τ)	Bond (τ0)
X direction Top bar σc=0.934 < 70 OK Y Direction Bottom bar σc=1.426< 70 OK	X Direction Top bar σs=55.839<1800 OK Y Direction Bottom bar σs=85.278<1800 OK	X Direction Top bar τ=0.187 <4 OK Y Direction Bottom bar τ=0.24<4 OK	X Direction Top bar τ0=1.184 <14 OK Y Direction Bottom bar τ0=1.748<14 OK

(f) Arrangement of reinforce bar



(2) Foundation Design of Post Insulator

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical Load	Horizontal Load	Moment	
					Mx	My
Long Period	X	Foundation	2.874	-	-	-
		Structure	0.245	-	-	-
		Total	3.119	-	-	-
	Y	Foundation	2.874	-	-	-
		Structure	0.245	0.1.84	0.551	-
		Total	3.119	0.184	0.551	-
Short Period	X	Foundation	2.874	0.287	-	0.114
		Structure	0.245	0.173	-	0.694
		Total	3.119	0.46	-	0.808
	Y	Foundation	2.874	0.287	0.114	-
		Structure	0.245	0.337	0.867	-
		Total	3.119	0.624	0.981	-

(b) Stability Analysis

Long Period (Eccentric Distance)	Short Period (Eccentric Distance)
X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0 Y Direction ex = ΣMy / Σv = 0.177 ey = ΣMx / Σv = 0	X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.259 Y Direction ex = ΣMy / Σv = 0.315 ey = ΣMx / Σv = 0

Judgement of Stability

Long Period	Short Period
X Direction 6.ex/L + 6.ey/B = 0 < 1 OK	X Direction 3.ex/L + 6.ey/B = 0.6 < 1 OK
Y Direction 6.ex/L + 6.ey/B = 0.82 < 1 OK	Y Direction 3.ex/L + 3.ey/B = 0.73 < 1 OK

(c) Table of bending moment and shearing stress

Condition	Direction	Bending Moment		Shearing Stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long Period	y Direction	0.048	0.088	0.317	0.586
Short Period	y Direction	0.101	0.141	0.669	0.939
*		0.067	0.094	0.446	0.626

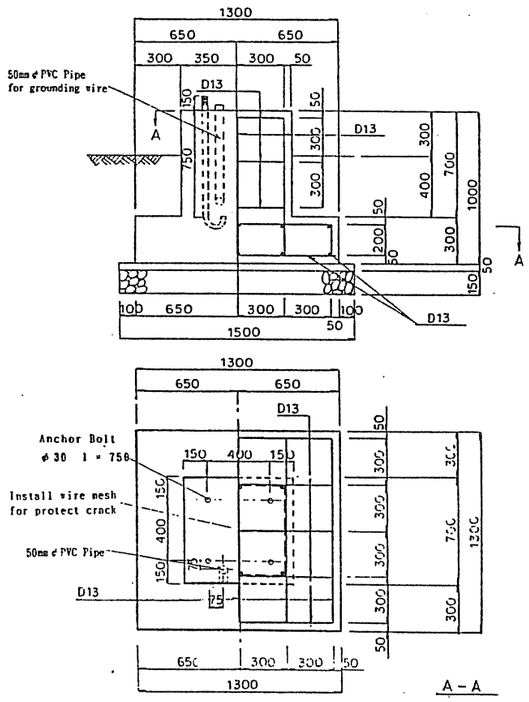
(d) Calculation of Reinforcement Content

X Direction	Y Direction
Top bar As1=M/(σsa.j.d)=0.17 D13@300 = 4.223 D13@300 = 4.223 cm² >0.170 cm² OK	Bottom bar As2 = M/(σsa.j.d) = 0.239 D13@300 = 4.223 D16@300 = 4.223 cm² >0.239 cm² OK

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement Bar (σs)	Shearing (τ)	Bond (τ0)
X direction Top bar σc=1.138 < 70 OK Y Direction Bottom bar σc=1.596< 70 OK	X Direction Top bar σs=68.019<1800 OK Y Direction Bottom bar σs=95.43<1800 OK	X Direction Top bar τ=0.191 <4 OK Y Direction Bottom bar τ=0.268<4 OK	X Direction Top bar τ0=1.434 <14 OK Y Direction Bottom bar τ0=2.013<14 OK

(f) Arrangement of reinforce bar



(3) Foundation Design of Circuit Breaker

(a) Design Condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical Load	Horizontal Load	Moment	
					Mx	My
Long Period	X	Foundation	14.419	-	-	-
		Structure	9.854	1.632	-	4.834
		Total	24.273	1.632	-	4.834
	Y	Foundation	14.419	-	-	-
		Structure	2.714	1.592	-	5.956
		Total	17.133	1.592	-	5.956
Short Period	X	Foundation	14.419	1.442	-	0.966
		Structure	2.714	3.142	-	10.526
		Total	17.133	4.584	-	11.492
	Y	Foundation	14.419	1.442	0.966	-
		Structure	2.714	1.04	5.162	-
		Total	17.133	2.482	6.128	-

(b) Stability Analysis

Long Period (Eccentric Distance)	Short Period (Eccentric Distance)
X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.199 Y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.348	X Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.671 Y Direction ex = ΣMy / Σv = 0.358 ey = ΣMx / Σv = 0

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0.32 < 1 OK y Direction 6.ex/L + 6.ey/B = 0.56 < 1 OK	x Direction 3.ex/L + 6.ey/B = 0.54 < 1 OK y Direction 3.ex/L + 3.ey/B = 0.77 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long period	x Direction	0.748	1.582	1.475	3.119
Short period	y Direction	0.186	0.267	1.24	1.777
Exchange to long period		0.124	0.177	0.827	1.185

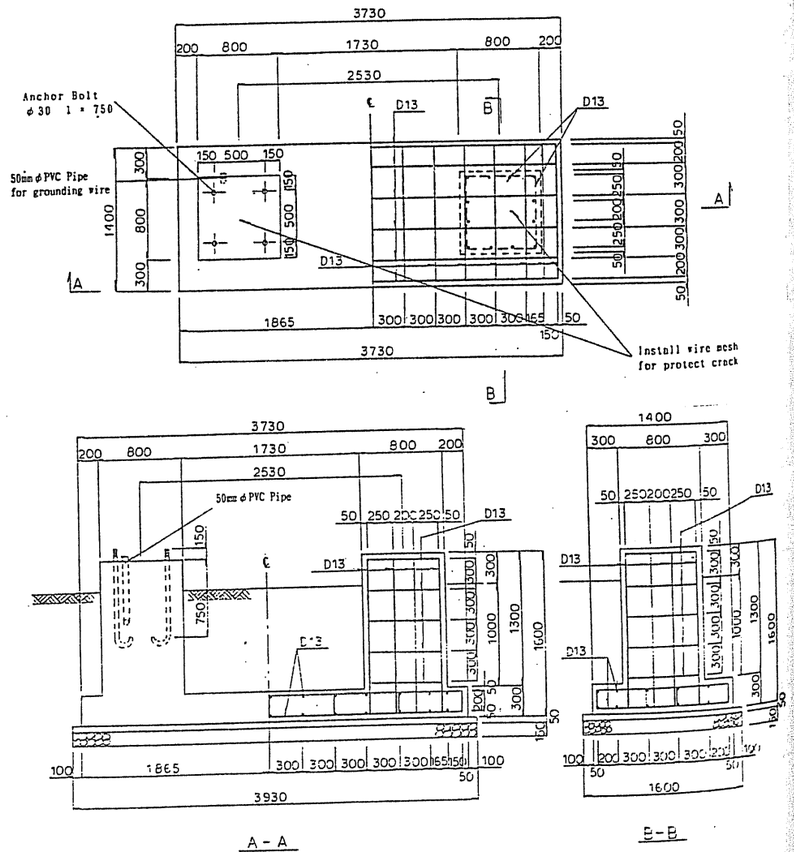
(d) Calculation of reinforcement bar

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=4.018 D13@300 = 4.223 D13@300 = 4.223 cm² >4.018 cm² OK	Top bar As1=M/(σsa.j.d)=0.315 D13@300 = 4.223 D13@300 = 4.223 cm² >0.315 cm² OK Bottom bar As1=M/(σsa.j.d)=0.45 D13@300 = 4.223 D13@300 = 4.223 cm² >0.450 cm² OK

(e) Check of unit stress of reinforcement bar

Concrete (σc)	Reinforcement bar (σs)	Shearing (τ)	Bond (τ0)
x direction Top bar σc=26.86 < 70 OK y direction Top bar σc=3.105 < 70 OK Bottom bar σc=3.005 < 70 OK	Top bar σs=1606<1800 OK Top bar σs=125.88<1800 OK Bottom bar σs=179.69<1800 OK	Top bar τ=1.337 <4 OK Top bar τ=0.355 <4 OK Bottom bar τ=0.508 <4 OK	Top bar τ0=10.03 <14 OK Top bar τ0=2.659 <14 OK Bottom bar τ0=3.811 < 14 OK

(f) Arrangement of reinforce bar



(4) Foundation Design of Disconnection Switch

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load	Moment	
					Mx	My
Long period	x	Foundation	15.453	-	-	-
		Structure	2.112	-	-	-
		Total	17.565	-	-	-
	y	Foundation	15.453	-	-	-
		Structure	2.112	0.858	2.112	-
		Total	17.565	0.858	2.112	-
Short period	x	Foundation	15.453	1.546	-	0.562
		Structure	2.112	0.519	-	2.112
		Total	17.565	2.065	-	2.674
	y	Foundation	15.453	1.546	0.562	-
		Structure	2.112	2.265	5.079	-
		Total	17.565	3.811	5.641	-

(b) Stability Analysis

Long period (Eccentric distance)	Short period (Eccentric distance)
x Direction ex = ΣMy / Σv = 0 < 1 ey = ΣMx / Σv = 0 < 1 y Direction ex = ΣMy / Σv = 0.12 < 1 ey = ΣMx / Σv = 0 < 1	x Direction ex = ΣMy / Σv = 0 < 1 ey = ΣMx / Σv = 0.152 < 1 y Direction ex = ΣMy / Σv = 0.321 < 1 ey = ΣMx / Σv = 0 <1

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0 < 1 y Direction 6.ex/L + 6.ey/B = 0.1 < 1	x Direction 3.ex/L + 6.ey/B = 0.34 < 1 y Direction 3.ex/L + 3.ey/B = 0.13 < 1

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long period	x Direction	-	0.32	-	0.5
	y Direction	-	0.025	-	0.169
Short period *	x Direction	-	1.075	-	1.68
Short Period *			0.717		1.12
	y Direction	0.004	0.038	0.024	0.258
		0.003	0.025	0.016	0.172

* Exchange to long period condition

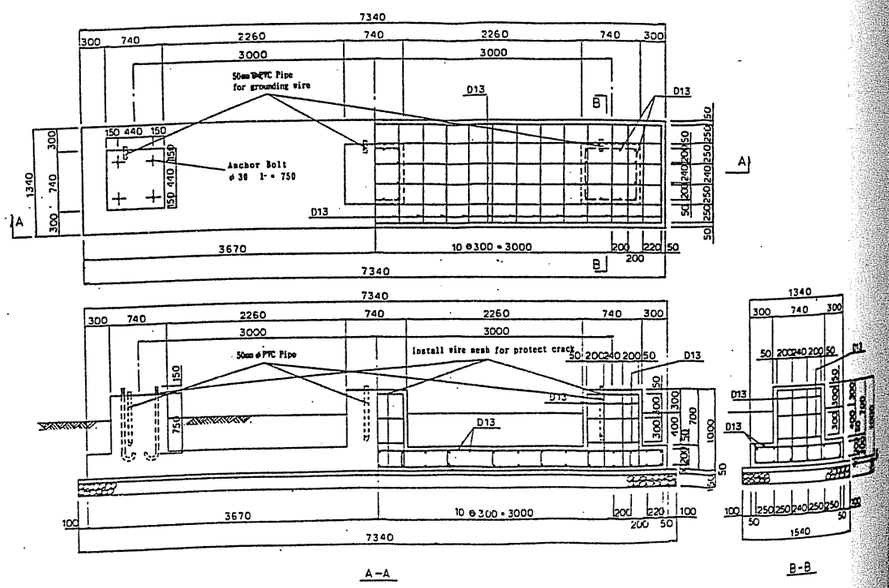
(d) Calculation of reinforcement bar

x Direction	y Direction
Top bar -	Top bar As1=M/(σsa.j.d)=0.008 D13@300 = 4.223 D13@300 = 4.223 cm² >0.008 cm² OK
Bottom bar As1=M/(σsa.j.d)=1.821 D13@300 = 4.223 D13@300 = 4.223 cm² >1.821 cm² OK	Bottom bar As2 = M/(σsa.j.d) = 0.063 D13@300 = 4.223 D13@300 = 4.223 cm² >0.063 cm² OK

(e) Check of unit stress of reinforcement bar

Concrete (σc)	Reinforcement bar (σs)	Shearing (τ)	Bond (τ0)
x direction Top bar - Bottom bar σc=12.174 < 70 OK	x Direction Top bar - Bottom bar σs=727.908<1800 OK	x Direction Top bar - Bottom bar τ=0.48 <4 OK	x Direction Top bar - Bottom bar τ0=3.60<14 OK
y direction top bar σc=0.051 < 70 OK Bottom bar σc=0.424 < 70 OK	y Direction Top bar σs=3.046<1800 OK Bottom bar σs=25.38<1800 OK	y Direction Top bar τ=0.007 <4 OK Bottom bar τ=0.074 <4 OK	y Direction Top bar τ0=0.051<14 OK Bottom bar τ0=0.553 < 14 OK

(f) Arrangement of reinforce bar



(5) Foundation Design of Neutral Resister

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load	Moment	
					Mx	My
Long period	x	Foundation	5.954	-	-	-
		Structure	1.367	0.388	-	0.541
		Total	7.321	0.388	-	0.541
	y	Foundation	5.954	-	-	-
		Structure	1.367	0.051	0.133	-
		Total	7.321	0.051	0.133	-
Short period	x	Foundation	5.954	0.595	-	0.149
		Structure	1.367	1.275	-	1.734
		Total	7.321	1.87	-	1.883
	y	Foundation	5.954	0.595	0.149	-
		Structure	1.367	0.051	0.133	-
		Total	7.321	0.646	0.282	-

(b) Stability analysis

Long period (Eccentric distance)	Short period (Eccentric distance)
x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.074 y Direction ex = ΣMy / Σv = 0.018 ey = ΣMx / Σv = 0	x Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.257 y Direction ex = ΣMy / Σv = 0.039 ey = ΣMx / Σv = 0

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0.44 < 1 OK	x Direction 3.ex/L + 6.ey/B = 0.77 < 1 OK
y Direction 6.ex/L + 6.ey/B = 0.04 < 1 OK	y Direction 3.ex/L + 3.ey/B = 0.04 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long period	x Direction	-	0.028	-	0.359
	y Direction	-	0.034	-	0.447
* Short period	x Direction	- 0.016	0.03	- 0.208	0.388
	y Direction		0.064	-	0.826

* Exchange to long period condition

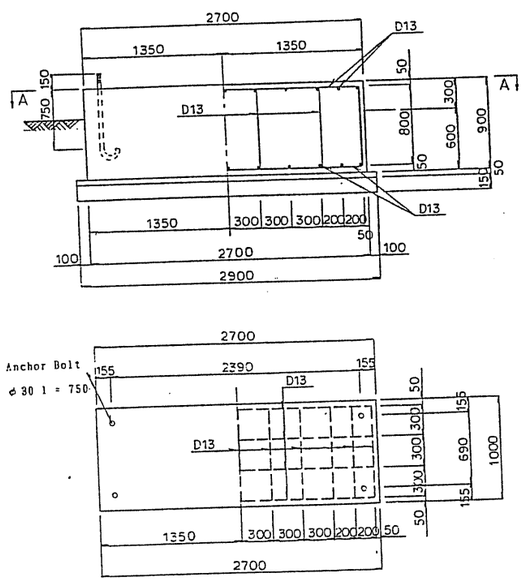
(d) Calculation of reinforcement bar

x Direction	y Direction
Top bar As1=M/(σsa.j.d)=0.023 D13@300 = 4.223 D13@300 = 4.223 cm² >0.023 cm² OK Bottom bar As1=M/(σsa.j.d)=0.042 D13@300 = 4.223 D13@300 = 4.223 cm² >0.042 cm² OK	Top bar As1=M/(σsa.j.d)=0 D13@300 = 4.223 D13@300 = 4.223 cm² >0 cm² OK Bottom bar As1=M/(σsa.j.d)=0.09 D13@300 = 4.223 D13@300 = 4.223 cm² >0.09 cm² OK

(e) Check of unit stress of reinforcement bar

Concrete (σc)	Reinforcement bar (σs)	Shearing (τ)	Bond (τ0)
x direction Top bar σc=0.272 < 70 OK Bottom bar σc=0.509 < 70 OK y Direction Top bar σc=0 < 70 OK Bottom bar σc=6.58 < 70 OK	x Direction Top bar σs=9.004 <1800 OK Bottom bar σs=16.92<1800 OK y Direction Top bar σs=0 < 1800 OK Bottom bar σs=36.09<1800 OK	x Direction Top bar τ=0.05 <4 OK Bottom bar τ=0.092 <4 OK y Direction Top bar τ=0 < 4 OK Bottom bar τ=0.197 < 4 OK	x Direction Top bar τ0=0.37 <14 OK Bottom bar τ0=0.693<14 OK y Direction Top bar τ0=0<14 OK Bottom bar τ0=1.476 < 14 OK

(f) Arrangement of reinforce bar



A – A

(6) Foundation Design of STR 20 kV Cable Termination

(a) Design condition (See 6.10.2 (1) Gantry Pylon Type 1 on soil condition)

Condition	Direction	Description	Vertical load	Horizontal load	Moment	
					Mx	My
Long period	x	Foundation	4.794	-	-	-
		Structure	1.122	0.346	0.448	-
		Total	5.916	0.346	0.448	-
	y	Foundation	4.794	-	-	-
		Structure	1.122	0.346	-	0.448
		Total	5.916	0.346	-	0.448
Short period	x	Foundation	4.794	0.48	0.18	-
		Structure	1.122	1	1.388	-
		Total	5.916	1.48	1.568	-
	y	Foundation	4.794	0.48	-	0.18
		Structure	1.122	1	-	1.388
		Total	5.916	1.48	-	1.568

(b) Stability analysis

Long period (Eccentric distance)	Short period (Eccentric distance)
x Direction ex = ΣMy / Σv = 0.076 ey = ΣMx / Σv = 0 y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.076	x Direction ex = ΣMy / Σv = 0.265 ey = ΣMx / Σv = 0 y Direction ex = ΣMy / Σv = 0 ey = ΣMx / Σv = 0.265

Judgement of stability

Long period	Short period
x Direction 6.ex/L + 6.ey/B = 0.18 < 1 OK	x Direction 3.ex/L + 6.ey/B = 0.31 < 1 OK
y Direction 6.ex/L + 6.ey/B = 0.4 < 1 OK	y Direction 3.ex/L + 3.ey/B = 0.69 < 1 OK

(c) Bending moment and shearing stress

Condition	Direction	Bending moment		Shearing stress	
		Top bar	Bottom bar	Top bar	Bottom bar
Long period	x Direction	- 0.009	0.062	- 0.59	0.412
	y Direction	0.043	0.166	0.145	0.559
* Short period	x Direction	- 0.097	0.15	- 0.647	1.001
	y Direction	- 0.111	0.321	- 0.374	1.078

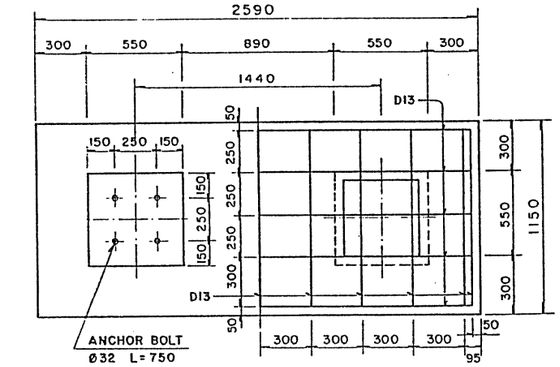
(d) Calculation of reinforcement bar

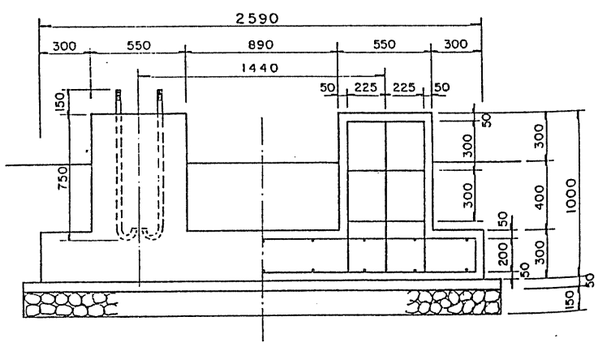
x Direction	y Direction
Top bar As1=M/(σsa.j.d)=0.023 D13@300 = 4.223 D13@300 = 4.223 cm² >0.023 cm² OK Bottom bar As2=M/(σsa.j.d)=0.254 D13@300 = 4.223 D13@300 = 4.223 cm² >0.254 cm² OK	Top bar As1=M/(σsa.j.d)=0.188 D13@300 = 4.223 D13@300 = 4.223 cm² >0.188 cm² OK Bottom bar As2=M/(σsa.j.d)=0.543 D13@300 = 4.223 D13@300 = 4.223 cm² >0.543 cm² OK

(e) Calculation of unit stress of concrete and reinforcement

Concrete (σc)	Reinforcement bar (σs)	Shearing stress (τ)
x direction Top bar σc=0.153 < 70 OK Bottom bar σc=1.698< 70 OK y Direction Top bar σc=1.256 < 70 OK Bottom bar σc=3.634 < 70 OK	x Direction Top bar σs=9.137 <1800 OK Bottom bar σs=101.521 <1800 OK y Direction Top bar σs=75.126 < 1800 OK Bottom bar σs=217.256 <1800 OK	x Direction Top bar τ=0.185 <4 OK Bottom bar τ=0.286 <4 OK y Direction Top barτ=0.107 < 4 OK Bottom bar τ=0.308 <4 OK

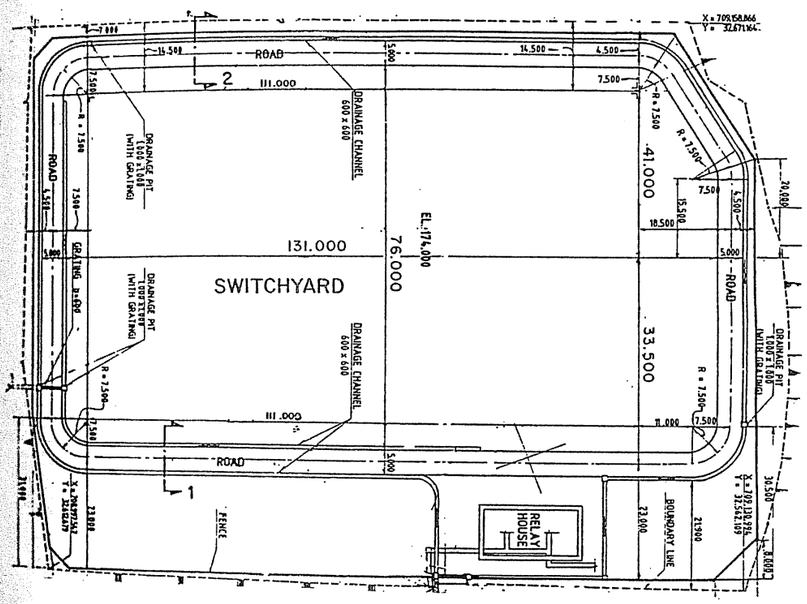
(f) Arrangement of reinforce bar





6.10.5 Plant Road and Access Road

An asphalt pavement road 5 m wide and 300 m long encircles the switchyard electrical equipment area. The cross slope was designed at 2% from the center to both side for drainage to the ditch constructed beside the whole plant road. The drainage capacity was calculated based on the rainfall intensity of 78.5 mm/hr that is equivalent to the past maximum record in the project catchment area. The access road to switchyard connects with the existing national road. The road is 5 m wide and the side drain ditch was constructed for drainage. The road grade was designed as level as possible to assist transport of heavy equipment to the switchyard.



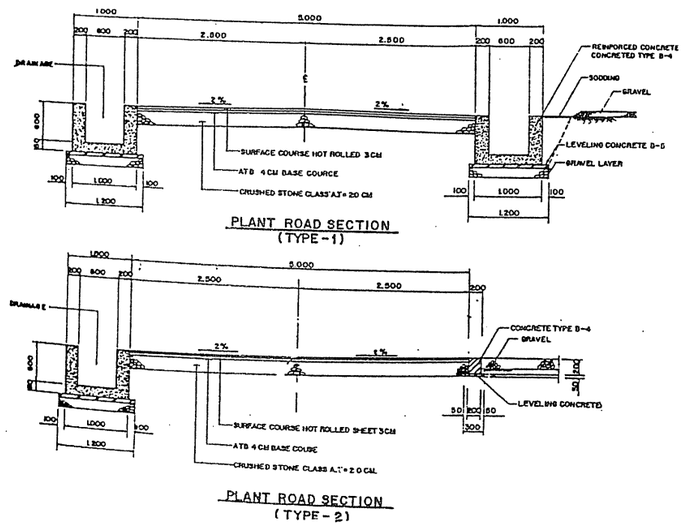


Fig. 1.6.10-3 Switchyard Plant Road

6.10.6 Slope Protection Wall Along Switchyard Access Road.

(1) General.

The most of the slope along the access road to the switchyard, located just under the transmission line tower No.2, is composed of soil and weathered rocks, which tend to be weakened especially under wet conditions. After excavation of the slope. some erosion and a small slope failure occurred due to rainfall. For the purpose of the future slope stability and the tower foundation stability, slope protection of concrete wall namely gravity type and leaning type, was designed and constructed in consideration of the geological and topographical condition.

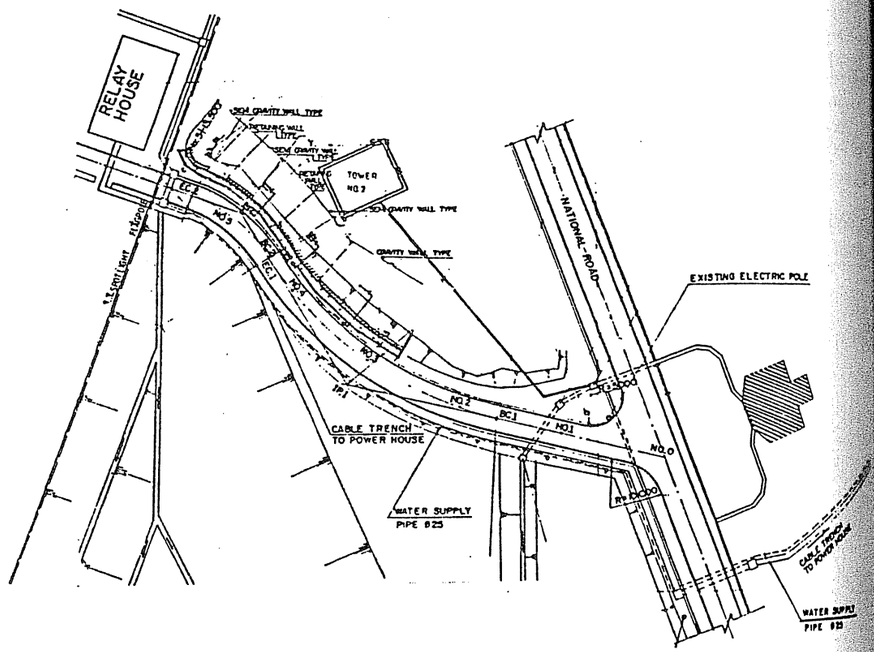


Fig. 1.6.10-4 Protection Area of Access Road to Switchyard

(2) Stability of gravity wall.

(a) Design Condition.

Material Properties	Others
Concrete. Unit weight (ɣ) 2.35 t/m³	Superimposed load.
Backfilling soil. Unit weight (ɣs) 1.90 t/m³	Transmission tower No.2 (q) 2.23 t/m²
Internal friction angle (Ø) 30°	Allowable bearing capacity (qa) 30 t/m²

(b) Calculation of earth pressure.

Coulomb's theory of earth pressure was basically applied. In case of a retaining wall with a backfilled ground surface, the coefficient of earth pressure acting on the wall is generally calculated by the following equation on condition that the degree of slide plane is determined optionally.

$$KA = \frac{\sin(\alpha-\varnothing)}{\cos(\alpha-\delta/3\ \varnothing - I)}\left[(1+2Ho/H)\,.\,(\cot\alpha + \tan I) + (Ho/H)^2(\cot\alpha - \cot\beta) + I/H^2\{Ho - H(\tan\beta + \tan\alpha\,.\,\tan\beta\,.\,\tan I)/(\tan\alpha - \tan\beta)\}\{Ho\cot\beta - (H+Ho)\cot - H\tan I\}\right]$$

if $\{Ho - H(\tan\beta + \tan\alpha\,.\,\tan\beta\,.\,\tan I)/(\tan\alpha - \tan\beta)\} < 0$, *then*

$\{Ho - H(\tan\beta + \tan\alpha\,.\,\tan\beta\,.\,\tan I)/(\tan\alpha - \tan\beta)\} = 0$

Where :
- KA : Coefficient of earth pressure.
- H : Height of retaining wall (= 3.45 m)
- Ho : ho + g/ɣ (= 8.0 m)
- ho : Height of backfilling from top of wall to top of backfilling (= 6.8 m)
- q : Superimposed load (= 2.23 t/m²)
- ɣ : Unit weight of soil (= 1.9 t/m³)
- Ø : Internal friction angle of soil (= 30°)
- α : The degree between slide plane assumed and horizontal
- β : The degree between surface of backfilling ground and horizontal (=5.1°)
- δ : Friction angle of wall face (δ = 2/3ø is standardized)
- I : The degree between wall's back and vertical (= 0.0°)

From a trial calculation the maximum value of the coefficient of earth pressure, KA = 0.3433 was found at the slope of slide plane, α = 33.8°. Then horizontal (KH, KV) and vertical components (PH, PV) of earth pressure were given as below.

KH = KA.cos (2/3 Ø + I) = 0.3226
KV = KA.sin (2/3 Ø + I) = 0.1174
PH = ½ ɣ KH.H² = 3.648 t/m
PV = ½ ɣ KV.H² = 1.327 t/m

(c) Stability Analysis.

Total of external forces and dead load

	Vertical force (t/m)	Horizontal force (t/m)	X (m)	Y (m)	Mx (tm/m)	My (tm/m)
Wall weight	9.831	-	0.957	-	9.413	-
Earth pressure	1.327	3.648	2.075	1.150	2.754	4.195
Total	11.158	3.648	-	-	12.167	4.165

Stability against overturning

The length of foundation B = 2.075 m. The effective point of the resultant force (*x*) was within the middle third so stability against overturning is assured.

$$x = \frac{Mx - My}{\Sigma V} = 0.174\ m,\quad e = |B/2 - x| = 0.324\ m < B/6 = 0.346\ m$$

Stability against compressive stress

The maximum compressive stress was less than the allowable bearing capacity of the ground, so stability against compressive stress is assured.

$$\left.\begin{matrix} q\ \max \\ q\ \min \end{matrix}\right\} = \frac{\Sigma V}{B}\left(1 \pm \frac{6.e}{B}\right) = \begin{cases} 10.42\ t/m^2 \\ 0.34\ t/m^2 \end{cases} < qa = 30\ t/m^2$$

Where : q max : Maximum Compression stress
q min : Minimum Compression stress.

Stability against sliding

The safety factor against sliding was larger than 1.5, so stability against sliding is assured.

$$Fs = \Sigma V \cdot \tan \varnothing / \Sigma H = 1.56 > 1.50$$

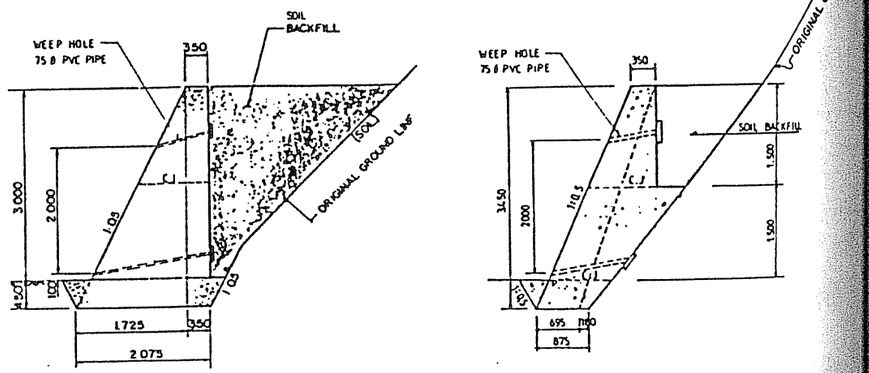


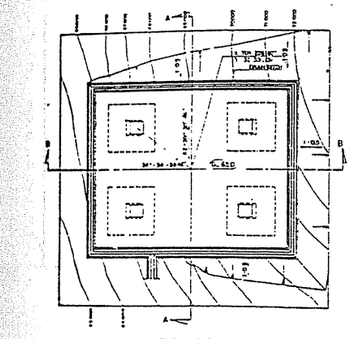
Fig. 1.6.10-5 Gravity Wall Along the Access Road to Switchyard

6.11 Transmission Line Tower Foundation.

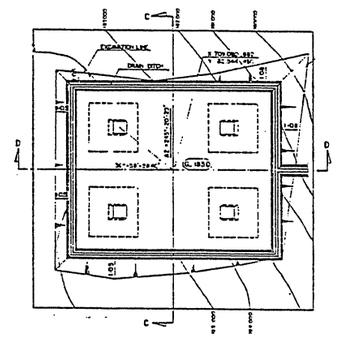
(1) General Layout.

The transmission line connecting the powerhouse with the switchyard has two transmission towers between. The location of transmission line tower No.1 located near the powerhouse in original design was relocated further to downstream side in order to get sufficient clearance between the transmission line and the ground surface and other structures. The tower foundations were originally designed on the condition that the bottom of the foundation would be CL to D class rock. However, the foundation design of the tower No.2 located near the switchyard was modified in consideration of the actual geological condition which was found to be soil during foundation excavation.

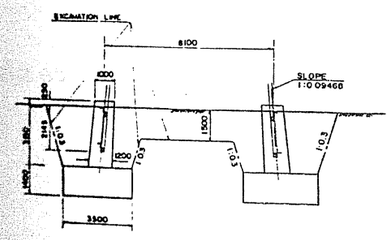
General arrangement of the tower foundation were shown in below figure.



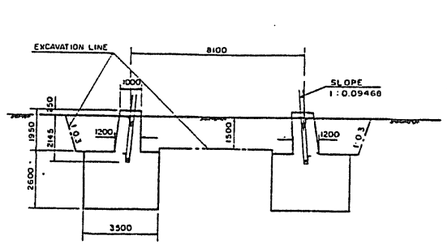
PLAN



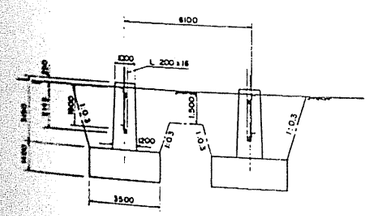
PLAN



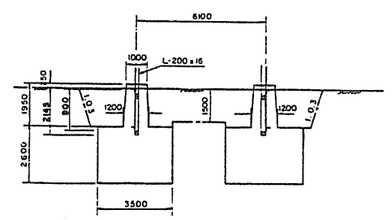
SECTION A - A



SECTION C - C



SECTION B - B
Tower No. 1



SECTION D - D
Tower No. 2

Fig. 1.6.11 Transmission Line Tower Foundation

(2) Design Condition.

Item	Tower No.1	Tower No.2
Load condition		
Compression force (C)	102.0 ton	102.0 ton
Uplift force (U)	90.4 ton	90.4 ton
Horizontal force (QB)	8.6 ton	8.6 ton
Unit weight of concrete (γc)	2.4 t/m³	2.4 t/m³
Geological condition		
Cohesion		
below slab (C1)	12.0 t/m²	5.0 t/m²
above slab (C2)	1.0 t/m²	1.0 t/m²
side of slab (C3)	12.0 t/m²	5.0 t/m²
Internal friction angle		
below slab (Ø1)	22.0°	15.0°
above slab (Ø2)	0.0°	0.0°
side of slab (Ø3)	22.0°	15.0°
Unit weight of soil		
below slab (γ s1)	2.2 t/m³	1.8 t/m³
above slab (γ s2)	1.7 t/m³	1.7 t/m³
side of slab (γ s3)	1.8 t/m³	1.7 t/m³
Coefficient of bearing capacity		
Nc	8.6	6.5
Nr	2.4	1.2
N%	6.5	4.7

(3) Stability Calculation.

(i) Compression bearing capacity.

The bearing capacity (qcr) the maximum soil reaction (σcmax) and the safety factor (Fs) were calculated as following equations.

qcr = 1/1.5 (α x C1 x Nc + β x γs2 x Df x Nq)
σcmax = P/A x μ
Fs = q cr / σc max.

Where;
- α : Coefficient of foundation shape (squared : α = 1.3)
- β : Coefficient of foundation shape (squared : β = 0.4)
- C1 : Cohesion below slab (t/m²)
- γ s1 : Unit weight of soil below slab (t/m³)
- s2 : Unit weight of soil above slab (t/m³)
- B : Width of slab (m)
- Df : Embedded depth of slab (m)
- Nc,Nr,Nq : Coefficient of bearing capacity determined from internal friction angle and gradient of resultant force
- P : Design vertical load (ton). P = C + Wc + Ws
- Wc : Total weight of concrete foundation (ton)
- Ws : Total weight of backfilling (ton)
- C : Design compression force (C = 102 ton)
- A : Area of slab bottom (A=B²=12.25m²)
- μ : The rate of increase for soil reaction acting on end of the foundation due to moment

$$\mu = \begin{cases} 1 + 6e/B \\ 2/3 \cdot (0.5 - e/B) \end{cases} \quad \begin{matrix} \text{(In case } e \le B/6) \\ \text{(In case } e > B/6) \end{matrix}$$

e : Distance between centroid of slab and working point of resultant force (m). e = QB x (h + t) / P
QB : Design horizontal force (QB = 8.6 t)

Calculation results are shown in the table below. From the calculation results, the safety factors for both towers are larger than 2.0, so the stability against compression capacity is assured.

Item		Tower No.1	Tower No.2	Remarks
q cr	(t/m²)	127.908	53.087	
Wc	(t)	50.333	82.118	
Ws	(t)	54.327	31.816	
P	(t)	206.660	215.934	
e	(m)	0.189	0.181	B/6 = 0.583
μ		1.325	1.310	
σc max	(t/m²)	22.346	23.103	≥ 2.0
Fs		5.723	2.297	

(ii) Uplift capacity.

The uplift capacity (q ty) and the safety factor (Fs) were calculated as following equations.

q ty = k (Wc + Ws + Rd + Rf), Fs = q ty / T

Where ; k : Coefficient of uplift capacity

$$k = \begin{cases} 1/1 + 6\,e\,B\,/\,(B^2 + b^2) \\ 0.667 \end{cases} \quad \begin{matrix} \text{(In case k} \ge 0.667 \text{)} \\ \text{(In case k} < 0.667 \text{)} \end{matrix}$$

Wc : Total weight of concrete foundation (ton)
Ws : Total weight of backfilling (ton)
Rd : Shear resistance from backfilling above slab (ton)
Rd = 1/1.5 x L x Dh x {C2 x 0.5 γs2 x Dh + tan ⊘2 / (1 + sin ⊘2)}
L : Distance round slab (4 x B = 14.0 m)
Dh : Earth covering of slab (m)
C2 : Cohesion above slab (t/m²)
γs2 : Unit weight of soil above slab (t/m³)
O2 : Internal friction angle of soil above slab (°)
Rf : Shear resistance from vertical surface of slab concrete
Rf = 1/1.5 x L x t x C3
t : Thickness of slab (m)
C3 : Cohesion side of slab (t/m²)
T : Design uplift force (T = 90.4 ton)

e : Distance between centroid of slab and working point of resultant force (m). e = QB x h {T - (Wc1 + Wc2)}
Wc1 : Weight of concrete column (ton)
Wc2 : Weight of slab located just under column (ton)
QB : Design horizontal force (QB = 8.6 t)
h : Height of column (m)

Calculation results were shown in the table below. The safety factor in both towers were larger than 2.0, so that the stability against uplift capacity is assured.

Item	Tower No.1	Tower No.2	Remarks
q ty (ton)	185.201	182.711	
Wc (t)	50.333	82.118	
Ws (t)	43.462	25.453	
Rd (t)	27.067	15.867	
Rf (t)	156.800	121.333	
k	0.667	0.746	
e	0.355	0.221	
Wc1	9.173	5.678	
Wc2	4.838	8.986	
Fs	2.048	2.021	≥2.0

(4) Structural Calculation.

Calculation for reinforcement was carried out only for transmission tower No.1 and applied for the reinforcement arrangement for the both tower No.1 and No.2 because the slab and column of the tower No.2 are thicker than the tower No.1

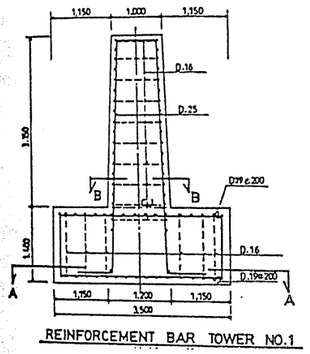
(i) Design of slab.

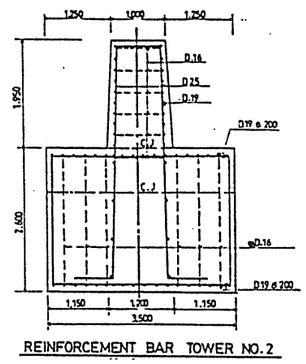
Item	Top side	Bottom side	Remarks
Design load	Uplift force	Compression force	
Maximum bending moment (tm/m)	6.31	8.37	
Maximum shearing force (t/m)	8.27	10.94	
Width of member (cm)	100	100	
Effective height (cm)	130	130	
Concrete cover for steel (cm)	10	10	
Reinforcement content (cm²/m)	14.33	14.33	
	D19 @ 200	D19 @ 200	
Steel ratio	0.00110	0.00110	
Compressive stress of concrete(kg/cm²)	4.8	6.3	<σca=70.0
Tensile stress of steel bar (kg/cm²)	358	475	<σsa=1800
Shearing stress of concrete (kg/cm²)	0.9	0.9	<τ a=4.0
Bond stress of concrete (kg/cm²)	3.0	3.0	<τoa<14.0

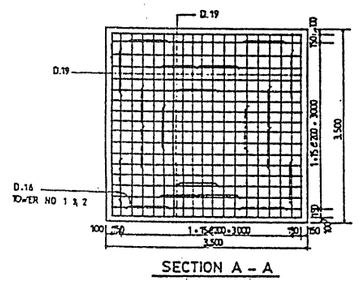
(ii) Design of column.

Item	Top	Bottom	Remarks
Bending moment (t.m)	23.94		
Uplift force (t)	81.23		
Horizontal force (t)	7.60		
Size of section (cm)	100 x 100	120 x 120	
Concrete cover for steel (cm)	10	10	
Reinforcement content (cm²)	101.34	101.34	
	D 25 @ 300	D25 x 30	
Spacing (cm)	16.127	20.000	
Compressive stress of concrete(kg/cm²)	-	15.5	< σca = 70
Tensile stress of steel bar (kg/cm²)	892	1577	< σ sa = 1800
Shearing stress of concrete (kg/cm²)	1.0	-	< τ a = 4.0
Bond stress of concrete (kg/cm²)	2.0	-	< τ oa = 14.0

(iii) Arrangement of reinforcement.







6.12 Access Road to Main Structures

(1) General

The access road for transporting equipment and material to the dam and powerhouse site connects with the national road about 600 m downstream of the dam site. About 1.5 km the national road had been newly constructed to detour the project construction site before the project started. At the connection point of the national road and the access road, the entrance gate and guardhouse were built as main entrance to the site. The access road was also designed for use for the operation and maintenance of the powerhouse and dam facility after the start of commercial operation.

The formation level and alignment of the road was determined in consideration of the balance of excavation, backfill and smooth traffic. The access roads were conveniently divided into the following three parts : (i) Access road to dam (Main entrance to dam crest on left bank, 885m long), (ii) Access road to boat handling (The end of dam access road to boat handling site, 251m long) and (iii) Access road to powerhouse (A branch from dam access road to powerhouse, 300m long).

The access road construction was basically divided into four parts, base course class B, base course class A, asphalt treated base (ATB), and hot rolled sheet (HRS). Some modification on the base course was made depending on the trafffic load condition.

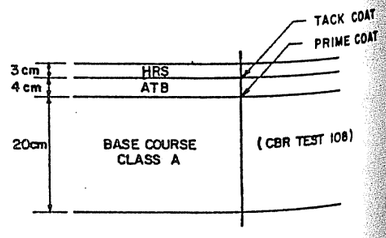
(2) Access Road to Dam

This access road was designed for transportation of heavy equipments and materials for dam construction. Spillway gate and intake gate were transported to dam and therefore, the road must support heavy loaded vehicles. The road is daily used for operation and maintenance of the facilities and utility of dam including spillway and intake operation.

The road was constructed with class A base course 20 cm thick to cater for heavy loads. The road is 885 m long and 5.50 m wide. Drainage was constructed on the right side of the road to catch the rainfall water from the road surface and the mountain side. At sta. 0+325, a culvert was installed to drain the water to the river side of the road. On the right mountain side of the road lighting was installed. Gravel was laid and compacted on the shoulder because the road has a steep gradient and soil shoulder would erode and be difficult to maintain.

The access road was constructed in 5 stages :

- Base course class A, thickness is 20 cm (for heavy load)
- Prime coat
- Asphalt Treated Base (ATB), thickness is 4 cm
- Tack coat
- Hot Rolled Sheet (HRS), thickness is 3 cm



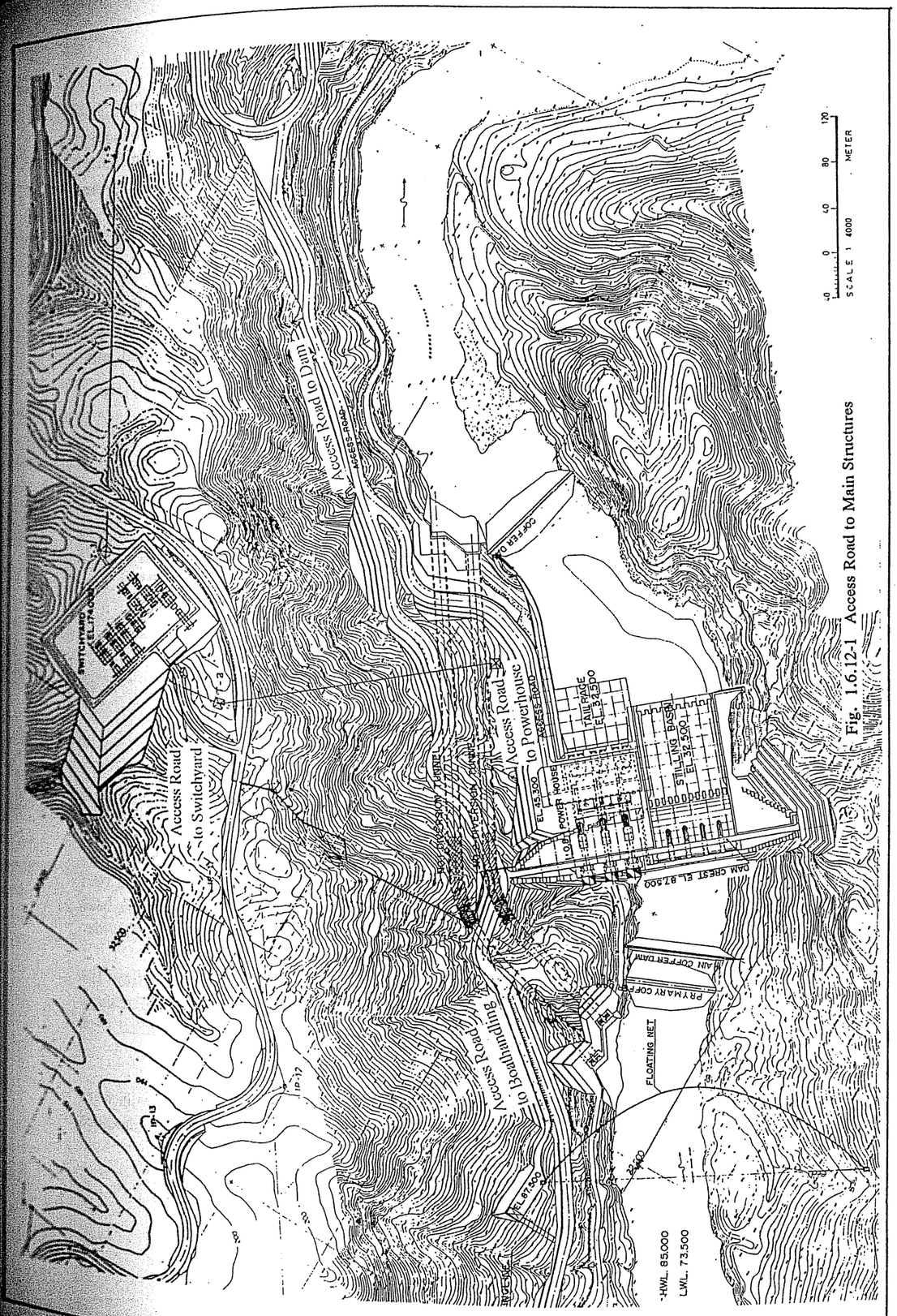


Fig. 1.6.12-1 Access Road to Main Structures

(3) Access Road to Boat Handling

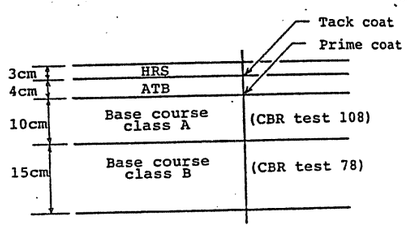
This road was designed to access the boat handling site where boat warehouse, logboom with floating net and jib crane were installed. In the warehouse are found rails, hoist and inclined trolley which are used for taking the boat in and out of the reservoir. Foundation of logboom and jib crane which can lift the wood and debris up to dump truck are located near the boat house area. The access road starts from the end of the dam access road, with 251 m long and of 3.50m wide.

Drainage was constructed on the right side of the road to catch rain water from the road surface and the mountain side. At sta 0+sta no: 0-10 and sta no: 6, a culvert was installed to lead drainage water to reservoir side. Road lighting was installed with buried cable underground on the right mountain side of the road.

Since on the left side of road is reservoir with steep cliff, guard rails were installed all along the road for safety of road users.

The access road was constructed in six (6) parts :

- Base cone class B, thickness is 15 cm
- Base course class A, thickness is 10 cm
- Prime coat
- Asphalt Treated Base (ATB), thickness is 4 cm
- Tack coat
- Hot Rolled Sheet (HRS), thickness is 3 cm



(4) Access Road to Powerhouse

The access road was designed for transportation of heavy loads such as turbine, generator, transformers and common equipment and materials to the powerhouse site. The road is also daily used for operation and maintenance of powerhouse after the start of commercial operation. As the road is to be used for heavy traffic, class A base course with 20 cm of thickness was applied. The road is 300 m long and 5.50 m wide.

Drainage was installed on the right side of the road to catch and drain water from the road surface and the mountain side. At Sta. 0+150, a culvert was installed to discharge the water to the river side. On the mountain side, road lighting was installed with buried cable underground.

The left (river side) of the road is partly steep cliff and in some parts, guard rails were installed for safety of road users. To beautify and reduce erosion on the shoulder surface by rainfall, turf was planted on both sides of the road shoulder.

The access road construction was constructed in the following five stages :

- Base course class A, thickness is 20 cm (for heavy load)
- Prime coat
- Asphalt Treated Base (ATB), thickness is 4 cm
- Tack coat
- Hot Rolled Sheet (HRS), thickness is 3 cm.

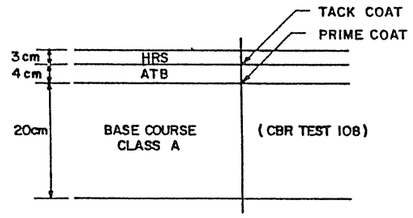
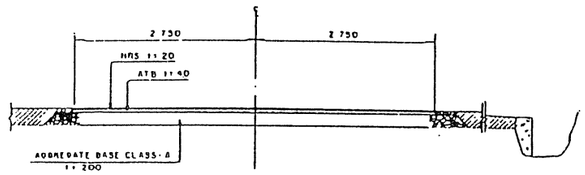
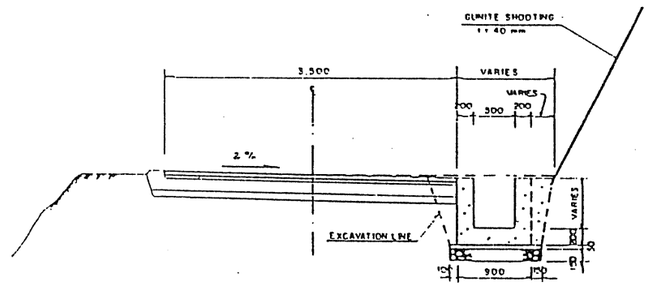


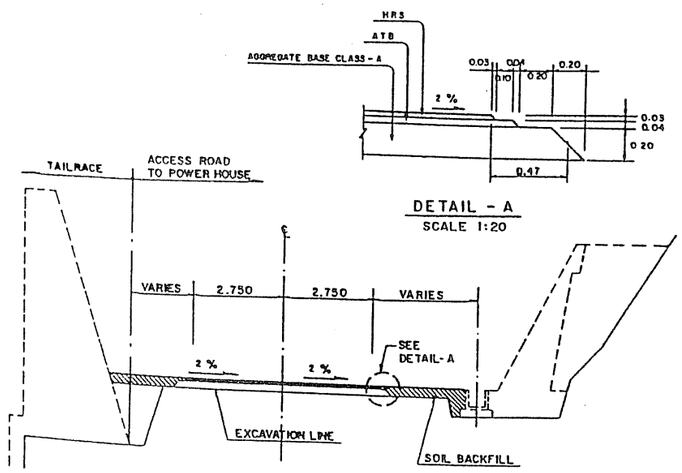
Fig. 1.6.12-2 Typical Cross Section of Access Road



Access Road to Dam



Access Road to Boathandling



Access Road to Powerhouse

(5) Slope Protection Along Access Road to Powerhouse

(a) General

In order to keep safety traffic, the slope protection, namely leaning type retaining wall, was newly proposed and designed along the access road to powerhouse.

Most of the slope along the access road to powerhouse was composed of weathered rocks, but the slope located just under the transmission tower No. 1 was composed of soil and highly weathered rocks which tended to be weakened under a wet condition. After excavation of the slope, some erosion and slope failure actually occurred and continued at this site. If the leaning type wall was build up on the erosioned ground as original design, backfilling would be required and earth pressure due to the backfilling would cause the wall unstable. This was judged from the result of stability calculation.

For the purpose of the slope stability in consideration of the tower foundation stability, the type of leaning wall originally designed for this slope protection was changed to gravity type as more reliable slope protection.

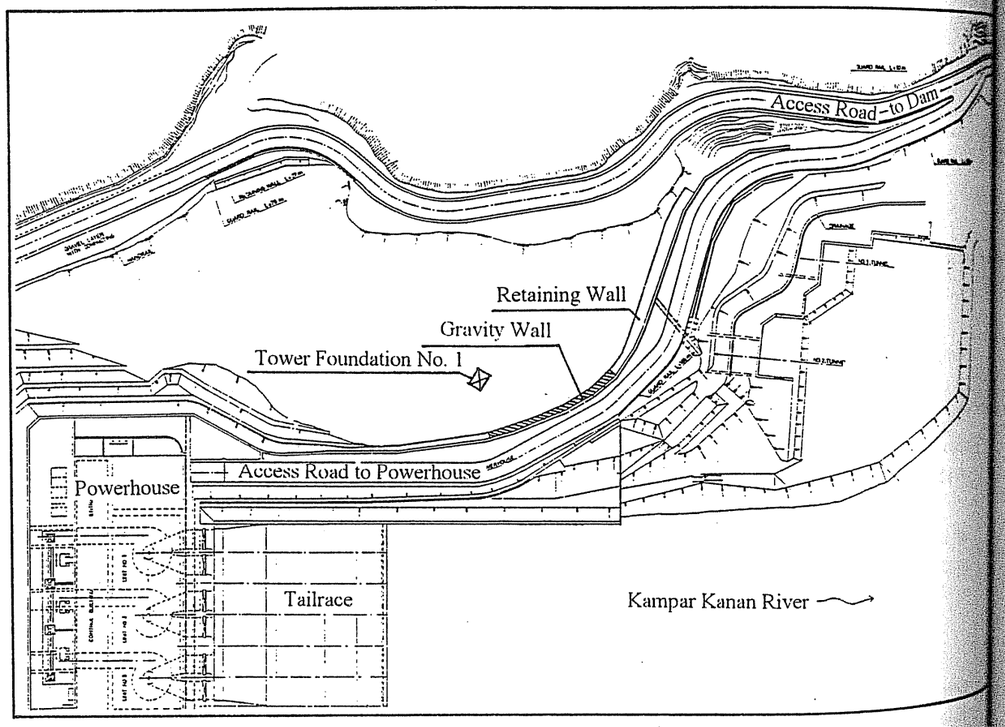


Fig. 1.6.12-3 Gravity Wall Protection Site

(b) Stability calculation of gravity wall

(i) Design load and material property

Concrete	Unit weight	2.35 t/m³
Backfilling soil	Unit weight	1.90 t/m³
	Internal friction angle	30°
Foundation	Allowable bearing capacity	30 t/m²
Surcharged load	Transmission tower No. 1	2.23 t/m²
Earth pressure	Coefficient of active earth pressure	0.199
	Active earth pressure	3.367 t/m
	(Horizontal)	3.299 t/m
	(Vertical component)	0.677 t/m

Note : Calculation method is by coulomb's theory of earth pressure and condition of earth pressure is as follows ;

Height of retaining wall (H)	: 4.22 m
Height of backfilling from top of wall to top of backfilling (ho)	: 4.85 m
The degree between surface of backfilling ground and horizontal (β)	: 22°
Fiction angle of wall face (δ)	: 2/3∅
The degree between walls back and vertical (I)	: -8.5°

(ii) Total of external forces and dead load

Load Items	Vertical force (t/m)	Horizontal force (t/m)	X(m)	Y(m) (tm/m)	Mx	My (tm/m)
Wall weight	10.510	-	1.477	-	15.208	-
Earth pressure	0.677	3.299	1.998	1.407	1.339	4.642
Total	11.187	2.299	-	-	16.547	4.642

(iii) Stability against overturning

The effective point of the resultant force was within the middle third so stability against overturning is assured.

$$X = \frac{Mx - My}{\sum V} = 1.064m, \quad e = |B/2 - x| = 0.18m < B/6 = 0.295m$$

Where, B : Length of foundation (=1.757m)
X : Working point of the resultant force (m)
e : Eccentricity (m)

(iv) Stability against sliding

The safety factor against sliding was larger than 1.5 so stability against is assured.

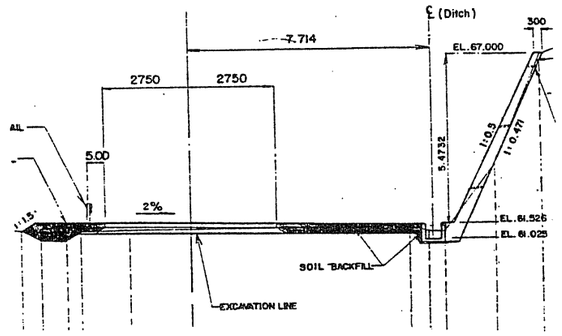
$$Fs = \sum V \cdot \tan\varnothing / \sum H = 1.95 > 1.5$$

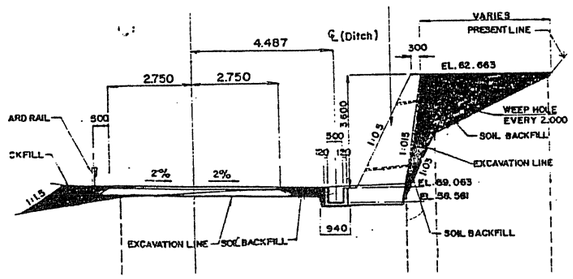
(v) Stability against compressive stress

Maximum and minimum compressive stress acting on the ground surface, q max, q min. were less than the allowable bearing capacity of the ground. So stability against compressive stress is assured.

$$\left.\begin{matrix} q\ \max \\ q\ \min \end{matrix}\right\} = \frac{\Sigma V}{B}\left(1 \pm \frac{6.e}{B}\right) = \begin{cases} 10.30\ t/m^2 \\ 2.43\ t/m^2 \end{cases} < qa\ =\ 30 t/m^2$$

(c) Typical cross section of concrete wall protection





6.13 Drainage System of Powerhouse Area

(1) General

The drainage system in the powerhouse area is especially important and has to work efficiently and smoothly in order to avoid a flood occurring in the powerhouse area that would cause serious damage to electric equipment during operation of the power station.

The drainage system was designed in consideration of the surrounding catchment area within the dam body, guide wall of spillway, tailrace and steep slope of left bank.

The drainage design was based on the following case which was considered very likely to occur in the future, that is, in case a flood occurs in the reservoir catchment area approximately equivalent to the 10 year probable flood 2000 m^3/s scale and the past maximum hourly rainfall of 78.5 mm/hr scale occurs locally in the powerhouse area simultaneously.

It is considered that a large flood more than the past maximum flood mentioned above in the reservoir catchment area and the past maximum hourly rainfall locally in the powerhouse area will not occur simultaneously, or if it should happen, this worst case will be very rare and it is not practical to consider such an extreme condition.

(2) Drainage system

The drainage systems consist of the dam gallery drainage, drain pit inside the powerhouse and tailrace pit located just downstream of the powerhouse which have the capacity to retain the drained water and discharge it to the tailrace bay. The system diagram and the water pump capacity installed in the sump pits are shown in the figure 1.6.13-1 and table 1.6.13-1.

(a) Rainfall water in the powerhouse catchment area

The powerhouse is located just downstream of the dam and surrounded by the dam body, guide wall of spillway, tailrace and steep slope of the left bank. The rainfall water in the catchment area must be collected and discharged to tailrace pit and drained to the downstream tailrace bay.

The powerhouse catchment area was checked based on the final layout of the drainage constructed, that is, CA=24,923 m^2 slightly larger than tender design CA=23,450m^2.

The Rational Formula was used to obtain the run-off volume for the hourly rainfall which was given for 78.5 mm/hr in (1) General mentioned above.

$$Q_{(78.5\ mm\ hr)} = 0.53\ m^3/s$$

(b) Drainage in the dam gallery

In order to collect the water percolating through the upstream face or seeping through the foundation, the drainage system comprising drain holes, side ditches and drain pits were provided in the dam gallery. The water collected in the drain pit located at EL. 34.5 in the gallery is pumped up to the surface into the side ditch drainage located just downstream of the dam and discharged to the tailrace water tank.

Two (2) drainage pumps are provided in the drain pit in the dam gallery. One pump is for normal operations, while the other serves as a stand-by in case the normal operating pump fails. The amount of the normal seepage and the drainage water is estimated approximately,

$$Q_{(dam)} = 0.3 \sim 0.4 \ m^3/min \fallingdotseq 0.01 \ m^3/s$$

(c) Drainage in the powerhouse

Three (3) station drainage pits were installed in the power station. The piping system is arranged between neighboring drainage pits so that water in each station drainage pit can overflow into the neighboring pit when the water level becomes higher by failure of the drainage pumps or an increase in drainage water into the drainage pit.

Each station drainage system consists of two (2) drainage pumping units located on the El. 27.30 m floor. The system raises the water from the drainage pit to discharge it into the tailrace water tank installed in EL. 42.0 m.

One (1) pump is for normal operations, while the other serves as a stand-by in case the water level in the pit become higher or the normal operating pump fails. The pumping units are also used for dewatering the draft tube and the spiral case for each unit. The amount of normal seepage and the drainage water which is collected by each drainage pit directed by a gravity will be approximately,

$$Q_{(P/H)} = 0.3 \sim 0.4 \ m^3/min \fallingdotseq 0.01 \ m^3/s$$

(d) Total inflow to the tailrace

From (b), (c), (d) mentioned above, the total inflow volume to the tailrace water tank is,

$$\text{Total} = Q\ (78.5\ mm/hr) + Q\ (dam) + Q\ (P/H) = 0.55\ m^3/s$$

(3) Discharge capacity to the tailrace

Drainage under gravity to the tailrace (gravity drain) depends on the function of tailrace bay water level caused by the spillway discharge to the tailrace bay. For a tailrace water level of EL. 41 - EL. 45, discharge capacity through the tailrace gravity drainage was obtained under the condition that the water level of the tailrace tank is EL. 45.0 corresponding to near top of the tank.

Discharge capacity of the tailrace drain tunnel pit (Gravity drain)

Discharge to tailrace bay m^3/s	Tailrace Water level EL	Discharge Capacity of Gravity Drain m^3/s	Remarks
350	41.0	1.09	Normal operation
1000	43.5	0.64	
1100	43.8	0.55	
1200	44.1	0.45	
1300	44.3	0.40	
1350	44.4	0.35	
1400	44.5	0.30	
1450	44.6	0.25	
1500	44.8	0.20	
1650	45.0	0.00	

As an alternative severe case, study was done on the condition that the 10 year probable flood with peak of 2000 m^3/s scale occurred in the reservoir catchment area and the past maximum rainfall 78.5 mm/hr occurred locally in the powerhouse area simultaneously. When 1000 m^3/s discharges to the downstream out of 2000 m^3/s under the condition that 1000 m^3/s volume will be stored in the reservoir as retention effect.

$$Q \text{ in } (78.5) = 0.55\ m^3/s < 0.64\ m^3/s \text{ (Gravity drain)}$$

Therefore, the drainage system can cope with the situation mentioned above. Two (2) drainage pumps with 500 l/min capacity are installed at the tailrace drain pit. One pump is for normal operation, while the other serves as a stand-by in case of emergency or the normal operating pump fails.

Fig. 1.6.13-1 Diagram of System Drainage

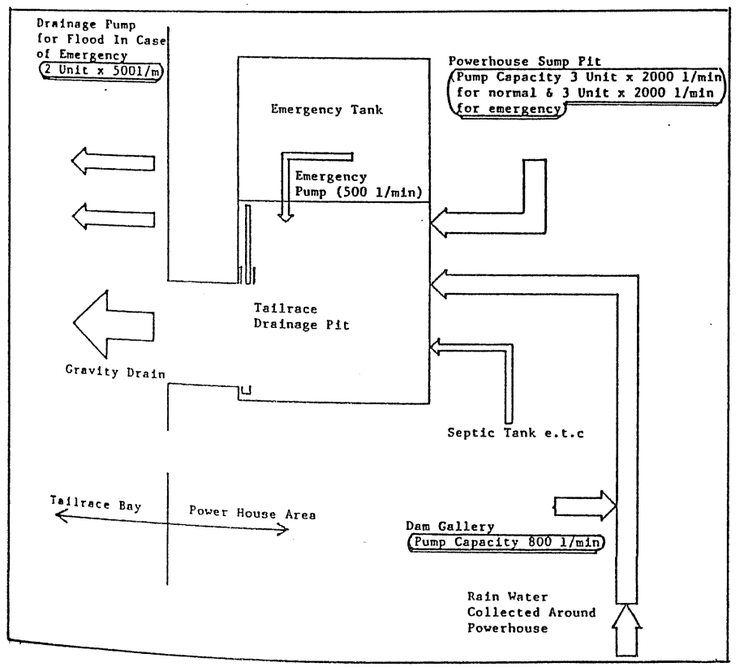
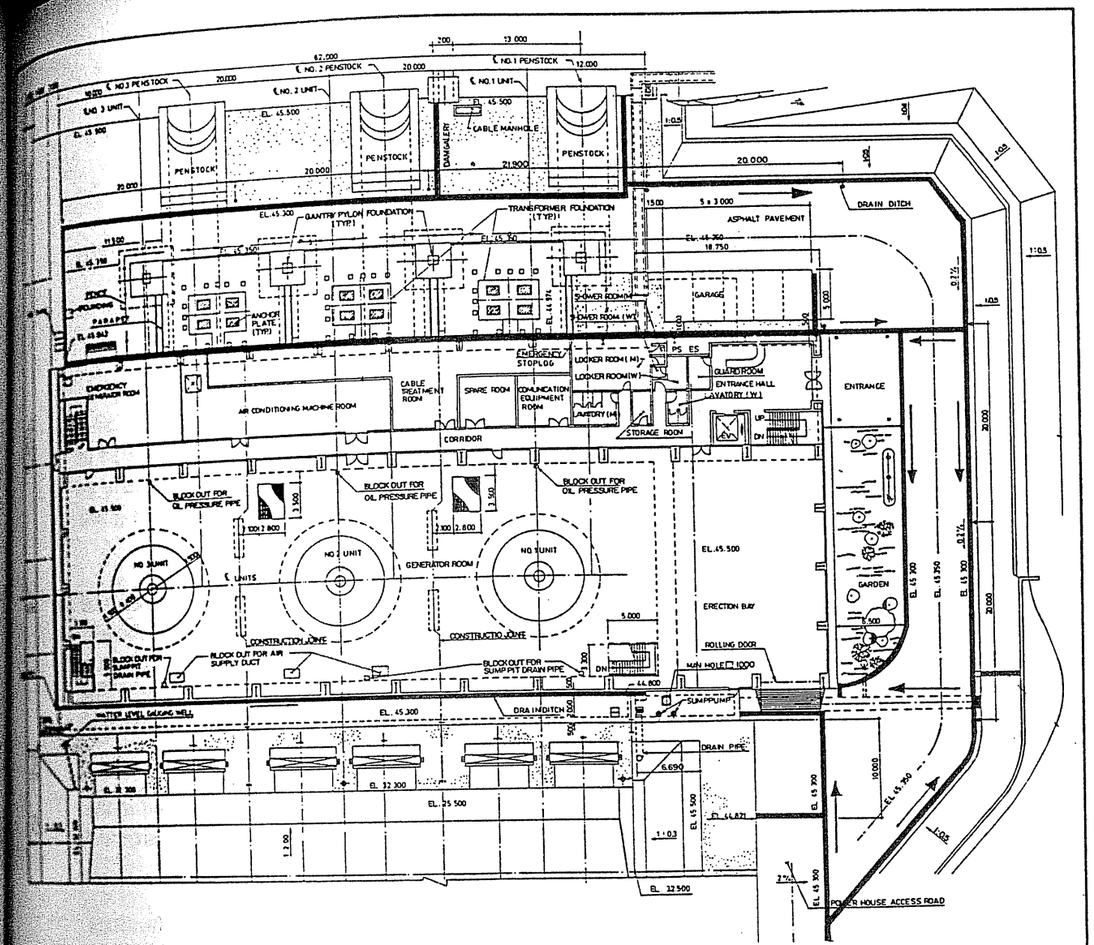
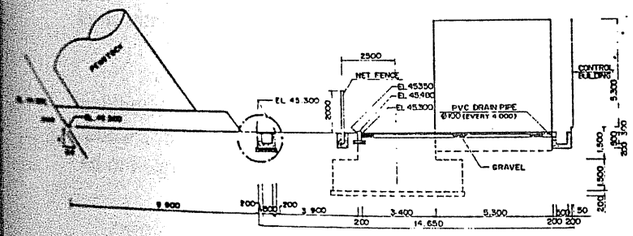
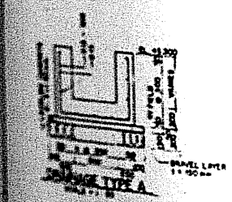


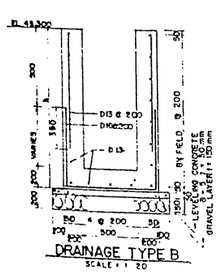
Table 1.6.13 Pump Capacity of Tailrace Sumpit

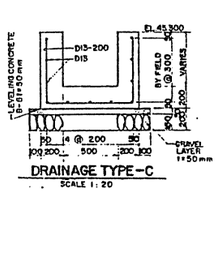
Drainage System at the Tailrace	
Water tank	Concrete structure
	Length 9.5m, width 2.0m
	Height 2.3m
Pump type	Submerged pump with float switch
Delivery head of pump	17.0m (EL. 59.5 - EL. 42.5)
Pump capacity	0.5m³/min
Pump motor	7.5KW, AC 230V, 50Hz
Number of pump	2 sets (1 set, contingency)
Drain pipe	Steel pipe Φ 80mm
Drain pipe length	51m (25.5m x 2 lines)
Drain gate	Knife type value
Gate dimension	Circle type Φ 0.5m
Total weight of gate	0.212 ton
Number of gate	1 gate
Manhole	Concrete precast 1000mm x 1000mm
Emergency Water Tank	
Water tank	Concrete structure
	Length 60.25m width 1.72m (bottom)
	Height 3.6m
Pump type	Submerged pump with float switch
Pump capacity	0.5 m³/min
Pump motor	2.2KW, AC 230V, 50Hz
Number of pump	1 set
Drain pipe	Steel pipe Φ 80mm
Drain pipe length	5m
Manhole	Concrete precast 950mm x 950mm
	750mm x 750mm











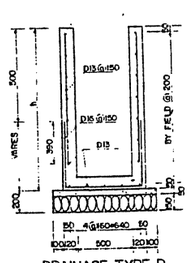


Fig. 1.6.13-2 General Layout of Drainage

6.14 Other Facilities

6.14.1 Water Supply Structure

The water supply facility was constructed for the purpose of supplying cooling water and fire hydrant water to the powerhouse, switchyard and other locations.

(1) Water Supply Intake Structure

(a) General layout

The preferred site for water supply intake pump with vertical intake tower is located at the upstream face of the dam on the left bank side. Alternative sites were examined as mentioned below i) and ii). These two alternative plans, however, are not as good compared to the site finally selected at the upstream face of dam.

i) A site located adjacent to the Boat Handling Facility about 300m upstream of dam was examined, however, it was decided this site is unsuitable especially considering difficulty of maintenance of the pump equipment such as taking on and off and changing the equipment, because of the inclined site condition. The water quality immediately adjacent to the boat handling facility could also be bad there.

ii) A vertical water intake tower of independent (stand-alone) type located in the reservoir was also studied, however this structure becomes quite large size and very costly.

The water supply intake structure has to provide necessary size and space to facilitate maintenance work (taking off and installing the pump and pipe with crane, inspection by diver). This structure has to accommodate the fluctuation of the reservoir water level from L.W.L 73.5 to H.W.L 85.0.

The thickness of slab and walls were designed to meet the necessary design condition of structure itself and unexpected load, for example, the load by crashing of driftwood. In consideration of these conditions, the thickness of slab and wall was assumed to be 50 cm based on the structural calculation and practical experience.

Intake Pump Installed

-	Type	:	Submerged type
-	Delivery head	:	60m
-	Delivery discharge	:	500 liter/minute
-	Diameter	:	80 mm
-	Motor capacity	:	11 kW
-	Power source	:	AC 230v, 50Hz
-	Number of installation	:	2 sets

(b) Design of structure

i) Design condition

Unit weight concrete	2.5 t/m³	Allowable stress	
Load : Superimposed	1.0 t/m²	Reinforcement bar	τsa=2000 kg/cm² (SD 35)
People group	0.3 t/m²		τsa=1000 kg/cm² (SD 35)
Own weight	1.25 t/m²	Concrete	τsa= 90 kg/cm² (Type B1,B2)

ii) Junction between dam body and pump pit

- Weight load : W = Load + Slab + Wall = 291.3 t
- Area of reinforcement bars : Total ΣAs = 424.0 m²
- Shearing stress intensity : τs = W/Σ As = 687 kg/cm² <τsa = 1000 kg/m²

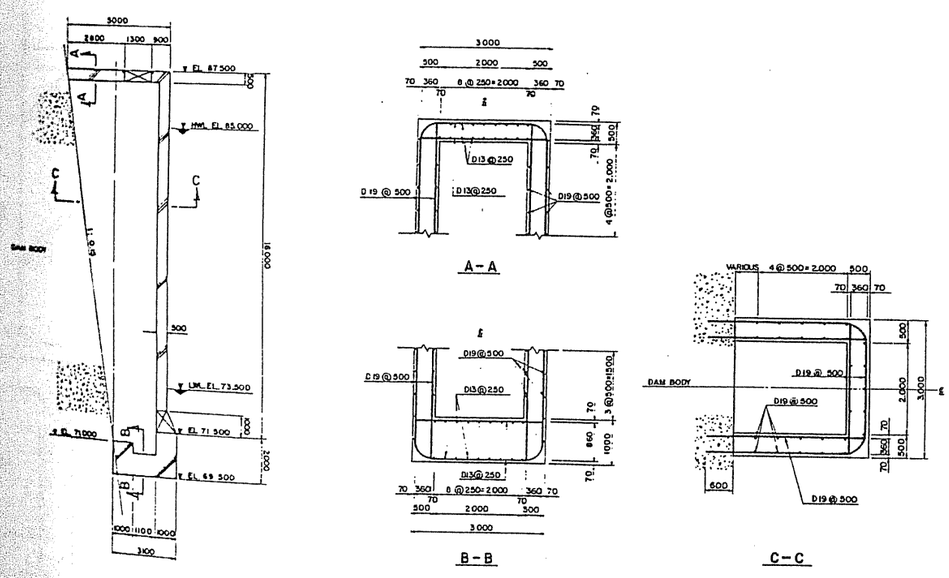
iii) Upper slab

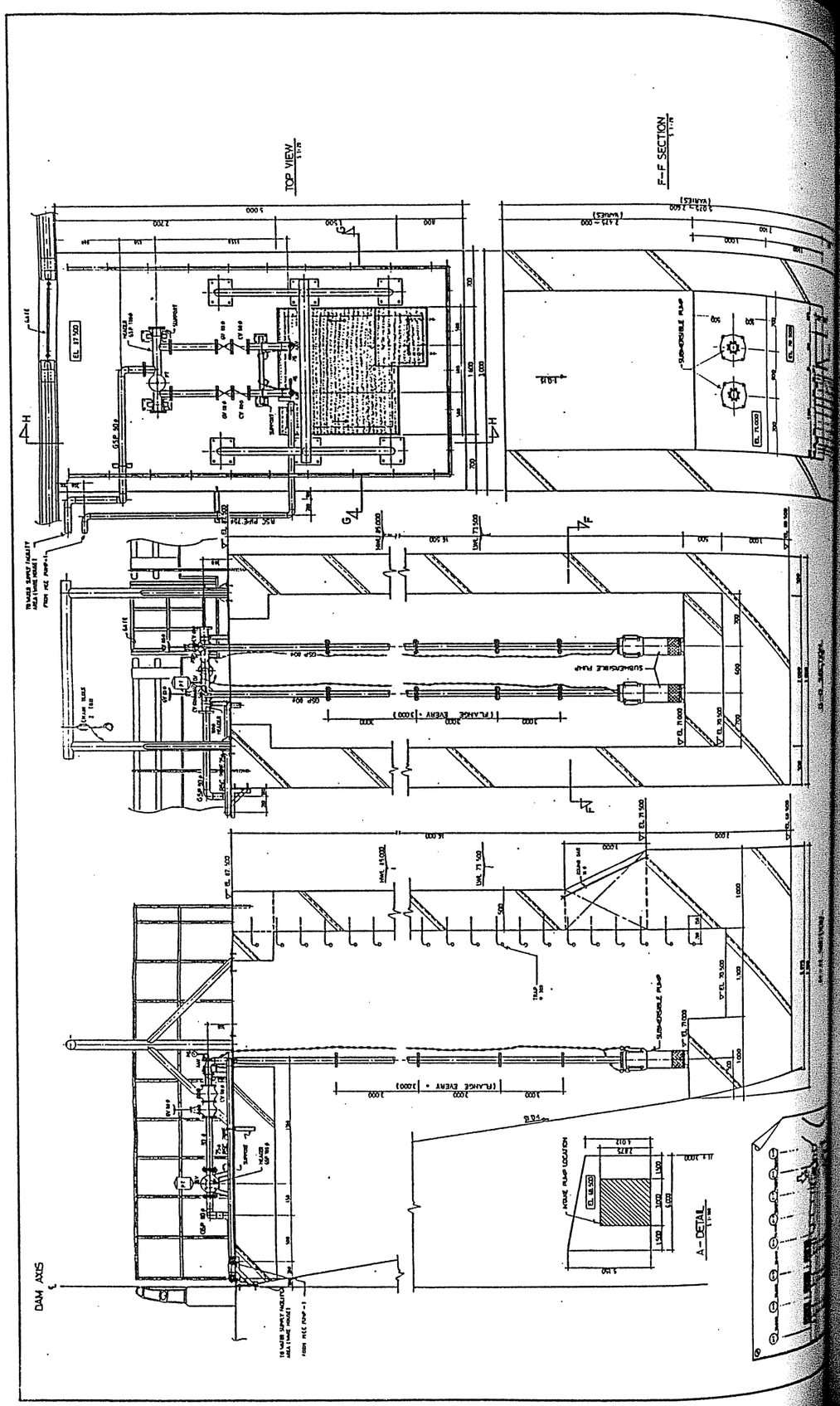
Upper slab was considered as a fixed beam supported by walls. From the design condition, total load q = 2.55 t/m². Moments at end part (A) and center part (B) are as follows.

$M_A = ql^2/6(k+2) = 0.3125$ t.m/m, $M_B = 1/8 \cdot q\, l^2 + M_A = 1.68$ t.m/m

Reinforcement was checked at the center part of large section force.

τc = 2M/KJbd² = 6 kg/cm² < τca = 90 kg/cm² OK
τs = M/As.JD = 881 kg/cm² < τsa = 2000 kg/cm² OK





(2) Water Supply Warehouse

The warehouse for housing the treatment equipment is located at EL. 87.5 on the left bank of the dam site. The warehouse site was originally planned at the mountain side of the access road just downstream of the dam, however, the space of this area is very limited and the large scale batching concrete plant had been installed adjacent to the area during the whole construction period. Therefore, the warehouse site was moved upstream about 50m where there was enough space for the warehouse and the access road to boat handling area.

The main features of the warehouse are it prefabricated steel structure with 7m wide and 30m long. These dimensions, changed from the original design that was 5m wide and 20m long because the housing space for treatment equipment in the original dimensions was found to be very tight and narrow and there needed some more space for future maintenance and repairing works. An additional pump was also installed to supply water the to entrance guard house.

The several equipment installed in the warehouse are as follows.

a) Pressure tank

Type	: Steel tank automatic air supply equipment
Capacity and max pressure	: 1.0 m^3, 10 kg/cm^2
Dimension and number of tank	: Diameter 1.0 m, high 1.5 m, 2 set

b) Raw water tank

Type and capacity	: Steel tank (JIS G 3101), 30 m^3
Dimension and number of installation	: Width 3m, length 5m, height 2m, 1 set

c) Rapid sand filter

Pump type	: Multistage pump 3.7 kW x 2 set
Capacity and dimension	: 20 m^3/min, Diameter 2.3, height 1.5m
Filter material and sand filter	: Anthracite 4 m^3, Sand 1.2 m^3 (Diameter 0.45 mm to 0.75 mm)
Control panel and number of installation	: Width 0.6m, length 0.4m, height 1.25m, 2 set

d) Filter tank

Type and capacity	: Steel tank (JIS G3101), 30 m^3
Dimension and number of installation	: Width 3.0m, length 5.0m, height 2.0m, 1 set

e) Pump

Pump type and number of installation	: Self priming multistage pump, 2 set
Deliver head and discharge	: 120m, 25 liter/min

f) Control panel

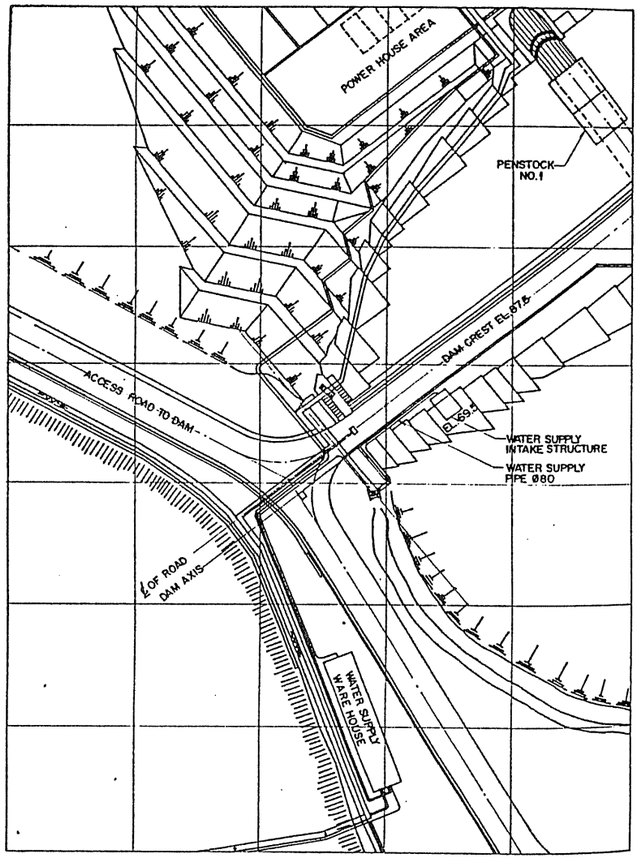
Power supply for the control panel was supplied through the distribution panel installed in the powerhouse.

Type : A self standing type with a door
Dimension : width 1.0m, length 0.5m, height 2.0m

g) Piping

The main supply piping from the warehouse to delivery point is as follows :

Delivery point	Approx. Distance	Pipe diameter
Intake pump	150 m	80 mm
Electric equipment	400 m	65 mm
Powerhouse building facility	400 m	80 mm
Switchyard	400 m	20 mm



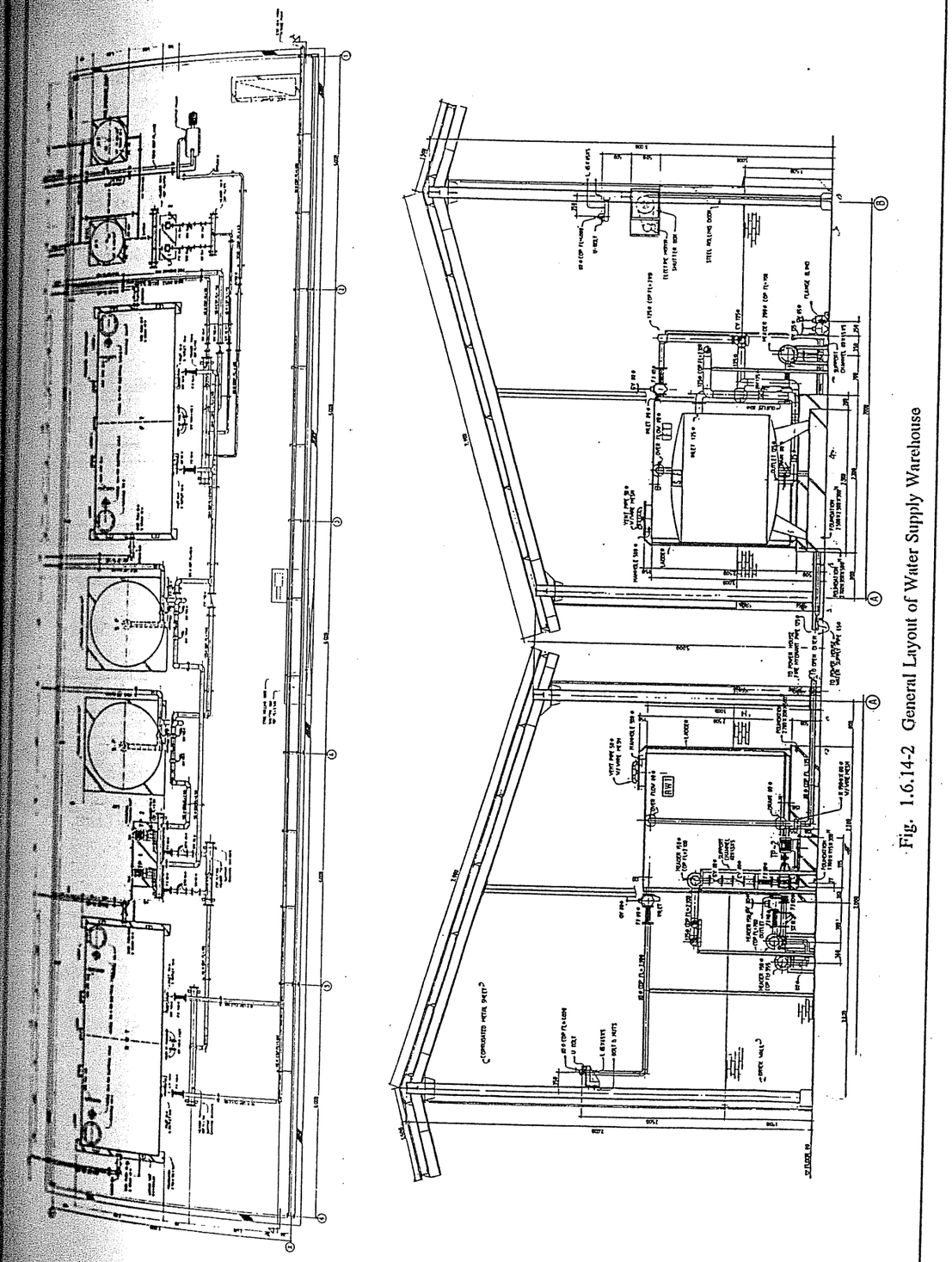


Fig. 1.6.14-2 General Layout of Water Supply Warehouse

6.14.2 Boat Facilities

The boat facilities such as boat, warehouse, inclined trolley and jib crane and so on are located in the left bank about 300m upstream of dam.

(a) Boats

For monitoring and inspection of the dam and reservoir area, two (2) boats with 40 horse power engines and two (2) boats with 150 horse power engines were provided at the boat facility site. The boats are made from fibreglass. The boat dimension is as follows :

40 horse power engine		150 horse power engine	
Gross capacity	: 1.88 ton	Gross capacity	: 2.2 ton
Boat weight	: 350 kg	Boat weight	: 400 kg
Engine capacity	: 698 cc	Engine capacity	: 2600 cc
Engine speed	: 5500 rpm	Engine speed	: 5000 rpm
Fuel tank capacity	: 24 liter	Fuel tank capacity	: 24 liter

(b) Warehouse

The warehouse can store two boats, a hoist, inclined trolley with winch and lighting facilities. A rolling door was installed at the entrance of the warehouse.

Type of structure	:	Prefabricated steel structure
Width, length, height	:	8.5m (W), 15.0m (L), 9.0m (H)

(c) Hoist

The hoist facilitates loading and unloading the boats to and from the inclined trolley and other transport means, and aid in maintenance and overhaul works. The hoist is of a single beam chain hoist type suspended from the bottom flange of the runaway beam. The hoist is electrically operated.

Type of hoist, range and design load	:	Overhead travelling type, 6m, 2t
Hoisting and travelling speed	:	8.4 m/min and 21 m/min
Motor	:	0.3 kw, Ac 230 V, 50 Hz

(d) Incline trolley

Incline trolley transports a boat on rails from EL. 70.5 m to the warehouse at EL.87.5m. The dimension of the trolley and the winch is as follows :

Trolley type of structure and design load	:	Steel structure, 3.5 ton
Width, length, height	:	4.0m (W), 3.5m (L), 1.5m (H)
Winch design load and hoisting speed	:	7.5 ton, 2 m/min
Wire rope	:	Ø 22.4 mm JIS G 3525 (Galvanized) length 55m
Winch motor	:	5.5 kW, AC 230 V, 50 Hz

(e) Jib crane

The jib crane is installed at EL. 87.5m adjacent to the log boom anchor for lifting up logs and debris which are trapped by the log boom.

Type of crane	:	Post type
Boom height and lifting capacity	:	5m, 3 ton
Hoist range	:	19m (EL. 92.5 to EL 73.5)
Horizontal rotation angle and boom radius	:	340 degree, 5m

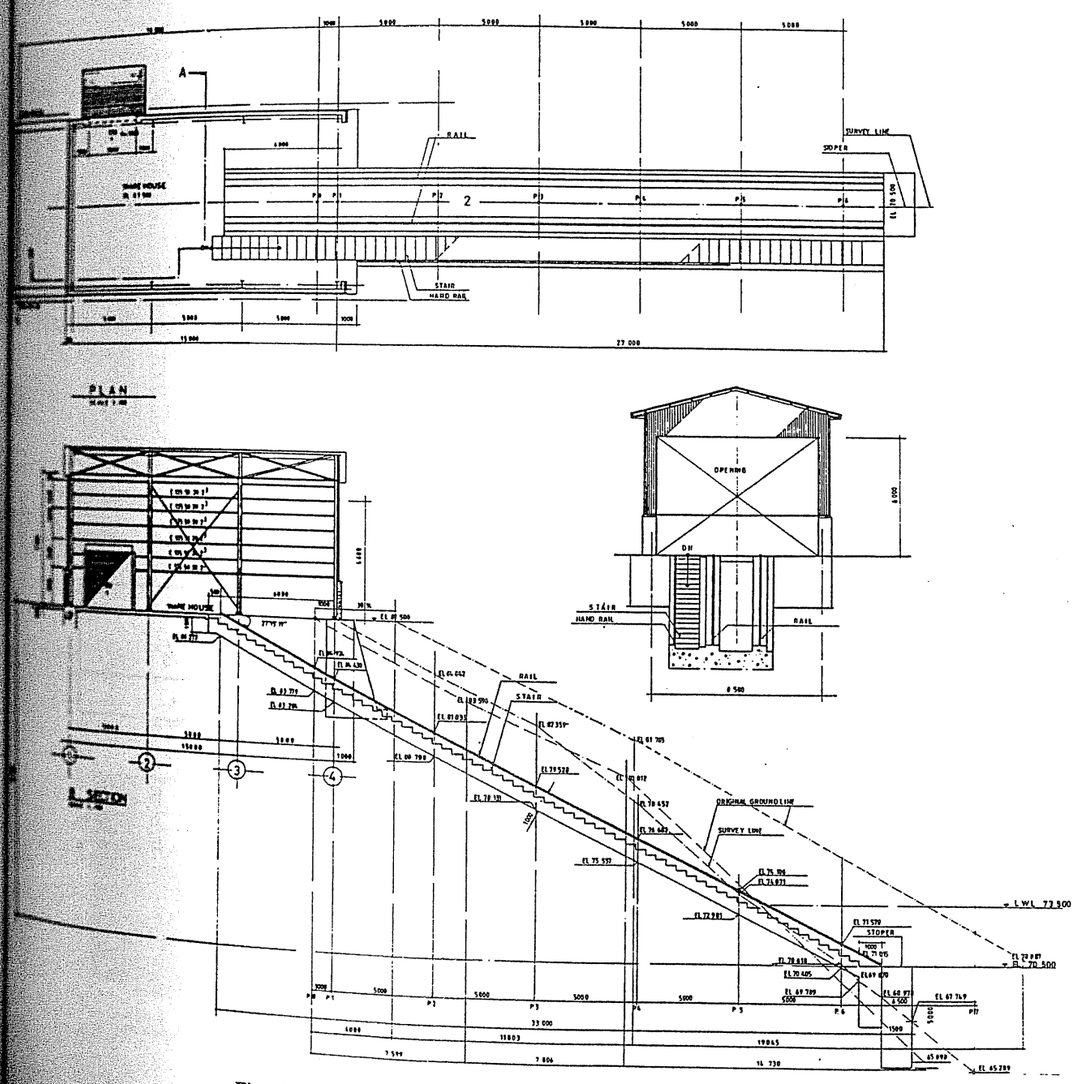


Fig. 1.6.14-3 General Layout of Boathandling

6.14.3 Log boom

The log boom consists of water tight floats connected together by a wire rope anchored at the left and right banks.

Float styrofoam	:	dia. 660 mm, length 940 mm
Cham	:	dia. 22 mm, length 1000 mm
Main rope and nylon rope	:	dia. 30 mm and dia. 12 mm
Anchor block	:	material concrete class B4, height 2000 mm, width 2500 mm, length 2500 mm

Design of log boom foundation

a) Resistance force

For against inflow water, $P = ½.Pw\ Vw^2\ Cw\ A$ (Kgf)

where, Pw : Unit weight of water (= 102 kgf/m³)
Vw : Inflow water's velocity (= 1.0 m/s)
Cw : Coefficient = 1.0 (= 0.154 m²)
A : Area of lower part from water surface

For water wave's load, $P = ½.Pw\ U^2\ Cw\ A$ (Kgf)
where, U : Velocity of water particle (m/s)= $(\pi\ H/T)\ e^{KZ}$, $K = 2\pi/L$, $Z = H/Z$
H : Wave height = $0.30\ [1-\{1+0.004\ (gF/Va^2)^{½}\}^{-2}]Ua^2/g$
g : 9.8 m/s
Ua : Wind velocity (m/s) = 10 m/s
F : Fetch distance (m) = 1700 m
L : Wave length (m) = $gT^2/2\pi$
T : Cycle = $1.37\ [1-(1+0.008\ (gF/Ua^2)^{½})^{-2}]\ 2\pi\ Va/g$

From the above expression, the resistance force is as follows,

	Against inflow water (kg/m)	Water wave's load (kg/m)
Driftwood Nylon net Total	P1 = 25.50 P2 = 10.996 Pin = P1+P2 = 36.496	P3 = 14.267 P4 = 6.152 Pw = P3 + P4 = 20.419
Total Pt	Pt = Pi + Pw = 56.915 kg/m	

(b) Tension force of main rope

$T = WL/(8\cdot\delta)\cdot(L^2+16\delta^2)^{½} = 10583.173$ kg

where, W : Resistance force = 56.915 kg, L : Distance to another anchor = 2500
δ : Deflection of main rope = L.n (n : 0.286)

(c) Stability of anchor block

Design and loading condition

Anchor block		Soil	
Height	H = 2.0 m	Unit weight	γs = 1.9 t/m³
Width and length	B = L = 2.5 m	Soil pressure	Ps = 0t
Unit weight	γc = 2.25 t/m³	Bearing capacity	Qa = 30 t/m²
		Friction coefficient	μ = tan ϕ=0.6

Load item	V (t)	x(m)	V.x	H (t)	y (m)	H.y
Self Weight	29.375	1.25	36.719	0	1.0	0
Tension load of rope	0	0	0	10.583	1.0	10.583
Σ (ton)	29.375		36.719	10.583		10.583

Stability check

- Against sliding, $F = \mu \times ZV / ZH = 1.665 > 1.5$ OK
- Against overturning, $d = (\Sigma V.x - \Sigma H.y) / Zv = 0.890 > b/3 = 0.833$ OK
  $e = b/z - d = 0.360 < b/6 = 0.417$ OK
- Against bearing capacity, $q = \Sigma V/bc\ (1 \pm 6e/b) = 8.761$, $Q/q = 3.424 > 3.0$

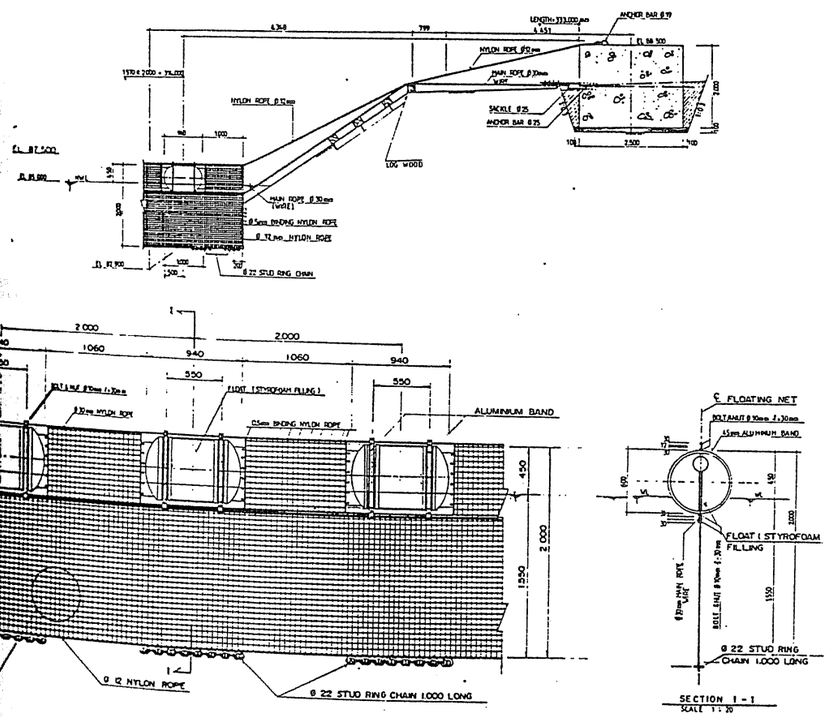


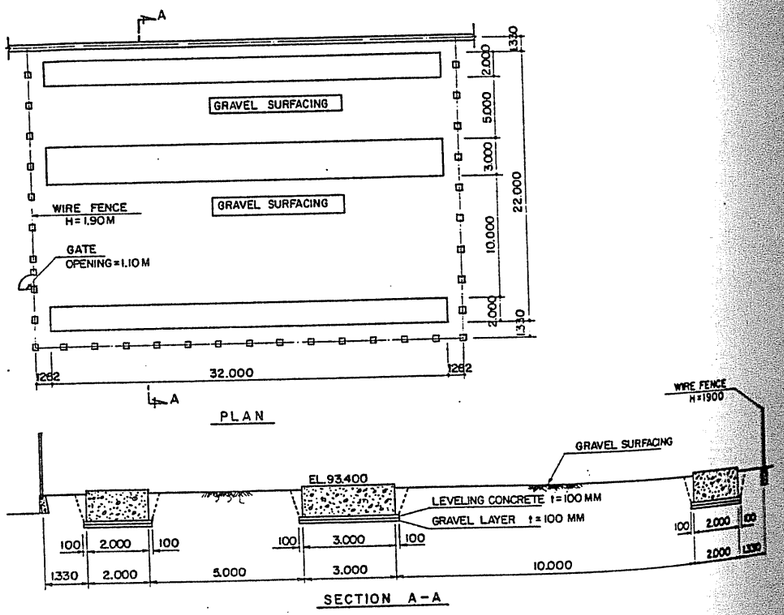
Fig. 1.6.14-4 General Layout of Logboom and its Foundation

6.14.4 Stoplog yard

The warehouse for spillway and intake stoplogs was originally designed for the left bank near the dam, but the stoplog area was moved to the project entrance area behind the guardhouse in consideration that the dam site space is very limited and its surrounding area is topographically unfavourable for further excavation.

The stoplog warehouse was also deleted and changed to the open yard type supported on concrete slab foundation. The area is covered by gravel and enclosed with a wire fence. It would be easy and practical for the large spillway stoplogs to be taken out and in from the opened yard compared to the warehouse. The stoplog yard is 10m wide and 20m long.

Spillway stoplog		Intake stoplog	
Size	: 11m (L), 1.5m (W)	Size	: 6m (L), 1.2m (W)
Quantity	: 12 panels	Quantity	: 20 panels
Total weight	: 122 ton	Total weight	: 100 ton



6.15 Muara Takus Buddhist Remains

(1) General

The remains of a Buddhist temple are located about 1 km south from Muara Takus Desa along the Kampar Kanan River, about 25 km west from the dam site in a straight line. The Temple, consisting of six remains were believed to be built in the 11th through to 12th century and they have been rehabilitated since 1977.

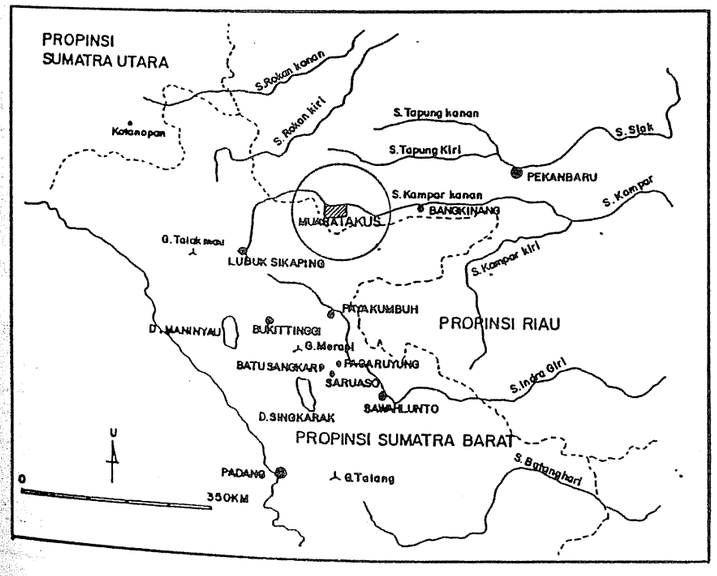
Since it is important from a historical point of view, study of the reservoir water level was carried out in accordance with a basic policy for preserving the remains so that they will not be submerged nor be affected by execution of this project.

The survey and study for preservation of the remains were carried out by PLN and the Ministry of Education and Culture. It was concluded that special countermeasure for preservation of the remains would not be required because the high water level of reservoir was set at EL. 85.00 and the temple situated at EL 86.25 m would not be submerged by reservoir impounding.

It is predicted that the highest water level of the reservoir for the flood occurrence of 200 year return period could reach to EL. 84.70 m according to spillway gate control at the Kotapanjang dam. As for back water arising in the flood time, the water level would rise to EL. 86.10m that is 0.15 m lower than the elevation of the Temple site.

However, in the light of fact that there are evidences indicating that the temples had been surrounded by a wall embankment in the past, it was necessary to study the need for protection of the surrounding area with a new embankment, especially against back water arising in the flood time. Therefore, the study was done for the riverbank and the temple foundation.

Fig 1.6.15-1 Location of Muara Takus Buddhist Remains Site



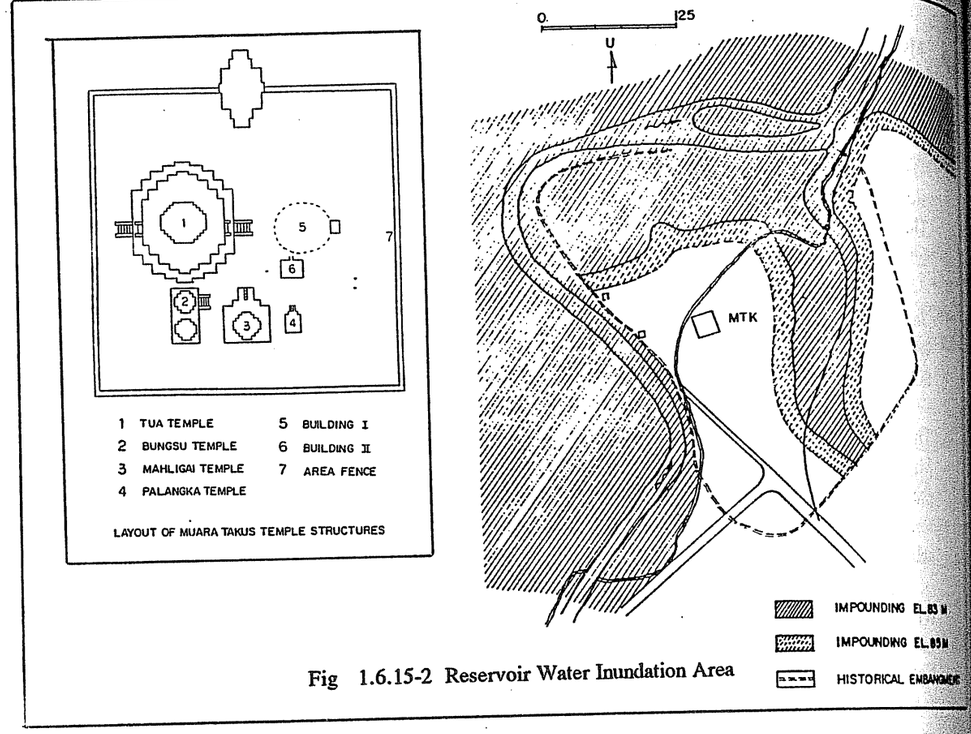


Fig 1.6.15-2 Reservoir Water Inundation Area

(2) Riverbank Stability

It may be necessary to stabilize and ported the slope along the riverbank at the western part of Muara Takus Historical Site in order to avoid land sliding due to reservoir water. There are some parts, however, where protection work is not required due to good geological condition with very steep or nearly vertical cliff riverbank. A slope stability calculation was done to check possibility of land slide a long the riverbank.

The most critical slope section of the river bank was selected for a slip circle analysis. At present, the river bank slope is self-supported and there is no sign of landslide. In consideration of this present condition and the numerical value which is generally used for various types of foundation, the characteristics of the foundation were assumed as shown in the table below. The geological condition was classified by boring log carried out at Muara Takus Temple site and a geological site survey along the riverbank.

Geological Condition and Property

Geological condition	Depth (m)	Unit Weight (t/m^3)	Angle Friction (")	Chesion (t/m^2)
Backfill and Capping	0~0.25	1.90	25	0.0
Yellow Clay	0.25~2.0	1.90	30	1.0
Sandy Clay	2.0~3.0	2.00	30	3.0
Coarse Sand	3.0~4.5	2.00	35	4.0
Gravel Layer	4.5~7.0	2.10	35	5.0
Tuff	7.0 ~	2.10	40	5.0

The safety factor 1.2 is generally used for slip circle analysis.

$$Fs = \frac{Mr}{Ms} = \frac{R.\Sigma(C.l + W\ Cos\ \alpha + \tan\ \phi)}{\Sigma WX + \Sigma Fn.\gamma h + \Sigma Fv.x} \geq 1.2$$

where ; Ms : Slide moment (t/m^2)
Mr : Resistance moment (t/m^2)

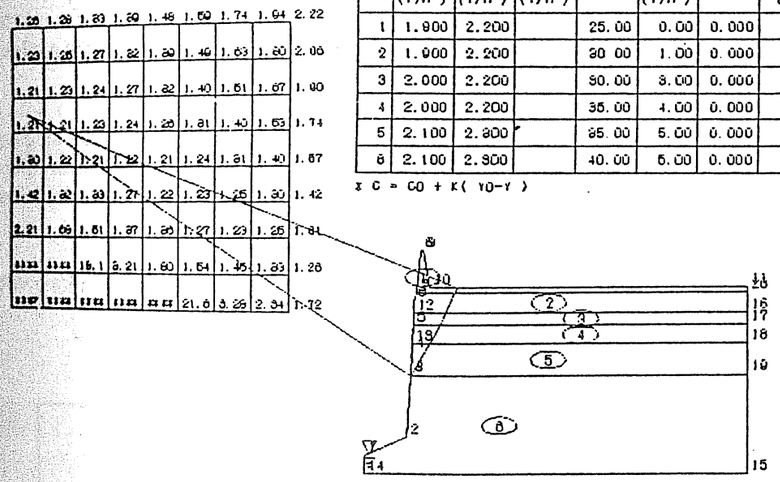
The calculation was done for the most critical condition of land slide occurrence with possibility that the underground water table level rises by seepage inflow from reservoir water and it rains at the same time.

	X (m)	Y (m)	R (m)	FS	MR (T.m/m)	MS (T.m/m)
1	-28.50	101.50	38.68	1.202	1245.	1035.

※ SOIL CONDITION & SEISMIC COEFFICIENT

	GAMMAT (T/m^3)	GAMMAS (T/m^3)	GAMMA' (T/m^3)	PHAI	CO (T/m^2)	K	YO (m)	KH
1	1.900	2.200		25.00	0.00	0.000	0.0	
2	1.900	2.200		30.00	1.00	0.000	0.0	
3	2.000	2.200		30.00	3.00	0.000	0.0	
4	2.000	2.200		35.00	4.00	0.000	0.0	
5	2.100	2.300		35.00	5.00	0.000	0.0	
6	2.100	2.300		40.00	6.00	0.000	0.0	

※ C = CO + K(YO-Y)



In this critical condition. the safety factor show 1.202. Therefore, there is no possibility of land slide occurrence of river bank on a large scale after impounding of reservoir, because the value of safety factor increase in proportion to reservoir water rise and the safety factor will become more than 1.2.

As for the fluctuation of reservoir water level, its speed is very small because the reservoir capacity is very large for the required reservoir operation discharge, about 0.1 m/day at around HWL 85.0 m. 0.2 m/day at around LWL 73.5 at maximum. Also reservoir water level is normally operated up to EL. 83.000 m, then there is little chance that reservoir water level will become higher than EL. 85.000 and will not directly impact on the riverbank. From the above consideration, the riverbank protection would not be required at this site.

(3) Temple for Foundation

In consideration of the soil condition and the reservoir high water level of EL. 85.0 m, the permeability zone of the temple foundation might be improved by consolidation grouting or concrete wall construction embedded around the temple structure and decrease water following through the foundation and partially increase soil bearing capacity.

Concrete wall construction, however, would need a rather large scale excavation in the limited space of the temple site, and the existing drainage system must be removed or newly constructed. Therefore, the grouting method is more practical and favorable in consideration of this site condition.

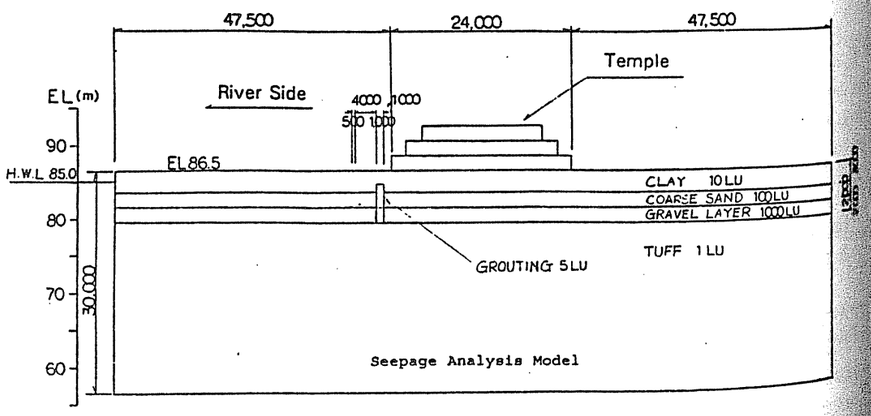
In order to check permeability improvement of the temple foundation by the countermeasure of consolidation grouting, a seepage analysis was carried out for the temple foundation.

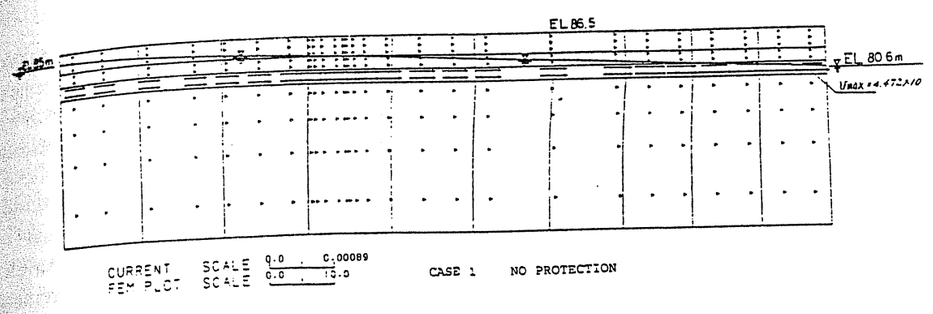
(a) Seepage Analysis of the Temple Foundation

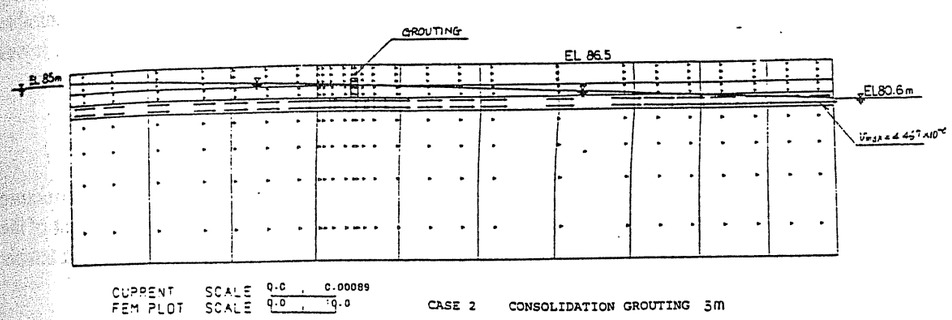
The shortest distance from riverside to the temple structure was chosen as a critical section for the seepage flow analysis. For simplification of the calculation, the analysis area is conveniently modeled as shown in the figure below.

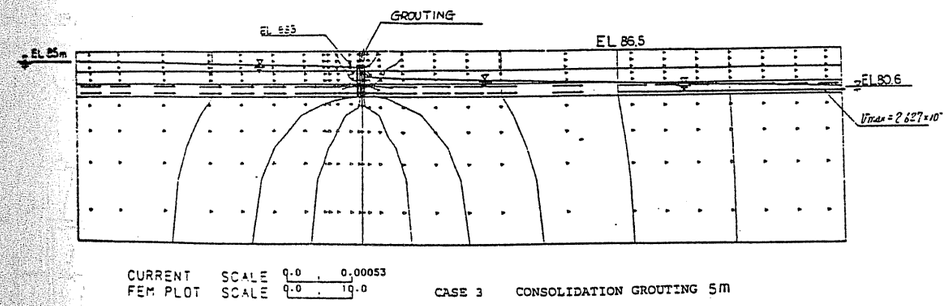
The boundary condition set water level EL. 85.0 (HWL) for upstream reservoir side and water level EL. 80.6 (NWL) for downstream back site. It is assumed that the underground water level of EL. 80.6 m will be kept constant around Muara Takus Temple during reservoir operation.

The dimension of consolidation was assumed 1.0 m wide and 3.0 m - 5.0 m long with target permeability of 5 Lu.









The result of Seepage Analysis

Cuse No.	Protection Method	Water Level under the Temple		Max. velocity (m/s)
1	Present condition	EL. 83 m	-	4.472 x 10^0
2	Grout for EL. 86.25 0 EL. 81.5	EL. 83 m	0 m (lowered)	4.467 x 10^0
3	Grout for EL. 86.25 - EL. 79.5	EL. 82 m	1 m (lowered)	2.627 x 10^0

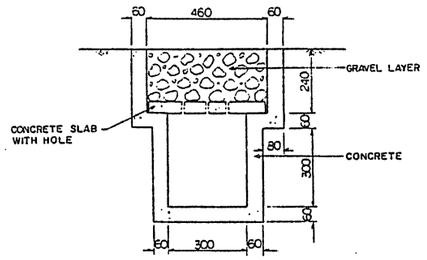
In the case of the present condition without countermeasure, water level under the temple is at EL. 83 m. In the case of grouting down to EL. 81.5 m, the water level of the temple foundation remained at EL. 83 m that is the same as present condition. For grouting down to EL. 79.5 m, water level of the temple foundation become EL. 82.0 m that is lowered about 1 m compared to present condition.

From the calculation result, there is no remarkable improvement by grouting around the temple structure. It is considered that there is a gravel layer 2 m thick, 1000 Lu under EL. 81.5m where grouting can not demonstrate its effect. Also, high pressure by grouting injection into the foundation surrounding the temple structure may cause unstable foundation conditions and an adverse effect on the temple structure. Therefore, it was concluded that foundation improvement of the temple would not be required.

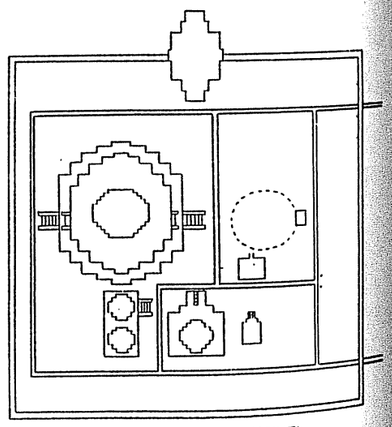
(b) Drainage System and Wells

From the seepage analysis, it was concluded that the grouting around the temple structure was not so effective and special countermeasures as not taken for the improvement of the foundation. It is predicted, however, that there might be some influence on the temple foundation by the reservoir water especially during heavy rainfall.

For protection of water infiltration to the temple foundation, two wells were constructed outside of the temple to pump the water up to the drain ditch along the perimeter of the temple site. The water level of the temple site is also observed by the installation of the wells what are results of well monitoring.



EXISTING DRAINAGE SYSTEM


DETAIL OF DRAINAGE CHANNEL

(c) Access Road to Muara Takus

The existing village road about 480 m long and 5.0 m wide branches from the provincial road provicles access to the historic Muara Takus Temple site. The road however, often flooded and traffic gets suspended in the rainy season. Based on the discussion with Ministry of Education and Culture and PLN, the road improvement was done for side ditch drain, raising the low sections of road masonry wall construction was made at the branch with the provincial road.

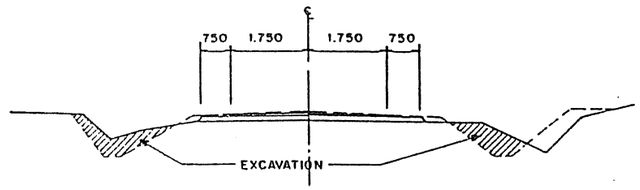
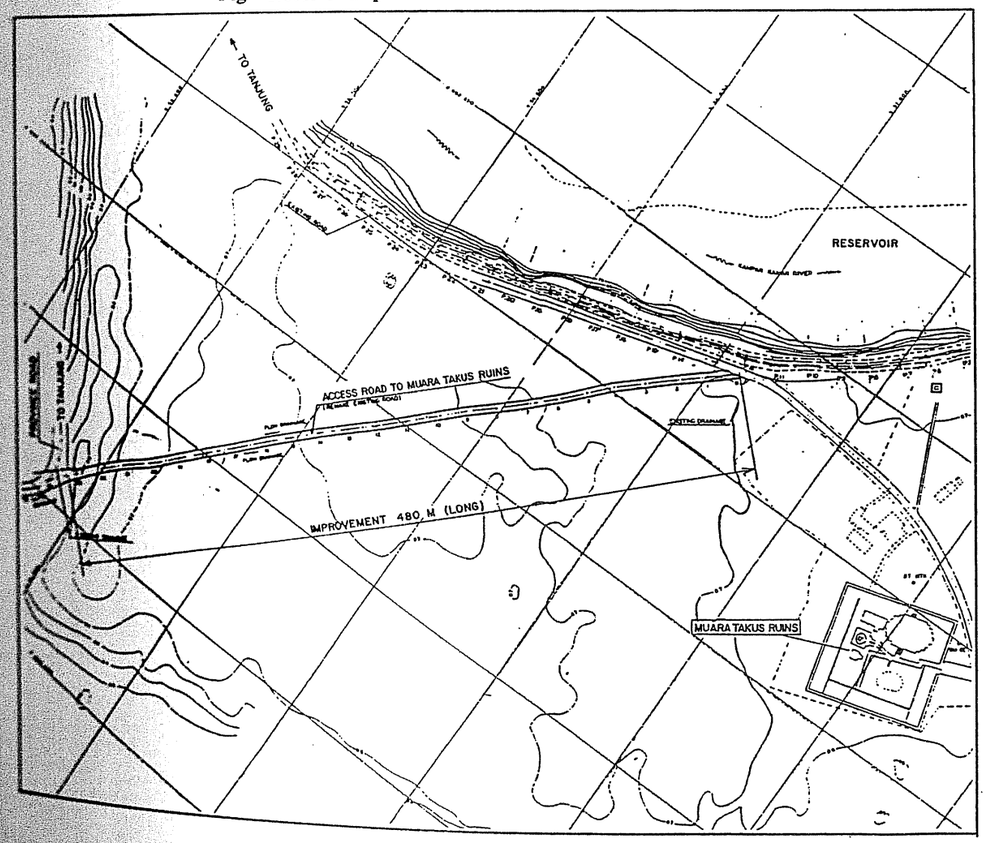


Fig 1.6.15-3 Improvement of Access Road to Muara Takus



PART 2

CONSTRUCTION OF CIVIL WORKS

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Chapter 1 GENERAL.

The Civil Works Contract, Lot I was awarded on 19th August 1991, and concluded on 23rd May 1992 between PT. PLN (PERSERO) and HAZAMA CORPORATION - PT. BRANTAS ABIPRAYA JOINT OPERATION under the Contract No. 036.PJP/922/1992/M. The Civil Works contract had twelve key dates shown below. The contract was on the basis of Bill of Quantities, and the contract period was 62 months from 16th October, 1992 till 15th December, 1997. The works were actually completed on 15th November, 1997 and taken over to the Employer PLN one month earlier than the scheduled key date. The project layout and construction progress of works are shown in Fig.2.1.1 and Fig. 2.1.2 respectively.

Table 2.1.1 Contract Key Date

Part of Works	Contract Schedule Date	Completion Date
(1) Partial completion of dam ready for dam inside penstock installation	15 Mar. 1995	15 Oct. 1994
(2) Partial completion of dam ready for intake gate installation	01 Mar. 1996	17 Jan. 1996
(3) Partial completion of dam ready for spillway gate installation	01 Jan. 1996	14 Aug. 1995
(4) Partial completion of dam ready for trash rack installation	01 Mar. 1996	17 Jan. 1996
(5) Partial completion of dam ready for trash rake installation	01 Mar. 1996	09 Feb. 1996
(6) Tailrace works ready for tailrace gate installation	01 May. 1995	28 Mar. 1995
(7) Tailrace works ready for tailrace gantry crane installation	01 Mar. 1996	20 Jan. 1996
(8) Powerhouse sub-structure concrete ready for draft tube liner installation	15 Dec. 1994	01 Dec. 1994
(9) Powerhouse works ready for overhead crane installation	01 Sep. 1995	09 Aug. 1995
(10) Powerhouse of substructure concrete ready for turbine and generator installation	01 Aug. 1996	16 July 1996
(11) Switchyard foundation ready for equipment installation	01 Jun. 1995	01 Jun. 1995
(12) Completion of the whole works (Time for Completion)	15 Dec. 1997	15 Nov. 1997

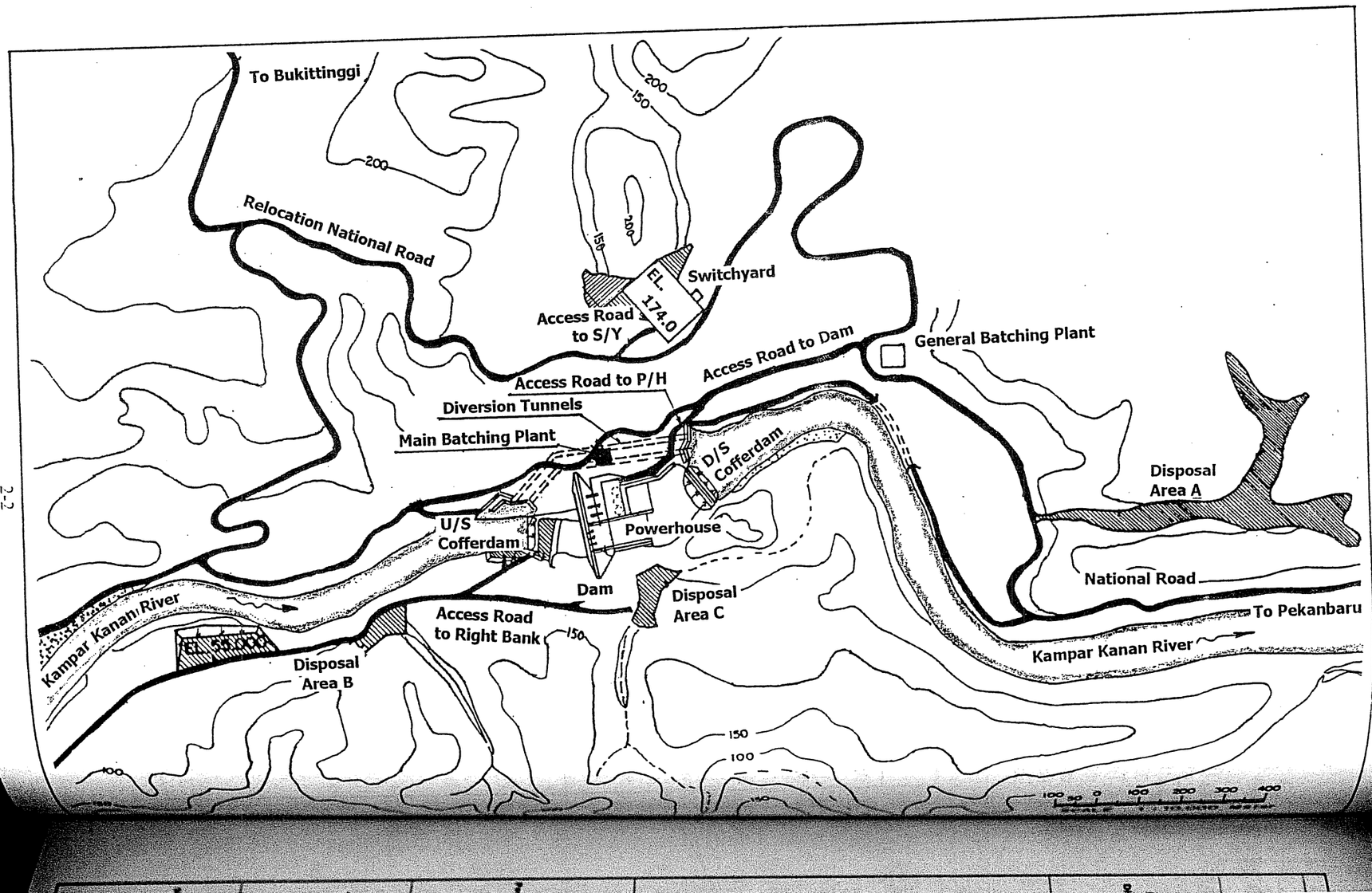
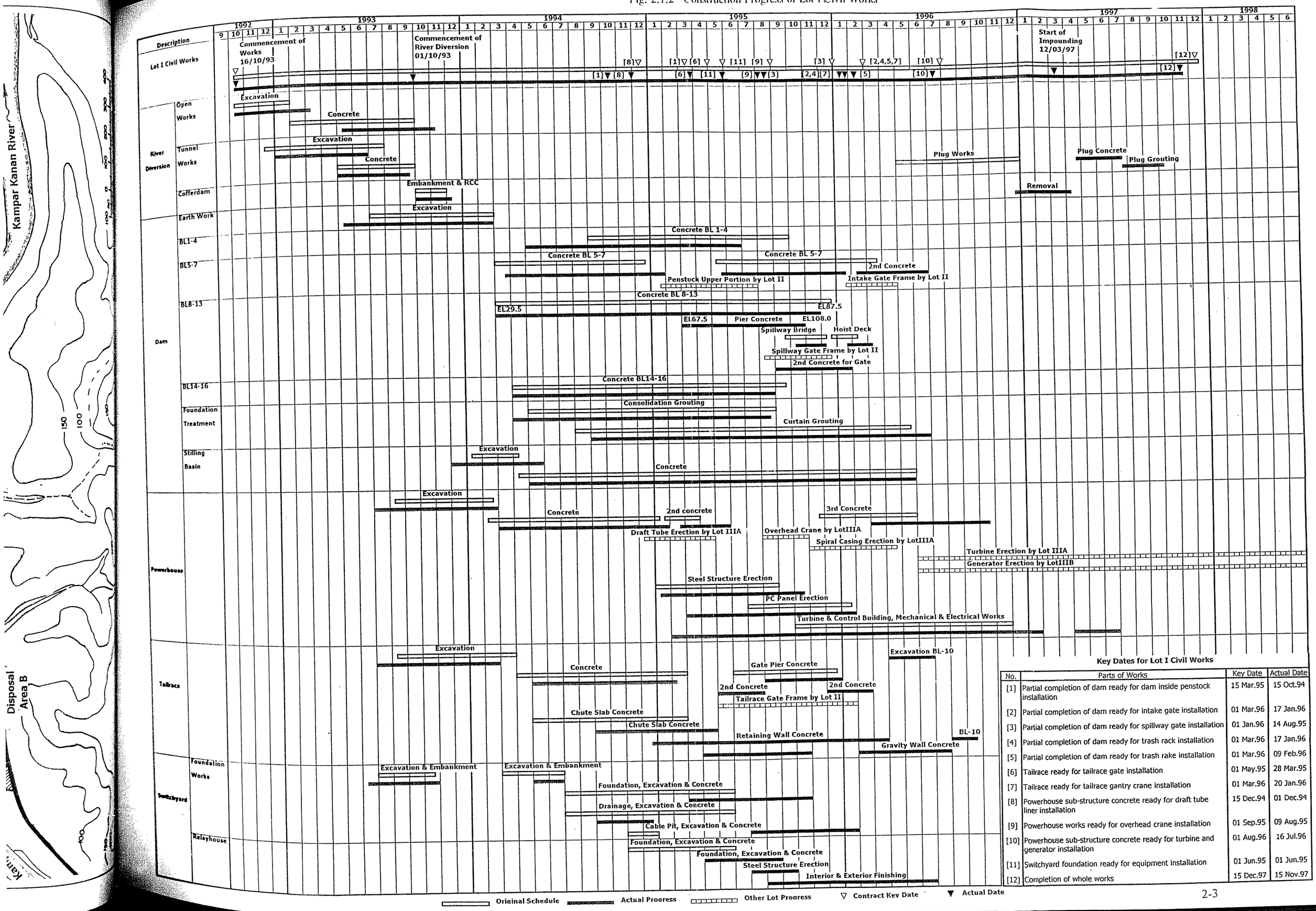

To Bukittinggi
Relocation National Road
Switchyard
EL. 174.0
Access Road to S/Y
Access Road to P/H
Access Road to Dam
General Batching Plant
Diversion Tunnels
Main Batching Plant
D/S Cofferdam
U/S Cofferdam
Powerhouse
Dam
Disposal Area A
National Road
To Pekanbaru
Kampar Kanan River
Disposal Area C
Access Road to Right Bank
Disposal Area B
EL. 55.000
200
150
100

Fig. 2.1.2 Construction Progress of Lot I Civil Works



Key Dates for Lot I Civil Works

No.	Parts of Works	Key Date	Actual Date
[1]	Partial completion of dam ready for dam inside penstock installation	15 Mar.95	15 Oct.94
[2]	Partial completion of dam ready for intake gate installation	01 Mar.96	17 Jan.96
[3]	Partial completion of dam ready for spillway gate installation	01 Jan.96	14 Aug.95
[4]	Partial completion of dam ready for trash rack installation	01 Mar.96	17 Jan.96
[5]	Partial completion of dam ready for trash rake installation	01 Mar.96	09 Feb.96
[6]	Tailrace ready for tailrace gate installation	01 May.95	28 Mar.95
[7]	Tailrace ready for tailrace gantry crane installation	01 Mar.96	20 Jan.96
[8]	Powerhouse sub-structure concrete ready for draft tube liner installation	15 Dec.94	01 Dec.94
[9]	Powerhouse works ready for overhead crane installation	01 Sep.95	09 Aug.95
[10]	Powerhouse sub-structure concrete ready for turbine and generator installation	01 Aug.96	16 Jul.96
[11]	Switchyard foundation ready for equipment installation	01 Jun.95	01 Jun.95
[12]	Completion of whole works	15 Dec.97	15 Nov.97

1.1 Contractor's Organization and Manpower Deployment

Organization for the execution of the civil works is shown in Fig. 2.1.3. Manpower deployment for the works was mostly on the two-shift basis, with the working hours as shown below :

Shift	Working hours	Break
Day	7:00 - 18:00	12:00 - 13:00 *
Night	19:00 - 6:00	24:00 - 1:00

(* 11:30 - 13:30 on Friday for the religious service)

The aggregate man-days for the civil work were 1,106,056 with the peak of 26,656 man-days recorded in December 1994, and the ratio of key person to local staff and labourers was 1:5.7 : 17.7 as shown in Fig. 2.1.4.

1.2 terial

The principal materials used for the civil work areas are summarized as shown in Table 2.1.2.

(a) Cement

The total quantity of cement used for the works was 112,120 tons, which consisted of 51,137 tons of Portland cement (ASTM-TYPE I) and 60,983 tons of Moderated heat cement (ASTM-TYPE II), 75% in the form of bulk cement and the remainder of bag cement. Cement was supplied solely by Semen Padang Corp. in Padang about 250 km away from the dam site and the maximum quantity of deliveries exceeded 6,900 tons a month.

To dispose of deliveries in such quantity, one 320-ton silo for general concrete batching plant and one 630-ton silo for dam concrete batching plant were provided at the project site.

(b) Steel Bars

Reinforcement steel bars used for the works were all of the deformed type and supplied mainly by Krakatau Steel Corp., Cilegon in the suburbs of Jakarta. There were no difficulties in local procurement of any size of bars. The total quantity of reinforcement bars was 5,066 tons.

(c) Explosive

The total quantity of explosives including dynamites and ANFOs was 124 tons. Dynamites and detonators used for the works were made in Indonesia, and ANFO was imported from Singapore.

(d) Fuel, Oil and Grease

The fuel used for construction equipment was all diesel oil. Gasoline was used only for passenger cars. The total consumption of these fuels was 9,835kl. These were all local supplies. The total consumption of oil and grease used for construction machine was 227kl and 14 tons, respectively. These were all local supplies.

(e) Transportation of Construction Equipment

Transportation of construction equipment from abroad was made by sea, except for urgent deliveries made by air. Freighters used for transportation totaled 10,498 m^3 with the total tonnage of 2,158 ton.

Table 2.1.2 Principal Materials Used for Civil Works

Item		Unit	Quantity
Diesel oil		kl	9,432
Gasoline		kl	403
Engine oil		kl	141
Hydraulic oil		kl	56
Gear (Transmission) oil		kl	24
Compressor oil		kl	6
Grease		kg	14,428
Dynamite		kg	116,000
ANFO		kg	8,200
Electric detonator		pcs	85,480
BAG cement	Small bag type I (40 kg/bag)	bag	59,596
	Small bag type I (50 kg/bag)	bag	11,935
	Small bag type II (40 kg/bag)	bag	20
	Big bag type I (1 ton/bag)	bag	15,042
	Big bag type II (1 ton/bag)	bag	10,662
BULK cement	Bulk type I	ton	33,114
	Bulk type II	ton	50,320
Admixture		ton	339
Reinforcing bars		ton	5,066
Rock bolt		ton	153
Steel materials		ton	1,25
Welded wire fabric		kg	14,967
Wire mesh		kg	28,716
PVC water stop	W = 200mm	m	6,152
	W = 400mm	m	1,770
Plywood		pcs	21,922
Timber		m3	1,915

1.3 Main Construction Equipment

The results of mobilization of main equipment used for the civil works are as tabulated in Table 2.1.3 and illustrated in Fig. 2.1.5.

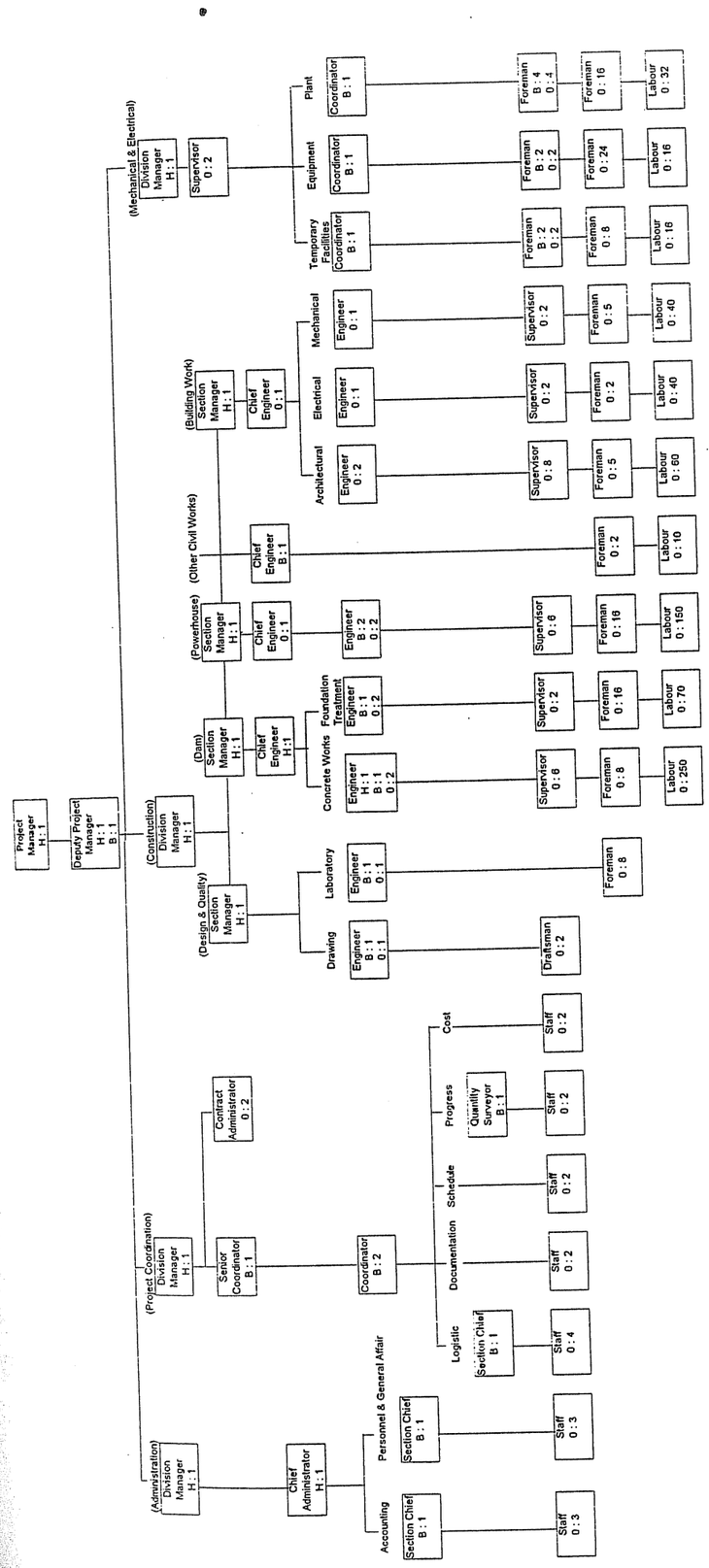


Fig. 2.1.3 Organization of Lot I Civil Works Contractor

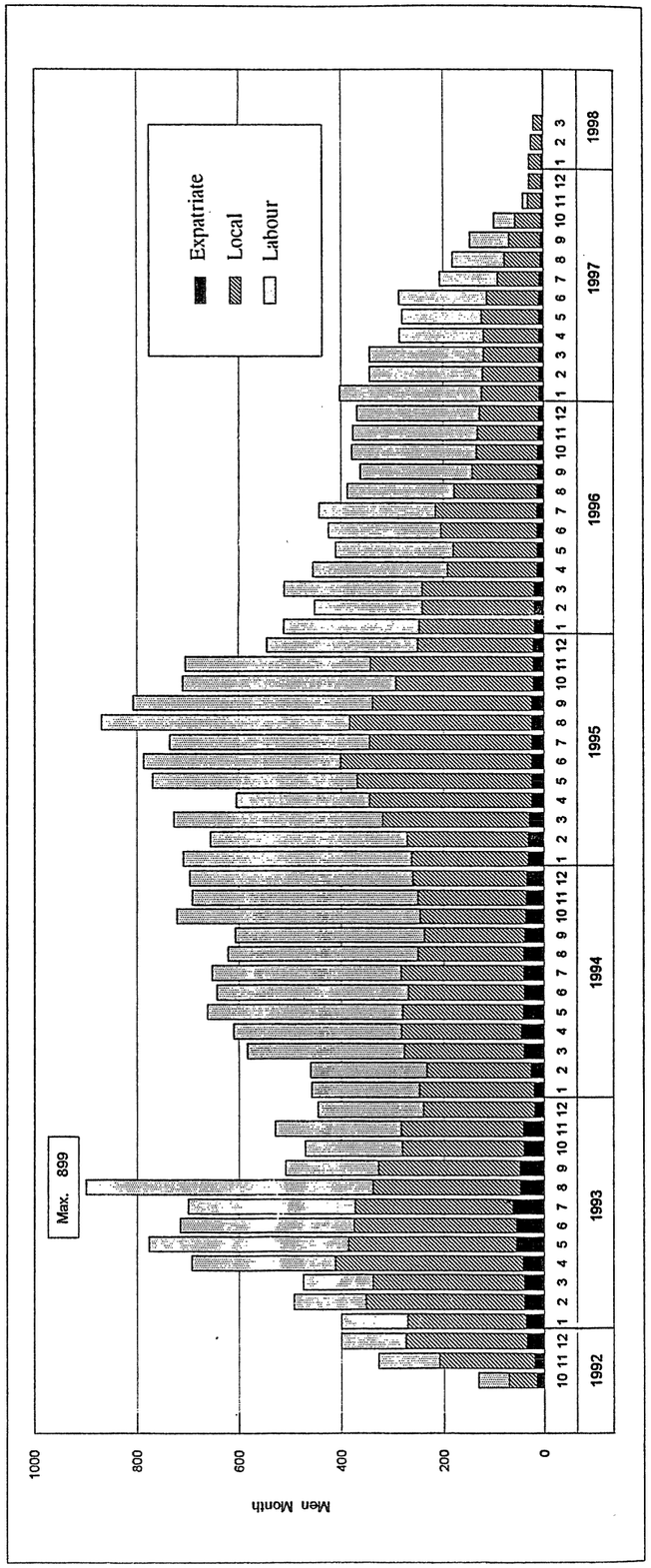


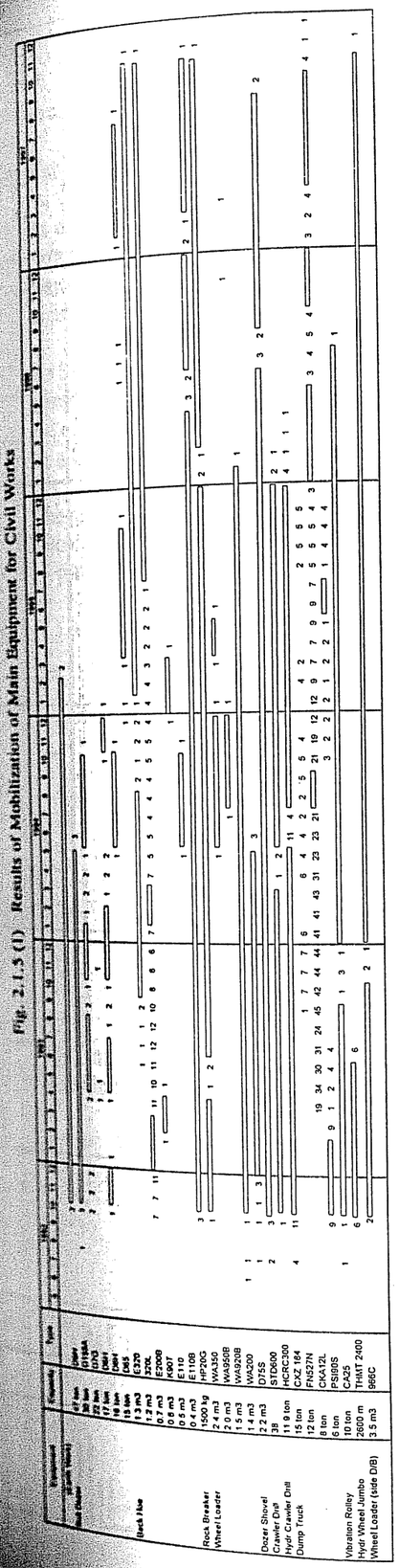
Fig. 2.1.4 Man - Day for Civil Works by Month

Table 2.1.3 Main Equipment Used for Civil Works

Description	Capacity	Unit-Months	Max.No. of Units per a Month	Accumulated Capacity
(Earth Works) Bulldozer	47T	60	2	6044 T.U.M
	30T	64	3	
	22T	34	2	
	15-17T	35	3	
Backhoe	1.3-1.2 m^3	91	3	297 m^3 U.M
	0.7 m^3	254	12	
	0.6-0.4 m^3	16	1	
Rock breaker	150 kg	158	3	
Wheel Loader	2.4 m^3	94	2	330 m^3 U.M
	2.0 m^3	13	1	
	1.4-1.5 m^3	55	2	
Dozer Shovel	2.2 m^3	60	3	
Crawler Drill	5T	118	3	1328 T.U.M
Hydraulic Crawler Drill	11.9T	62	2	
Dump Truck	12-15T	453	18	13360T.U.M
	6-8T	886	44	
Vibration roller	10T	49	3	
Hydraulic Wheel Jumbo	2 boom	60	6	
Wheel Loader (side D/B)	3.5m^3	76	2	
(Concrete Works) Batching Plant	34 m^3/h	21	1	90054 m^3/hU.M
	60 m^3/h	58	1	
	90 m^3/h	54	1	
Crushing Plant	150 t/h	63	1	
Washing Plant	80 t/h	63	1	
Aggregate Plant	100 t/h	53	1	
Water Cooling Unit	8 m^3	47	1	
Cable Way	13.5T	33	1	
Tower Crane	150Tm	17	1	
Agitator Truck	3-4.5 m^3	363	7	1123 m^3 U.M
Concrete Pump truck	100 m^3/h	126	3	
Viback	Ø130x4	23	1	
Vibrator	Ø52-130	2144	38	
Shotcrete machine	10-24 m^3/h	175	5	2288 m^3/h U.M
Shotcrete robot	12-24 m^3/h	72	2	1416 m^3/h U.M
Mortal Mixing Pump	2 m^3/h	89	2	

Description	Capacity	Unit-Months	Max.No. of Units per a Month	Accumulated Capacity
(Drilling & Grouting)				
Drilling Machine	125-150m	404	11	54125m U.M
Drilling Pump	40-62 l/min	145	5	8352 l/min U.M
Grouting Pump	105-120 l/min	306	8	32565 l/min U.M
High Speed Mixer	400-600 l/min	58	2	17800 l/min U.M
Grout Mixer	400 l/min	123	3	
Agitator Mixer	400 l/min	84	2	
Colloidal Mixer	400 l/min	44	2	
Mortal Grout Pump	105 l/min	10	2	
(Service Equipment)				
Crawler Crane	4.8-80T	98	2	7731 T.U.M
Truck Crane	3-20T	178	4	
Rough Terrain Crane	25T	50	1	
Mobile Crane	5T	44	1	
Truck Trailer	30T	43	1	
Water Truck	5 kl	42	1	
Fuel Truck	8 kl	41	1	
Ordinary Truck	4-8T	66	2	
Pick Up	0.7T	443	9	
Station Wagon	9 men	393	7	
Jeep Hardtop	6 men	46	1	
Micro Bus	22 men	55	1	
Motor Cycle	115 cc	647	15	
Motor Grader	13.5T	13	1	
High Lift Truck	5T	66	2	
(Other Equipment)				
Generator	590 kVA	179	4	227225 kVA U.M
	455 kVA	94	2	
	250 kVA	144	3	
	195 kVA	124	2	
	100-150 kVA	140	3	
	15 kVA	81	3	
Air Compressor	17-18.5 m^3	327	8	6206 m^3 U.M
	7 m^3	27	1	
Submerged Pump	8B	435	8	
	6B	352	6	
	4B	500	10	
	3B	202	4	
	2B	643	14	
Drain Pump	12 m^3/min	42	1	
Arc Welder	250 A	60	2	
Radio Communication		177	3	
Telecommunication		54	1	

Note : T.U.M : Ton-Unit - Months



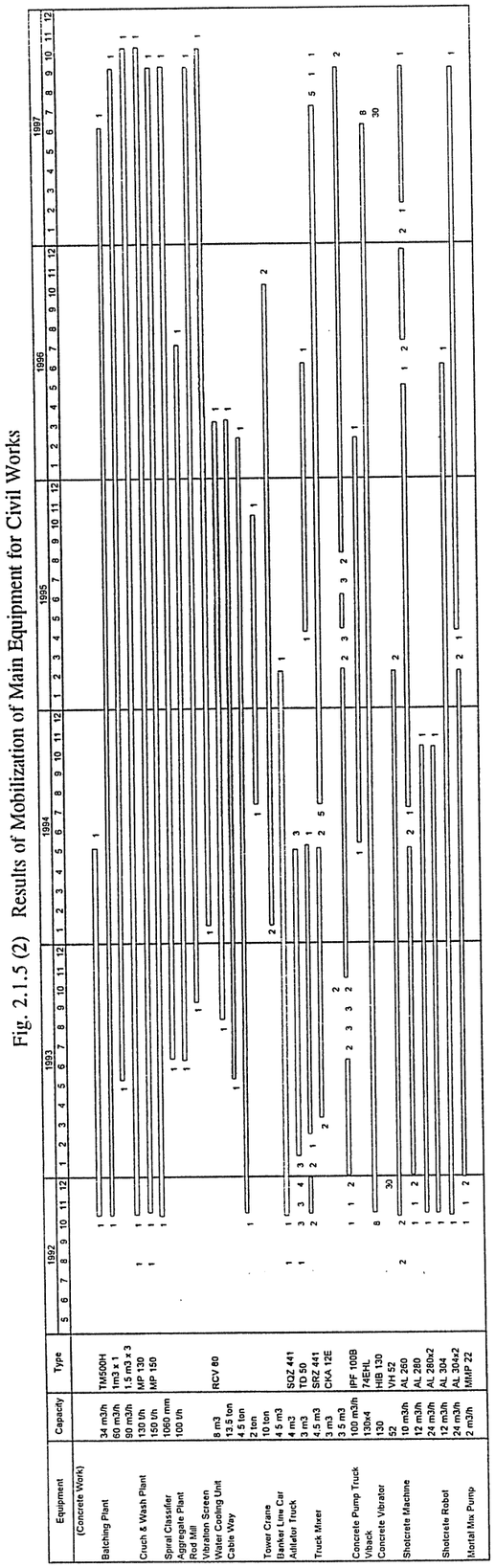


Fig. 2.1.5 (3) Results of Mobilization of Main Equipment for Civil Works

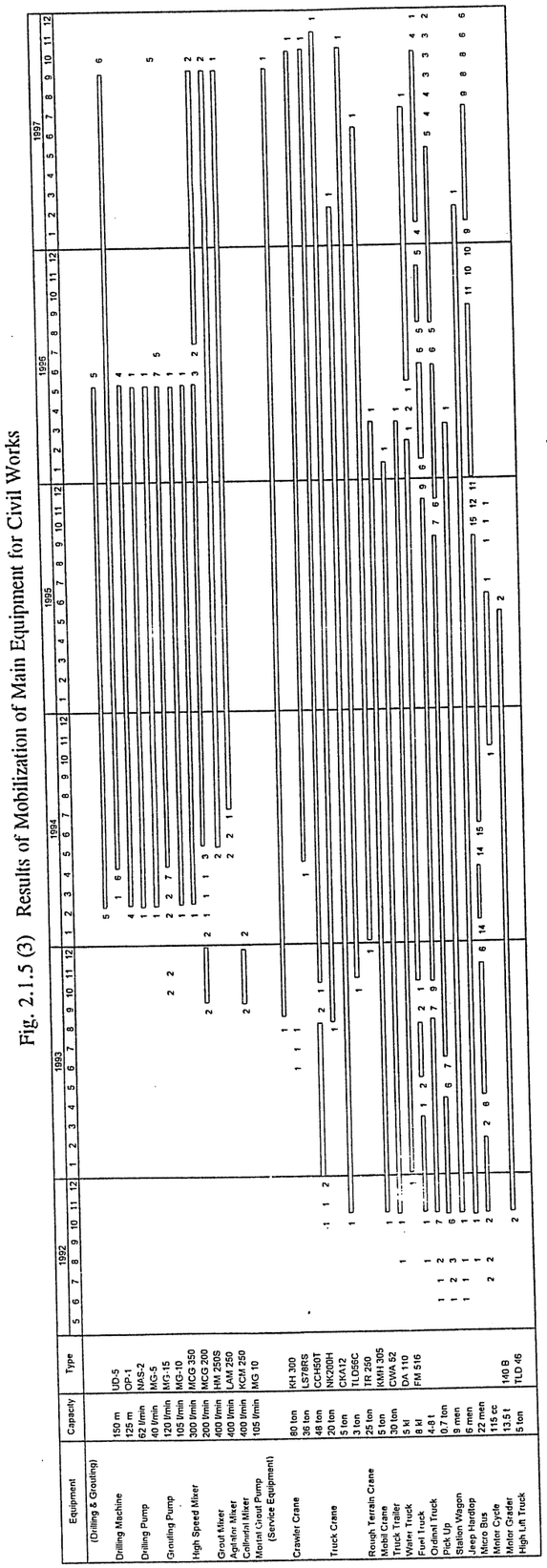
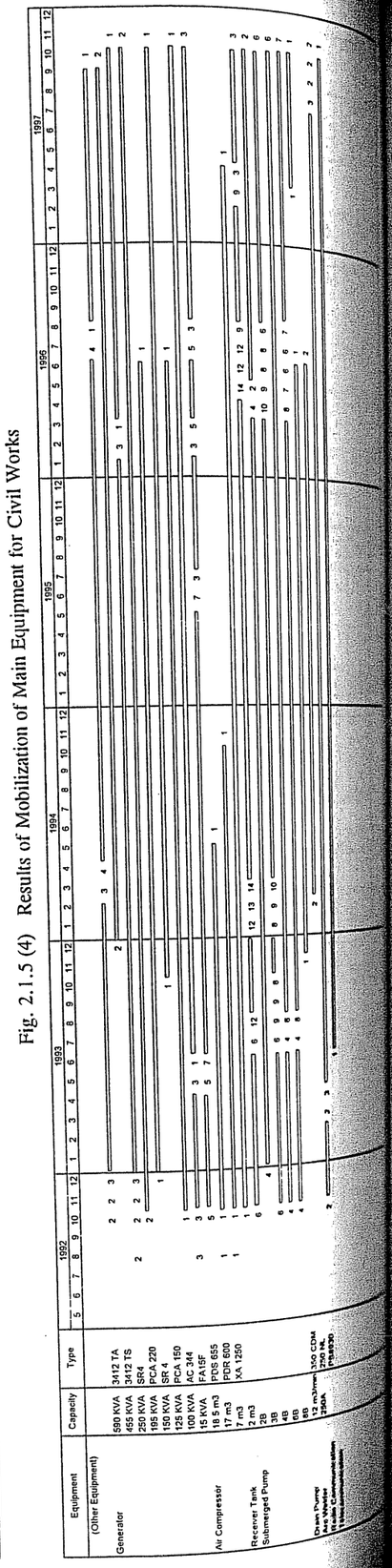


Fig. 2.1.5 (4) Results of Mobilization of Main Equipment for Civil Works



1.4 Project Safety Control and Accident Prevention

1.4.1 General

In order to complete such as a large-scale project like Kotapanjang Hydroelectric Power Project in a relative short period and safely in a rocky cliff of the Kampar Kanan river, the first priority was given toward the safety-precautions during construction.

As for construction methods, in order to obtain maximize efficiencies, mechanization (utmost use of equipment and machineries) had to be applied and as its consequence, the construction techniques which minimize construction accidents and hazards had to be adopted accordingly.

The project safety control and accident prevention program had to be initiated by taking into consideration the safety problems peculiar to this project which are summarized as follows:

i) Communication gap between Foreign Experts or Engineers of the Contractors and Indonesian workers. Relatively low safety-mind of local workers.

ii) Scarcity of skilled workers especially who have experience in underground construction works and in handling of explosives.

iii) Lack of familiarity and policing of regulations on safety and sanitation and of an enforceable license-system of operators. Difficulty to obtain safety devices and tools (as local workers are unaccustomed to use such tools).

iv) Health control of workers (especially against hot and humid working site conditions).

v) Traffic accident prevention.

The fundamental principles of the project Safety-Committee were established by considering the above factors further as follows :

a) To facilitate smooth progress of construction works by the Contractors, making proper decision of safety without delay.

b) To develop a safety - consciousness that is "Safety is Everybody's Business" among all the workers through good labor management and their skillful work.

c) To promote an "essential" safety control by providing good care of construction machinery and equipment, and by completely providing the job with all the necessary safety devices and tools.

d) To attain successful safety work and progress of technical skills by promotion of standard working methods and frequent holding of safety meetings as well as education of the workers in safe habits and practices, and

e) To encourage labor's self-education and to enhance their moral by maintaining good labor relations and communications under clean job sites and desirable working conditions.

1.4.2 Safety Committee Organization of Kotapanjang Hydro Electric Power Project

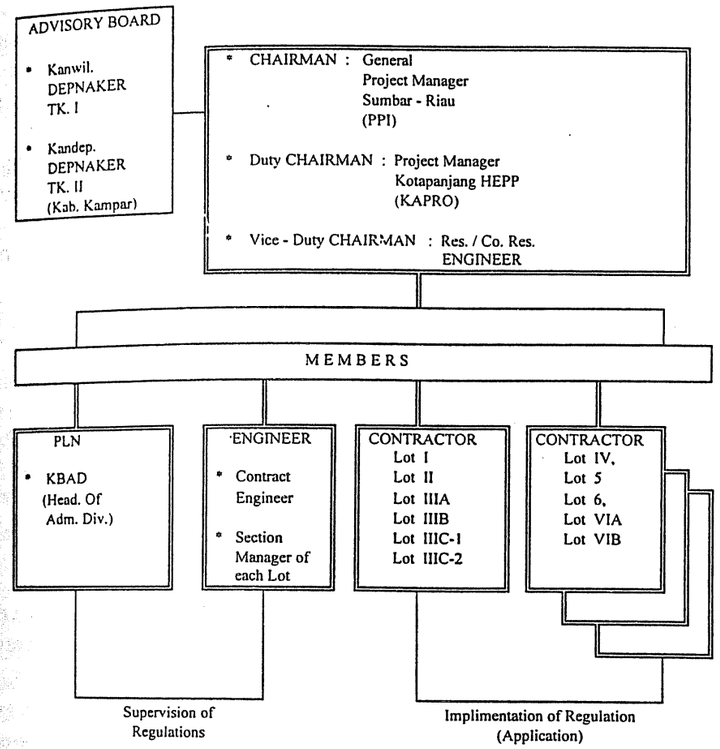
(1) Committee for Safety and Sanitation :

To make necessary plans and execute measures for the safety and sanitation for the project, a "Committee for Safety and Sanitation" (hereafter called "the Committee") was established in 16 October 1992, when the work started. The committee arranged a monthly meeting composed of key supervisory personnel from the Employer (PLN), the Engineer and all the Contractors of each Lot with the following membership :

Chairman	:	General Project Manager (PPI Pikitring Sumbar-Riau)
Duty Chairman	:	PLN Site Manager (KAPRO)
Vice Duty Chairman	:	Resident / Co-Res. Manager of the Engineer (TEPSCO)
Advisory Board	:	Kandep. DEPNAKER (Provincial) and Kandep. DEPNAKER (Kampar District)
Executive Members	:	PLN (KBAD), ENGINEER (Contract Engineer + Chief Engineer of each Lot) and Project Manager / Deputy Manager of each Contractor.

The organization of the Safety committee is shown in Fig. 2.1.6. The committee has five subordinate subcommittees for (i)Traffic Control, (ii) Explosive Control, (iii) Sanitary Control, (iv) Facilities Control and (v) Award for Safety Control of Field Work.

Fig. 2.1.6 Safety Committee Organization of Kotapanjang Hydroelectric Power Project



(2) Subcommittee for Traffic Control

The members of this subcommittee consisted of some members of the Committee and those engaged in the Construction works (personnel of PLN, TEPSCO and Contractors) appointed by the Chairman of the Committee.

Regular meetings were held at least once a month. The subcommittee performed planning and enforcement of traffic control and measures for traffic accidents in relation to vehicles and construction equipment such as :

a) Preparation of regulations for traffic control in the project area.
b) Posting of traffic sign boards in the project area.
c) Decision of speed limit.
d) Check method of periodical service of vehicles and equipment.
e) Education of drivers and operators.
f) Transmittal method of traffic control to other vehicles than those working for the Project.
g) Control by PLN guardsman (SATPAM).
h) Measures in case of traffic accidents, etc.

(b) Subcommittee for Explosive Control

The membership and frequency of regular meetings were similar to the subcommittee for traffic control. This committee engages in the following activities to prevent accident caused by transportation, handling, storing and using explosives :

a) Study and review of GOI rules and regulations on explosives handling and usage.
b) Establishment of the Project autonomous criteria on Transportation, storing, handling and use of explosives according to GOI rules regulations.
c) Preparation of "Manuals for Explosives Control" for local workers concerned and letting them have full knowledge there of.
d) Other necessary matters related to explosive control.

(c) Subcommittee for Sanitation Control

The membership and frequency of regular meetings were similar to the above committees. The purpose of this subcommittee was to provide and keep all workers engaged in the construction work with clean and Sanitary working environments. Major duties of the subcommittee are itemized as follows :

a) To exercise health care and check-up for workmen.
b) To inspect, check and arrange for Sanitary Conditions in the labor camp site.
c) To provide for corrective and remedial measures against adverse working environment caused by dust, noise, vibration, poisonous gas, etc.
d) To prevent epidemics and other diseases.
e) To provide for other necessary measures as occasions demand.

(d) Subcommittee for Facilities Control and Award of Safety Control

These two subcommittees were set up in the later stage of construction works. The purpose the of subcommittee for facilities control was to regularly check the completed building facilities for their proper functions and to draw up appropriate remedial measures and actions in case of trouble and failure to the parties concerned.

The subcommittee for an award of safety control was established to bestow a special awarded (merit of certificate and plaque) on the members who gave outstanding performance and results in fulfillment of requirements of the committee membership. Eventually all the members of the committee were presented with this special award.

The safety committee meeting was held in principle on the last Tuesday of every calendar month. Safety patrol was usually carried out in the morning and followed if necessary by a Safety Meeting after the patrol. An unannounced patrol might be carried out if necessary.

In the meeting the result of the working safety patrol was reviewed and discussed, and each subcommittee reported the respective activities in the month. Especially major accidental occurrences were investigated by all the members of the committee and appropriate instructions were issued to reduce re-occurrences of the accidents or loss in future.

The first safety patrol was conducted in 6th October, 1992 and the last one carried out in 30th December, 1997. Covering 63 months, this system having brought about outstanding performance for safety control.

(2) Safety Personnel of Contractor

The Contractor in each Lot provided a full time safety Engineer throughout the entire construction stages of the Project who generally belonged to personnel section in Administrative Divisions. This Safety Engineer and his subordinates were in charge of industrial Safety and Sanitation to handle the following matters :

a) Planning and administration of industrial Safety.
b) Accident compensation.
c) Planning and administration of hygiene, sanitation and community health.
d) Planning and administration of tropical communicable disease control and anti-epidemic measures.
e) Studies and review of health laws and regulations concerning hygiene, sanitation and community health.
f) Matters concerning procedures stipulated in the same GOI health laws and regulations.
g) Government relations concerning industrial safety and Sanitation (DEPNAKER).

(3) Rantau Berangin Clinic

The Contractor under Lot I of Civil works established a medical treatment clinic in Rantau Berangin permanent camp site by recruiting medical personnel including a doctor, nurses and technicians. The function of Rantau Berangin clinic was :

a) To plan and administer medical treatment and health check.
b) To exercise medical examination, treatment and hospitalization, and
c) To provide injured persons with emergency first aid treatment.

The clinic was also open to the other Contractors staff who paid for the services they received. The major function of the clinic was to give out-patient rather than in-patient treatment.

1.4.3 Contractor Safety Organization

Besides establishment of the powerful Committee for safety and sanitation as discussed above, each Contractor for each Lot established for his own well organized safety control organization at the beginning of construction work. One example of this (HAZAMA BRANTAS J.O) is shown in Fig.2.1.7.

The Project Manager of each Contractor had the supreme responsibility for safety control as the "Executive Safety Manager". Each section chief of the Contractor's construction office also held responsibilities for safety within each work division. A full-time foreman in charge of the safety was also assigned in each section.

The Contractor's Safety Committee Organization also had a safety patrol and meeting system as presented in Table 2.1.4. Regular (monthly) safety patrol was held to check all the work conditions in view of safety and to detect potential hazards, dangerous conditions and unsafe construction work.

The result of the safety patrol was discussed at the safety-meetings and any safety hazards pointed out were made good immediately. The safety meetings were held daily or monthly depending on the situation. Safety measures decided at the meetings were executed immediately without fail.

In general most injuries and accidents occur to workers who have had less than 3 (three) months of experience on the job. Therefore, the soundest safety program begins with workers education and job training.

Educational work was carried out continuously to make all workers safety-minded. Safety education required for the work were safety training courses such as training of special machine operators, acquiring proper licenses, etc.

These safety education classes were reinforced with safety meetings of tool-box type, which is a short safety session at the beginning of work each day. In addition to the daily meetings, it was very effective to hold a general safety meeting once a month attended by all the workers to elevate proper safety consciousness.

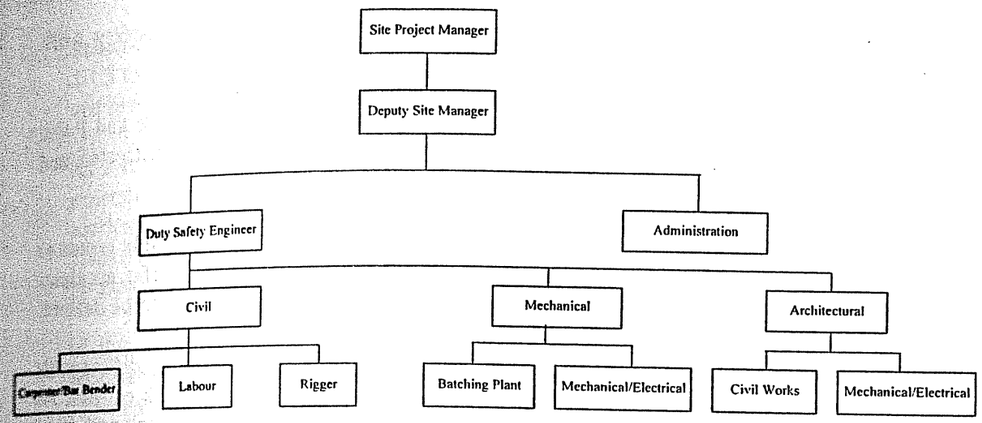


Fig. 2.1.7 The Safety Organization Chart of Lot I Civil Works Contractor

Table 2.1.4 Contractor Safety Meetings and Trainings

Class	Frequency	Location	Participants	Remarks
Safety Conference Project	Once a month	Job Site	Member of Safety	To let all Contractors fully know key points of Safety Control.
Safety Commitee Patrol	Each last Tuesday of the month	Dam Site & Vicinity	All member of Safety Committee	Inspection, correction and guidance of safety facilities and working methods.
Contractor's Safety commitee discussion	Occasional	Contractor's office or Job Site office	Member of Contractor's Safety Committee	Discussion liaison and coordination of Committee suggestions, safety control, etc.
General Safety Meeting	First day of each month	Job Site	All Supervisory staff and workers	To let all employees fully know key points of Safety Control.
Safety Patrol	Occasional	Job Site & Camps	All Safety Engineers	Inspection, correction, and guidance of safety facilities and working methods.
Daily Safety Meeting (Tool-box Type)	Daily (at the beginning & ending of works)	Job Site & Tool Box.	Respective supervisor and Foreman	Safety direction, checking, reviewing and coordination of daily works.
Foreman education & training	At the beginning of the works	Office or Job Site	Foreman	Instruction of foreman's duties, etc.
Drivers & Operators	Occasional	Office or Job Site	Drivers and Operators	Instruction of Safe driving, and safe operating of heavy equipments.
Field Training & License Test	Occasional	Office or Job Site	Corresponding workers	Blasting works, erection of forms and scaffold, crane lifting operation, grinding & cutting, welding, low-voltage circuit, dewatering , etc.

1.4.4 Accomplishment of Safety Control

The result of the Construction Safety Control and performance of safety work operations for the Project are briefly reviewed below.

a) The access roads mainly used for aggregate haul road from Kuok and Pulau Gadang Sites to respective job sites were in general well constructed and maintained.

b) Sign of notice, warning, speed limit, direction, etc. were appropriately posted to control traffic and warn of overhead lines, etc.. However traffic accidents occurred often and the accident prevention performance left a little to be desired.

c) The greatest hazards in the work were related to the height differentials experienced in the project site which featured steep slopes of the rock canyon. As a result, great emphasis was placed on protection from falling rock and other objects, and on precautions to prevent falls by the works themselves. Benches, barricades, rock bolts and anchors, wire netting, fence etc. were employed for protection of workers at lower elevations.

d) Adequate facilities for storage and handling of explosives were provided and maintained properly. The transportation and use of explosives were well controlled with no resulting accidents. All major blasting in the diversion tunnels were performed by safe blasting methods. Adequate ventilation was provided during tunnel excavation and tests for toxic gases were conducted at frequent intervals.

e) Placing concrete using cable ways for the dam and a tower crane for the powerhouse were controlled by remote control with appropriate communications system. This method of placement was both efficient and controlled in a safe manner.

f) Overhead lighting on the job for dam construction was good, well maintained and contributed materially to safe and efficient operations during hours of darkness.

g) Safety precautions for electrical power lines and equipment were vigorous, and injuries and accidents from this source were relatively small in number and minor in severity.

h) Extensive on site equipment and machinery repair facilities, coupled with the regular program of inspection and maintenance. considerably reduced accidents and injuries due to equipment failure.

1.4.5 Accident Statistics

Statistical data of the Contractor's lost time due to accidents are presented in Table 2.1.5 to 2.1.8, to show a comparison of two important safety indices (frequency rate and severity rate) of the Project civil works and similar works in Japan.

As a result of the extensive accident prevention program, civil contractors workers experienced an overall average accident frequency rate of only 0.706 (lost time accidents per-million man-hours of exposure) during the Project Construction Period. This rate was only 1/8 th of the average one of 5.55 experienced in similar Civil Works in Japan.

In this regard the Project Safety Control was highly successful. Of the total 39 accidents reported, injuries from strike by or cut by hand tools had the largest share of 33% and injuries from the traffic accidents had the next largest share of 21%. Injuries from these two causes accounted for about 54% of the lost time accidents.

A total of 4 fatalities occurred on the civil works and mechanical & electrical works during approximately of 6 years tenure, almost all of which resulted from errors in human judgement or freak accidents. Of these total fatalities, 1 case was caused by falling object, 1 case was caused by traffic accident in the diversion tunnel, 1 case was caused by electrical hazard and 1 case was caused by falling and drowning at tailrace. Thus the greatest hazards in the works were related to the height differentials involved in construction beside struck by or cut by falling metal object or hand tools..

The resulting overall average accident severity rate (lost time days per thousand man -hours) amounted to 0.551 which was about 0.27 only of the average one of 2.06 experienced in similar works in Japan as shown in Table 2.1.8. In other words the Project had relatively less injuries and less frequency as well.

There were no fatalities and only one accident reported during construction of the transmission line even though the work involved long extended high level works such as tower erection uphill and downhill, stringing works, etc from Payakumbuh in West Sumatera Province until Pekanbaru in Riau Province.

In summary, there were 39 job injuries causing lost time (including 4 fatalities) to all Contractor's workers while performing 55,237,421 man-hours of hazardous construction. The Contractor's had a rather good accident record throughout the job taking into account the type of construction involved.

Table 2.1.5 Lost Time Accident by Job Site
Contractor : HAZAMA - BRANTAS JO

	1992			1993			1994			1995			1996			1997			Total		
	F	S	L	F	S	L	F	S	L	F	S	L	F	S	L	F	S	L	F	S	L
Main Dam	0	3	0	0	0	1	0	1	3	1	1	8	1	1	3	0	0	0	2	6	15
Powerhouse	0	0	0	0	0	1	0	0	3	0	0	5	0	0	3	1	0	0	1	0	12
Diversion tunnels	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
Notional road vicinity dam	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	0
Total	0	5	0	1	0	2	0	1	6	1	1	13	1	1	6	1	0	0	4	8	27

Note : F : Fatalities
S : Seriously injured (16 days or longer to heal completely)
L : Lightly injured (1 to 15 days to heal completely)

Table 2.1.6 Number of Injured/Fatalities by Type of Injury
Contractor : HAZAMA BRANTAS JO

Type of Injury	1992			1993			1994			1995			1996			1997			Total		
	F	S	L	F	S	L	F	S	L	F	S	L	F	S	L	F	S	L	F	S	L
Falls of persons - Same level										0	0	2							0	0	2
Falls of persons - One level to another										0	1	2	0	1	1	1	0	0	1	2	3
Strike by falling objects													1	0	0				1	0	0
Strike by flying, rolling, sliding objects	0	1	0							0	0	3	0	0	2				0	1	5
Stepping in or on objects																			0	0	0
Caught in or between																			0	0	0
Strains or sprains-lifting																			0	0	0
Strike by or cut by hand tools							0	1	5	0	0	4	0	0	3				0	1	12
Poisoning, infection or burned	0	2	0																0	2	0
Explosions																			0	0	0
Traffic accidents	0	2	0	1	0	2	0	0	1	0	0	2							1	2	5
Others										1	0	0							1	0	0
Total	0	5	0	1	0	2	0	1	6	1	1	13	1	1	6	1	0	0	4	8	27

Table 2.1.7 Lost - Time Accidents Summary by Lot-I Civil Works
Contractor : HAZAMA-BRANTAS JO

YEAR		Man days	Man-hours	Lost Time		Frequency rate	Severity rate
				accidents	days		
1992	2nd half	60,975	600,603.00	5	90	8.325	0.150
1993	1st half	582,600	5,808,522.00	1	7,500	0.172	1.291
	2nd half	678,750	6,950,400.00	2	20	0.288	0.003
1994	1st half	642,150	6,691,203.00	2	20	0.299	0.003
	2nd half	754,050	8,045,713.00	5	60	0.621	0.007
1995	1st half	859,500	8,955,990.00	2	40	0.223	0.004
	2nd half	783,000	8,002,260.00	13	7,620	1.625	0.952
1996	1st half	522,750	5,211,818.00	2	20	0.384	0.004
	2nd half	426,150	4,210,362.00	6	7,560	1.425	1.796
1997	1st half	49,500	477,680.00	0	0	0.000	0.000
	2nd half	31,500	282,870.00	1	7,500	3.535	26.514
	Total	5,390,925	55,237,421.00	39	30,430	0.706	0.551

Note : 1. Frequency rate = Lost time accidents per million man-hours
2. Seventy rate = Lost time days per thousand man-hours
3. One death is counted as corresponding to a lost time of 7,500 days

Table 2.1.8 Comparison of Frequency and Severity Rate of KOTAPANJANG PROJECT and Similar Project in Japan

YEAR of Construction		Frequency Rate		Severity Rate	
		KTPJ Project	Similar Works in Japan	KTPJ Project	Similar Works in Japan
1992	2nd half	8.325	6.460	0.150	3.530
1993	1st half	0.172		1.291	
	2nd half	0.288	6.580	0.003	2.300
1994	1st half	0.299		0.003	
	2nd half	0.621	7.060	0.007	3.060
1995	1st half	0.223		0.004	
	2nd half	1.625	3.990	0.952	0.960
1996	1st half	0.384		0.004	
	2nd half	1.425	3.650	1.796	0.470
1997	1st half	0.000	Not yet	0.000	Not yet
	2nd half	3.535	published	26.514	published
Total of whole period average		0.706	5.550	0.551	2.060

Notes : Comparison is made of main Civil Works Lot I HAZAMA BRANTAS JO.

Chapter 2 TEMPORARY FACILITIES

Temporary facilities for the civil works were provided for the purpose of effective and economic execution of the construction. The principal temporary facilities provided involve the following.

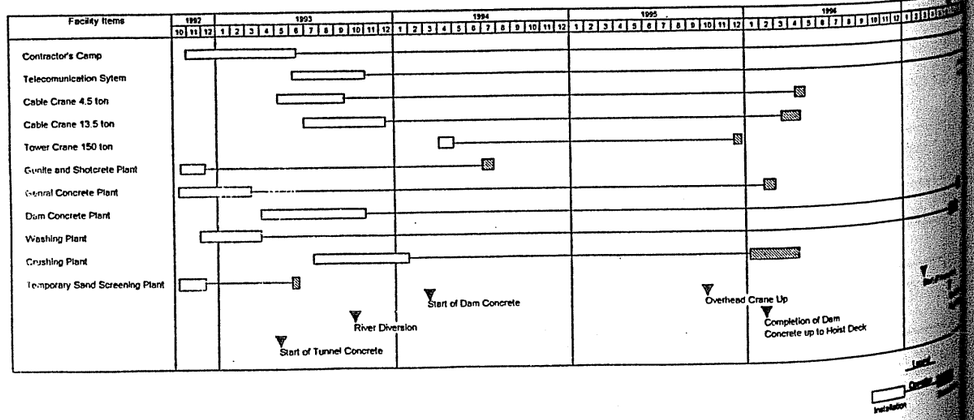
- Production of concrete
- Temporary building (Base camp, workshop, powder magazine)
- Cable crane
- Tower crane
- Telecommunication facilities
- Power supply facilities
- Water supply facilities
- Oil supply facilities
- Air supply system
- Dewatering system

The temporary facilities were planned on the basis of following basic considerations.

- Locations of the facilities were determined considering accessibility, economical installation, good convenience for the works.
- Capacity of the facilities were enough for the execution of each work item including loss and allowance for the work efficiency.
- Operation mechanism of the facilities was simplified, safe and made easy to handle by the less trained local workman.

Their location and the progress record of installation and dismantlement are as shown in Fig. 2.2.1 and Fig. 2.2.2 respectively.

Fig. 2.2.2 Progress of Principal Temporary Facilities



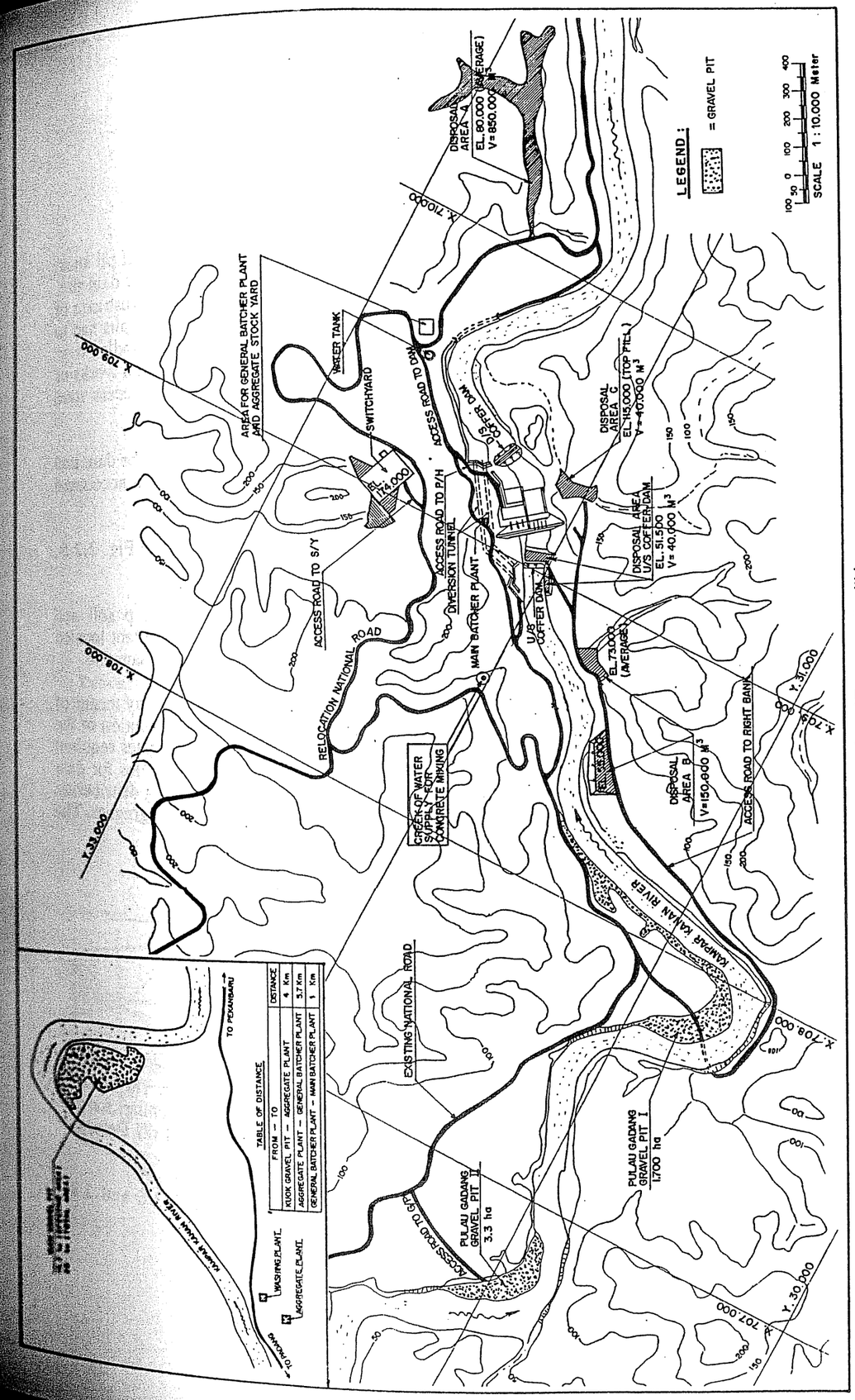


Fig. 2.2.1 Location of Temporary Facilities

2.1 Production of Concrete

2.1.1 Method of Production

(1) Production of Concrete Aggregates

Sand and gravel for concrete aggregates were extracted from three gravel pit sites, namely, Pulau Gadang Gravel Pit I & II located about 4 km upstream of the dam site, and Kuok Gravel Pit located at the Kampar Kanan River about 10 km downstream of the dam site. These locations are shown in Fig. 2.2.1. Both Pulau Gadang pits had to be operated in spite of a long way from the stock yard due to a delay in handing over of the Kuok Pit. Sand and gravel from them were mainly used for diversion works of tunnel and cofferdam concrete and base coarse of existing and new access road construction from March, 1993 till March 1994.

The raw material of aggregates from Kuok Pit were mostly extracted for the dam and the powerhouse concrete from March, 1994 till December, 1995, which accounted for 78% of the whole extraction.

The available extraction area of gravel pits are as shown in Fig. 2.2.3 to Fig. 2.2.5, and the monthly extraction of raw materials is tabulated in Table 2.2.1.

Original gravels were gathered by excavator after removal of the top soil and transported to the temporary stockyard near the washing and crushing plant located along the existing national road approximately 6km downstream of the dam site.

The production of aggregates for general concrete was carried out only by means of washing and screening without crushing because the actual grading distribution of the raw material extracted from Kuok Pit was almost similar to that of aggregates required for the general concrete. On the other hand, the production of aggregates for dam concrete was carried out by means of crushing in consideration of grading distribution of the raw material and production balance of each grading of coarse aggregate. The capacity of each plant was determined as follows.

Item	Washing Plant	Crushing Plant
Max. Q'ty of required aggregate per month	20600 t	46200 t
Max. Q'ty of production of aggregate per month	21200 t	47630 t
Max. Q'ty of extraction of raw material per month	34000 t	51780 t
Working days per month	26 days	26 days
Working hours per day	16 hours	16 hours
Required Capacity	80 t/hr	125 t/hr

The stocked and transported materials were washed and screened twice at the washing plant with a capacity of 80 ton/hr, for the general concrete aggregates. The crushing plant had a capacity of 150 ton/hr with triple crushing and twice screening, for dam and powerhouse structure concrete aggregate, which were produced till December, 1995.

Detail of the washing and crushing plant are as shown in Fig. 2.2.6 and 2.2.7 respectively.

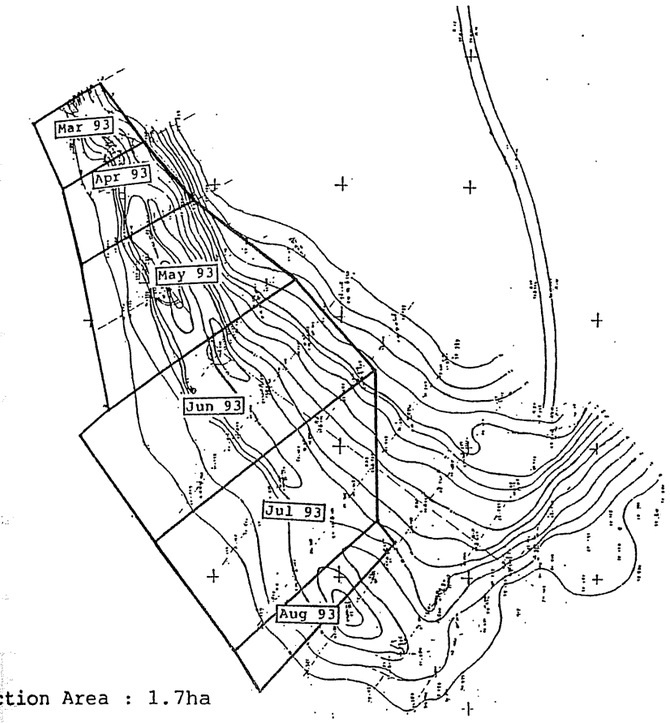


Fig. 2.2.3 Pulau Gadang Gravel Pit I Extraction Plan

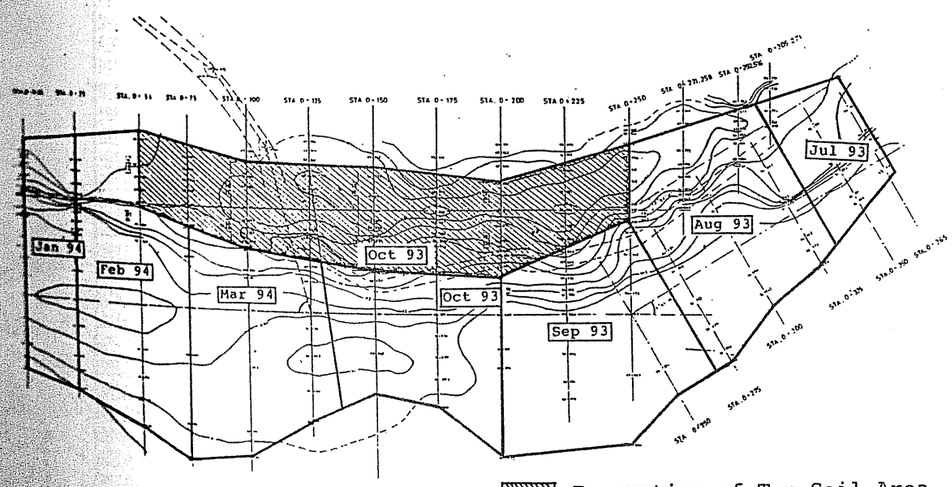


Fig. 2.2.4 Pulau Gadang Gravel Pit II Extraction Plan

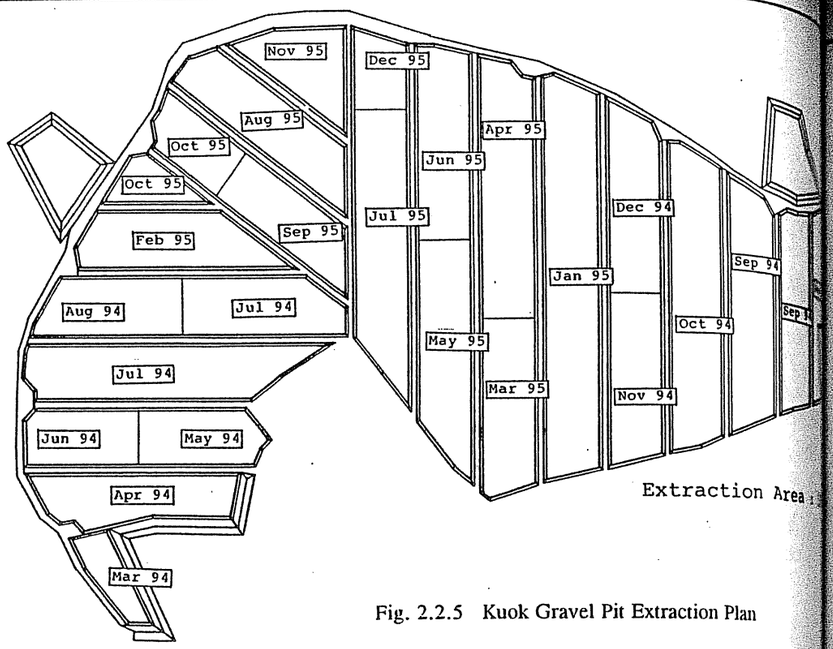
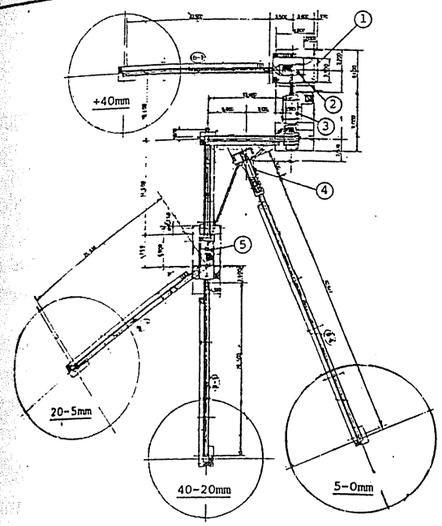


Fig. 2.2.5 Kuok Gravel Pit Extraction Plan

Table 2.2.1 Records of Raw Material Extraction

Period		Raw Material Extraction (M3)			Cumulative (M3)		
		P. Gadang	Kuok	Total	P. Gadang	Kuok	Total
Up to							
1993	3	614	-	614	614	-	614
	4	14,632	-	14,632	15,246	-	15,246
	5	18,527	-	18,527	33,773	-	33,773
	6	21,616	-	21,616	55,339	-	55,339
	7	29,886	-	29,886	85,275	-	85,275
	8	21,591	-	21,591	106,366	-	106,366
	9	15,138	-	15,138	122,004	-	122,004
	10	20,505	-	20,505	142,509	-	142,509
	11	-	-	0	142,509	-	142,509
	12	-	-	0	142,509	-	142,509
1994	1	-	-	0	142,509	-	142,509
	2	7,422	-	7,422	149,931	-	149,931
	3	11,023	99	11,122	160,954	99	161,053
	4	-	43,651	43,651	160,954	43,750	204,704
	5	-	26,083	26,083	160,954	69,833	230,787
	6	-	29,522	29,522	160,954	99,355	260,309
	7	-	45,639	45,639	160,954	144,994	305,948
	8	-	32,111	32,111	160,954	177,105	338,059
	9	-	37,972	37,972	160,954	215,077	376,031
	10	-	40,441	40,441	160,954	255,518	416,472
	11	-	34,289	34,289	160,954	289,807	450,761
	12	-	31,012	31,012	160,954	320,819	481,773
1995	1	-	32,331	32,331	160,954	353,150	514,104
	2	-	21,558	21,558	160,954	374,708	535,662
	3	-	20,031	20,031	160,954	394,739	555,693
	4	-	30,933	30,933	160,954	425,672	586,626
	5	-	28,132	28,132	160,954	453,804	614,758
	6	-	25,780	25,780	160,954	479,584	640,538
	7	-	27,129	27,129	160,954	506,713	667,667
	8	-	12,554	12,554	160,954	519,267	680,221
	9	-	14,451	14,451	160,954	533,718	694,672
	10	-	14,227	14,227	160,954	547,945	708,899
	11	-	12,543	12,543	160,954	560,488	721,442
	12	-	6,925	6,925	160,954	567,413	728,367



No.	Parts Name	Capacity	Q'ty	Remarks
1	Raw Hopper	21 m³	1	3500 (W) x 2000 (L) x 3000 (H)
2	1st Screen	120 t/h	1	1500 x 3600 x 1D 40mm Mesh
3	Drum Washer	100 t/h	1	ϕ 1800 x 4500 5mm Mesh
4	Classifier	80 t/h	1	YS - 48S ϕ1200 x 7100
5	2nd Screen	100 t/h	1	1800 (W) x 4800 (L) x 2D 20mm, 5mm Mesh

Fig. 2.2.6 Washing Plant Layout

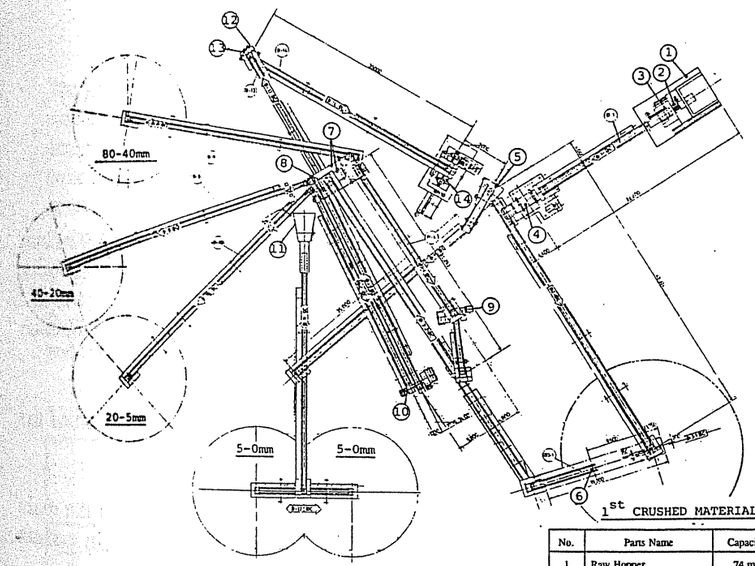
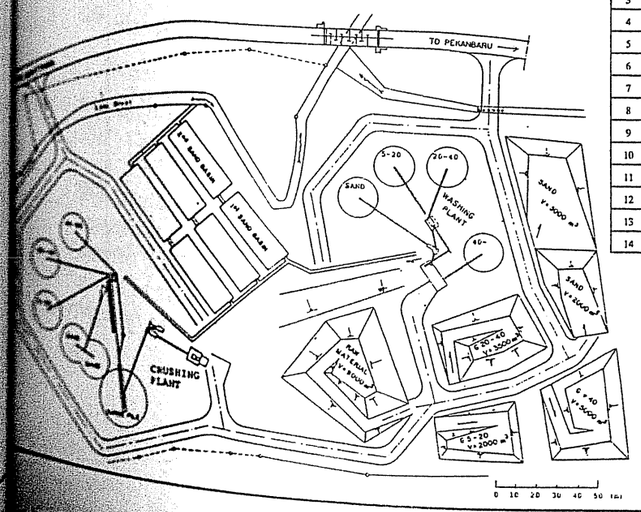


Fig. 2.2.7 Crushing Plant Layout

No.	Parts Name	Capacity	Q'ty	Remarks
1	Raw Hopper	74 m³	1	4500 (W) x 4000 (L) x 4150 (H)
2	Grizzly Feeder	160 t/h	1	EGF 1200 x 3000
3	1st Crusher	70 t/h	1	ST 2536 OSS : 50 mm
4	Scrubber	150 t/h	1	ϕ1800 x 3600 (L)
5	Classifier	40 t/h	1	KA 42 ϕ1060
6	Vibrating Feeder	100 t/h	2	FD - 50 - 90L
7	1st Screen	200 t/h	1	1500 x 3000 x 2D
8	2nd Screen	150 t/h	1	2100 x 4800 x 2D
9	2nd Crusher	130 t/h	1	KM 1512 L CSS : 11 mm
10	3rd Crusher	95 t/h	1	KFS 1350 L CSS : 6 mm
11	Classifier	80 t/h	1	KA 48 (ϕ1220)
12	Raw Sand Hopper	4 m³	1	2200 (W) x 2200 (H) x 2000 (H)
13	Vibrating Feeder	50 t/h	1	FD - 40 - 90 L
14	Rod Mill	36 t/h	1	612



Aggregate Plant Layout

(2) Production of Concrete

The concrete batching plant consisted of three concrete production units, one main batching plant with a capacity of 90 m^3/hr for dam concrete mixed with Moderated heat cement, Type II, one general batching plant with a capacity of 30 m^3/hr for general concrete, mixed with Portland cement, Type I and one shotcrete production unit with a capacity of 34 m^3/hr for gunite.

(a) Main Batching Plant

The main batching plant produced the concrete for the dam and the diversion plug.

Assuming that a maximum monthly concrete volume was 29,400 m^3 to be placed for 22 days per month, the required capacity of the batching plant was determined as follows :

Max. Monthly placing quantity	: 29400 m^3
Number of placing days per month	: 22 days
Average quantity placed per day	: 1337 m^3
Max. Quantity placed per day	: 1738 m^3
Operating hours per day	: 20 hours
Max. Quantity placed per hour	: 87 m^3/hr

According to the above study, 1.5 m^3 x 3 units of front charge mixer of pneumatic tilting type (capacity 90 m^3/hr) was selected. Compared with this plan, actual peak monthly rate of placing was about 28,700 m^3 against the planned rate of 29,400 m^3. This peak production occurred in August 1994 and during this period, full production capacity of the batching plant was utilised.

The main batching plant was installed on the left bank about 50 m downstream of the dam. This location was selected for the following reasons.

- The shuttle train line was available to transport the concrete bucket from the main batching plant and the loading and unloading area of the cable crane in a short distance.
- It was easy to transport concrete material (cement and aggregate etc.) to the batching plant.
- The area for the batching plant could be used until completion of the diversion plugging works at the final phase of the project.

A cement silo with a capacity of 630 ton and a cement warehouse with capacity of 170 ton were constructed by the side of the main batching plant and alongside the access road to the dam about 100 m downstream of the plant respectively. Their cement storage capacity corresponded to a concrete placing volume for about 3 days to lessen effect of any delay disruption to cement supply to the site. The cement was brought to the site in bags and bulk and then put into the cement silo.

Aggregate bins were installed on the hill side of the batching plant for the storage of aggregates. A temporary road was constructed to the aggregate bins so that the aggregate could be directly thrown into bins from dump trucks. The capacity of bins was about 1,000 m^3 which corresponded to concrete placing for 1 day. The aggregates stocked in bins were transported to the batching plant by belt conveyers.

(b) General Batching Plant

The general batching plant produced the general concrete for other structures except the dam and diversion plug. According to the concrete placing schedule, maximum placing volume per day was estimated at 470 m^3/day, so that 1.0 m^3 x 1 unit of turbine with twin shaft pugmil type batching plant (capacity 30 m^3/hr) was selected based on the following study assumptions.

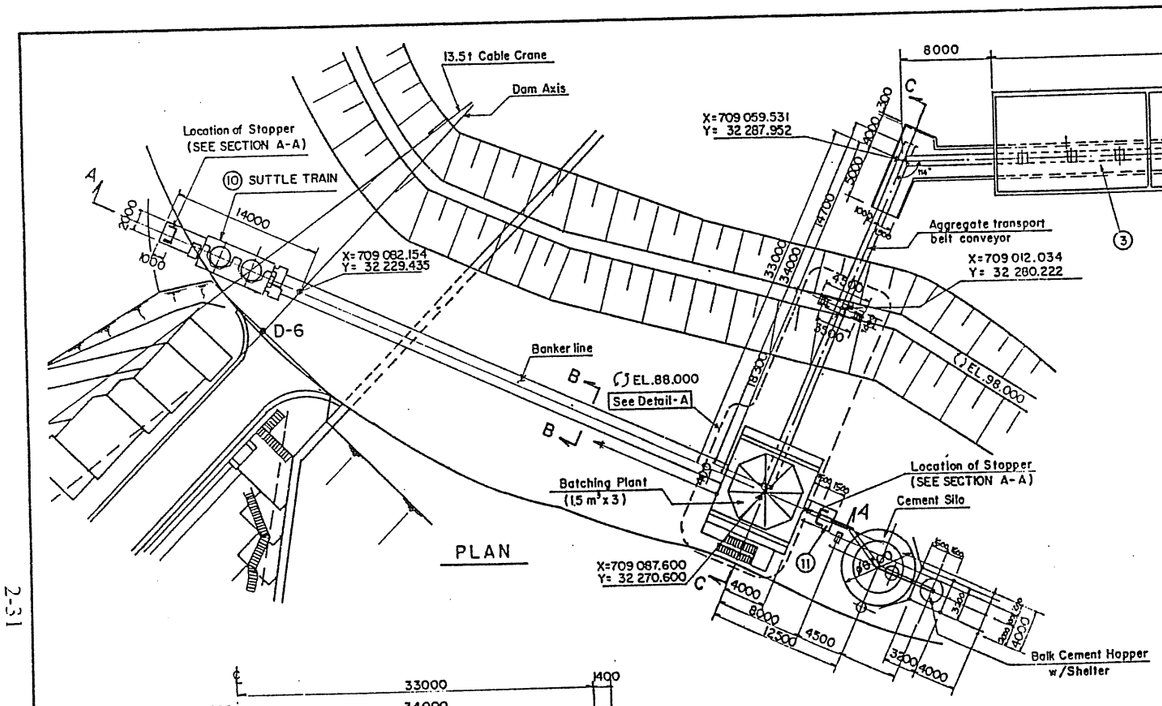
Max. Daily placing volume	: 470 m^3
Operating hours per day	: 20 hours
Max. Quantity placed per hour	: 24 m^3/hr

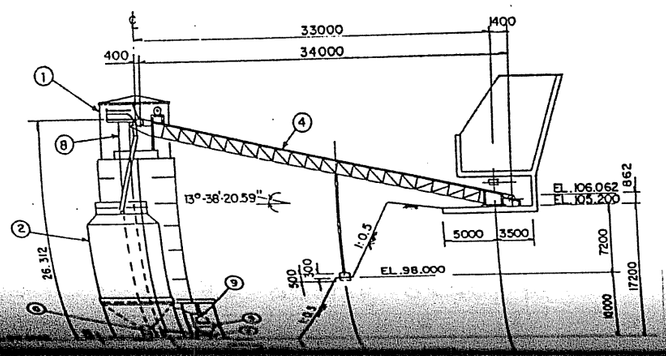
The general batching plant was installed at the entrance area of the site by the national road because there was not enough space available to install it in the main Site area. A cement silo with a capacity of 320 ton and a cement warehouse with capacity of 75 ton were constructed by the side of the general batching plant and alongside the national road respectively. Aggregate bins were installated alongside the national road and with the storage capacity of 150 m^3. The aggregate from the bins was transported to the batching plant by belt conveyers over the national road.

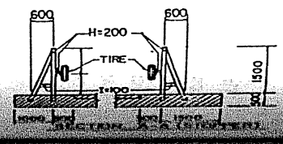
Both concrete batching plants and shotcrete plant are shown in Fig. 2.2.8 to Fig. 2.2.10 and the following tables.

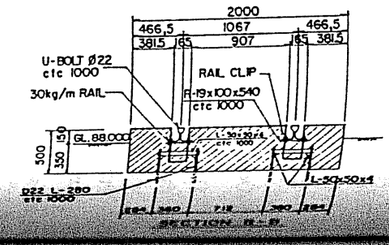
Specification of Concrete Plant

Description	Specification		
	Main	General	Shotcrete
Batching Plant			
- Capacity of Mixer	3 x 1.5 m^3	1 x 1.0 m^3	1 x 0.5 m^3
- Type of Mixer	Tiltling	Turbine	Turbine
- Mixing Capacity	90 m^3/hr	30 m^3/hr	17 m^3/hr
- Cement Silo	630 ton	320 ton	10 ton
Cement Storage			
- Type of structure	Wooden Type	Wooden Type	Use the cement storage for the general in common
- Floor area	440 m^2	193 m^3	
- Capacity	170 ton	75 ton	
- Floor finishing	Concrete	Concrete	
Aggregate Bin			
- Sand	588 m^3	50 m^3	80 m^3
- Coarse aggregate			
20- 5 mm	420 m^3	50 m^3	80 m^3
40-20 mm	420 m^3	50 m^3	-
80-40 mm	420 m^3	-	-

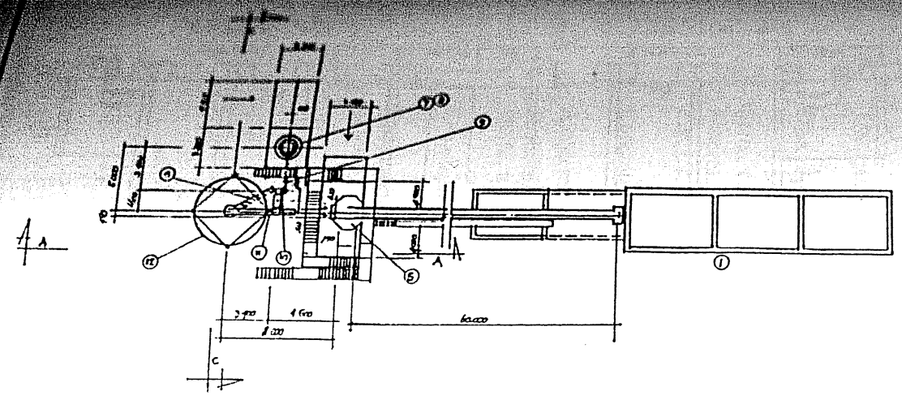




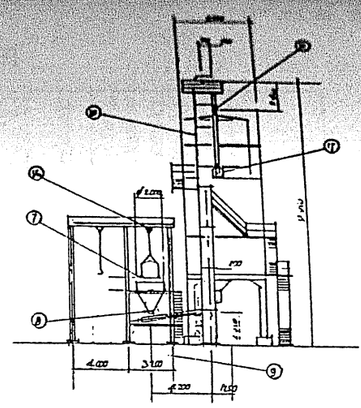




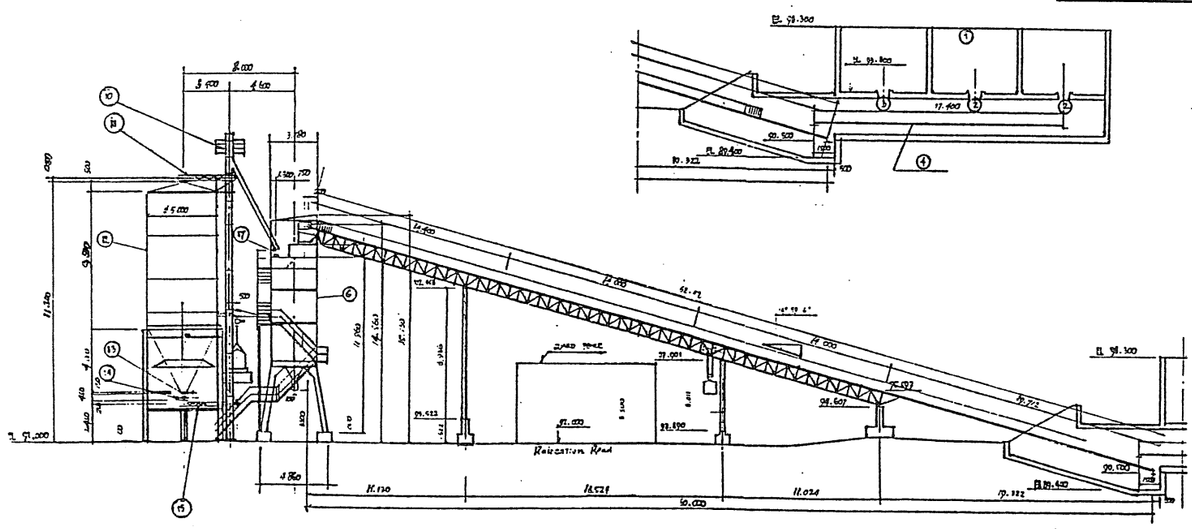
No.	Parts Name	Capacity	Q'ty	Remarks
1	Batching Plant	90 m³/h	1	
2	Cement Silo	630 t	1	
3	No. 1 Belt Conveyor	270 t/h	1	900 (W) x 55000 (L)
4	No. 2 Belt Conveyor	270 t/h	1	900 (W) x 35000 (L)
5	Cement Supply Hopper		1	
6	Rotary Feeder	40 t/h	1	
7	Screw conveyor	40 t/h	1	4600 (L)
8	Bucket Elevator	40 t/h	1	27500 (H)
9	Hoist	2 t	1	
10	Shuttle Train		1	Electric Motor
11	Screw Conveyor	40 t/h	1	5100 (L)



PLAN



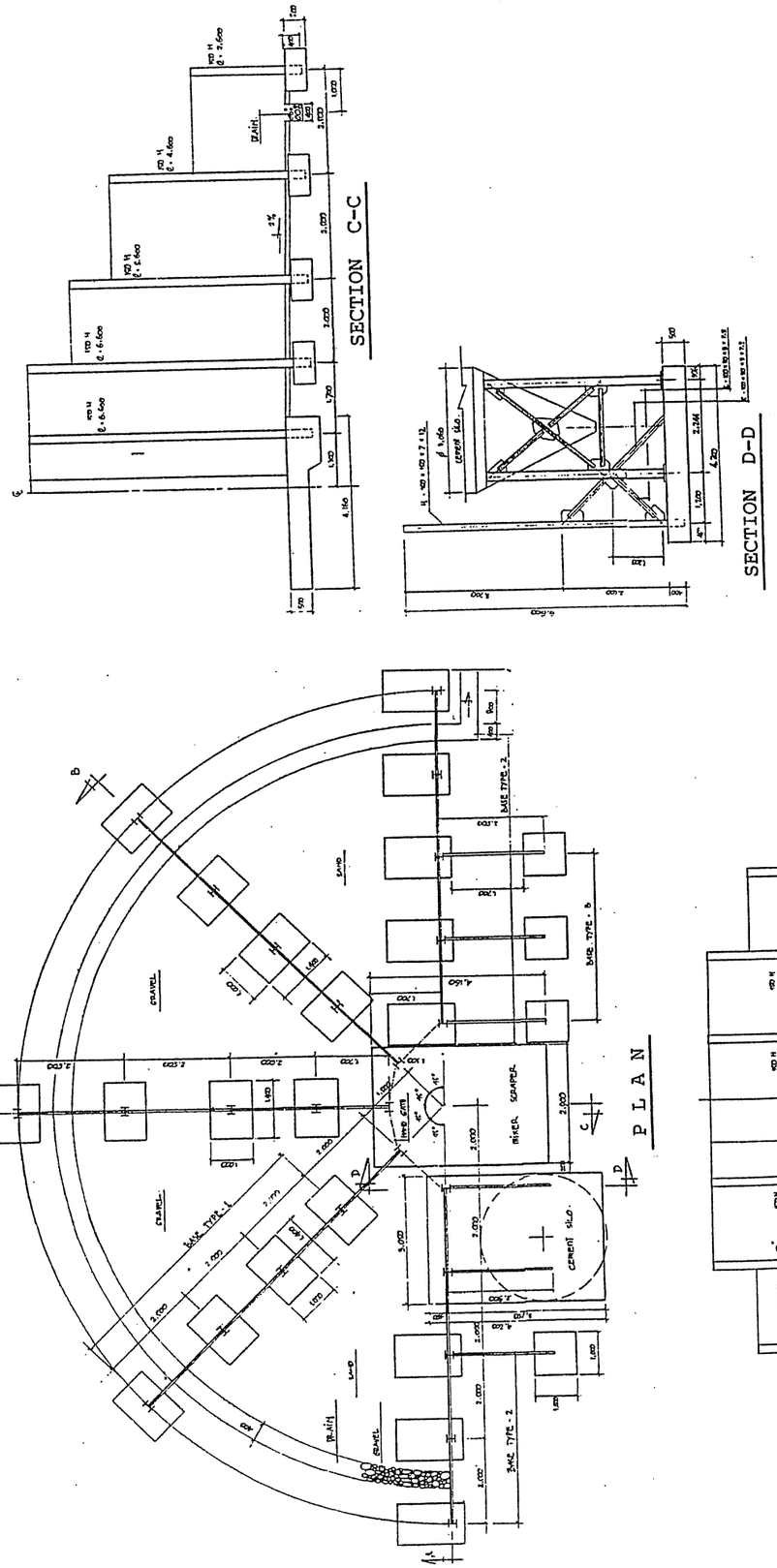
SECTION C-C



SECTION A-A

No.	Parts Name	Capacity	Q'ty	Remarks
1	Aggregate Stock Bin	60 m³	3	6000 (W) x 4200 (L) x 4300 (H)
2	Vibrating Feeder	200 t/h	2	F - 44 BD† 762 (W) x 1219
3	Tunnel Cut Gate		1	□ 600
4	No. 1 Belt Conveyor	200 t/h	1	600 x 17400 (L)
5	No. 2 Belt Conveyor	200 t/h	1	600 x 62112 (L)
6	Batching Plant	60 m³/h	1	
7	Cement Tank	4 m³	1	ϕ200 x 2400 (H)
8	Hand Slide Gate		1	□ 300
9	No. 1 Screw Conveyor	20 t/h	1	□ 300 x 4300 (L)
10	Bucket Elevator	20 t/h	1	500 x 1000 x 18900 Belt Type
11	No. 2 Screw Conveyor	20 t/h	1	ϕ300 x 3400 (L)
12	Cement Silo	320 ton	1	ϕ5000 x 14000 (H)
13	Hand Slide Gate		1	□ 300
14	Rotary Feeder		1	
15	No. 3 Screw Conveyor	20 t/h	1	ϕ300 x 4000 (L)
16	Electric Hoist	2 ton	1	
17	Electric Dumper		1	

Fig. 2.2.9 General Batching Plant Layout



(c) Cooling Plant

For the purposes of controlling the temperature of the fresh concrete at placing, the cooling plant was provided to cool down the river water from 25°C to 5 °C to produce concrete of 27 °C just after mixing based on the calculation result of heat quantity as shown below.

Heat Quantity Calculation
(case of internal concrete /m³)

Material	Weight (kg/m³)	Specific Heat (kcal/kg°c)	Unit heat Quantity (kcal/m³°C)	Material Temperature (°C)	Heat Quantity (kcal/cu.m)
Coarse aggregate	1506	0.18	271	31	8401
Fine aggregate	639	0.18	115	31	3565
Cement	157	0.23	36	50	1800
Mixing water	101	1	101	5	505
Total	2403		523		14271
Remarks				Temperature after mixing 14,271/523=27.3°C	

(d) Transportation of Concrete

Dam concrete was transported using 4.5m³ buckets by battery car and a 13.5 ton cable crane. Concrete was placed directly on the site using rubber chutes for the confirmed parts. General concrete was transported by aditator trucks and placed mostly by concrete pump trucks. The flow chart for dam concrete is detailed as shown in Fig.2.2.11. The mix proportion for concrete and shotcrete is shown in Table 2.2.2.

(e) Concrete Laboratory

A concrete laboratory was installed in the site near the general batching plant area. The laboratory was equipped with a compressive testing machine, a portable concrete mixer, a Los Angeles machine and other equipment as shown in Fig. 2.2.12.

(f) Equipment and Facilities

Equipment and facilities for the concrete works of dam and general structures wire as follows :

Washing plant	80 t/h	1 unit	
Crushing plant	150 t/h	1 unit	
Sand screening plant		1 unit	(before installation of washing plant)
Batching plant	90m³/h (Main batching)	1 unit	
	30m³/h (General batching)	1 unit	
	34m³/h (Shotcrete)	1 unit	
Shuttle train line	(Battery car)	1 unit	
Cable crane	13.5t	1 unit	(one side traveling type)
Concrete bucket	4.5m³	3 units	
Concrete pump truck	100m³/h	3 units	(with boom)
Aditator truck	4.5m³	7 units	
Cooling plant	212,000 kcal/h	1 unit	
Concrete laboratory		1 set	(concrete test)

2.1.2 Material Used

(1) Cement

Indonesia manufactured portland cement (ASTM-TYPE I) for general concrete and moderated heat cement (ASTM-TYPE II) for dam concrete were used. The products of the local manufacturer satisfied ASTM standards, and their manufacturing capacity was enough to meet the requirements for dam construction. Delivery was made in the bulk form and in the bag form by cement lorries and trucks directly from the factory in Padang.

(2) Admixture

In view of the necessity for securing a long, stable supply of admixture, it was decided to purchase it from the Indonesian manufacture. Two kinds of the water-reducing and set-retarding type admixtures, Pozzolith 300R and Sika VZ were selected and used in combination with the air entraining agent, Pozzolith 303A and Sika AER respectively to supplement entrained air supply to concrete.

(3) Water

River water purified passing through the cofferdam from the main river to conform to the requirements of the Specifications was used as the mixing water. Since the water temperature was as high as 27-30°C, it was cooled down to 5 °C by the cooling plant before mixing, except for river diversion concreting works.

(4) Aggregates

Concrete aggregates, both coarse and fine were made of crushed and screened materials. No particular problem was encountered in achieving the specific gravity, grade-distribution and other qualities according to the requirements of the Specifications.

(5) Records on Concrete Production

Records on concrete and shotcrete/gunite production are tabulated as shown in Table 2.2.3 and Table 2.2.4.

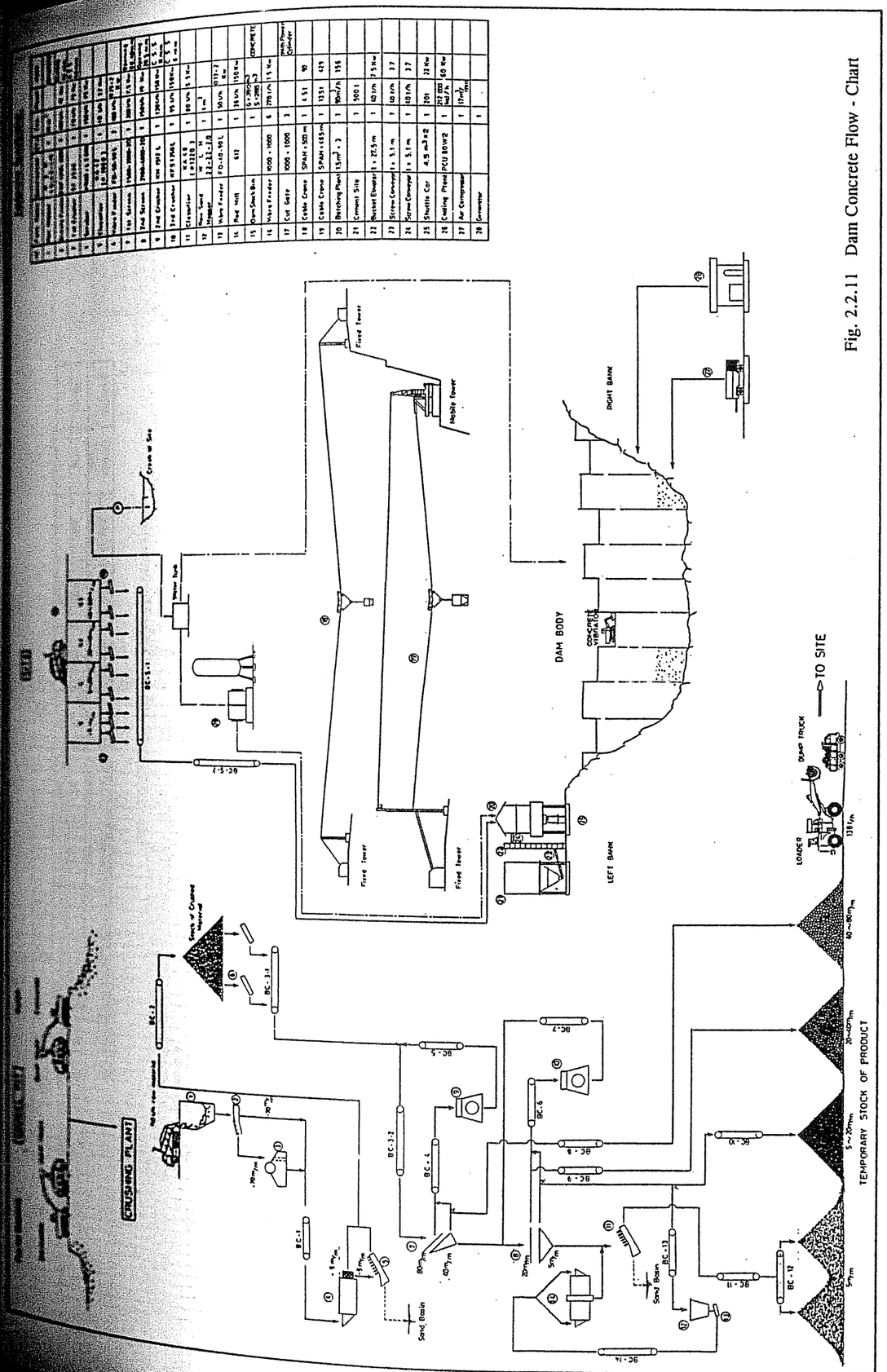


Fig. 2.2.11 Dam Concrete Flow - Chart

Table 2.2.2 (1) Concrete Mix Proportion

TYPE I CEMENT

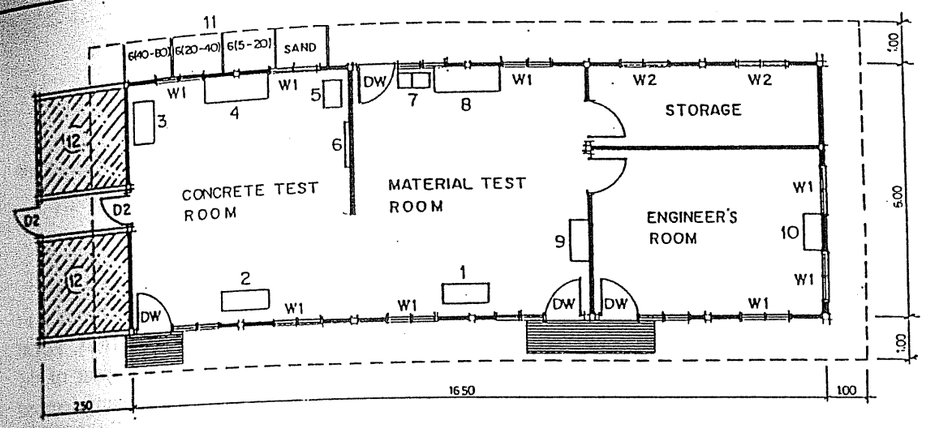
Concrete Type	σ ck (kg/cm2)	G MAX (mm)	SLUMP (Cm)	AIR (%)	W/C (%)	s/a (%)	Water (Kg)	Cement (Kg)	AGGREGATE (Kg) Sand	G 20-5	G40 -20	G80 -40	ADMIXTURE (cc) WAR	AE * 10	REMARK
B 1	230	40	3 ± 2	4.5 ± 1	45	35	135	298	668	552	677	-	894	536	
		40	3 ± 2	4.5 ± 1	47	35	135	290	671	554	630	-	870	609	
B 1 - R 1		40	3 ± 2	4.5 ± 1	49	37	137	280	710	538	660	-	840	590	
B 1 - R 4		40	3 ± 2	4.5 ± 1	47	37.5	153	325	709	526	643	-	1275	-	
B 2	260	40	12 ± 2	4.5 ± 1	51	37.5	153	300	636	517	631	-	1176	-	
B 2 - R 1	260	40	12 ± 2	4.5 ± 1	49	36	141	290	684	541	664	-	870	609	
B 2 - R 2	260	40	12 ± 2	4.5 ± 1	51	37	143	280	699	533	654	-	840	504	
B 2 - R 3	260	40	12 ± 2	4.5 ± 1	48	36	136	286	690	546	670	-	858	601	
B 3	240	40	3 ± 2	4.5 ± 1	51	37.5	153	300	717	532	651	-	1177	-	
B 4	240	40	12 ± 2	4.5 ± 1	55	37.5	153	278	693	522	633	-	1090	-	
B 4 - R 1	240	40	12 ± 2	4.5 ± 1	53	37	142	268	709	537	659	-	804	482	
B 4 - R 2	240	40	12 ± 2	4.5 ± 1	53	37	137	258	717	543	666	-	774	464	
B 5	220	40	3 ± 2	4.5 ± 1	53	37	137	279	325	997	-	-	837	335	
B 6	210	20	16 ± 2	4.5 ± 1	61	45	170	287	317	1028	-	-	861	344	
B 7	230	20	10 ± 2	4.5 ± 1	55	44	158	308	739	1042	-	-	942	370	
B 7 - R 1	230	20	10 ± 2	4.5 ± 1	51	43	157	305	739	1042	-	-	942	370	
B 8	120	20	3 ± 2	4.5 ± 1	85	40	153	180	305	1199	-	-	540	216	
B 8 - R 1	120	20	3 ± 2	4.5 ± 1	60	45	155	260	349	1026	-	-	780	312	
B 9 - R 3	35	20	3 ± 2	-	102	43	170	166	964	1037	-	-	415	-	
B 11	500	20	10 ± 2	-	103	55	185	130	1025	336	-	-	540	450	
PC - R 1	260	5	17 ~ 20	-	55	100	272	490	741	979	-	-	5880	-	RH - 716
MB 2	260	5	17 ~ 20	-	55	100	272	495	1467	-	-	-	1485	44.6	
MB 4	240	5	17 ~ 20	-	58	100	272	470	1486	-	-	-	1410	42.3	

NOTE : WAR = SIKA VZ
AE = SIKA AER

TYPE II CEMENT

Concrete Type	σ ck (kg/cm2)	G MAX (mm)	SLUMP (Cm)	AIR (%)	W/C (%)	s/a (%)	Water (Kg)	Cement (Kg)	AGGREGATE (Kg) Sand	G 20-5	G40 -20	G80 -40	ADMIXTURE (cc) WAR	AE * 10	REMARK
A 1	330	80	3 ± 1	3 ± 1	47	26	101	215	539	466	539	544	789	405	
A 1 - R 1	330	80	3 ± 1	3 ± 1	47	26	101	215	547	465	545	545	648	390	
A 2	220	80	3 ± 1	3 ± 1	61	29	101	166	615	459	529	533	588	302	
A 2 - R 1	220	80	3 ± 1	3 ± 1	55	28	101	184	588	460	536	536	551	441	
A 3	280	40	8 ± 1	4.5 ± 1	49	36	137	280	694	549	673	-	1.008	424	
A 3 - R 1	280	40	3 ± 2	4.5 ± 1	50	36	135	270	694	553	678	-	956	486	
A 3 - R 2	280	40	4 ± 1	4.5 ± 1	50	36	125	250	714	565	693	-	900	450	
A 4	280	40	12 ± 2	4.5 ± 1	48	36	142	308	790	541	664	-	1.069	450	
A 7	330	20	10 ± 2	4.5 ± 1	51	43	157	308	790	1044	-	-	1.109	550	
MA		5	-	-	45	100	245	545	1440	-	-	-	1.962	1100	

NOTE : WAR = Pozzolith 300 R
AE = Pozzolith 303 A



1. Compressive Testing Machine.
   Product : Macross Corporation, Japan.
   Model : TC - 202/224.
   Cap : 100 t.
2. Portable Concrete Mixer.
   Merk : TRINDO.
   Cap : 0.12 m3
3. Los Angeles Machine.
   Product : Mekan Babakan Tujuh.
   Model : AG-700
4. Concrete Desk.
5. Sieving Machine.
   Product : MBT
6. White Board.
7. Electric Oven (2 nos).
   Merk : Memmert, Germany.
   Model : GE-171
8. Concrete Desk.
9. File Locker.
10. File Locker.
11. Aggregate Bin.
12. Sample Curing Basin.

Fig. 2.2.12 Concrete Laboratory Layout

Table 2.2.3 Records of Concrete Production

Month	Production (M3)			Cumulative (M3)		
	General BP	Main BP	Total	General BP	Main BP	Total
Jan 92	123	-	123	123	-	123
Feb	1,795	-	1,795	1,918	-	1,918
Mar	5,944	-	5,944	7,862	-	7,862
Apr	13,016	-	13,016	20,878	-	20,878
May	13,933	-	13,933	34,311	-	34,311
Jun	9,631	-	9,631	44,442	-	44,442
Jul	2,287	351	2,287	46,378	351	46,729
Aug	16,074	14,907	16,074	47,545	15,258	62,803
Sep	460	158	460	47,846	15,417	63,263
Oct	614	-	614	48,460	15,417	63,877
Nov	191	4	191	48,647	15,421	64,068
Dec	8,889	5,956	8,889	49,880	21,077	70,957
Jan 93	13,328	11,802	13,328	51,606	32,679	84,285
Feb	17,445	15,015	17,445	54,036	47,694	101,730
Mar	23,043	19,819	23,043	58,060	66,713	124,773
Apr	31,553	26,272	31,553	63,341	92,985	156,326
May	34,078	28,742	34,078	68,677	121,727	190,404
Jun	30,432	27,800	30,432	71,509	149,327	220,836
Jul	29,707	24,400	29,707	76,816	173,727	250,543
Aug	29,440	25,563	29,440	80,673	199,310	279,983
Sep	23,845	19,087	23,845	85,451	218,377	303,828
Oct	23,157	18,948	23,157	89,662	237,323	326,985
Nov	17,758	12,886	17,758	94,734	250,009	344,743
Dec	18,113	16,728	18,113	100,119	260,737	360,856
Jan 94	18,722	12,829	18,722	106,012	273,566	379,578
Feb	17,575	12,866	17,575	110,632	286,521	397,153
Mar	13,843	10,272	13,843	114,243	296,793	411,036
Apr	12,930	9,217	12,930	117,956	306,010	423,966
May	11,654	7,459	11,654	122,151	313,469	435,620
Jun	8,496	5,730	8,496	124,917	319,199	444,116
Jul	6,832	3,460	6,832	128,269	322,679	450,948
Aug	4,231	2,459	4,231	130,070	325,109	455,179
Sep	2,173	2,173	2,173	130,070	327,282	457,352
Oct	706	788	706	130,070	328,048	458,118
Nov	981	981	981	130,070	329,009	459,079
Dec	1,253	1,253	1,253	130,070	330,262	460,332
Jan 95	1,821	1,821	1,821	130,070	332,083	462,153
Feb	2,630	2,630	2,630	130,070	334,713	464,783
Mar	1,950	1,950	1,950	130,070	336,663	466,733
Apr	2,376	2,376	2,376	130,070	339,039	469,109
May	1,427	1,427	1,427	130,070	340,466	470,536
Jun	1,249	1,249	1,249	130,070	341,715	471,785
Jul	736	736	736	130,070	342,451	472,521
Aug	467	467	467	130,070	342,948	473,018
Sep	407	407	407	130,070	343,355	473,425
Oct	435	435	435	130,070	343,790	473,860
Nov	309	309	309	130,070	344,099	474,169
Dec	378	378	378	130,070	344,477	474,547
Jan 96	506	506	506	130,070	344,983	475,053
Feb	2,194	2,194	2,194	130,070	347,177	477,247
Mar	2,786	2,786	2,786	130,070	349,963	480,033
Apr	2,033	2,033	2,033	130,070	351,996	482,066
May	148	148	148	130,070	352,144	482,214

Table 2.2.4 Records of Shotcrete / Gunite Production

Period		Production (M3)			Cumulative (M3)		
		Shotcrete	Gunite	Total	Shotcrete	Gunite	Total
Up to							
1992	10	-	61	61	-	61	61
	11	24	45	69	24	106	130
	12	76	108	184	100	214	314
1993	1	630	95	725	730	309	1,039
	2	1,407	231	1,638	2,137	540	2,677
	3	1,140	150	1,290	3,277	690	3,967
	4	971	-	971	4,248	690	4,938
	5	505	-	505	4,753	690	5,443
	6	588	-	588	5,341	690	6,031
	7	56	-	56	5,397	690	6,087
	8	10	55	65	5,407	745	6,152
	9	53	61	114	5,460	806	6,266
	10	-	194	194	5,460	1,000	6,460
	11	-	237	237	5,460	1,237	6,697
	12	-	162	162	5,460	1,399	6,859
1994	1	-	161	161	5,460	1,560	7,020
	2	-	283	283	5,460	1,343	6,803
	3	-	119	119	5,460	1,962	7,422
	4	-	169	169	5,460	2,131	7,591
	5	-	91	91	5,460	2,222	7,682
	6	-	66	66	5,460	2,288	7,748
	7	-	-	-	5,460	2,288	7,748
	8	-	-	-	5,460	2,288	7,748
	9	-	-	-	5,460	2,288	7,748
	10	-	-	-	5,460	2,288	7,748
	11	-	-	-	5,460	2,288	7,748
	12	-	-	-	5,460	2,288	7,748
1995	1	-	-	-	5,460	2,288	7,748
	2	-	-	-	5,460	2,288	7,748
	3	-	-	-	5,460	2,288	7,748
	4	-	-	-	5,460	2,288	7,748
	5	-	60	60	5,460	2,348	7,808
	6	-	92	92	5,460	2,440	7,900
	7	-	67	67	5,460	2,507	7,967
	8	-	101	101	5,460	2,608	8,068
	9	-	109	109	5,460	2,717	8,177

2.2 Miscellaneous Facilities

Temporary facilities are as described below except for production of concrete and their location, layout and detail are shown in Fig. 2.2.13 to Fig. 2.2.24.

(1) Contractor's Base Camp

The Contractor's base camp facilities were located approximately 5km downstream of the dam site. The camp facilities comprised of Expatriate Personnel Quarters, Local Staff Quarters, Labour Quarters and Office Area. During the first half year of the works, the main office and quarters were temporarily set up at Bangkinang located approximately 15km downstream of the dam site. Field offices were additionally provided at several locations such as near the dam site, tailrace of riverbed and motor pool area of upstream area.

(2) Workshop

The principal workshops located at two places. The main workshop at the motor pool area was responsible for repair and store keeping of construction equipment and vehicles, assembling and disassembling of equipment from abroad, and fabrication of forms and hardwares, while other sub-workshop was located at the construction site for simple repair and adjustment of equipment.

(3) Powder Magazine

The powder magazine was provided in compliance with the requirements of the respective local regulations. The magazine was made up of three houses for store keeping of dynamite, ANFO and detonators used for tunnel and open-cut works. The maximum store keeping quantity of the magazine was set at 116 tons of dynamite and 8.2 tons of ANFO, the monthly magazine consumption is tabulated as shown in Table 2.2.5.

(4) Cable Crane

(a) 13.5 ton Cable Crane

A 13.5 ton cable crane was installed for transportation of dam concrete, concrete forms and other materials including penstock and intake gates of Lot II contractor Metal Works.

A cable crane of which one side was fixed on the left bank and the other side was movable by a traveling tower set on the right bank, was installed over the dam site. In consideration of the dam site topography and the dam features, especially crest length of the dam. Assuming the maximum rate of dam concrete placing was 1720 m^3/day and the cycletime of 188 sec., the capacity of buckets was decided at 4.5 m^3 (1720 m^3/day x hr/3600 sec.x188 sec.). The lifting capacity of the cable crane was 13.5 ton and span length was 465m.

The cable crane anchors and run way on left bank and right bank respectively were of concrete gravity type having concrete volume of 310 m^3 at the left bank and 1830 m^3 at the right bank. Excavation and concrete placing for anchors and run way on both sides were executed from the access roads and a 4.5 ton cable crane.

Details of plan profile and run way are shown in Fig. 2.2.17, and the specification of 13.5 ton cable crane is described below.

Specification of 13.5 ton Cable

Item	Specification		
Type	One side traveling cable crane		
Load	13.5 ton (Concrete bucket 4.5m^3)		
Span	465m (between the fixed tower and traveling tower)		
Fixed Tower	Height 11m		
Traveling Tower	Height 3.8m Run way length 120m, Rail gage 5m		
Wire Rope	Main rope	Ø 70mm	locked coil
	Hoisting rope	Ø 25mm	stranded rope
	Traversing rope	Ø 22.4mm	stranded rope
	Hanger rope	Ø 14mm	textile core
	Take-up rope	Ø 31.5mm	
Designed Speed	Hoisting up (full bucket)	90 m/min	
	Hoisting up (empty bucket)	180 m/min	
	Hoisting down	120 m/min	
	Traversing	300 m/min	
	Traveling	15 m/min	
Motor Capacity	Hoisting up/down	220 kw/unit	
	Traversing	185 kw/unit	
	Traveling	13 kw/unit	
	Take-up	11 kw/unit	

(b) 4.5 ton Cable Crane

A 4.5 ton cable crane was installed just upstream of the 13.5 ton cable crane for the installation works for the 13.5 ton cable crane and for transportation of miscellaneous materials from the left bank to the right bank.

(5) Tower Crane

A 150 ton.m tower crane was installed in the tailrace area to transport construction materials and equipment for the powerhouse and tailrace. The crane was also used by Lot II Contractor (Metal Works) and Lot IIIA (Turbine equipment) Contractors for their erection of the tailrace gate and draft tube and overhead crane respectively.

Specification of 150 t.m Tower Crane

Item	Specification	
Type	Jib Crane (JL-150)	
Post Height	43m (foundation 2.5m)	
Jib Length	50m	
Capacity	Radius 2.5 - 10m	10 ton (Max)
	Radius 50m	2.4 ton
Designed Speed	Hoisting up (10 ton)	20 m/min
	Hoisting up (3 ton)	60 m/min
	Rotating mechanism	0.6 r/min (r-radius)
	Trolley Traveling	50 m/min
	Traveling	23.5 m/min
Motor Capacity	Hoisting up/down	45kw / unit
	Traversing	37.5kw / unit
	Climbing	7.5kw / unit

(6) Telecommunication Facilities

Telecommunication system of Lot I Contractor consisted of 5 telephone lines and a radio link between PT.Telekom Pekanbaru and PLN Rantau Berangin through existing PT. Telekom Repeater Station in Kualu and the dam site office near the Switchyard. A TDMA radio link connected PLN Rantau Berangin, Bangkinang camps, Lot I Contractor's base camp, and dam site office, base equipment with PABX function located in PLN Rantau Berangin camp area for providing 70 telephone extensions and overhead cable to connect all other locations to the base station of the TDMA radio terminal stations.

(7) Power Supply Facilities

Electric power for the construction site and Lot I Contractor's camp were provided by a series of generators with the total installed capacity 4,830 kVA. Distribution line is with 220 voltage and 50 Hz. The maximum power demand of construction equipment was about 2,360 kVA at the dam site.

Power Supply Facilities

Location	Generator Capacity	Quantity
Base camp	250 kVA	1 unit
Crushing/washing plant	455 kVA	2 units
	100 kVA	1 unit
Dam site	590 kVA	4 units
	250 kVA	1 unit
Motor pool area	250 kVA	1 unit
	125 kVA	1 unit
(Others)	195 kVA	2 units
	150 kVA	1 unit
	15 kVA	3 units
Total		17 units

(8) Water Supply Facilities

Water supply for concrete, grout mixing, curing and cleaning works at the dam site were provided by 2 or 8 submersible pumps (capacity 8 m^3/min) from the upstream tributary before river diversion and from the upstream cofferdam after river diversion respectively.

(9) Oil Supply Facilities

Oil supply facilities were located at five places as described in the table below. In addition, one 8kl tank lorry was provided for the 2-day supply at the maximum rate of usage.

Oil Supply Facilities

Location	Description	Capacity	Quantity	Remarks
Motor Pool	Fuel Tank (Diesel)	10kl	3 set	Supply for Common use Vehicles & Dump Truck etc.
	Fuel Tank Rolly	8kl	1 unit	Supply for Heavy Equipment & Generators (go around in the site)
Crushing Plant	Fuel Tank (Diesel)	10kl	1 set	Supply for Generators
Cable Crane Yard (Left Bank)	Fuel Tank (Diesel)	10kl	1 set	Supply for Generators
Kuok Gravel Pit	Fuel Tank (Diesel)	5kl	1 set	Supply for Dump Trucks used for Aggregate Transportation
Base Camp B	Oil Store House (gasoline & lubricant)	24m^2	1 store	Supply for Vehicles. Store Lubricant and other oils (Hyd oil etc.)

(10) Air Supply System

Compressed air was supplied for various work including concreting work, rock surface cleaning and removal of laitance. The table below shows location and specifications of compressors.

Air Supply Facilities

Location	Type	Specification	Unit
Dam site	Engine drive rotary compressor portable type	7.0 m^3/min, 17.0 - 18.5 m^3/min	9 units

(11) Dewatering System

Dewatering systems were required in order to keep work areas in a dry condition. The rainfall and seepage from the construction site were drained off by submersible pumps. A sump pit was provided in the upstream of the dam with two units of 8 inch submersible pumps to discharge surface water and inflow water for curing, mixing and washing from the dam and grout plant. In the downstream area, a sump pit near the downstream cofferdam was also provided to discharge water from stilling basin and powerhouse area through submersible pumps.

No.	Description	Area (m²)
1	Main Office	471
2	Meeting Room	50
3	Ware House	302.5
4	Living Quarters 1	589.5
5	Dining Room & Kitchen 1	210
6	Living Quarters 1A	315
7	Family House 1	107
8	Family House 2	72
9	Japanese Bath Room	31.5
10	Living Quarters 2	497
11	Canteen	153
12	Labour Quarters	504
13	Dining Room & Kitchen 2	108
14	Toilet	108
15	MUSHOLLA (Praying House)	155
16	Guard House	6.5
17	Guest House	30
18	Guest House	15
19	Fuel Storage	15
20	Water Tower	10
21	Hot Water Tower	180
22	Garage (3 Nos)	

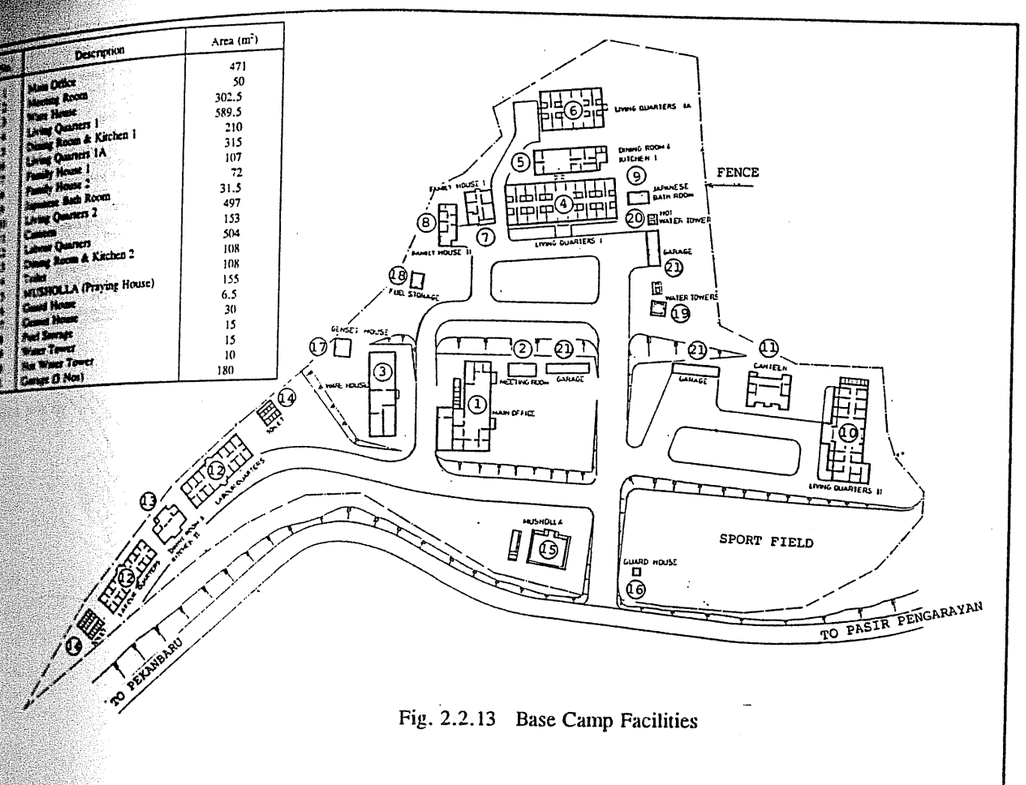


Fig. 2.2.13 Base Camp Facilities

No.	Description	Area (m²)
1	Meeting Room	40
2	Operator Room for 13.5 ton Cable Crane	2.5
3	Dam Site Office	50
4	Dam Site Work Shop	110
5	Dam Site Laboratory	40
6	Toilet	6.5

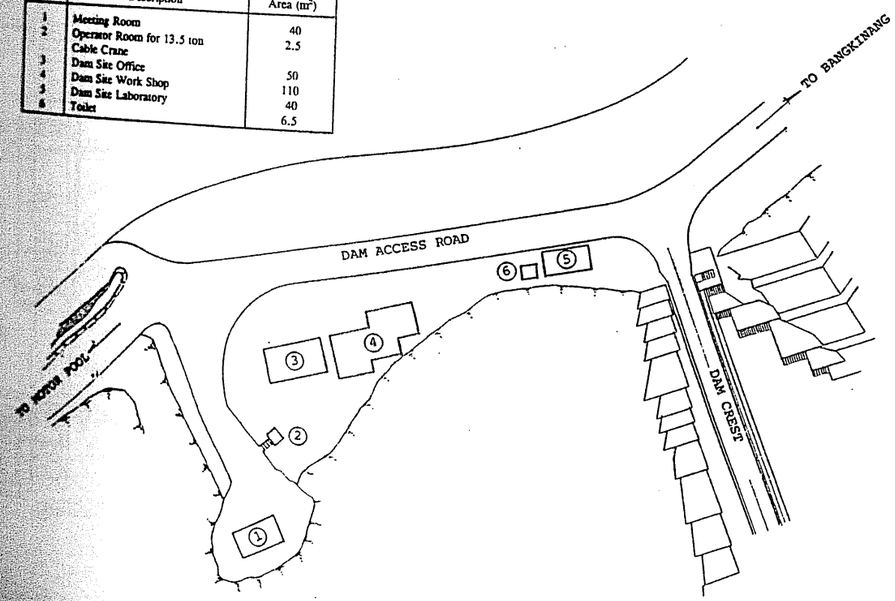
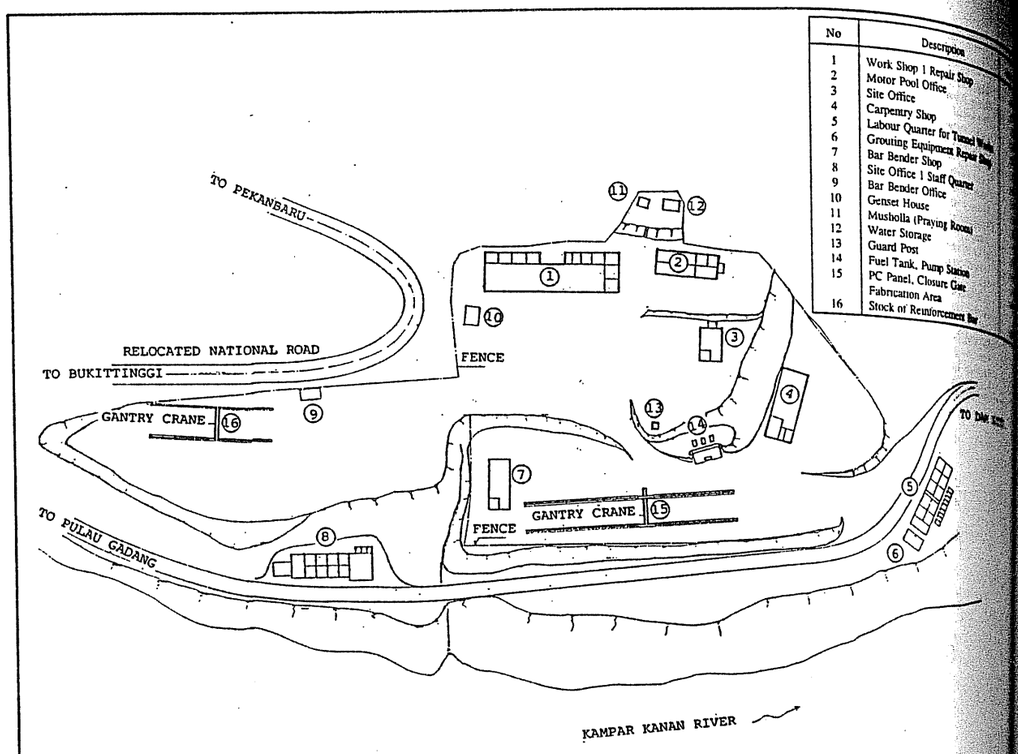


Fig. 2.2.14 Field Office at Dam Site



No	Description
1	Work Shop 1 Repair Shop
2	Motor Pool Office
3	Site Office
4	Carpentry Shop
5	Labour Quarter for Tunnel Works
6	Grouting Equipment Repair Shop
7	Bar Bender Shop
8	Site Office 1 Staff Quarter
9	Bar Bender Office
10	Genset House
11	Musbolla (Praying Room)
12	Water Storage
13	Guard Post
14	Fuel Tank, Pump Station
15	PC Panel, Closure Gate
16	Fabrication Area
	Stock of Reinforcement Bar

Fig. 2.2.15 Field Office at Motor Pool

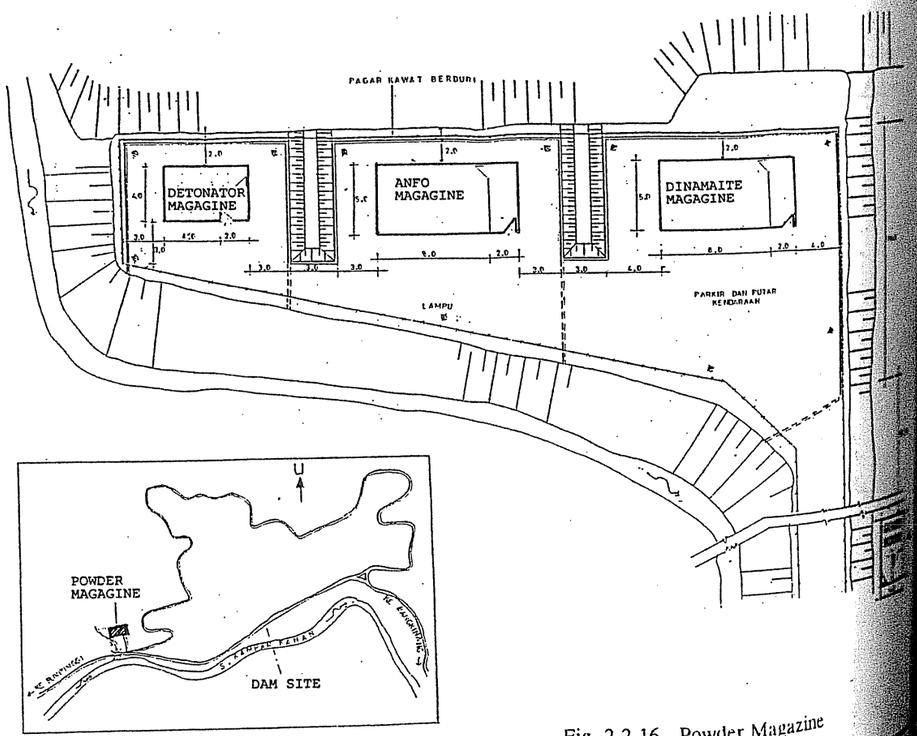


Fig. 2.2.16 Powder Magazine

Table 2.2.5 Explosive Used for Civil Works

(Unit : kg or pcs)

Year	Month	Dynamite			Ammonium	Electric
		Dam Tunnel	Powerhouse	Total	Nitrate (ANFO)	Detonator
1992	October	187.80	0.00	187.80	240.00	425.00
	November	1,149.4	0.00	1,149.40	0.00	724.00
	December	2,044.00	458.24	2,502.24	0.00	1,477.00
1993	January	438.80	6,238.80	6,677.60	0.00	7,078.00
	February	190.80	8,597.88	8,788.68	0.00	7,432.00
	March	9.20	17,060.91	17,070.11	0.00	14,265.00
	April	8.00	5,910.27	5,918.27	0.00	6,167.00
	May	672.00	3,698.95	4,370.95	0.00	4,055.00
	June	0.00	7,079.00	7,079.00	0.00	6,371.00
	July	0.00	2,353.20	2,353.20	0.00	1,562.00
	August	0.00	1,270.00	1,270.00	0.00	780.00
	September	0.00	1,223.40	1,223.40	0.00	737.00
	October	0.00	3,486.60	3,486.60	775.00	1,037.00
	November	0.00	7,922.75	7,922.75	6,260.00	3,911.00
	December	13,300.00	0.00	13,300.00	925.00	3,562.00
1994	January	18,930.00	0.00	18,930.00	0.00	5,132.00
	February	5,369.00	0.00	5,369.00	0.00	8,347.00
	March	1,821.00	0.00	1,821.00	0.00	2,078.00
	April	2,580.00	308.00	2,888.00	0.00	2,363.00
	May	0.00	2,172.00	2,172.00	0.00	4,556.00
	June	0.00	236.00	236.00	0.00	1,560.00
	July	0.00	0.00	0.00	0.00	0.00
	August	0.00	0.00	0.00	0.00	0.00
	September	0.00	480.00	480.00	0.00	780.00
	October	0.00	804.00	804.00	0.00	1,081.00
TOTAL		46,700.00	69,300.00	116,000.00	8,200.00	85,480.00

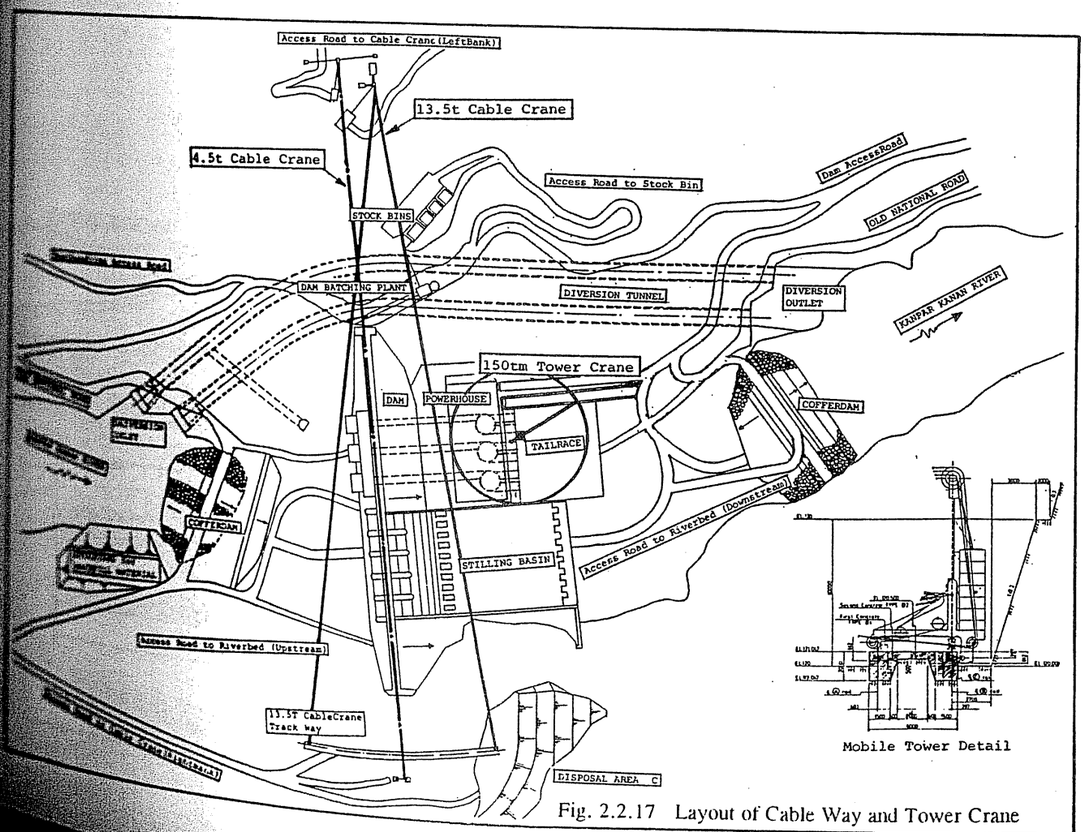


Fig. 2.2.17 Layout of Cable Way and Tower Crane

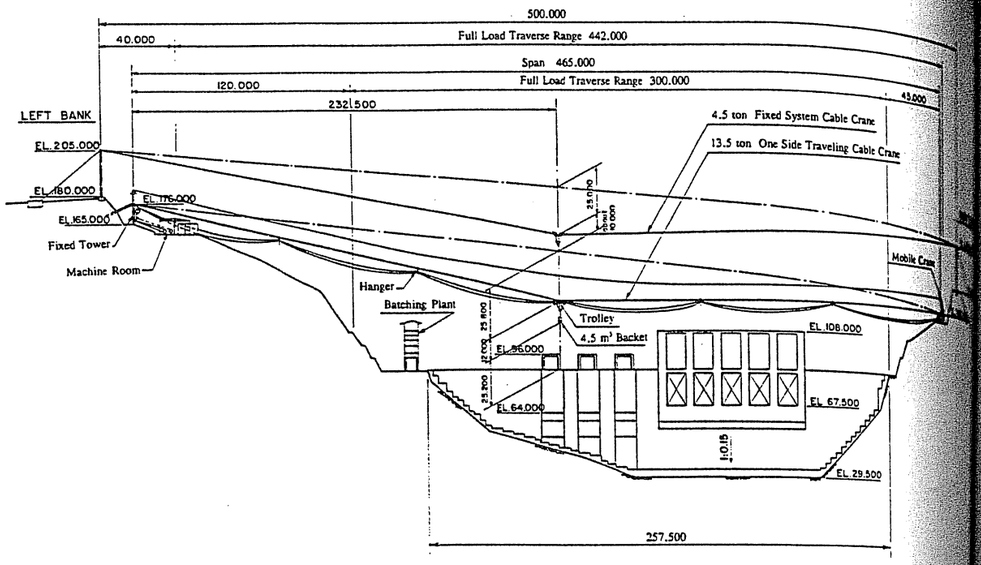


Fig. 2.2.18 13.5 ton and 4.5 ton Cable Crane Profile

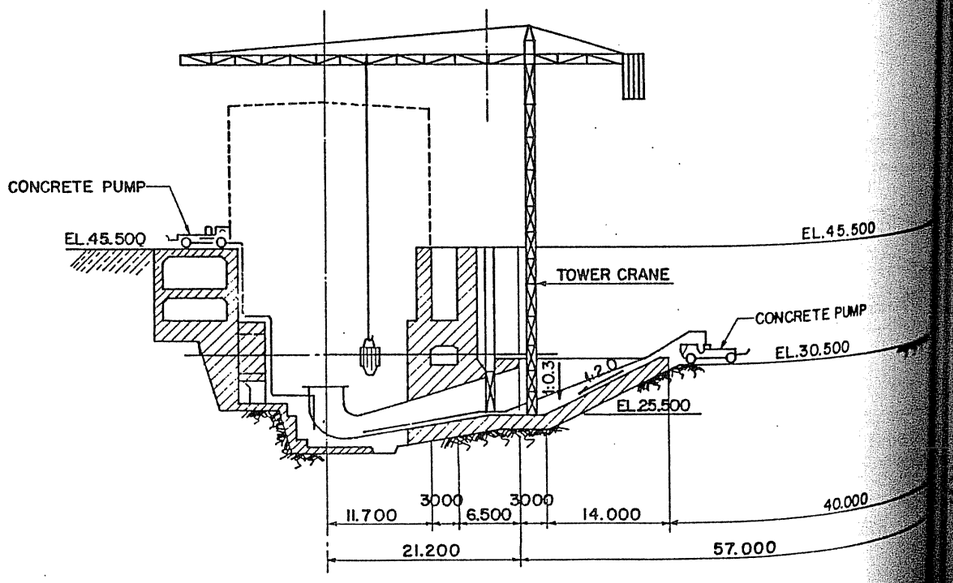


Fig. 2.2.19 Tower Crane on Powerhouse Longitudinal Section

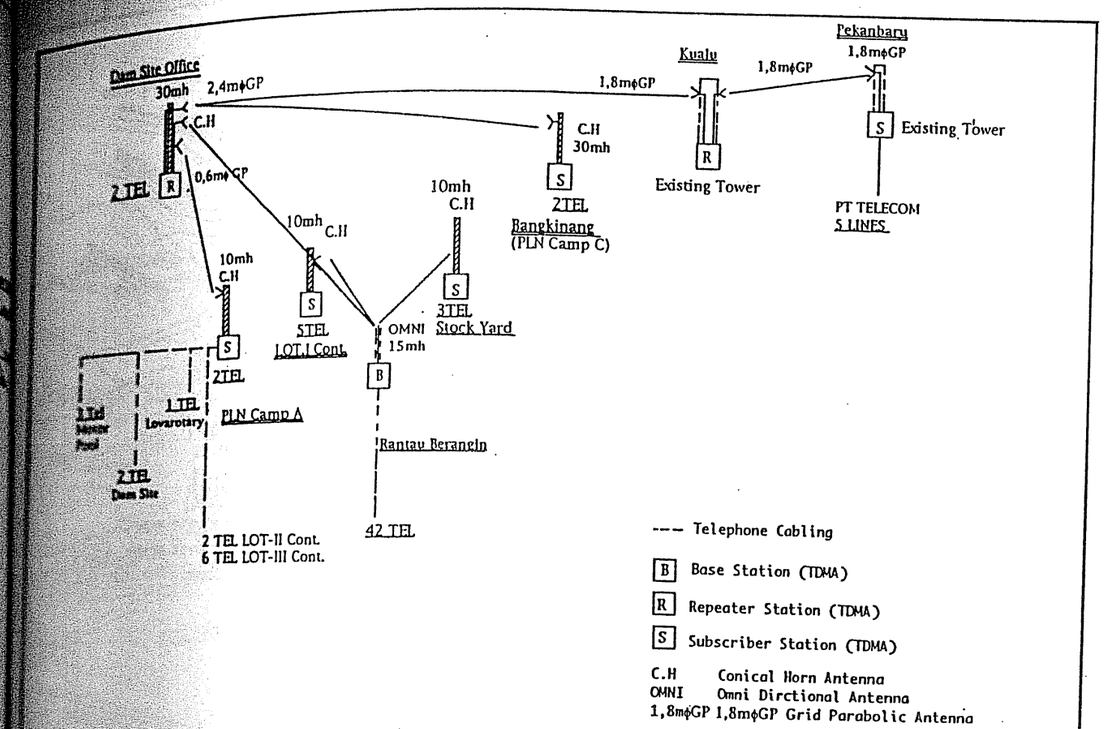


Fig. 2.2.20 Configuration of TDPMA Radio Telephone System

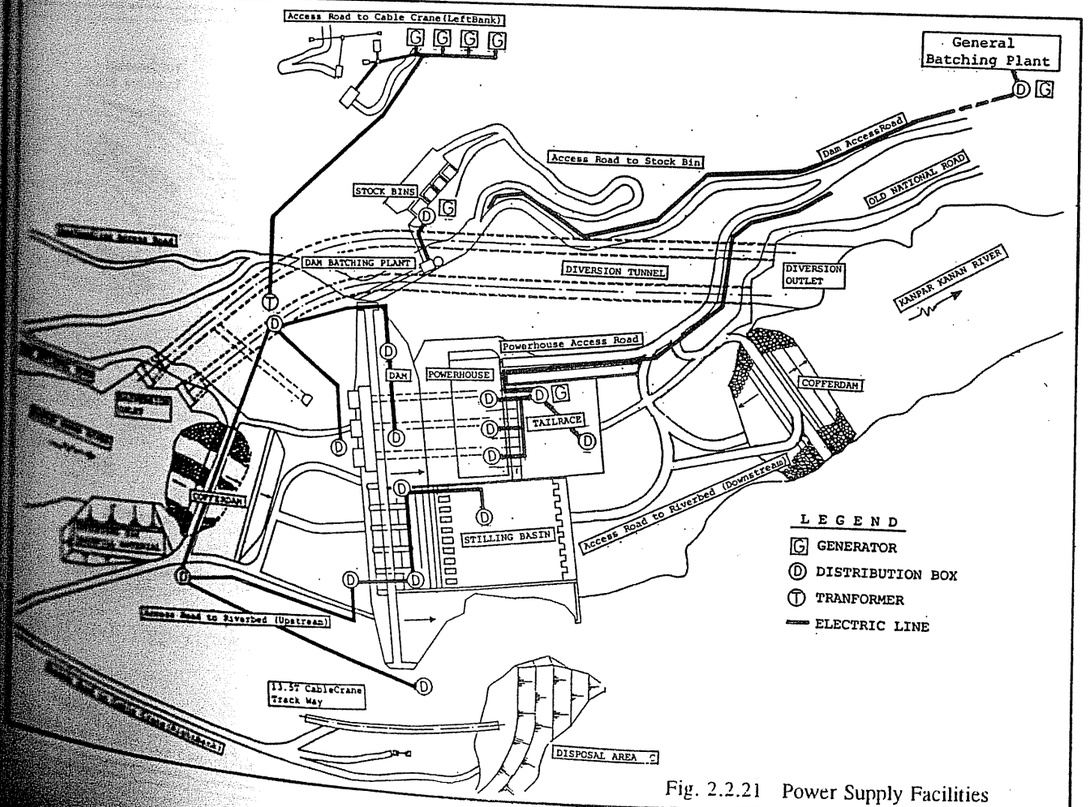


Fig. 2.2.21 Power Supply Facilities

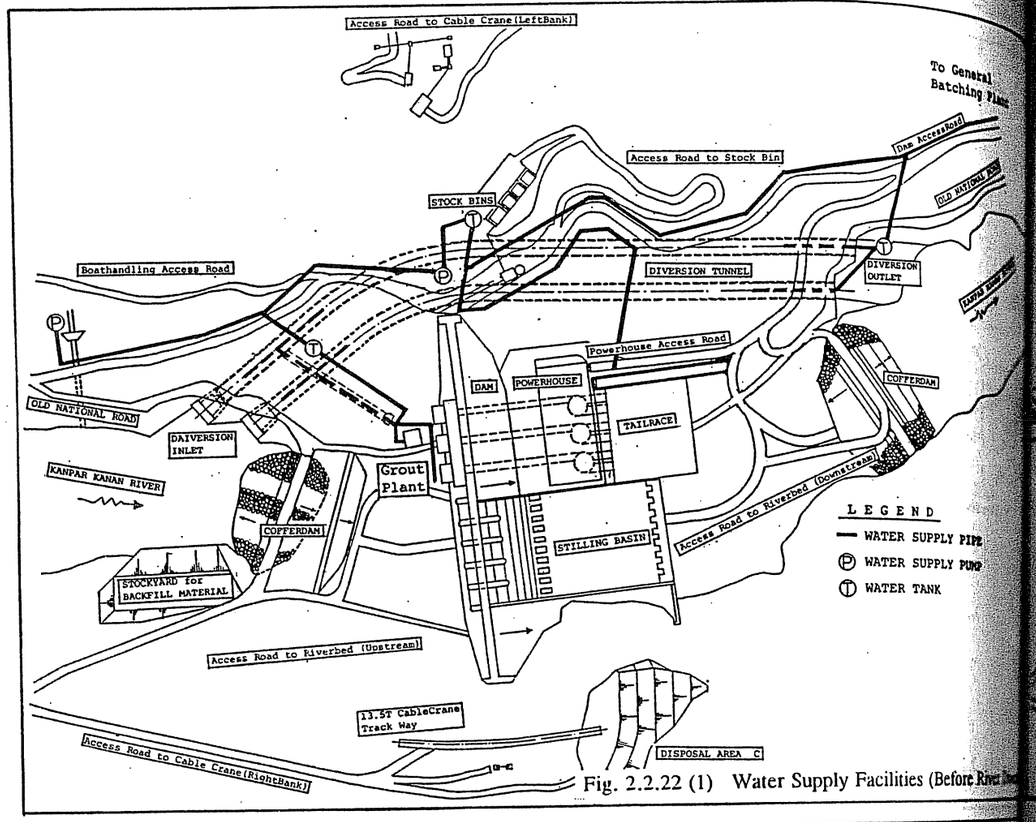


Fig. 2.2.22 (1) Water Supply Facilities (Before

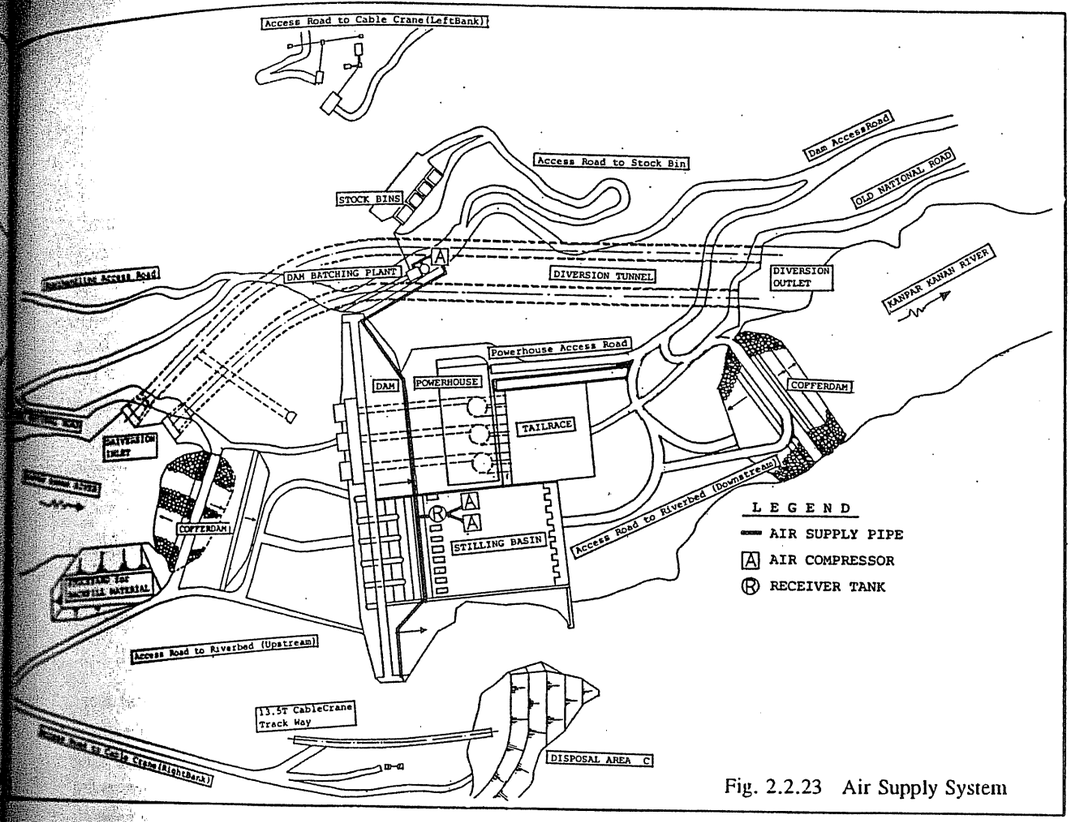


Fig. 2.2.23 Air Supply System

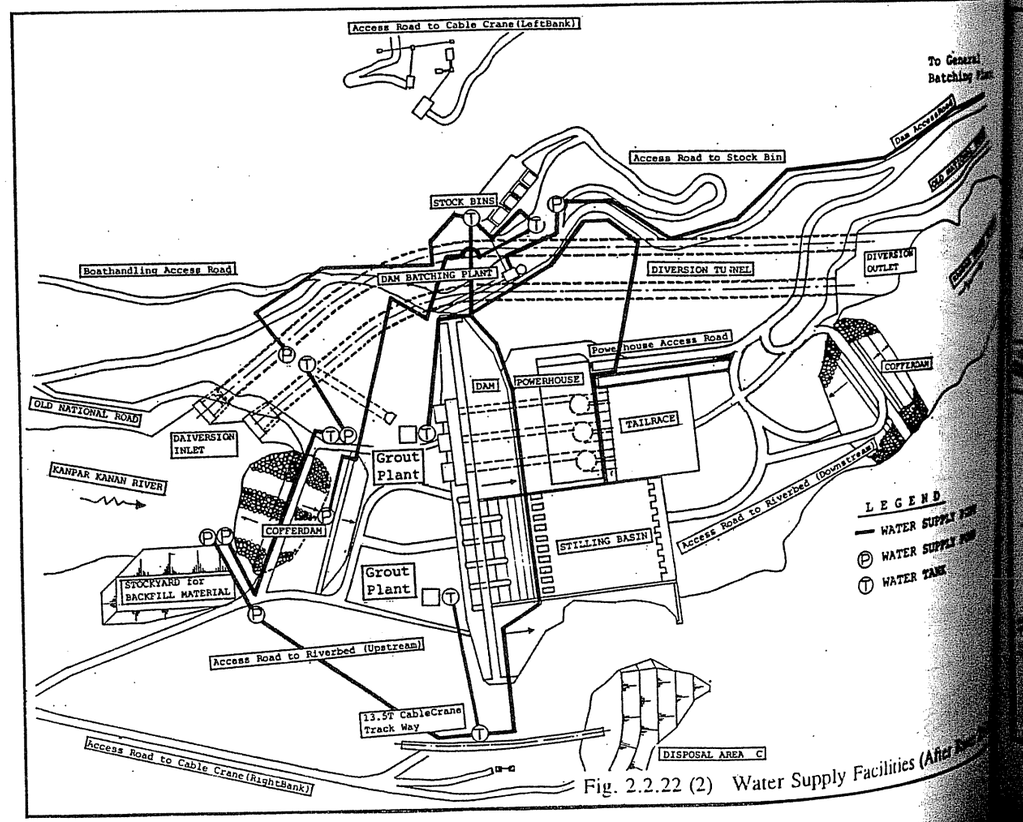


Fig. 2.2.22 (2) Water Supply Facilities (After

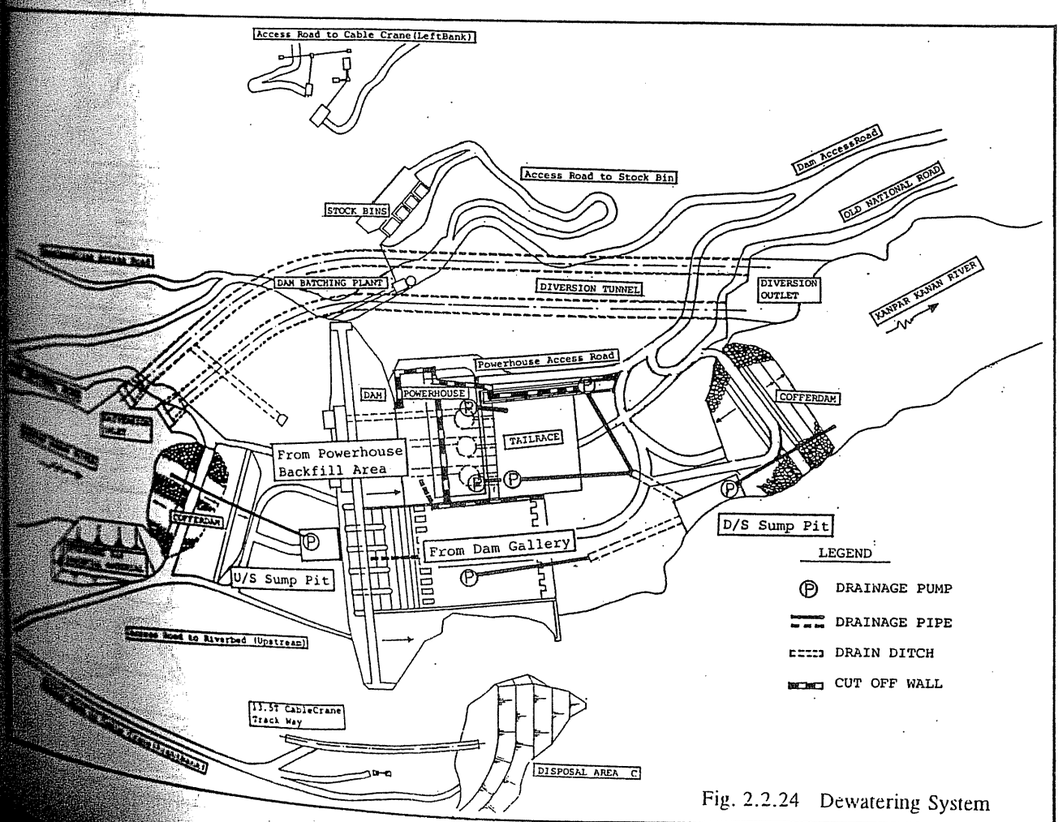


Fig. 2.2.24 Dewatering System

Chapter 3 ROADS AND DISPOSAL AREA

3.1 Permanent Roads and Temporary Access Roads

The permanent roads were built for this project to give access to the main structures powerhouse, boat handling and switchyard. The powerhouse and boat handling access roads branch from the dam access road, and the switchyard access road from the existing national road. The excavation for the dam access road had been completed mostly by an other Contractor before commencement of Lot-I Civil Works. These roads were constructed prior to the construction of each structure and were used for construction purpose. Afterwards, all roads including the dam access road were paved by asphalt treated based and hot rolled sheet with 4 cm and 3 cm thickness respectively at the final stage of the project in 1997 to be available for operation and maintenance in future.

For Muara Takus access road, improvement in the existing village road about 480 m long 5.0 m wide branched from the provincial road was done based on the discussion with Ministry of Education and Culture and PLN, works generally comprised making side drain ditch, raising low road level and construction for masonry wall.

Temporary unpaved roads were constructed for access to the gravel pits, disposal area, cable crane, stock bin and riverbed during construction. The temporary cable crane access road to right bank was modified and extended from 1,050 m to 2,750 m long because the crossing point at the Kampar Kanan river had to be changed further to the upstream point due to delay of commencement of the Civil Works for the rainy season.

The location, construction schedule and specifications of the roads were shown in Fig. 2.3.1, Fig. 2.3.2 and Table 2.3.1 respectively.

Fig. 2.3.2 Construction Progress of Roads

	Work Items	1992	1993	1994	1997
		10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9 10 11 12	1 2 3 4 5 6 7 8 9
Permanent Road	Powerhouse Access Road		Excavation		Pavement
	Swithyard Access Road		Excavation		Pavement
	Swithyard Plant Road			Excavation	Pavement
	Dam Access Road		Widening & Lowering		Pavement
	Boat Handling Access Road			Excavation	Pavement
	Muara Takus Access Road				
Temporary Access Road	Pulau Gadang Access Road				
	Kuok Access Road				
	Access Road to Disposal B				
	Access Road to Cable crane (R/B)				
	Access Road to Cable crane (L/B)				
	Access Road to Stock Bin				
	Access Road to River Bed (U/S)				
	Access Road to River Bed (D/S)				

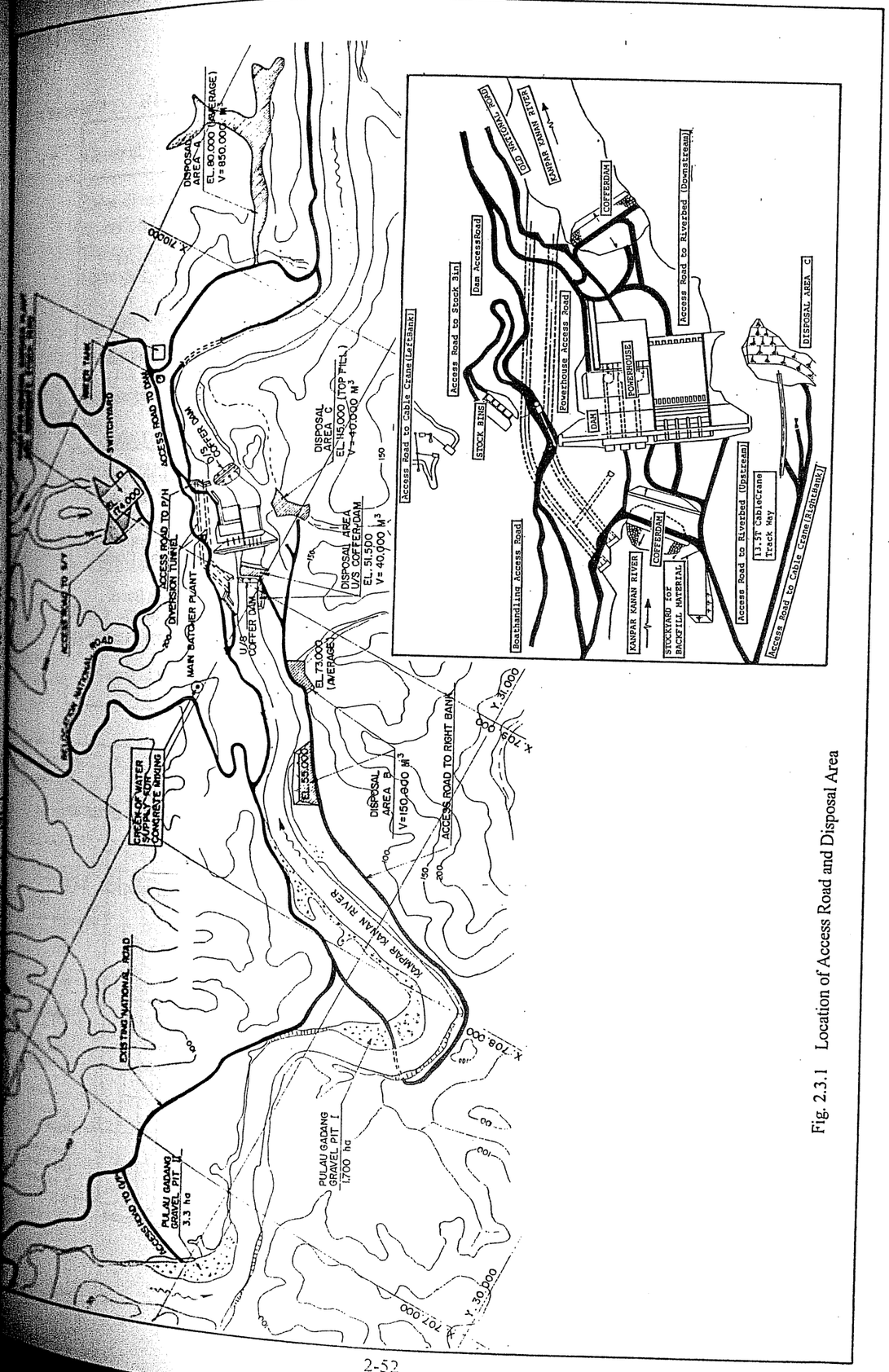


Fig. 2.3.1 Location of Access Road and Disposal Area

Table 2.3.1 Permanent Roads & Temporary Access Roads

Road	Length (m)	Width (m)	From - To
1. PERMANENT ROAD :			
(1) Powerhouse Access Road	322	6.0	New Dam Access Road to Powerhouse
(2) Switchyard Access Road	113	5.0	New National Road to Switchyard
(3) Switchyard Plant Road	433	5.0	Switchyard Area
(4) Dam Access Road	885	5.5	National Road to Dam
(5) Boat Handling Access Road	251	3.5	Existing Dam Access Road to Boat Handling
(6) Muara Takus Access Road			New Provincial Road to Muara Takus
2.TEMPORARY ACCESS ROAD:			
(1) Pulau Gadang Access Road	645	6.0	Old National Road to Pulau Gadang Gravel Pit
(2) Kuok Access Road	1489	8.0	National Road to Kuok Gravel Pit
(3) Access Road to Disposal A	700	4.5	National Road to Disposal Area A
(4) Access Road to Cable Crane (Right Bank)	2,750	6.0	Pulau Gadang to Cable Crane's mobile tower at right bank
(5) Access Road to Cable Crane (Left Bank)	200	6.0	New National Road to Cable Crane's fixed tower at left bank
(6) Access Road to Stock Bin	300	6.0	Dam Access Road to Stock Bin of Main Batching Plant
(7) Access Road to Riverbed (Upstream)	320	4.5	Access Road to Cable Crane (Right Bank) to Dam
(8) Access Road to Riverbed (Downstream)	420	4.5	Powerhouse Access Road to Dam

3.2 Disposal Condition

The excavated rock and common material were transported to four disposal areas as shown in Figure 2.3.1. Distance and disposal volumes are shown below.

The Disposal A was used as office and work shop area for Lot II (metal works), Lot IIIA (turbine) and Lot IIIB (generator) from July 1994.

Disposal Area	Location	Distance from Site	Disposal Vol.
A	Left Bank Downstream	1.5	850,000 m³
B	Right Bank Upstream	1.0	150,000 m³
C	Right Bank	Directly	40,000 m³
-	Upstream Cofferdam	1.3	40,000 m³

Chapter 4 RIVER DIVERSION

4.1 General

River diversion works for the Kotapanjang dam site involved construction of two lined diversion tunnels and upstream and downstream cofferdams, as shown in Fig.2.4.1. The tunnels located in the left bank of the Kampar Kanan river have a 10m diameter horseshoe section 478.8m long for No.1 tunnel and 423.0m long for No.2 tunnel.

The upstream cofferdams consisted of the earth fill type for the primary cofferdam and roller compacted concrete type for the secondary cofferdam, located 140m upstream from the dam axis. The downstream cofferdam was of earth fill type and located 300m downstream from the dam axis.

Construction of the diversion tunnels commenced in October 1992 with excavation of the inlet portals. The tunnel excavation was completed in June 1993 for No.1 tunnel and in July 1993 for No.2 tunnel. The average and maximum rate of advance achieved 4.03 m/day and 6.9 m/day respectively. Concrete lining was placed from April 1993 to September 1993. River diversion was carried out on October 1, 1993 followed by construction of upstream and downstream cofferdams for two (2) months.

Work quantities of major items are summarized in Table 2.4.1 and actual work progress are also shown in Fig.2.4.2.

Fig. 2.4.2 Work Progress of Diversion Tunnels

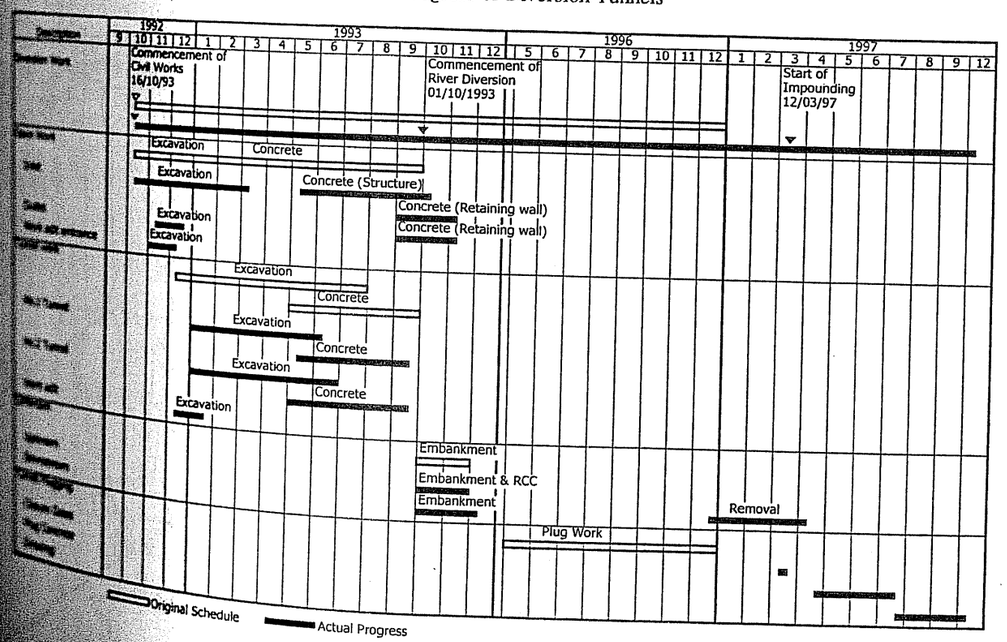


Fig. 2.4.1 (2) Profile of Diversion Tunnels

Table 2.4.1 Work Quantity of Diversion Works

Description	Quantity	Description	Quantity
Diversion tunnels and portals		(Tunnel works)	
(Inlets and outlets)		Excavation	2035 m^3
Open excavation, common	21517 m^3	Rock bolt	179 pcs
Open excavation, rock	89369 m^3	Shotcrete	892 m^2
Rock bolt	1438 pcs	Concrete	420 m^3
Gunite	7044 m^2	Backfill	1615 m^3
Concrete	8803 m^3		
Backfill	7882 m^3	Cofferdams	
(Tunnel)		(Upstream)	
Tunnel Excavation	109668 m^3	Primary, fill type	18698 m^3
Rock bolt	3460 pcs	Secondary, RCC type	14286 m^3
Shotcrete	25723 m^2	(Downstream)	
Concrete lining	32859 m^3	Primary, fill type	31459 m^3
		Secondary, fill type	14393 m^3
Tunnel Adit		Diversion tunnel plug	
(Open Works)		Concrete	6259 m^3
Excavation common	574 m^3	Consolidation boring	1019 m
Excavation rock	6542 m^3	Consolidation grouting	40 ton
Rock bolt	6 pcs	Curtain boring	1456 m
Gunite	213 m^2	Curtain grouting	31 ton

4.2 Open Works for Diversion Tunnels

Open works for the diversion tunnels consisted of inlets and outlets, and the tunnel adit portal was located at approximately 60m downstream from the inlet portals. The tunnel adit was planned to give access from the upstream side to enable excavation from upstream towards the downstream direction before completion of excavation of the inlet to shorten the excavation schedule.

(1) Excavation

Temporary cofferings for the construction of open works were constructed along the existing national road around the inlet and outlets portals, as shown in Fig.2.4.3 and Fig.2.4.4. The temporary cofferings were built at EL.45.0, 10m up from the riverbed, to avoid overtopping during construction for the river discharge of 140 m^3/s.

The foundation rock of the inlet bases for No.1 and No.2 tunnels was very hard massive and jointed dacitic tuff, therefore the geological condition of the open works for the diversion tunnels was generally stable. Open excavation for the diversion tunnels commenced on October 16, 1992, and completed on March 16, 1993.

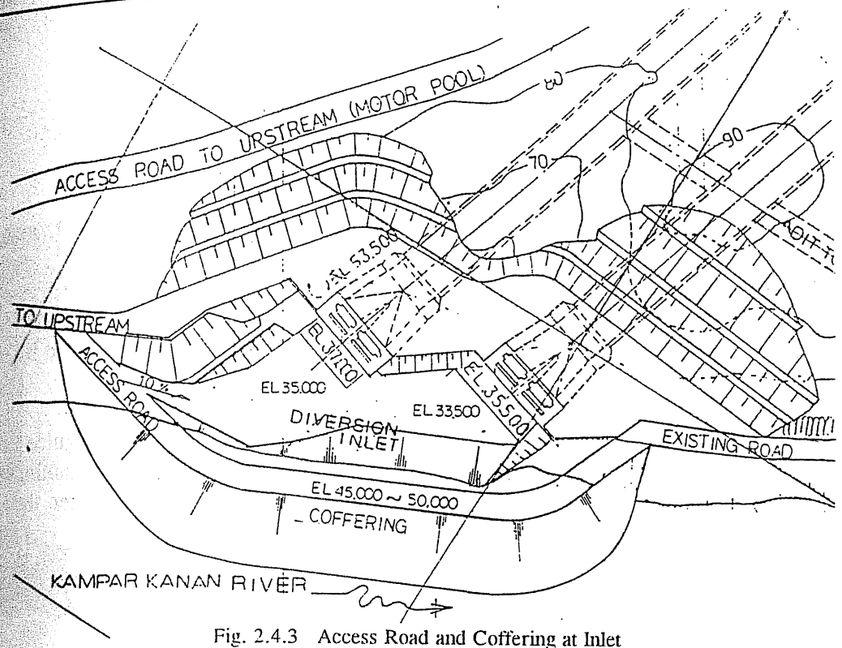


Fig. 2.4.3 Access Road and Coffering at Inlet

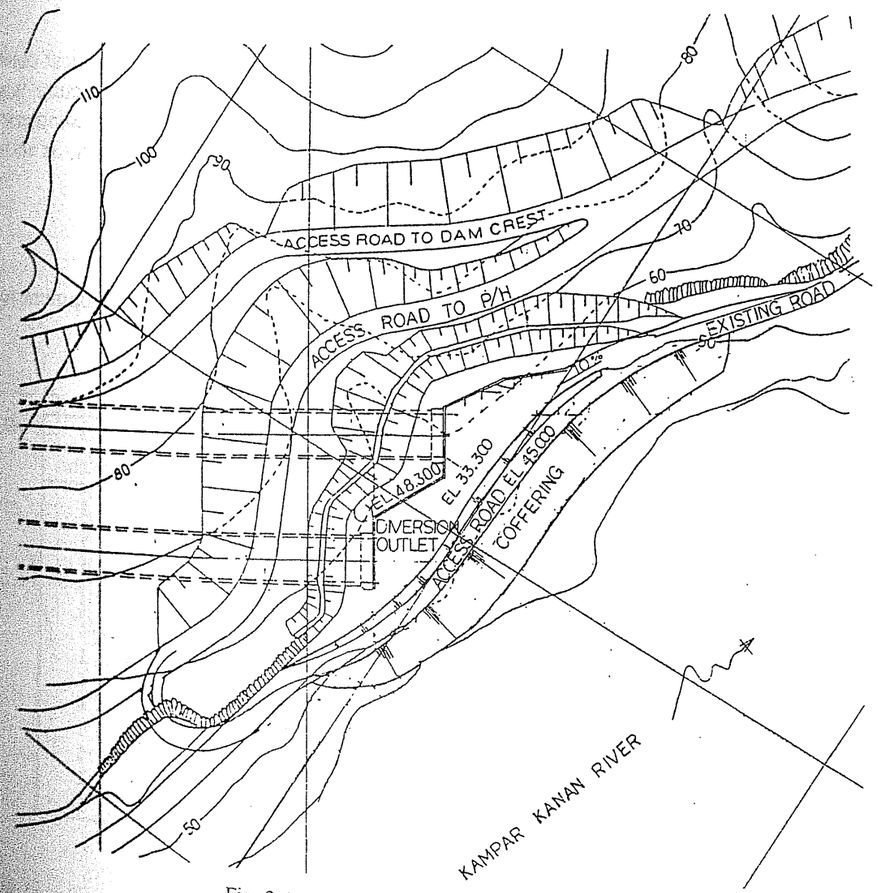


Fig. 2.4.4 Access Road and Coffering at Outlet

(2) Placing Concrete

(a) Inlet

Concrete placement for the inlet structures commenced in May 1993, and was completed in September 1993.

Concreting was mainly divided into four blocks such as the invert, side walls, center pier and top structures. The forms and reinforcement bars were installed by using conventional scaffolding.

After completion of the inlet structures, the secondary concrete for gate guide groove was placed in September 1993. Concrete placement for the inlet retaining walls commenced in August 1993 and completed in November 1993 after the river diversion.

Fig. 2.4.5 to Fig. 2.4.7 show progress of the inlet structure concrete, second stage concrete and retaining wall respectively.

(b) Outlet

The outlet portals were eliminated from construction, because geological condition around the outlet area was stable, therefore the concrete outlet structure was not necessary to be constructed. Concrete works for the retaining walls commenced in August 1993 and completed in October 1993 after the river diversion, as shown in Fig.2.4.8.

(3) Construction performance data

The summarized tables below are main performance data such as equipment for the works and the progress rate of open excavation.

Main Equipment for Open Works

Operation	Equipment	Capacity	Quantity Inlet	Quantity Outlet
Drilling	Hydraulic Crawler Drill	165 kw	1	3
	Pneumatic Crawler Drill	135 kw	1	0
Excavation	Excavator	1.3 m^3	1	2
		0.7 m^3	1	1
	Bulldozer with Ripper	45 ton	1	1
		32 ton	2	1
	Hydraulic Breaker	1,500 kg	1	1
Loading	Wheel Loader	2.4 m^3	1	1
	Dozer Shovel	2.2 m^3	1	8
Hauling	Dump Truck	11 ton	8	1
Rock Bolt	Mortal Mixer Pump	2 m^3/hr	1	1
Gunite Shooting	Shotcrete Machine	12 m^3/hr	1	1
	Dump Truck with Vessel	11 ton		1

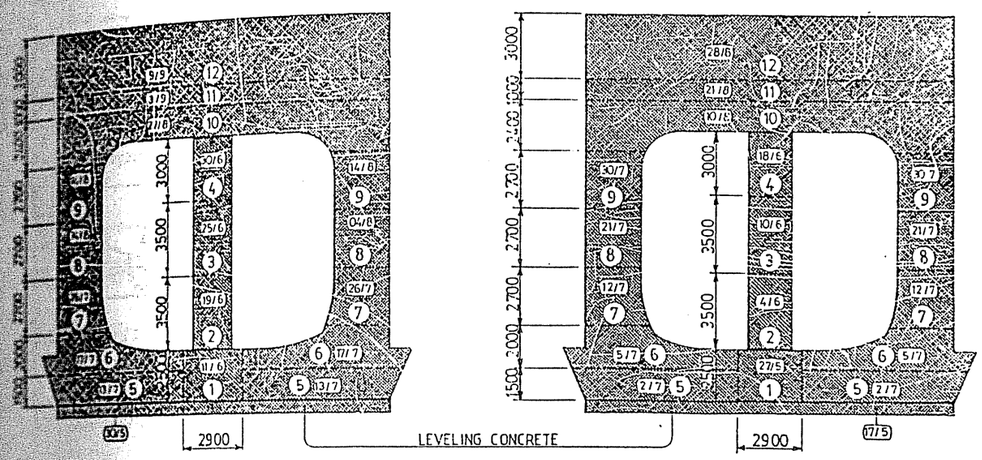


Fig. 2.4.5 Inlet Structure Concrete Progress

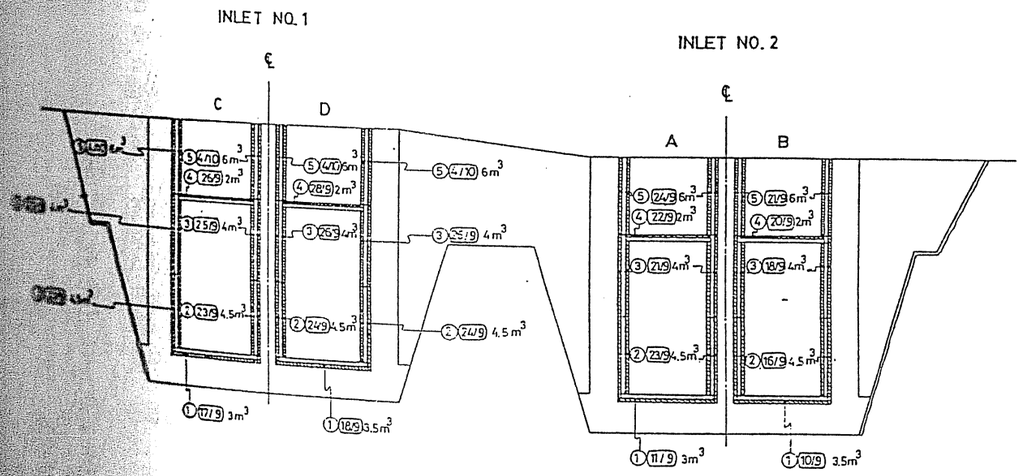


Fig. 2.4.6 Monthly Progress Second Stage Concrete for Gate Guide

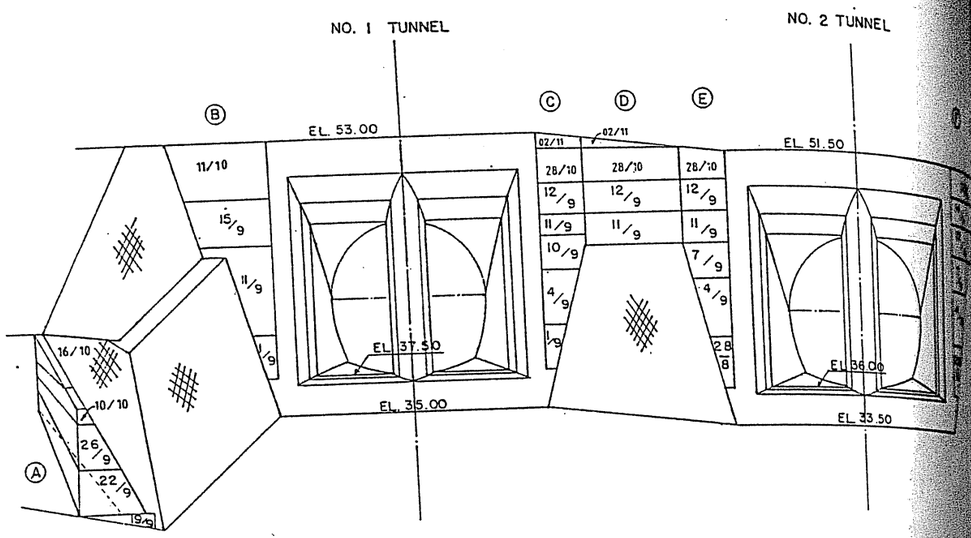


Fig. 2.4.7 Inlet Retaining Wall Concrete Progress

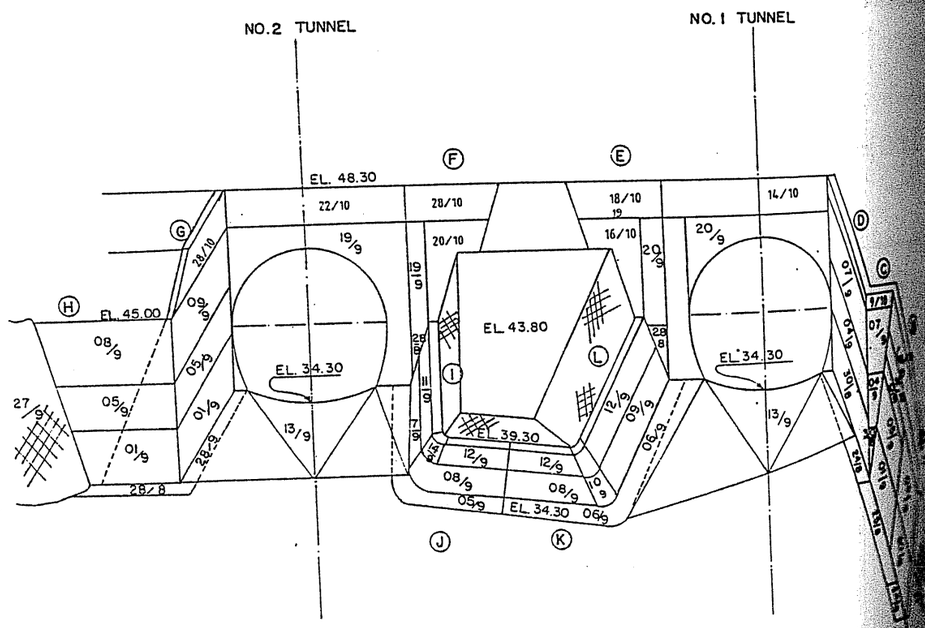


Fig. 2.4.8 Outlet Retaining Wall Concrete Progress

Result of Open Excavation and Concrete Works

		Volume (m^3)			Working days	m^3/day	Person/ 100 m^3
		Common	Rock	Total			
Excavation	Inlet	11,957	52,818	64,775	142	456	4.39
	Outlet	9,560	36,551	46,111	43	1,072	4.33
	Adit	574	6,542	7,116	33	216	5.67
Concrete	Inlet	-	-	6,537	51	128	84
	Outlet	-	-	2,266	33	69	58

4.3 Diversion Tunnels Works

4.3.1 General

Two diversion tunnels were constructed in the left bank of the dam site for the length of 500m approximately to divert the design discharge of 1300m3/s.

Construction of the diversion tunnels commenced in October 16, 1992, six and half months later than the original plan which planned to divert the Kampar Kanan River in the dry season between June and July of 1993, therefore all the diversion work had to be completed within one year.

To meet this requirement, the following countermeasures were selected to shorten the construction schedule of the tunnels works.

(i) An additional tunnel adit was constructed at 60km downstream from the inlet to allow for access from the upstream side of the tunnels during the construction. By this adit, open works for the inlet portals and excavation work for the tunnels from the inlet to the outlet were carried out concurrently, and the main dam construction work was accelerated. When the upstream cofferdam was slightly overtopped due to unexpected large flood in February 1995, the main dam having been increased the bulk of the dam temporarily could prevent the downstream work site from innundation by the flood.

(ii) The outlet portal structures were omitted to reduce concrete work before river diversion.

(iii) The elevation of No.1 tunnel inlet was raised 1.5m to EL.36.5 from the original design to proceed in the river diversion No.2 Tunnel firstly and reduce work quantity for related works such as cofferdam, and portal excavation and gunite.

Principal dimensions of the diversion tunnels are shown in the Table 2.4.2.

Table 2.4.2 Principal Dimension of Diversion Tunnels

Item	Dimension and Quantity
Tunnel Length and Gradient	No.1 : 478.8m 1/149.628 No.2 : 423.0m 1/248.829
Inner Cross Section	Diameter : 10.0m Horse Shoe Shape Sectional area : 82.9 m^2
Cross Section for Excavation	Upper half : 54m^2 Lower half : 60m^2
Shotcrete	Thickness : 5cm, 10cm, 15cm
Rock bolt	Length : 3.0 m
Lining Concrete	Thickness : 80 cm

4.3.2 Excavation

(1) Excavation method and progress

Tunnel excavation was carried out in halves by driving the upper half first, and upon completion, the lower half next, because of the large cross section. Tunnel driving of upper half was done from downstream face and junction of the tunnel adit in each diversion tunnel. After completion of excavation of upper half, the excavation of lower half of the tunnels was carried out from upstream and downstream faces towards around tunnel station 200 m.

As shown in the tunnel progress charts of Fig.2.4.9, excavation of the upper halves commenced in January 1993 and were completed in April 1993. Following completion of the upper halves excavation, the lower halves excavation commenced in April 1993 and were completed on July 4, 1993 for No.1 tunnel and June 30 1993 for No.2 tunnel.

The tunnels were excavated 197m from the inlet and 279m from the outlet of the No.1 tunnel, while 164m from the inlet and 256m from the outlet of the No.2 tunnel. The rock condition along the diversion tunnels was hard and massive dacitic tuff, except 2m^3 of highly cracked rock collapsed from the mountain side shoulder at station 382m from the No.1 inlet and slightly weathered dacitic tuff found at station 246m from the No.2 inlet.

(2) Tunnel support

The tunnel support system was classified into five types according to the geological condition at design stage. Actually, the tunnel No. 1 and No. 2 was mainly located in generally CH class rock and good rock with rating of 80-61, class II based on the geomechanic classification. Therefore, support type O was mainly applied as shown in the table below.

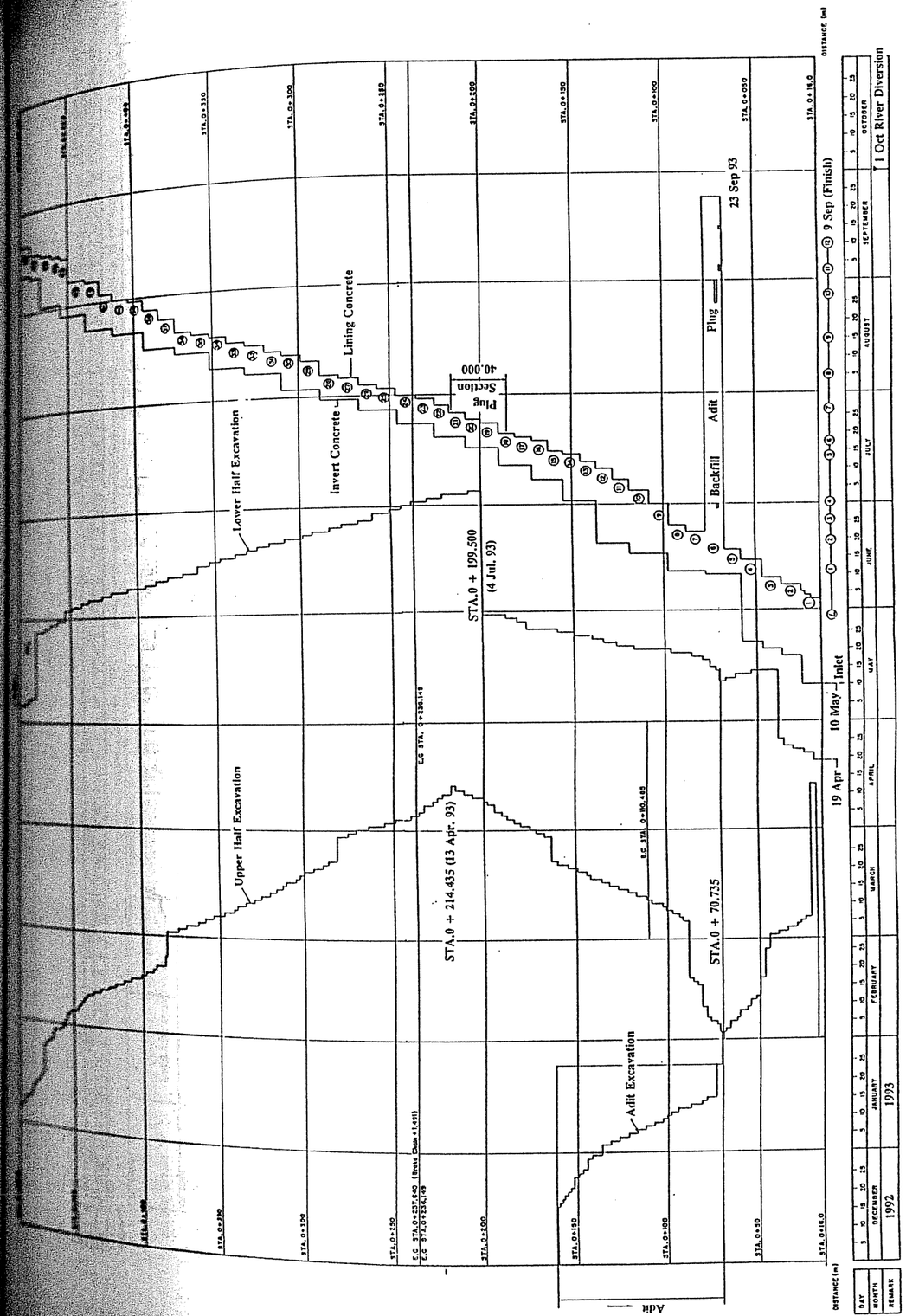
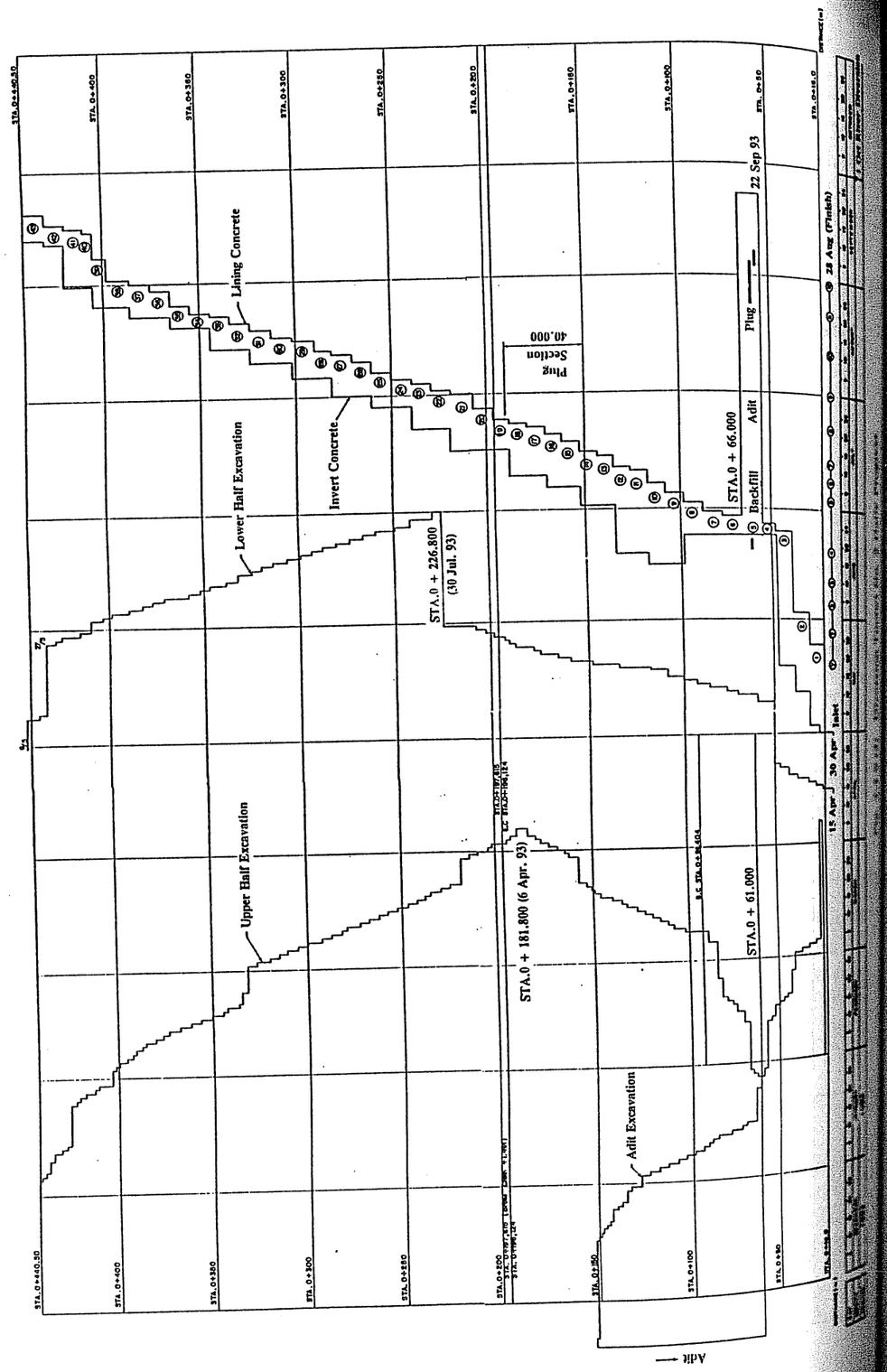


Fig. 2.4.9 (1) Diversion Tunnel No. 1 Daily Progress

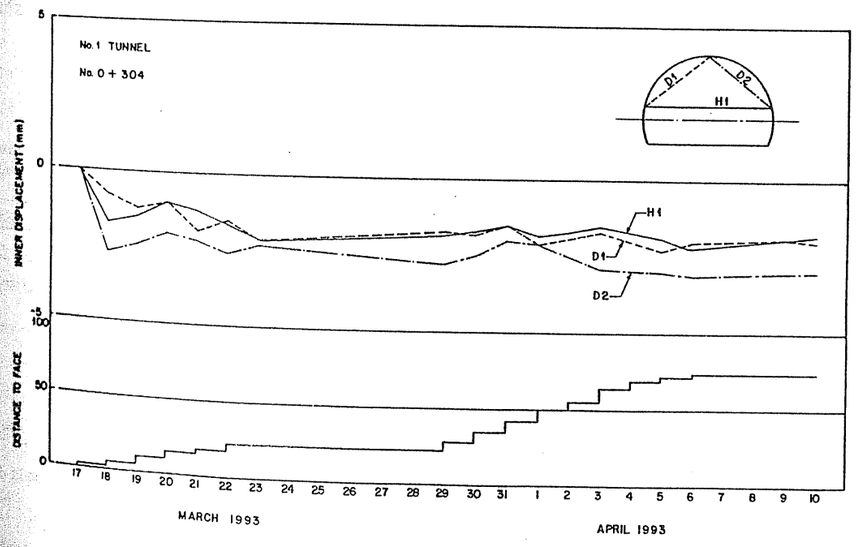


Support Pattern	Tunnel No.1		Tunnel 2		Total	
	Length (m)	(%)	Length (m)	(%)	Length (m)	(%)
0	418.2	87	328.1	78	746.3	83
I	38.9	8	60.4	14	99.3	11
II	18.2	4	34.5	8	52.7	6
III	0.0	0	0.0	0	0.0	0
IV	3.5	1	0.0	0	3.5	0
Total	478.8	100	423.0	100	901.8	100

For the purpose of confirming the stability of the ground around the tunnel opening after supporting, measurement of displacement of the inner section was carried out. The following figure illustrates the result of measurement at STA 0+304.0 m in the diversion tunnel No. 1 in where support type Ø was applied under condition of rock class II. The displacement D1, D2 and H1 increased in proportion to the progress of tunnel excavation and reached about 2-3 mm (shrinkage) on 5-6 days hence from completion of the tunnel excavation of the gauging section. When the face of tunnel excavation was about 22 m distant from the gauging section and the distance was equivalent to 2.2 D (diameter of the tunnel). Afterwards, the displacement became stable and no harmful deformation generated around tunnel surface and support members.

Other gauging sections showed the same tendency for displacement movement as the gauging section mentioned above and maximum displacement was 7.8 mm at STA 0+51.3 m in the diversion tunnel No. 1.

Therefore, the tunnel support system applied to the diversion tunnel was judged to be adequate.



(3) Construction performance data

The summarized tables below are the main performance data such as summary of tunnel excavation works, equipment for the works and temporary facilities for tunnelling works.

Tables 2.4.3 shows the progress of excavation : The average progress of the upper half section was 4.0m/day in both tunnels, and the progress of the lower half section were 7.6 - 8.0m/day in the No.1 tunnel and 6.5 - 7.4 m/day in the No.2 tunnel. Drilling pattern for the diversion tunnels is shown in Fig.2.4.10 and the standard cycle time for the upper half excavation is also shown in Fig.2.4.11.

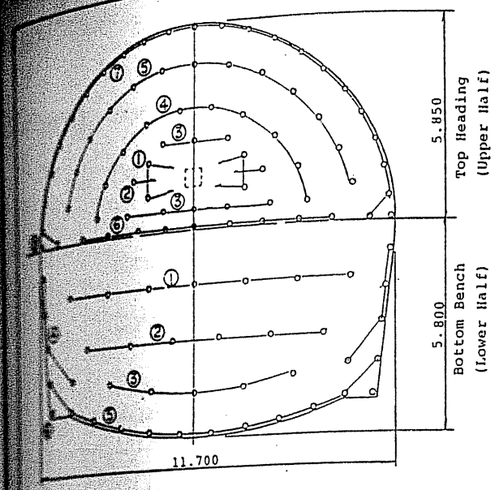
Table 2.4.3 Excavation Progress Data for Diversion Tunnels

NO. 1 TUNNEL

Items / Year Month	Upper Half Section						Lower Half Section					
	Direction from Inlet			Direction from Outlet			Direction from Inlet			Direction from Outlet		
	Length (m)	Work Darys	m/day	Length (m)	Work Days	m/day	Length (m)	Work Darys	m/day	Length (m)	Work Days	m/day
1992 November				2.0	1	2.0						
1992 December				37.7	16	2.4						
1993 January	1.0	1	1.0	71.4	17	4.2						
1993 February	43.1	12	3.6	120.6	25	4.8						
1993 March	109.2	25	4.4	45.154	9	5.0	23.5	5	4.7			
1993 April	43.26	10	4.3				160.009	19	8.4	46.0	7	6.6
1993 May										225.3	27	8.3
1993 June										24.0	3	8.0
1993 July												
Total/Average	196.56	48	4.1	278.874	68	4.1	183.509	24	7.6	295.3	37	8.0

NO. 2 TUNNEL

Items / Year Month	Upper Half Section						Lower Half Section					
	Direction from Inlet			Direction from Outlet			Direction from Inlet			Direction from Outlet		
	Length (m)	Work Darys	m/day	Length (m)	Work Days	m/day	Length (m)	Work Darys	m/day	Length (m)	Work Days	m/day
1992 November				2.0	1	2.0						
1992 December				37.7	15	2.5						
1993 January	1.0	3	3.3	71.7	18	4.0						
1993 February	35.1	9	3.9	128.0	25	5.1						
1993 March	99.0	25	4.0	17.009	5	3.4	33.0	7	4.7			
1993 April	19.825	5	4.0				175.509	21	8.4	35.0	8	4.4
1993 May										179.5	37	8.5
1993 June												
1993 July												
Total/Average	163.925	42	3.9	255.709	64	4.0	208.509	28	7.4	214.5	45	8.0



Data per Round (Average)

Items		Top Heading	Bottom Bench
Section are	(m^2)	53.8	60.5
Advance	(m)	2.3	2.8
No. of holes	(holes)	71	40
Explosive	(kg)	82.8	37.0
Powder factor	(kg/m^3)	0.67	0.22
Detonator factor	(pcs/m^3)	0.67	0.22

Fig. 2.4.10 Drilling Pattern for Diversion Tunnel

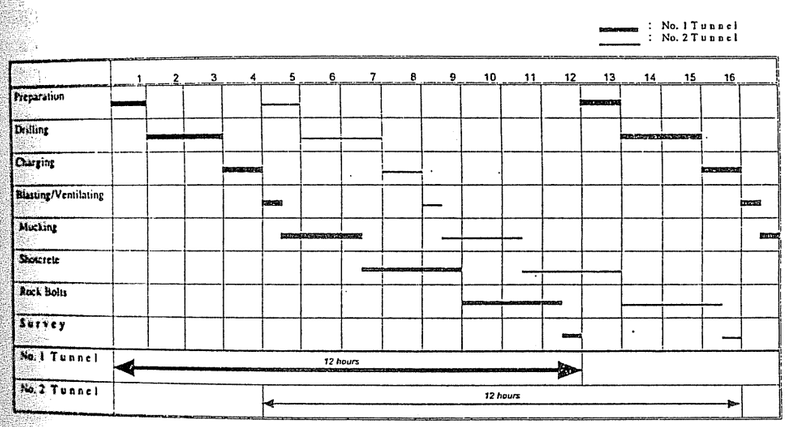


Fig. 2.4.11 Standard Cycle Time for Upper Half Excavation (Type 0) (Excavated Direction from Outlet)

Main Equipment for Tunnel Excavation

Operation	Equipment	Capacity	Quantity Inlet	Quantity Outlet
Drilling	Hydraulic wheel jumbo, 2 booms	20 ton	2	4
Loading	Wheel loader	3.5 m^3	2	
Hauling	Dump truck	15 ton	6	
Trimming	Excavator	0.7m^3	1	1
Shotcrete placing	Hydraulic breaker	1500 kg	1	2
	Shotcrete machine	12 m^3/hr	3	
	Sprayer robot	12 m^3/hr	3	
Rock bolting	Aditator Truck	4.5 m^3	2	
	Hydraulic wheel jumbo, 2 booms	20 ton	same as drilling	
Monitoring	Mortal mixer pump	2 m^3/hr	2	
	High roof truck	5 t	2	

Summary of Tunnel Excavation Works

		No. 1 Tunnel	No. 2 Tunnel	Total
Tunnel length (m)	(Upper Half)	475.4	419.6	895.0
	(Lower Half)	478.8	423.0	901.8
	(Total)	954.2	842.6	1796.8
Excavation volume (m^3)	(Upper Half)	27314	24171	51485
	(Lower Half)	30863	27320	58183
	(Total)	58177	51491	109668
Working Days (days)	(Upper Half)	116	106	222
	(Lower Half)	61	61	122
	(Total)	177	167	344
Progress Rate (m/day)	(Upper Half)	4.10	3.96	4.03
	(Lower Half)	7.85	6.93	7.39
	(Total)	5.39	5.05	5.22
Explosives (kg/m^3)	(Upper Half)	0.62	0.62	0.62
	(Lower Half)	0.20	0.21	0.21
	(Total)	0.40	0.41	0.40
Detonator (Pcs/m^3)	(Upper Half)	0.56	0.57	0.57
	(Lower Half)	0.19	0.22	0.21
	(Total)	0.37	0.39	0.38
Excavated volume (m^3/shift)	(Upper Half)	118	114	116
	(Lower Half)	253	224	238
	(Total)	164	154	159
Manpower (person/shift)	(Upper Half)	20.6	19.8	20.2
	(Lower Half)	24.8	32.6	28.7
	(Total)	22.0	24.5	23.2
Manpower (persons/$100m^3$)	(Upper Half)	17.5	17.4	17.4
	(Lower Half)	9.8	14.6	12.1
	(Total)	13.4	15.9	14.6

Temporary Facility for Diversion Tunnel Works

Operation	Equipment	Capacity	Quantity Inlet	Outlet
Air ventilation	Blower	37 kW x 2,100 m^3/min	2	2
	Vynil air delivery-hose	Ø 1200 mm		
Air supply	Air compressor	18.5 m^3/min	1	2
	Air supply pipe	Ø 100 mm		
Electrical supply	Generator	195 kVA	1	0
		590 kVA	1	2
Water supply	Submerged pump	3.7 kW 500 l/min	1	1
		3.7 kW 250 l/min	2	2
	Water storage tank	5 m^3	1	1

4.3.3 Concrete Lining

(1) Concrete lining method and progress

Following excavation, concrete lining work commenced on May 10, 1993 for No.1 tunnel and April 30, 1993 for No.2 tunnel, as shown in Fig. 2.4.12.

Concrete placement was made in the direction from the inlets to the outlets.

Firstly the invert concrete was placed then travelling steel forms 10.5m long were installed on the invert to place the concrete for sidewalls and ceiling of the tunnels. On September 23, 1993 concrete placement for both the tunnels was completed.

(2) Lining Type

The tunnel lining was classified into four types according to geological condition at design stage. Actually, it was confirmed that the tunnel No. 1 and No. 2 was mainly located in CH class rock. Therefore, tunnel lining type II was mainly applied as shown in the table below.

Lining Pattern	Tunnel No. 1		Tunnel No. 2		Total	
	Length (m)	(%)	Length (m)	(%)	Length (m)	(%)
I	0.0	0	0.0	0	0.0	0
II	436.8	91	381.0	90	817.8	91
III	0.0	0	0.0	0	0	0
IV	42.0	9	42.0	10	84	9
Total	478.8	100	423.0	100	901.8	100

(3) Construction performance data

Summarized below are main performance data such as equipment for tunnel lining, arrangement of concreting equipment and progress rate of concrete lining works.

Arrangement of concreting equipment is shown in Fig.2.4.13. Standard cycle time of concreting for tunnel lining was estimated at 30 hrs/cycle.

Table 2.4.4 shows the concreting progress data for the diversion tunnels. Average daily progress of lining concrete was 4.3m/day for tunnel lining and 3.7m/day for the invert.

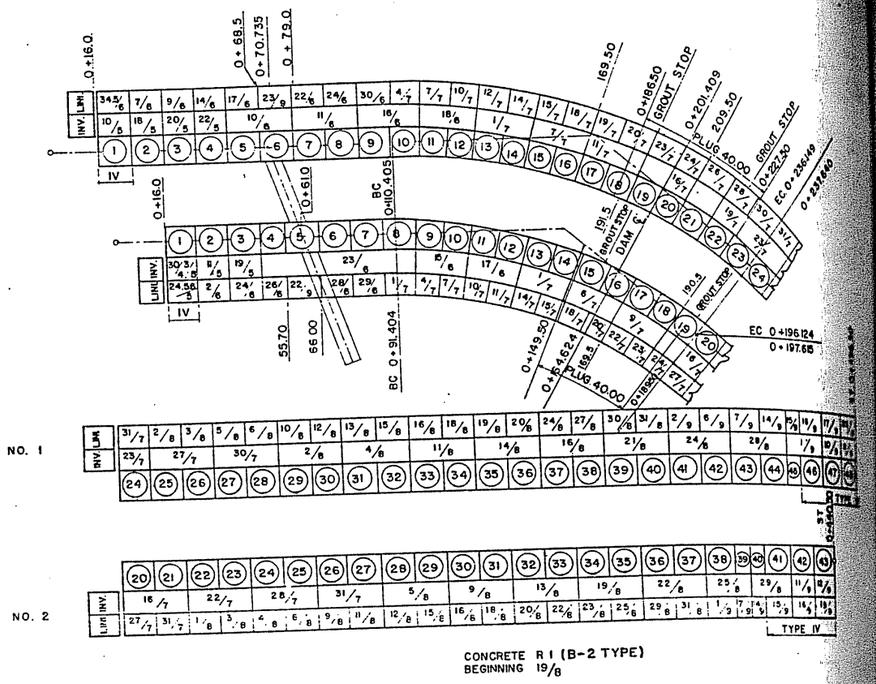
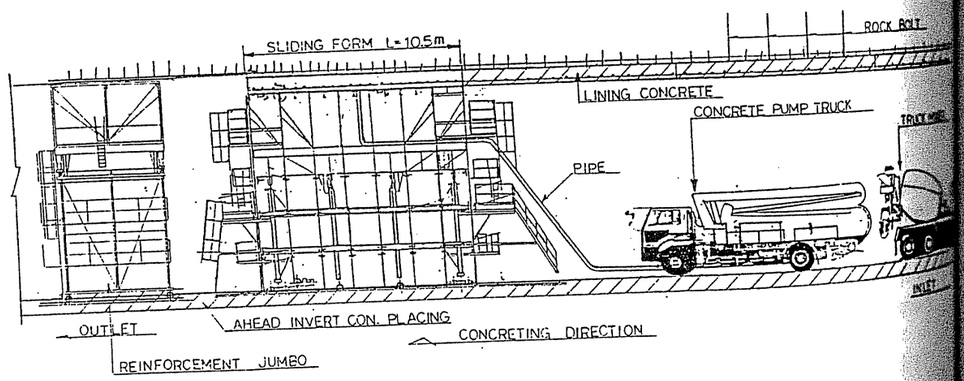


Fig. 2.4.12 Concrete Progress of Diversion Tunnels



Standard Cycle Time for Lining Concrete

Concrete placing 250 m^3 (25 m^3/h)	:	10 hrs
Curing (invert cleaning)	:	8 hrs
Form work 1. Removal	:	2 hrs
2. Transport	:	2 hrs
3. Cleaning	:	2 hrs
4. Setting (Bulk Board)	:	4 hrs
Preparation for concrete placing	:	2 hrs
1 Cycle Time		30 hrs

Fig. 2.4.13 Concrete Placing Method and Cycle Time

Main Equipment for Tunnel Lining

Operation	Equipment	Capacity	Quantity No. 1 T	Quantity No. 2 T
Installation of reinforcement bars	Reinforcement bar-jumbo	L = 6.0 m	1	1
Forming	Travelling steel form	L = 10.5 m	1	1
Concrete placing	Concrete pump truck	100 m^3/hr	2	
	Aditator truck	4.5 m^3	2	
		4.0 m^3	1	
		3.0 m^3	4	
Backfill grouting	Grout mixer	200 ltr x 2 bucket	2	
	Grout pump	7.5 kw	2	

Table 2.4.4 Progress Rate of Tunnel Lining Works

	No. 1 Tunnel Invert	No. 1 Tunnel Lining	No. 2 Tunnel Invert	No. 2 Tunnel Lining	Average Invert	Average Lining
Length (m)	478.8	478.8	423.0	423.0	901.8	901.8
Volume (m^3)	4,783	12,625	4,250	11,201	9,033	23,826
Working days	117	101	125	110	242	211
Progress rate						
(m/day)	4.1	4.7	3.4	3.8	3.7	4.3
(m^3/day)	40.9	125.0	34.0	101.8	37.3	112.9

Note : Working days includes days for reinforcement bars work, formworks, sliding, placing, curing and so on except site holidays.

(4) Adit Plug Concrete

After completion of the tunnel concreting, the junction of the diversion tunnels and the tunnel adit was backfilled with concrete and rock as shown in Fig.2.4.14.

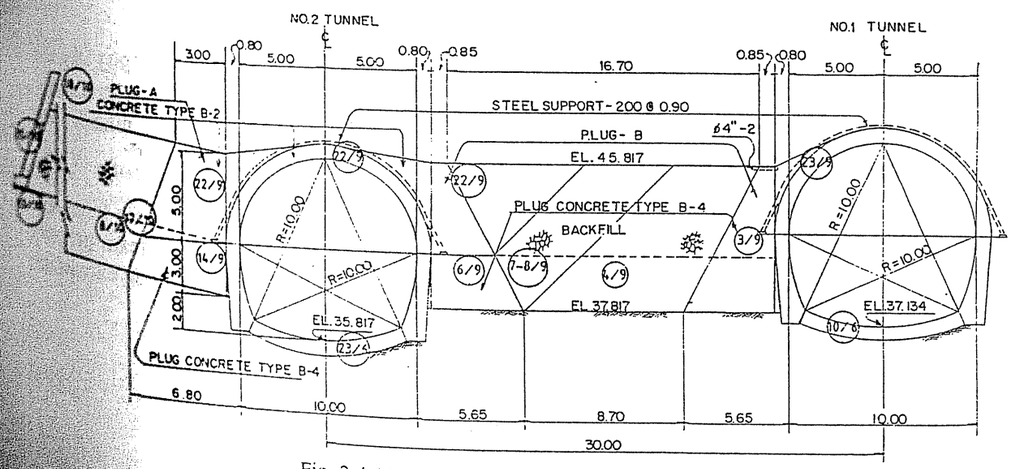


Fig. 2.4.14 Adit Plug Concrete Progress

4.3.4 Backfill Grouting

The backfill grouting was carried out by drilling through 50mm diameter steel pipes embedded in the lining concrete at the arch crown and both sides of the ceiling at 45 degrees.

Grouting started with cement grout of w/c = 1/3 under the maximum pressure 2kg/cm^2. When cement grouting exceeded 300 liter, then cement mortar was injected as shown in Table 2.4.5. A total injection of backfill grouting reached 41,250 liter with 24.8 ton of cement, for the 901.8 m long tunnels.

Table 2.4.5 Injection Data for Backfill Grouting of Diversion Tunnels

	Injection Volume of Cement Milk (l)					Injection Volume of Cement Milk (l)					Total Injection	
Hole Location	C-1	C-2	L	R	Sub Total	C-1	C-2	L	R	Sub Total	Total	Average (l/m)
Tunnel No. 1	1,712	4,714	1,184	1,476	8,545	1,875	17,275	400	200	19,750	28,295	
Tunnel No. 2	1,200	3,580	900	1,100	6,780	250	5,050	200	675	6,175	12,955	
Total	15,325					25,925					41,250	

Mix Proportion of Cement Milk and Mortar Grout

Material / Type	Water (kg)	Cement (kg)	Sand (kg)	W/C (by weight)	(by volume)
Cement Grout	800	614	-	1.30	4
Mortar Grout	417	596	1010	0.70	2

4.4 Execution of River Diversion

The Kampar Kanan river was diverted through the tunnels on October 1, 1993. By end of September, 1993, concrete placing in the diversion tunnels and inlet struc backfill grouting of the tunnels, plugging of the temporary adit tunnel construction of the retaining wall at the inlet and outlet were completed.

River flow was diverted by blasting and removal of cofferdams around the inle outlets. Immediately after opening of the diversion tunnels, the upstream p cofferdam was constructed across the river from the left bank by dumping exca spoil into the flowing river.

The river was closed off after six hours of embankment construction for the primary cofferdam, and all the river flow was then diverted through the tunnels. The downstream primary cofferdam was constructed in a similar way on October 2, 1993. Dewatering of the pond between the two primary cofferdams then started immediately.

4.5 Cofferdams

Cofferdam work consisted of construction of the primary and secondary cofferdams both upstream and downstream of the dam site, as shown in Fig. 2.4.15. Main features of each cofferdam are shown in the table below.

Main Features of Cofferdams

Item	Upstream Cofferdams		Downstream Cofferdams	
	Primary	Secondary	Primary	Secondary
Type	Rockfill	RCC Dam	Rockfill	Rockfill with center core
Crest EL. Embankment Volume	EL. 51.5 18,698m^3	EL. 51.5 14,286m^3	EL. 46.0 31,459m^3	EL. 45.3 14,393m^3

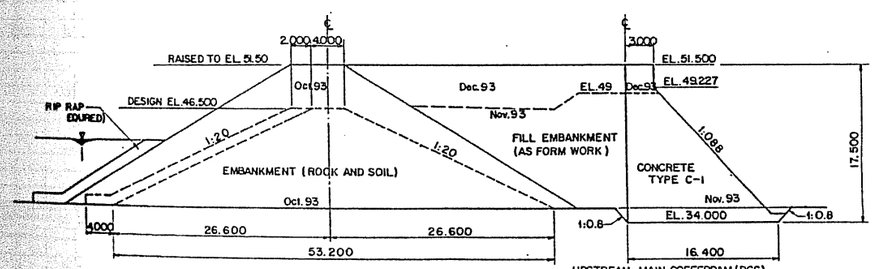


Fig. 2.4.15 Upstream and Downstream Cofferdam

4.5.1 Upstream Cofferdam

Construction of the primary cofferdam commenced on October 1, 1993 at the time of the river diversion. After embankment work reached to EL. 46.50 in design elevation, the crest elevation of the primary cofferdam was modified and risen up to EL. 51.50 by the Contractor to give the access to the secondary cofferdam to transport concrete material and reduce the risk of the overtopping during construction period of the secondary cofferdam. Embankment work comprised 18,698m^3 up to EL. 51.5 to divert the river flow through the diversion tunnels.

Following completion of the primary cofferdam, construction of the secondary cofferdam commenced and excavation was completed on October 27, 1993. Then concrete work commenced. Roller compacted concrete (R.C.C) type gravity dam was adopted for the secondary cofferdam to reduce the risk of overtopping during construction and damage to the cofferdam in the rainy season.

Concrete work for the secondary cofferdam commenced on October 27, 1993 and was completed on December 1, 1993, the concrete volume comprised 14,286m^3 up to EL.51.5. The progress is shown in Fig. 2.4.16.

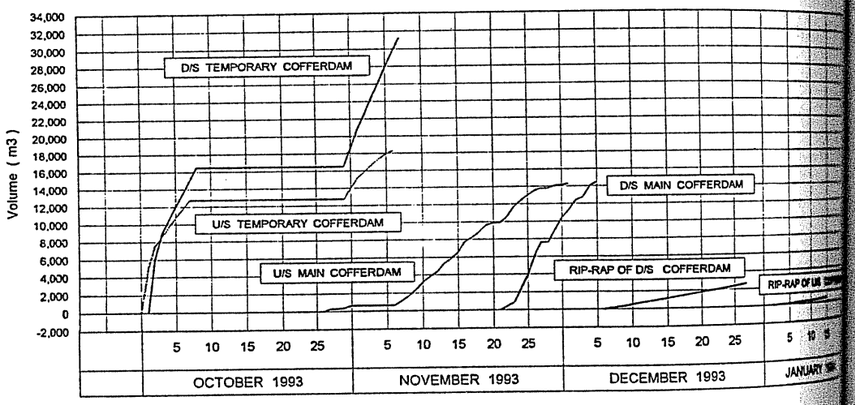


Fig. 2.4.16 Cofferdam Progress

Preliminary tests for concrete placement of roller compacted concrete were carried out by using a 10 ton vibration roller to determine the concrete mix proportion and the method of concrete placement as shown below.

Design Mix of Roller Compacted Concrete

Item	Specification	Item	Specification	Item	Specification
σck	70 kg/cm^2	W/C	87.5%	Sand	755 kg
σr	95 kg/cm^2	s/a	34%	G1 : 5-20	869 kg
Gmax	40 mm	Water	105 kg	G2 : 20-40	582 kg
VC	40-150 sec	Cement	120 kg		

Three passes of the vibration roller (10t class) produced RCC with 94% compaction. Thus for actual construction of the upstream secondary cofferdam, it was decided that RCC will be compacted to achieve 95% degree of compaction using vibration roller by 1 pass with no vibration and 3 passes with vibration. The degree of compaction was evaluated as following equation.

$$\text{Degree of compaction} = \frac{\text{in-site density by RI}}{\text{calculated density}} \times 100\ (\%)$$

The concrete placement was carried out continuously 24 hours each day by two shifts. An average of three layers were placed in a day.

The following criteria was applied to ensure binding of layers in accordance with ACI 207.5R - 89.X. {AST x TE ≤ 1,800, where AST: Average Surface Temperature (^{0}F), TE:Time of Exposure (hr)}.

No surface treatment of the previous layer was applied except for curing if the above condition was fulfilled. Mortar was placed on the previous layer for resumption of work . In most cases, the next layers were placed within 1,000 of AST x TE.

The actual site work was carried out as follows :

(a) Mixing of Concrete

The main dam batching plant located in the left bank of the dam axis was used. The batching plant capacity is 1.5 m^3 x 3 units of "front charge mixer of pneumatic tilting type" and mixing time was set at 90 secs.

(b) Foundation Surface Treatment

Prior to placing the RCC material, all the rock surface was covered with mortar. At the bottom of the river, the deep river deposits were replaced with conventional concrete.

(c) Transportation of Concrete

The dump trucks were used to transport the RCC material to the site. The access was constructed from the downstream to EL. 40.0 and from the upstream up to the crest elevation. Dump truck tires were cleaned with a pressure water jet prior to entering the cofferdam area.

(d) Spreading of Concrete

The dumped concrete was spread to a depth of 30 cm by using a backhoe. steel plate was welded to the bucket of the backhoe for effective spreading.

(e) Compaction of Concrete

The 10-ton vibration roller was used and compaction was carried out on each layer. One pass of compaction was done without vibration and three passes of compaction with vibration. The downstream edge and both banks were compacted by manual plate tamper.

The upstream side of the secondary cofferdam was backfilled with the embankment which was used for the form work of RCC. The downstream slope was compacted by the backhoe.

4.5.2 Downstream Cofferdam

Construction of the primary cofferdam commenced on September 25, 1993, just before the river diversion was made place. A total volume 31,459 m^3 of embankment was completed up to EL. 46.00 by the mid November, 1993 as shown in Fig. 2.4.14. The main sources of embankment materials were of excavation from the dam, tailrace and powerhouse.

The secondary cofferdam was constructed just upstream of the primary cofferdam which was incorporated in the secondary cofferdam as shown in the Fig. 2.4.15.

Construction of the secondary cofferdam commenced on November 21, 1993 and was completed on December 5, 1993. The total volume of impervious and pervious zones reached 14,393 m^3 up to EL. 45.30. The material used for the impervious zone was shown in Table 2.4.6.

Table 2.4.6 Material for Impervious Zone

Particle Size	Passing by Weight (%)
4.8 mm	50-100
0.074 mm	20-80

Embankment material was compacted in layers of 30 cm by 4 passes of a 10 ton class vibration roller. Excavated material of the dam foundation was used for the pervious zone compacted in layers of 1.0 m by 2 passes of D-7 class Bulldozer followed by 4 passes of 10 ton class vibration roller.

4.5.3 Floods During Construction

Several floods were observed during operation of the diversion tunnels from October 1993 to March 1997. Among these floods, the floods in February 1995 and February 1996 overtopped the upstream cofferdam and flooded the work area upstream of the dam.

In February 1995, the concrete placement of the dam reached up to EL. 53.5 which was the same elevation as the upstream cofferdam.

On 2nd February 1995, heavy rain was observed in the project area. The river water level gradually increased, then later the water level at the inlet reached EL. 53.9 at around noon and the river water started overtopping at the upstream cofferdam.

In the upstream area, the temporary equipment such as the water supply system for concrete production and the grout plant for foundation treatment were inundated. It took about one month to restore the normal site condition. The stilling basin, the powerhouse, the tailrace site, the main dam and the downstream cofferdam existed without overtopping of the river water, therefore there was no serious damage in the downstream area.

The largest flood during the construction occurred on 4th February 1996. The maximum discharge recorded 2140 m^3/s and the water level exceeded the upstream cofferdam level. Although the flood caused inundation in the upstream of the dam where water supply pipe lines and grout plant for foundation treatment were located, concrete placement had already reached the spillway crest elevation EL. 64.5, therefore no serious damage occurred for the dam and the powerhouse work.

4.5.4 Construction Performance Data

The summaried tables below are the main performance data such as equipments for this work and the progress rate of cofferdam construction.

Main Equipment for Cofferdam

Operation	Equipment	Capacity	Quantity
[Upstream Primary Cofferdam]			
Loading	Wheel Loader	3.5 m^3	2
	Excavator	1.3 m^3	2
Hauling	Dump Truck	11 ton	8
Spreading and Levelling	bulldozer	45 ton	2
		32 ton	2
Compacting	Vibration Roller	10 ton	1
[Downstream Primary and Secondary Cofferdam]			
Loading	Wheel Loader	3.5 m^3	1
	Excavator	1.3 m^3	1
Hauling	Dump Truck	11 ton	6
Spreading and Levelling	Bulldozer	45 ton	1
		32 ton	1
Compacting	Vibration Roller	10 ton	1
Rip Rapping	Excavator	1.3 m^3	1
[Upstream Secondary Cofferdam]			
Excavation & Loading	Excavator	1.3 m^3	2
		0.7 m^3	1
Hauling	Dump Truck	11 ton	8
Mixing	Dam Batching Plant	90 m^3/hr	1
Transportation	Dump Truck	11 ton	4
Spreading & Levelling	Excavator	0.7 m^3	1
	Bulldozer	32 ton	1
Compacting	Vibration Roller	10 ton	1

Progress rate of Cofferdam embankment

	Upstream		Downstream	
	Primary	Secondary	Primary	Secondary
Volume (m^3)	18,698	14,286	31,459	14,393
Working days	62	24	119	72
Progress rate (m^3/day)	302	595	264	200

4.6 Plugging of Diversion Tunnels

On 12 March 1997, the closure gates were installed at the inlets of the diversion tunnels to close the diversion tunnels and to commence impounding of the reservoir. After both tunnels were closed, a temporary coffering at the outlet of the tunnels was constructed for dewatering and to provide access for further works, plug concrete placement and grouting work in the tunnels.

On 6 October 1997, the final inspection of the plug work was conducted and it was confirmed that grouting work was completed. The Contractor commenced removal of the coffering and backfilled the outlet area.

4.6.1 Closure Gate Installation

Installation of the diversion closure gate was required to commence impounding of the reservoir. In consideration of the work progress of other Lot Contractor's work and the timing of the lowering and closing of the closure gate of the diversion tunnels in the rainy season, the start date of impounding was planned at the beginning of March 1997. Lowering of each gate was carried out as follows.

(a) Sequence of Gate Closure

The number of the gates are four. Namely, two sets each for the two diversion tunnels. The sill elevation of the Tunnel No. 1 is 1.5 metre higher than that of the tunnel No. 2.

From the condition of the closure gates mentioned above, it would be very difficult to lower all gates at the same lowering speed because there would be some difference from each other in the friction between grooves and the gate and in the hydraulic pressure acting on the gate.

Therefore, tunnel No. 2 was first closed and then tunnel No. 1 gate was closed

(b) Condition of Lowering of Gate

The conditions for lowering the gates were examined on the basis of design conditions below. It was planned that the gates would be lowered by their own weight against frictional resistance and buoyancy caused by the water level upto EL. 40.6, when river discharge was 200 m^3/s. Then it would be required to superimpose additional weight on the gate body when the water level would be higher than EL. 40.6. Therefore, the lowering of the gate was planned depending on the water level as follows.

(i) In the case of WL ≤ EL. 40.6, the lowering gate would be carried out without superimposed weight.

(ii) In the case of EL. 40.6 < WL ≤ EL. 41.5 (the design water level, river discharge was 280 m^3/s), the lowering gate would be carried out after additional weight added to gate body.

(iii) In the case of WL > EL. 41.5, the lowering should be postponed.

The actual lowering of the gate was carried out successfully with no additional weight on 12 March 1997 under the condition that water level was at EL. 39.8 and river discharge 100 m^3/s.

Weight	:	36 ton/gate
Dimensions	:	W5.8 m x H10.6 m x T0.7 m
Design Water Elevation	:	Front : EL. 85.000 m
		Rear : EL. 36.000 m
Closing Water Elevation	:	Front : EL. 41.500 m
		Rear : EL. 36.000 m
Coefficient of Friction	:	μ = 0.4 (metal v.s metal)

(c) Lowering of Gate Assemblies

(i) Test Installation

A trial of the diversion closure gate installation was carried out from September 1996 to March 1997 to confirm the function of gate assemblies gantries and gate grooves.

Elevations were marked on the gate grooves to check the closure condition of the gate, when the gate was installed.

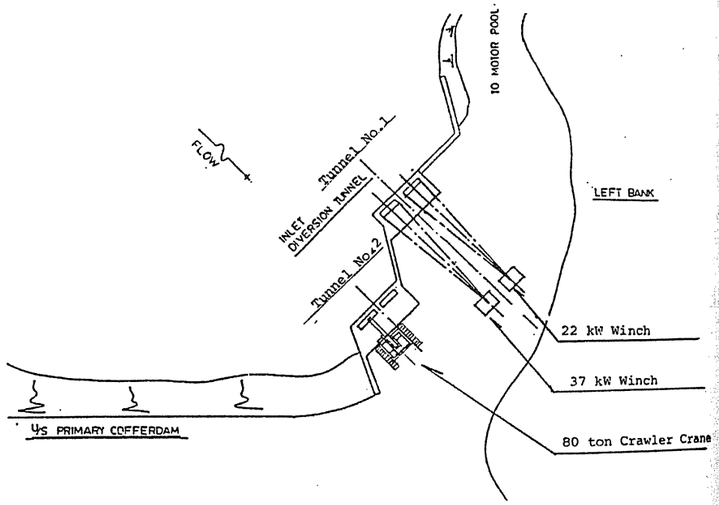
(ii) Lowering of Tunnel No. 2 Gate Assemblies

Just before lowering each gate assembly, possible deposit accumulating in the grooves was removed by using compressed air after lowering stoplogs in the inlet. Driftwood was also removed.

The installation of the closure gate commenced at 8:50 AM on 12 March 1997. The water level was at EL. 39.8 corresponding to 100 m^3/s discharge. Each 36 ton gate assembly was lowered by using a 80 ton crawler crane. Both gate assemblies were lowered one by one and the two gate installations were completed at 9:15 AM.

(iii) Lowering of Tunnel No. 1 Gate Assemblies

Following completion of the No. 2 tunnel, both gate assemblies were lowered simultaneously by a separate 22 kW winch and a 37 kW winch at 9:30 AM the same day. The No. 1 gate installation was completed at 9:40 AM. After confirmation of gate installation, the crawler crane and the winches were removed from the site.



Equipment Layout for Diversion Tunnel Closure

4.6.2 Plug Concrete

After completion of the downstream coffering in the outlets of the diversion tunnels, the temporary plug concrete 10m long was placed to secure the main plug work, as shown in Fig. 2.4.17. In the temporary plug, one drain pipe, Ø150mm, was embedded in lift-2 to divert upstream water to the downstream during the temporary plug concrete placing. Afterwards the drain pipe was closed by cock and cement mill.

Before concrete placing of the temporary plug, anchor bars of 25 mm in diameter 2m long were installed on the bottom of the diversion tunnel to obtain adequate shear strength between the tunnel lining concrete and the plug concrete.

Concrete volume of each temporary plug was 829 m^3, which was placed in five (5) lifts.

The main plug consists of three blocks each with five lifts of concrete. The grouting gallery is located in the lift 2 concrete. From the gallery, embedded pipes for consolidation and curtain grouting were installed to protect other embedded pipes, such as cooling and contact grouting pipes against cutting off by drilling.

Before placing the main plug concrete, chipping and washing of the tunnel lining concrete surface were carried out carefully to obtain adequate shear strength and bond strength between the tunnel lining concrete and the plug concrete. Split drain pipes were also installed to drain leakage water from the joints of the lining concrete.

Concrete placing was basically carried out 10m long by one lift, however, for the final lift located in the top of the plug concrete, the concrete placing was done in 3 stages each 3.3m long in one lift which was placed separately by temporary form made by steel mesh in order to avoid the void caused by not filling concrete due to lack of sufficient working space for concrete placement.

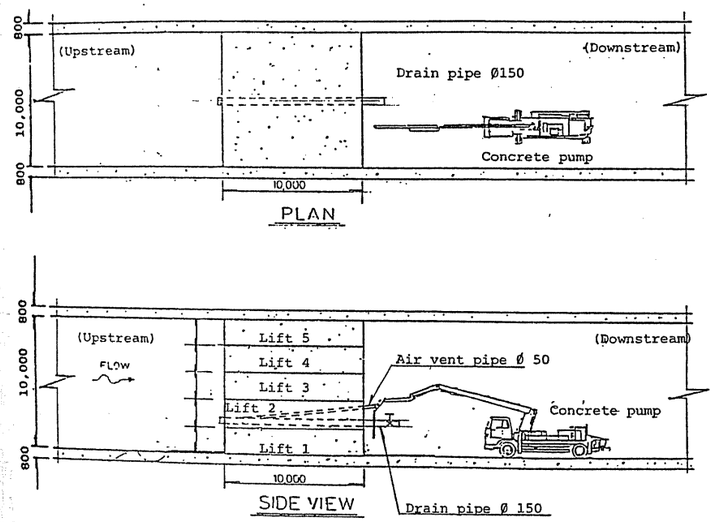


Fig. 2.4.17 Equipment Layout for Plug Concrete Work

Circulation of cooling water, temperature 16°C with discharge velocity 16 liter/min, commenced soon after concrete was placed. Cooling of concrete continued until the concrete temperature decreased to the range of 27-28°C which is the groundwater temperature in the tunnel.

4.6.3 Backfill grouting

(1) General

After completion of the main plug concrete, the backfill grouting works commenced on 28 July 1997. Backfill grouting was carried out to inject the possible voids between lining and the plug concrete. Grouting took place after the last lift of plug concrete was placed as shown in Fig. 2.4.18. The total quantity of the injection is shown in the table below.

Quantity of Backfill Grouting

Location	Volume (liter)	Cement (kg)
Tunnel No. 1	11,720	8,050
Tunnel No. 2	18,108	11,244

(2) Grouting Method

Grouting was injected from the grout supply pipe at pressure 2 kg/cm². The concentration of cement grout (w/c = 4) was injected primarily up to 300 liter. When the total volume exceeded 300 liter, the mortar grouting (c/s = 1) was injected as shown in table below.

Mix Proportion for Backfill Grouting

	Cement (kg)	Sand (kg)	Water (ltr)	Volume (ltr)
Cement Grout (w/c=4)	200	-	260	325
Mortar Grout (c/s=1)	513	434	666	1000

Note : specific gravity of cement and sand is 3.07 and 2.6 respectively.

(3) Grouting Result

At the early stages of the backfill grouting leakage of water was found from various embedded pipes, such as temporary drain pipes at the concrete lining joints as well as the embedded pipes for curtain and consolidation grouting. During injection, leakage of cement grout was observed from the downstream face of the plug concrete.

Additional backfill grouting was carried out from the embedded pipes with leakage water, since it was assumed that there were voids or gaps between concrete lining and plug concrete. After injection of backfill grouting, these leakages virtually ceased.

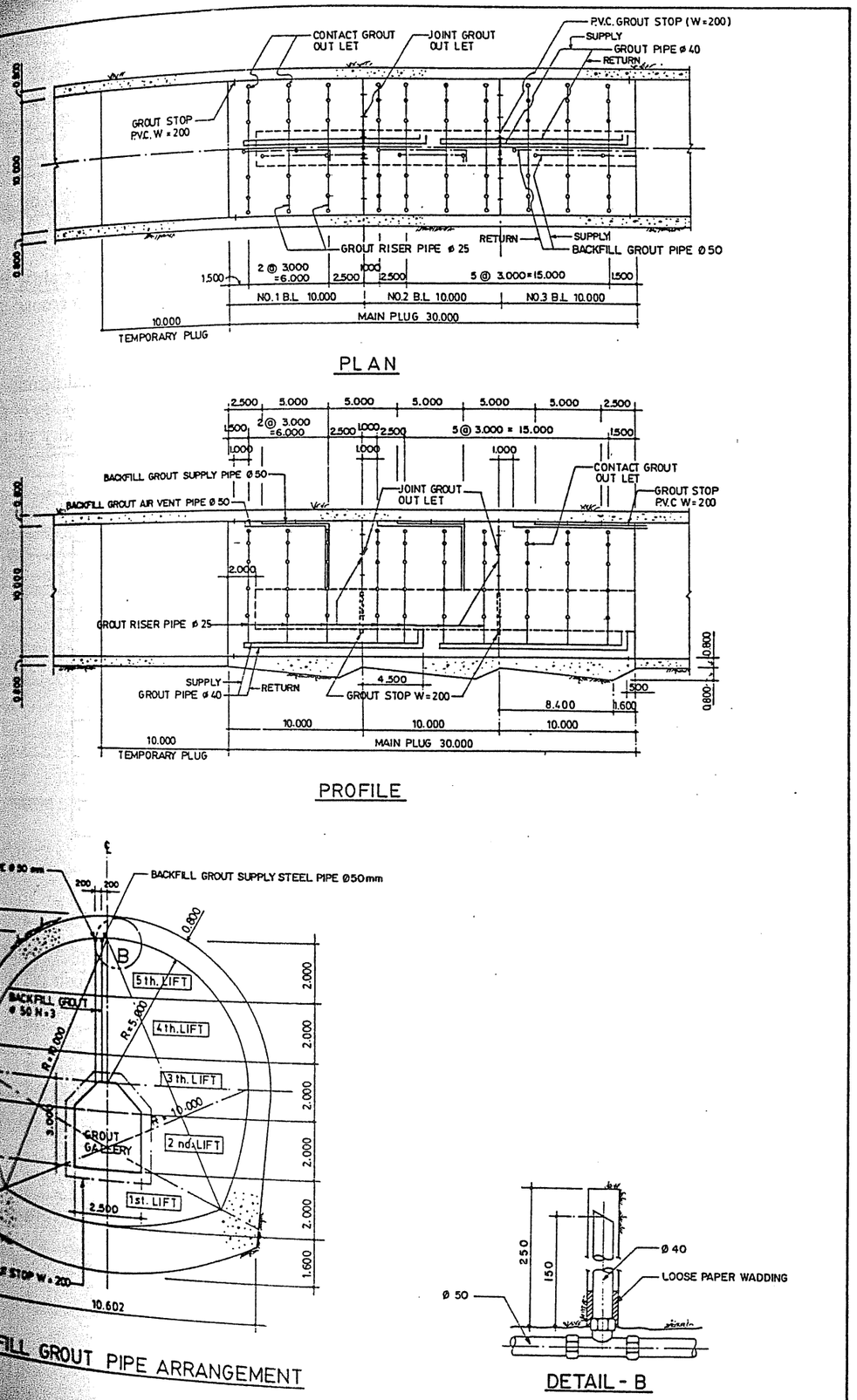


Fig. 2.4.18 Backfill Grout Pipe Arrangement

4.6.4 Consolidation Grouting

(1) General

After completion of backfill grouting, consolidation grouting for the diversion tunnels started on 2 August 1997 in tunnel No. 1 and on 29 July 1997 in tunnel No. 2.

Design of consolidation grouting was planned to place nine (9) sections along the grout gallery, each section with 8 holes high for every 45 degrees direction around the tunnel, as shown in Fig. 2.4.19.

Grouting for the design holes was completed in the end of August, and additional holes were carried out to check the high permeability areas. A total 39 holes of the check holes were injected by the end of September. The final quantity of the consolidation grouting is shown in the table below.

Quantity of Consolidation Grouting

Location	Stage	Drilling		Average Lugeon (Lu)	Cement Injection (kg/m)
		Concrete (m)	Rock (m)		
Tunnel No. 1					
Design Holes	72	143.2	216.0	17.1	84.3
Check Holes	22	106.8	68.7	13.4	28.3
Tunnel No. 2					
Design Holes	72	126.5	216.0	23.4	60.8
Check Holes	17	83.8	58.3	12.6	29.0
Total	183	460.3	559.9	18.7	65.2

(2) Grouting Method

Drilling and grouting were done from the grouting gallery. In the downstream area of each tunnel plug, grout pumps and mixers were installed. The main items of equipment are shown in the table below.

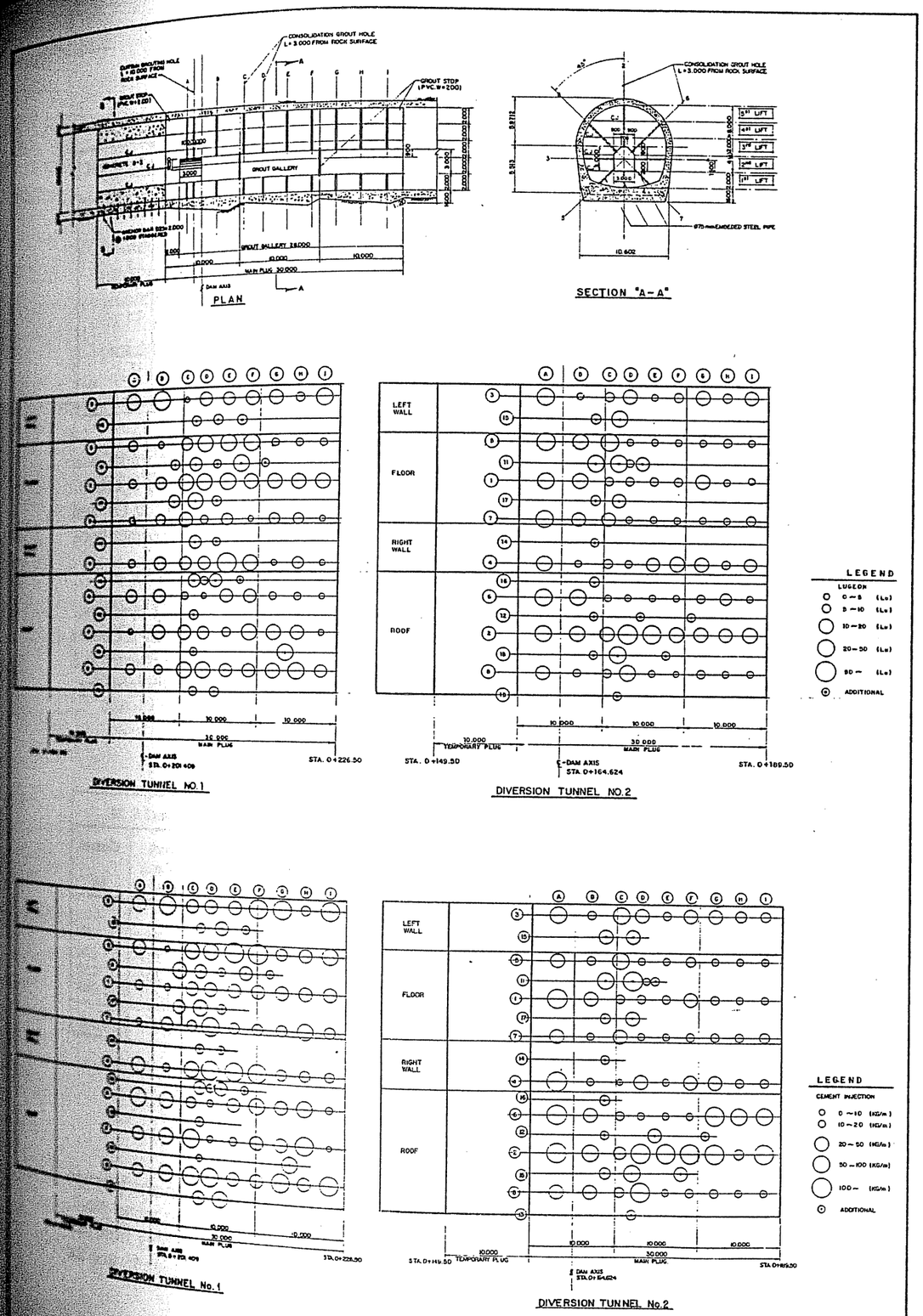


Fig. 2.4.19 Consolidation Grouting Results for Diversion Tunnel

Equipment List

Purpose	Name	Manufacturer / Model	Capacity / Spec.	Number	Power (kw)
Boring	Boring Machine	TONE : UD - 5	150	5	5.5
	Boring Machine	KOKEN : OP-1	150	3	5.5
	Boring Machine	KOKEN : OE-2L	150	3	5.5
	Boring Pump	TONE : NAS - 2	150	5	2.7
	Core barrel		ϕ 56 Single tube	7	
	Packer		ϕ 56 Expansion packer	10	
Plant	Grout Pump	KOKEN : MG - 1	Volume 40, 68, 105 ltr/m Pressure 70, 50, 30 kgf / cm^2	6	7.5
	Grout Mixer	KOKEN : LM-250	Agitating cap. 200 ltrs x 2	6	2.2
Plant	Grout Pump	KOKEN : MG - 1	Volume 55, 100, 160 ltr/m Pressure 70, 40, 25 kgf/cm^2	2	11
	Grout Mixer	KOKEN : HM - 2	Agitating cap.200 ltrs	2	5.5

Grouting pressure was 5 kg/cm^2. Mix proportion changed from the initial w/c according to the result of initial permeability by water pressure test before injection. Initial mix proportion was w/c= 8 when Lugeon value is less than 20 Lu, and w/c = 6 when Lugeon value exceeds 20 Lugeon. The mix proportion becomes thick according to the injected volume as shown in the table below.

The maximum injection volume was set at 3000 liter for one grouting operation. When injection exceeded 3000 liter, then regrouting was carried out after six hours from the first injection. Grouting was completed when injection rate reached 0.2 liter/meter/minute for 30 minutes. After completion of consolidation grouting, grouting holes were closed with grout mix w/c = 0.5.

Mix Proportions for Consolidation Grouting

Mix (By Weight) W/C	Cement kg	Water Liter	Batch Volume Liter	Less Than Lu. 20			More Than Lu. 20		
				No. Of Batches	Batch Volume Liter	Cement kg	No. Of Batches	Batch Volume Liter	Cement kg
8	20	160	166	3	498	60	-	-	-
6	30	180	190	3	570	90	3	570	90
4	40	160	173	3	519	120	3	519	120
2	80	160	186	3	558	240	3	558	240
1	120	120	159	5	795	600	9	1,431	1,080
TOTAL				17	2,940	1.110	18	3,078	1,530

(3) Result of Grouting

(a) Tunnel No. 1

After completion of grouting up the design holes, the average Lugeon and injection of each grouting section result is shown in Fig.2.4.19 and 2.4.20. In the tunnel No.1, Lugeon and cement injection were high in the central part of the main plug from line C to line F. The highest average Lugeon, 24.3 Lu was found in the Line E, and the

highest average injection 128.6 kg/m occurred in the Line D. Additional check holes were carried out for the locations (i) Lugeon more than 10 Lu. (adopted from the dam consolidation grouting), (ii) cement injection more than 50 kg/m. (adopted from the dam consolidation grouting) and (iii) holes with seepage.

In order to check these areas, additional check holes were drilled and a total of 22 holes were injected. Although some holes still had permeability more than 10 Lu, the secondary check holes were placed to confirm that low permeability was achieved. The final condition of the permeability is shown in Fig. 2.4.21 which indicates the probability equaled or exceeded 10 Lu is less than 15%.

(b) Tunnel No. 2

After completion of grouting up the design holes, high Lugeon and injection holes were found in the upstream side of the grout lines, such as A, B and D, and the top part of the tunnel from upstream to downstream direction, as shown in Fig. 2.4.19 and 2.4.20. Therefore in these areas, additional check holes were grouted.

The top part of the tunnel was improved by the design hole except one hole which was confirmed by secondary check holes to be less than 5 Lugeon. The lower part of the tunnel required several additional holes to confirm the final Lugeon was less than 10 Lu. The total number of the additional check holes required was 17 holes. The final condition of the permeability is shown in Fig.2.4.21 which indicates the probability equaled or exceeded 10 Lu is less than 15%.

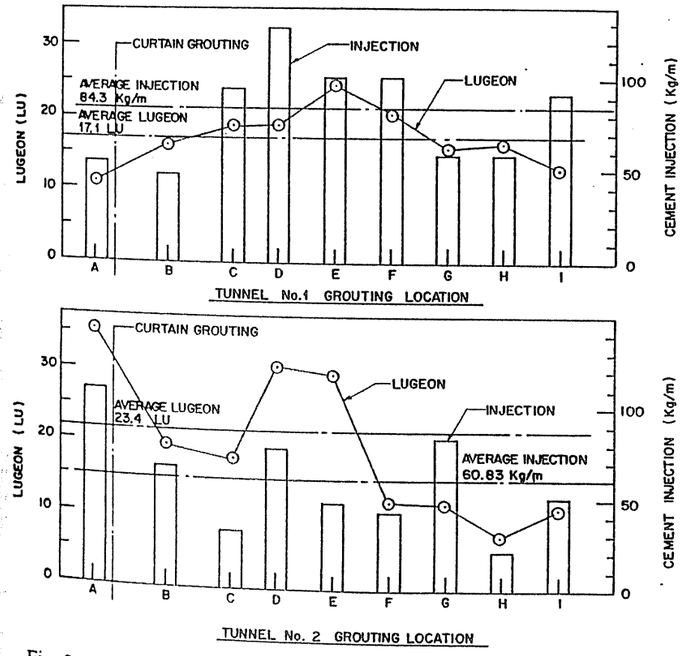
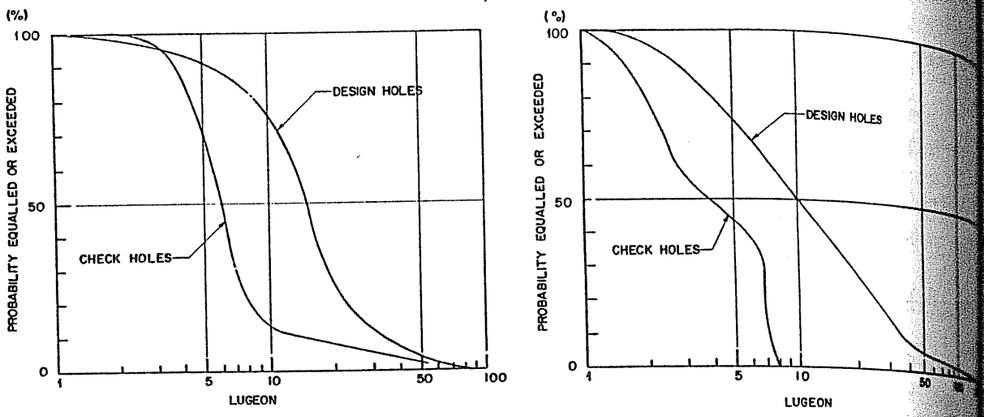


Fig. 2.4.20 Average Lugeon and Injection for Consolidation Grouting



Tunnel No. 1 Tunnel No. 2

Fig. 2.4.21 Lugeon Probability Equalled or Exceeded

4.6.5 Curtain Grouting

(1) General

Curtain grouting was commenced after consolidation grouting the design holes and was completed on 14 August 1997 in the tunnel No. 2 and on 25 August 1997 in the Tunnel No. 1. The curtain grouting line is located in the same line as the dam curtain grouting. A total of 36 design holes were drilled at every 10 degrees around the tunnel. The quantity of the curtain grouting is shown in the table below.

Quantity of Curtain Grouting

	Location	Stage	Drilling (m)		Average Lugeon	Cement Injection (kg/m)
			Concrete	Rock		
Tunnel No. 1	Primary	16	33.4	80.0	10.9	60.8
	Secondary	16	36.8	80.0	8.5	20.9
	Tertiary	40	93.7	200.0	5.9	24.3
	Additional	26	63.2	130.0	3.9	13.8
	Sub-total	98	227.1	490.0	6.6	26.9
Tunnel No. 2	Primary	16	33.4	80.0	9.2	38.14
	Secondary	16	36.3	80.0	12.3	30.76
	Tertiary	40	90.35	200.0	8.7	30.11
	Additional	28	78.95	140.0	3.6	32.13
	Sub-total	100	239.0	500.0	7.9	32.1
	TOTAL	198	466.1	990.0	-	-

(2) Grouting Method for Curtain Grouting

(a) Grouting Equipment

Equipment for curtain grouting was the same equipment as used for the consolidation grouting.

(b) Grouting Pressure and Mix Proportion

Grouting pressure was adopted to be 25 kg/cm^2, and mix proportion started from w/c = 10 for Lugeon less than 10 Lu and w/c = 8 for Lugeon more than 10 Lugeon as shown in the table below.

Mix (By Weight) W/C	Cement kg	Water Liter	Batch Volume Liter	Less Than Lu. 20			More Than Lu. 20		
				No. Of Batches	Batch Volume Liter	Cement kg	No.of Batches	Batch Volume Liter	Cement kg
10	10	100	103	6	618	60	-	-	-
8	20	160	166	3	498	60	3	498	60
6	30	180	190	3	570	90	3	570	90
4	40	160	173	3	519	120	3	519	120
2	80	160	186	3	558	240	3	558	240
1	120	120	159	5	795	600	9	1,431	1,080
TOTAL				23	3,558	1,170	21	3,576	1,590

(c) Permeability Test

Permeability tests were carried out at the primary holes to check the initial condition of the rock around the plug concrete.

(3) Result of Curtain Grouting

(a) Tunnel No. 1

High permeability and injection holes occurred mainly in the lower half of the tunnel. Especially in the lower half of the left hand side of the tunnel, several check holes were required to confirm that permeability in this area was improved to be less than 5 Lugeon, as shown in Fig. 2.4.22.

Additional check holes were carried out in the area having the following grouting data (i) tertiary holes more than 5 Lu with 100 kg/m or more injection, (ii) tertiary holes more than 10 Lugeon and (iii) tertiary holes more than 5 Lugeon and deems necessary to achieve final criteria; probability equaled or exceeded 5 Lugeon is less than 15%.

Generally, the Lugeon value decreased from the primary holes to check holes as the sequence of grouting proceeded, as shown in Fig. 2.4.23. Grouting injection was high only for the primary holes, 60.8 kg/m, then reduced to be 13.8 kg/m for the check holes.

Improvement of Lugeon is also obvious from the probability of Lugeon value as shown in the Fig.2.4.24. Final value of probability achieved equalled or exceeded 5 Lu is less than 15%.

(b) Tunnel No. 2

The high Lugeon of the tertiary holes were found in several directions such as top, right hand, lower right and left directions. In these areas, a total 16 check holes were drilled and injected as shown in Fig. 2.4.22.

Most of the high Lugeon holes of the tertiary holes were confirmed to be less than 5 Lugeon.

Average value of Lugeon decreased from the secondary to the check holes as the grouting sequence proceeded, as shown in Fig. 2.4.23.

The probability curve of the check holes, Fig. 2.4.24, shows improvement of permeability, and probability achieved equalled or exceeded 5 Lugeon less than 15%.

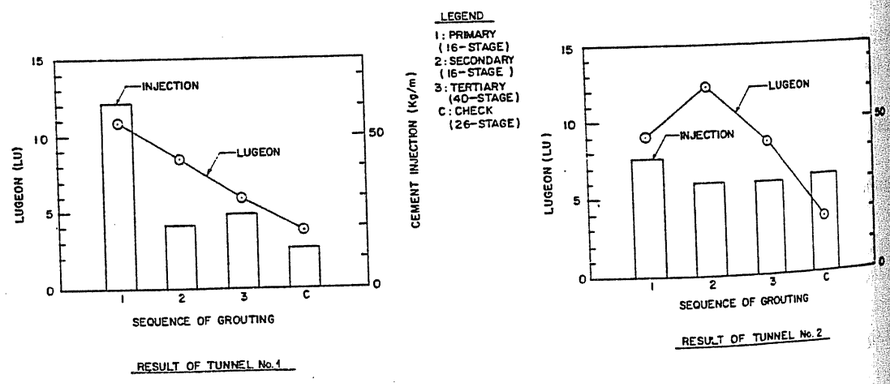


Fig. 2.4.23 Average Lugeon and Injection for Curtain Grouting

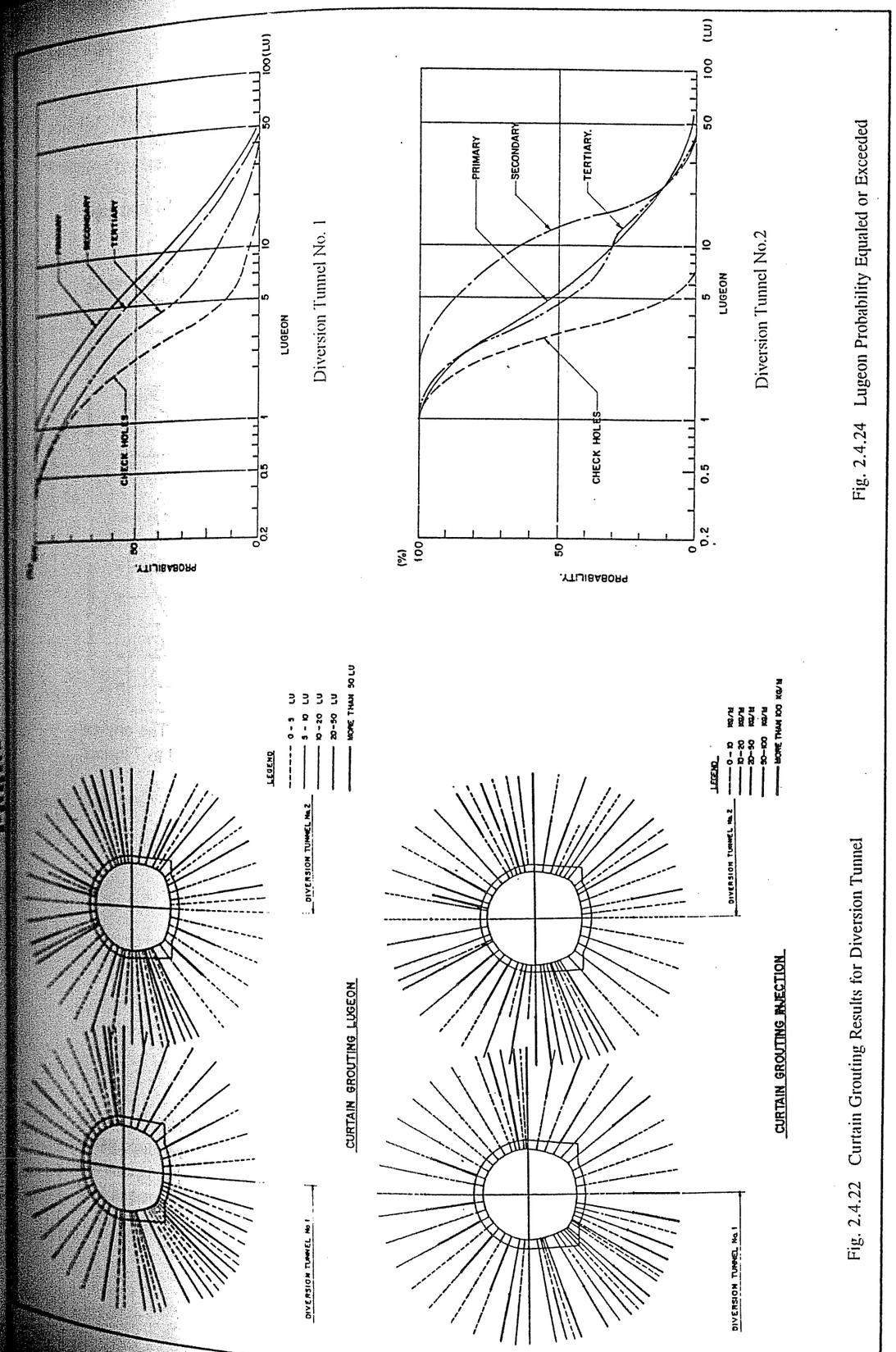


Fig. 2.4.24 Lugeon Probability Equaled or Exceeded

Fig. 2.4.22 Curtain Grouting Results for Diversion Tunnel

4.6.6 Contact Grouting

(1) General

Contact grouting was carried out to fill the gaps between concrete lifts and gaps between lining and plug concrete.

Injection was carried out after the temperature of the plug concrete reduced to a stable temperature, approximately 27°C, as shown in Fig. 2.4.25.

Grouting work was conducted in mid September 1997.

Total Quantity of Contact Grouting

Location	Injected Volume	Remarks	
Tunnel No. 1	330 liter	BL 2-3 BL 1-2	214 liter 116 liter
Tunnel No. 2	2000 liter	BL 2-3 BL 1-2	1945 liter 56 liter

(2) Grouting Method

Pressure and Mix Proportion

Grout was injected through embedded pipes as shown in Fig. 2.4.26. The original pressure of injection was 5 kg/cm², however the pressure was increased to 7 kg/cm² to fill gaps more efficiently.

Mix Proportion

Mix w/c	Cement (kg)	Water (liter)	Batch Volume (liter)
8	20	160	166
6	30	180	190
4	40	160	173
2	80	160	186
1	120	120	159

(3) Result of injection

The injection volume was generally low except BL 2-3 in Tunnel No. 2. During injection of backfill grouting, leakage of grout was found from the embedded pipe, therefore it was assumed that a large void between concrete lining and plug concrete was already filled by backfill grouting. Only remaining small gaps between plug concrete was filled by contact grouting, therefore injection volume was not large.

After injection, only a wetted surface of the concrete joint was observed and no running water was found from the perimeter of plug concrete and gaps between the concrete lifts.

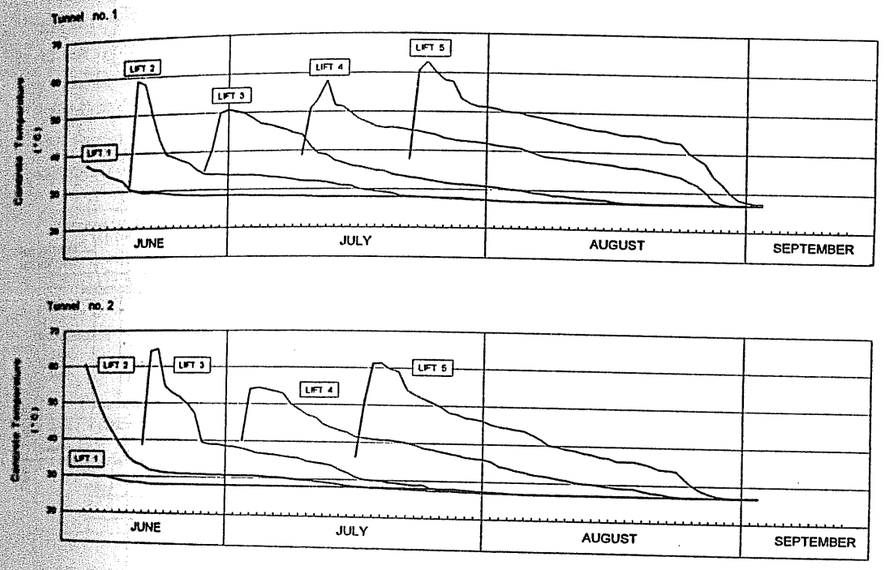


Fig. 2.4.25 Plug Concrete Temperature

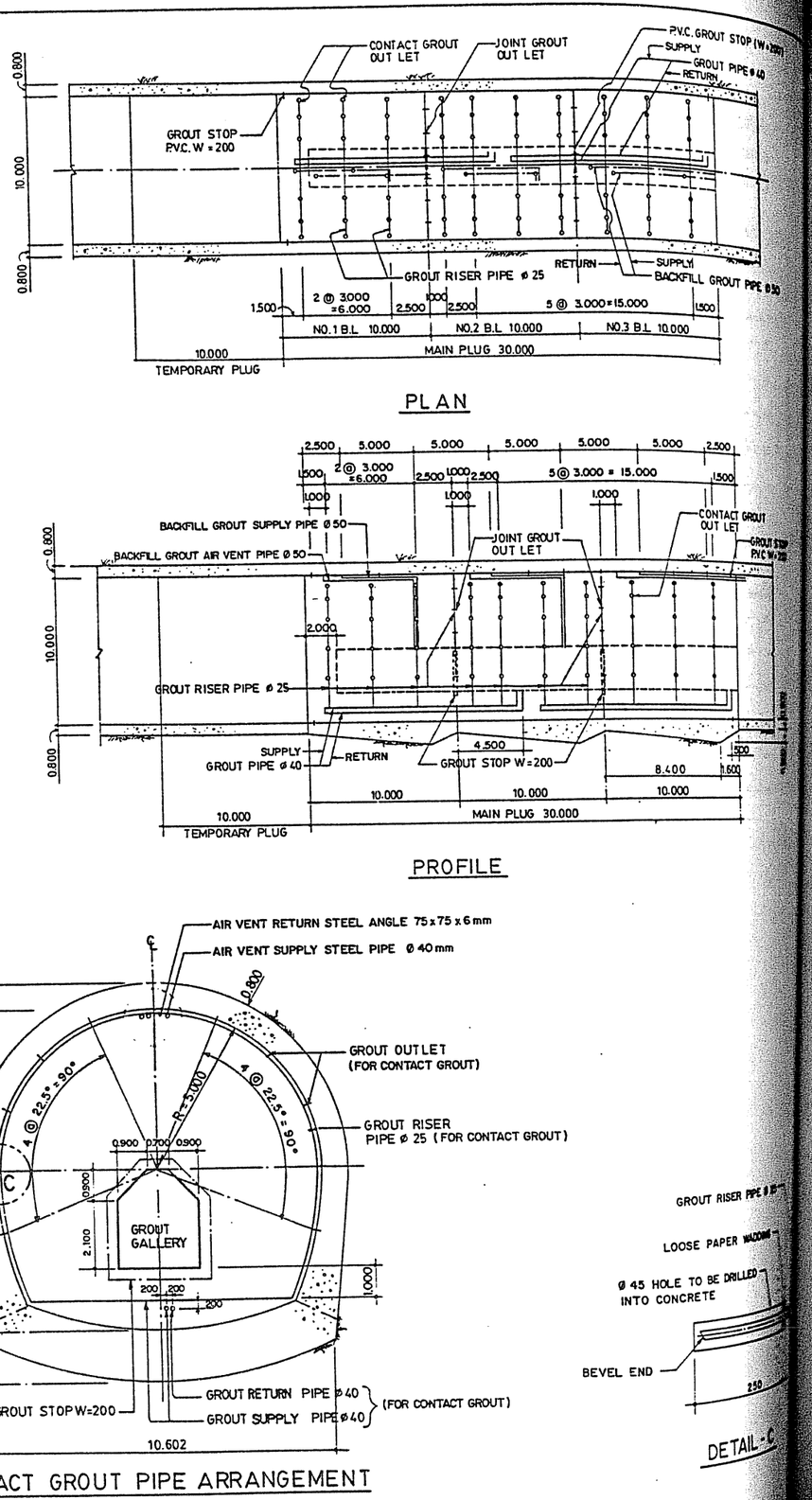
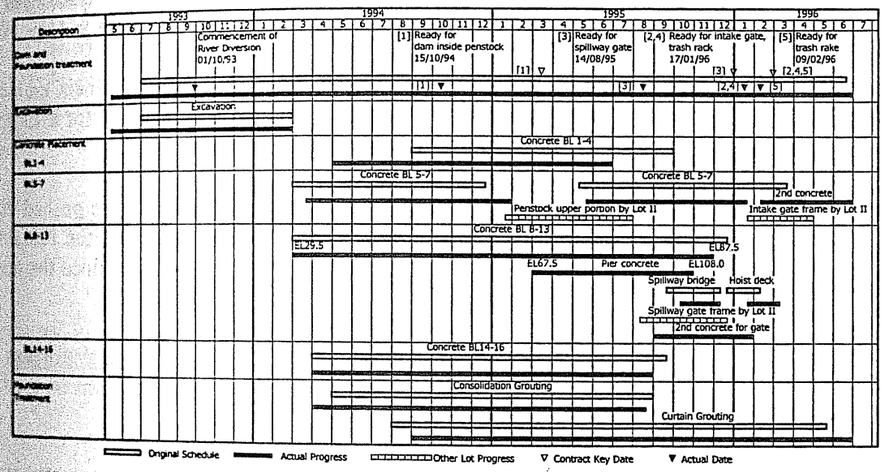


Fig. 2.4.26 Contact Grouting Pipe Arrangement

Chapter 5 DAM

The dam is of a concrete gravity type 58 m high and 257.5 m long at the crest with the total volume of concrete 301,335 m^3. The stilling basin is a horizontal apron type with vertical training walls and dentated sill at its end.

Excavation of the dam commenced in May 1993 on the right bank, and in July 1993 on the left bank. After completion of the river diversion in October 1993, excavation resumed in the river bed area. The total 285,350 m^3 of the dam excavation was completed by the end of February 1994 including additional excavation caused by modified excavation line based on the actual geological condition. After finishing the excavation and cleaning for the foundation rock, concrete placing for the dam started in March, 1994 and was completed in June 1996.



Work Progress for Dam Excavation and Concrete Placement

5.1 Excavation

5.1.1 Excavation Progress

(1) Left Abutment

Excavation of the dam left abutment commenced from the dam crest at EL. 88.0 in July 1993 and was completed in September 1993 up to EL. 52.0 where the existing national road is situated.

Prior to the commencement of the excavation of dam, widening and lowering the access road to the dam was carried out from October 1992 to July 1993 to enable installation of the dam concrete batching plant and the shuttle train line. The access road had been completed upto the dam crest before the start of Lot-I's contract, but some parts of the road were rather higher than the design elevation and not wide enough to pass equipment to install these facilities. Various temporary and permanent access roads to the

enough to pass equipment to install these facilities. Various temporary and permanent access roads to the powerhouse, stockbin, cable crane and riverbed were also constructed in this period for transportation of excavated materials and installation of each construction facility.

Hence, all the excavated materials of left abutment except the large sized materials were transported out via the above mentioned access roads and the existing national road to the disposal area "A" on the left bank downstream near the relocated national road without being pushed down to the river. The large sized rock material was selected and used for the rip rap material for the cofferdams.

(2) Right Abutment

Construction of the 2,750m access road to the cable crane from Pulau Gadang to the right abutment started in October 1992 and reached the right abutment in May 1993.

After completion of the access road, excavation for the dam right abutment including excavation for the 4.5 ton cable crane site at EL. 140 and the runway for a 13.5 ton cable crane on EL. 120 was carried out from May to October 1993.

In this period, half of the excavated materials of the left abutment were pushed down directly to the disposal area "C" on the right bank downstream and the remaining excavated materials were pushed down to the lower river bed portion since the access there by transportation equipment was not able to cross the river.

From October 1993 to January 1994 the dam excavation reached the dam crest EL. 87.5. Excavation continued down to EL. 47.5 which was the lowest point on the right abutment slope in January 1994.

Rock bolt and gunite works for slope stability were carried out on the excavated slope from EL. 137.5 to EL. 47.5 from November 1993 to April 1994.

(3) Riverbed Excavation

After completion of the excavation in the left abutment to EL. 52.0 and in the right abutment to EL. 47.5, the riverbed part of the dam excavation continued to EL. 39.5 the design elevation of the dam foundation, by the end of February 1994.

The excavated materials pushed down to the riverbed were transported out via the access road to river bed (upstream and downstream) to the disposal area "A", "B" and upstream cofferdam.

The dam excavation progress diagram and arrangement of hauling roads are shown in Fig. 2.5.1, Fig. 2.5.2 respectively.

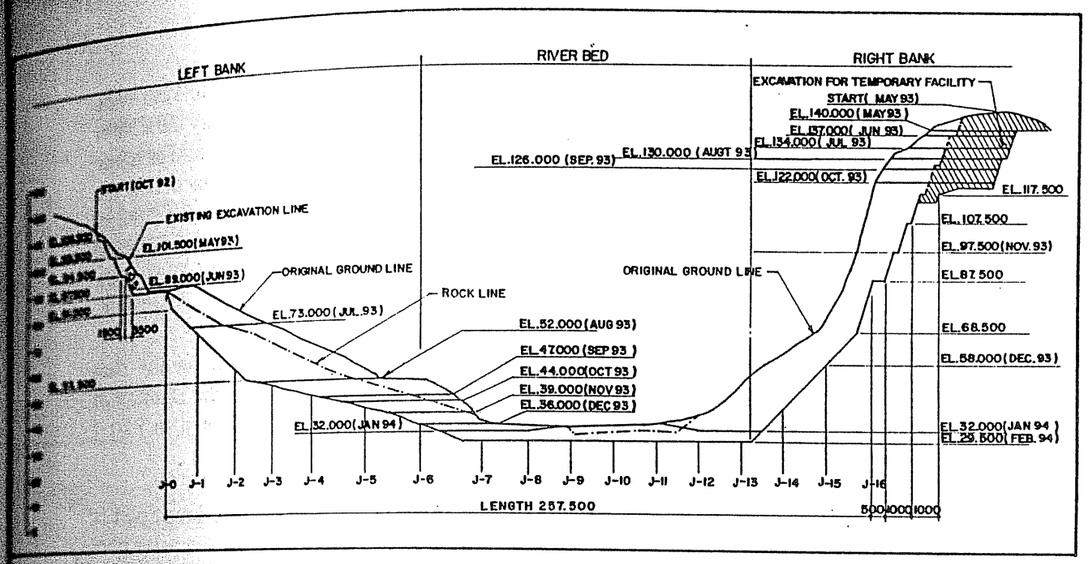


Fig. 2.5.1 Dam Excavation Progress

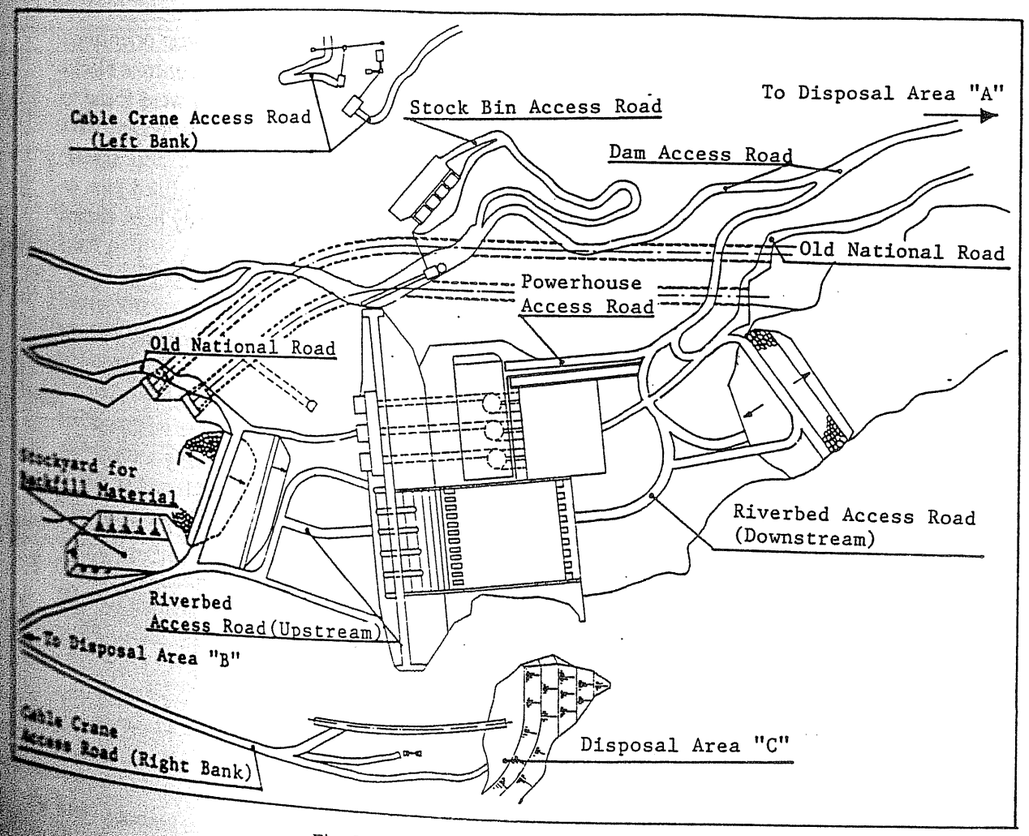


Fig. 2.5.2 Arrangement of Hauling Road

5.1.2 Modification for Excavation Line

The entire area to be occupied by the dam base should be excavated to firm rock capable of withstanding the design loads. As this dam is a concrete gravity dam 58 m high, the dam foundation rock was required to be CM class at river bed and at lower part of the both abutments, and CL class at the upper part in order to obtain the adequate shear strength and bearing capacity against the design loads assuring stability of each dam concrete block. The original design excavation line was determined based on the geological investigation and requirement for finding the necessary rock classifications mentioned above.

The geological condition and the distribution of rock classification on the dam foundation after excavation are described in Part 1. Chapter 2.1 and the stability of each dam block based on the final dam foundation is assured as shown in Part 1 Chapter 6.2 Main Dam.

The final excavation line was almost the same as the design except the lower part of the left abutment at BL. 5-7 and the river bed part of BL. 11. These two parts were modified for following reasons.

(1) Lower Part of Left Abutment.

Several numbers of fracture zone with mainly north-south and east-west orientation were found on the left abutment at about EL. 46-29.5, where the dam concrete blocks No. 5-7 are built up. The fracture zones with north-south orientation were found to extend from the upstream end to the downstream end of the dam foundation and were jointed to the other fracture zones oriented east-west. These were also several discontinuities distributed in the rock-mass in the form of joint, clay-filled joint or seams, quartz vein and brecciate zone. Moreover, this area had already been altered by hydrothermal process in the past.

The weak rock-mass caused by the effect of these discontinuities and the hydrothermal alteration was mainly distributed on the entire area of BL. 5's foundation and the downstream area of BL. 6-7's foundation.

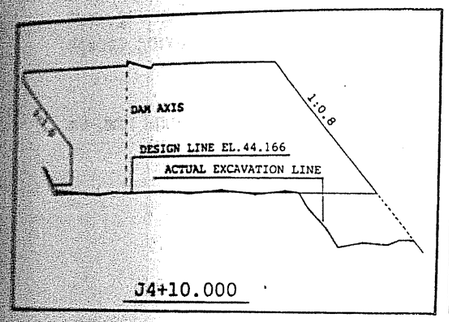
Therefore, in order to remove this weak rock-mass, the excavation line was modified and the weak rock-mass was excavated additionally down to moderately sound rock. The maximum excavated depth was about 10m at downstream part of BL. 5 as shown in Fig.2.5.3.

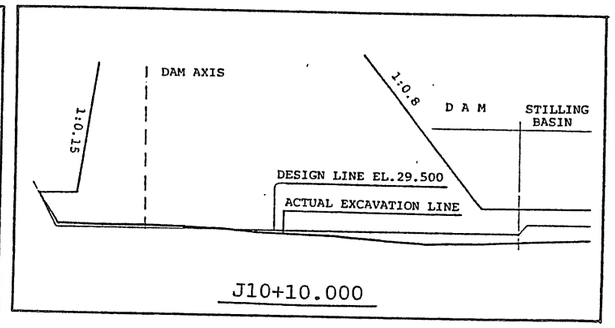
The additional excavated area of 2800 m^3 was replaced with dam concrete and additional reinforcement bars were installed at the edge of the foundation rock for reinforcing the dam concrete and rock surface as shown in Fig.2.5.4.

(2) River Bed

The river bed gravel deposits and completely weathered rocks were still found downstream of the river bed part at the design excavation elevation, EL. 29.5, where the dam concrete block No. 11 is built up.

Therefore, in order to assure the dam stability, these unsuitable materials were excavated additionally down to suitable foundation rock found at EL. 28.0. And these additional excavated areas of 1200 m^3 were replaced with dam concrete as shown in Fig.2.5.3.





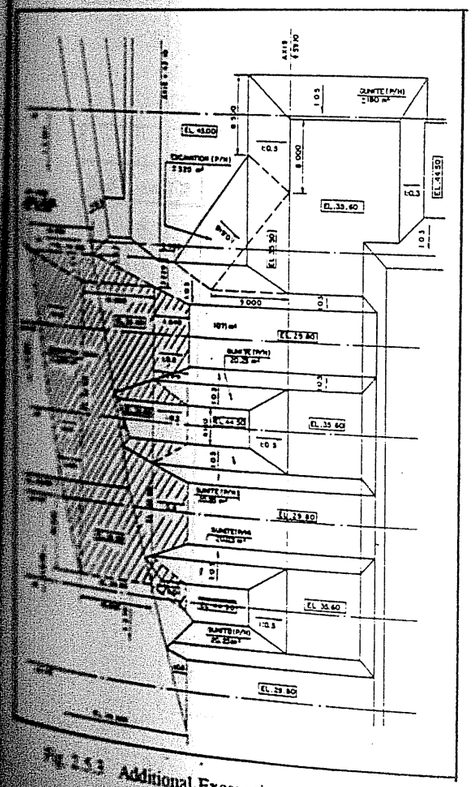


Fig. 2.5.3 Additional Excavation for Dam Foundation

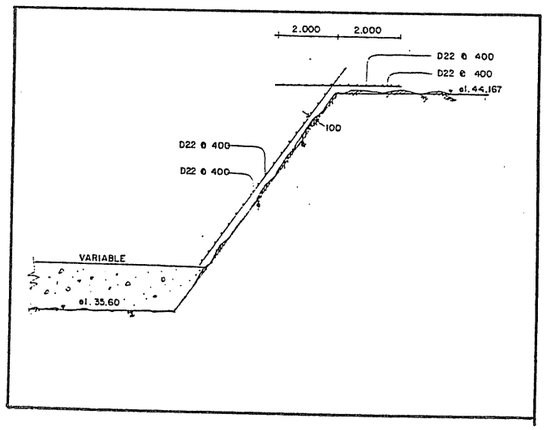


Fig. 2.5.4 Reinforcement at Dam Additional Excavation

5.1.3 Excavation Method

The excavation of the dam foundation has to be carried out in such a manner not to shatter or loosen the foundation rock and to give a smooth shape to the slope. Therefore, the excavation of the dam foundation was carried out by following two steps. The excavation method and construction equipment are shown in Fig. 2.5.5 and the table below respectively.

(1) Excavation for Bench

At 1st step, the excavation of the dam foundation was carried out by blasting leaving a temporary unexcavated rock cover of 30 cm from the design line by the bench-cut method from the upper part of the dam site to the riverbed. This is the usual method adopted for the dam foundation excavation works having the advantage of giving small damage to the bed rock, shaping the slope smoothly at ease, and having enough efficiency.

In accordance with the excavation location and the gradient of the ground line, the following two patterns were used.

	steep slope or adjacent to cover rock	gentle slope with excavation depth more than 8.5 m
Bench height	3 m	5 m
Spacing of hole	1.5 m	2.0 m
Volume of powder	less than 2.0 kg/m³	less than 2.5 kg/m³

Main Equipment for Dam and Stilling Basin Excavation

Operation	Equipment	Capacity	Quantity				
			Left Bank	Right Bank	River Bed	S. Basin	Tail
Drilling	Hydraulic Crawler Drill	165 kw	1	0	0	0	
	Pneumatic Crawler Drill	135 kw	0	1	1	1	
	Air Compressor	18.5 m³/min	0	1	1	1	
Excavation	Bulldozer with Ripper	45 ton	1	0	1	0	
		32 ton	0	1	1	1	
	Excavator	1.3 m³	1	1	(1)	(1)	
Trimming	Hydraulic Breaker	1.500 kg	1	1	(1)	(1)	
Loading	Wheel Loader	3.5 m³	1	0	1	0	
		2.4 m³	0	0	1	0	
	Dozer Shovel	2.2 m³	1	0	1	1	
Hauling	Dump Truck	11 ton	2	0	4	2	
		8 ton	18	0	24	6	
Spreading and	Bulldozer	25 ton					
Leveling	Vibration Roller	10 ton				1	
(Disposal Area)	Hand Breaker	20 kg	1	0	1	1	
Final Treatment	Pick Hammer	7.5 kg	1	2	1		
	Air Compressor	18.5 m³/min					
	Reservoir Tank	3 m³, 7 kg/cm²					

(2) Finishing Excavation

The bedrock was protected against damage by blasting and subsequent weathering by leaving 30 cm to be excavated for finishing. Finishing excavation was carried out by pick hammer, hand breaker and other tools up to next concrete placing height plus 30cm just before the concrete work. (refer to Fig. 2.5.5).

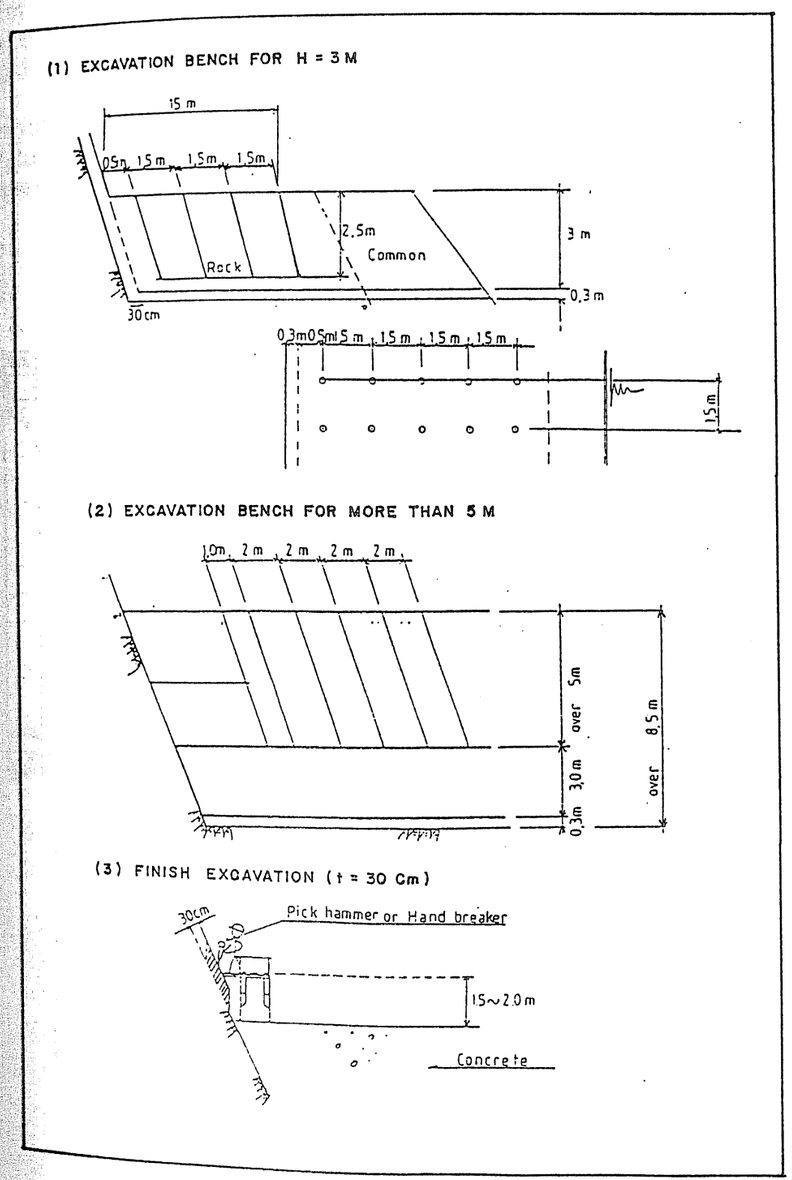


Fig. 2.5.5 Excavation Method

(3) Construction performance data

The summarized table below is the main performance data such as summary of the dam excavation works.

Monthly Excavation Progress and Progress Rate

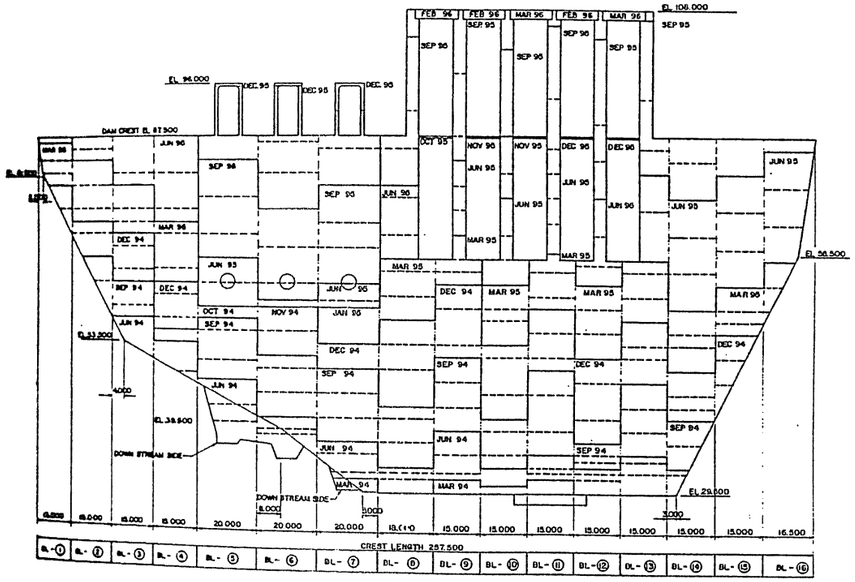
		Dam			Stilling Basin
		Left Bank	Right Bank	River Bed	
1993	Jul	9800	15700	0	0
	Aug	11000	14000	0	0
	Sep	5200	12300	0	0
	Oct	9600	13400	1200	0
	Nov	21600	17000	7300	13000
	Dec	16900	13400	18500	24300
1994	Jan	15400	13600	24200	23400
	Feb	11300	11100	19900	30900
	Mar	800	200	300	9800
	Apr	400	200	61	2800
	May	100	100	0	700
	Jun	100	100	0	500
	Jul	70	70	0	180
	Aug	45	45	0	100
	Sep	45	45	0	42
	Oct	45	45	0	0
	Nov	45	45	0	0
	Dec	45	41	0	0
Total		102495	111391	71461	105722
Total of Rock		87825	103621	65181	64325
Number of shift		546	565	200	274
Ave. Progress (m^3/shift)		188	197	357	386
Total Explosive (kg)		16270	17550	13760	10810
Explosive Usage ((kg/m^3 Rock)		0.19	0.17	0.21	0.17

5.2 Dam Concrete Placement

The Kotapanjang dam, required placement of a total concrete volume of 301,335 m^3 taking two years and five months from the completion of excavation in March 1994. The progress of concrete placement is shown in Fig. 2.5.6.

A layer method was adopted for concrete placement which required the maximum block of 18 m wide and 57.5 m long in the riverbed and 20 m wide and 55.3 m long at the intakes in the left banks. The maximum block volume was 1,891 m^3 at BL. 8 on EL.37.5 - EL.39.5.

The monthly maximum concrete placement amounted to 27,398 m^3 in August 1994. After placement of the dam body concrete, the secondary concrete was placed for steel embedment at the spillway gates, intake gates and intake trashrack.



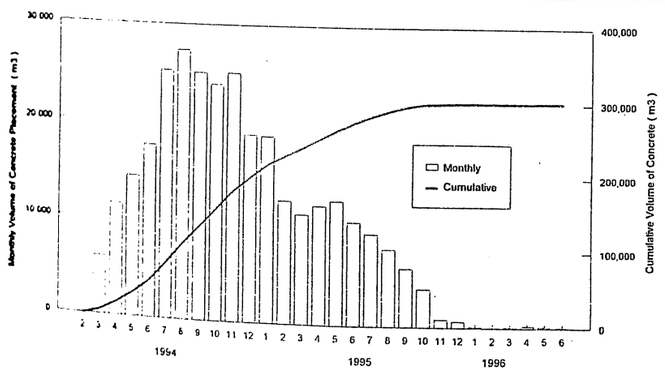


Fig. 2.5.6 Monthly Progress of Concrete Placement

5.2.1 Contractual Key Dates

Inspection was carried out for installation of the penstocks, intake gates and the spillway gates by Lot II contractor. The actual hand over from Lot I contractor to Lot II contractor is shown in the table below. It was confirmed that Lot I contractor completed construction work before the specified key dates.

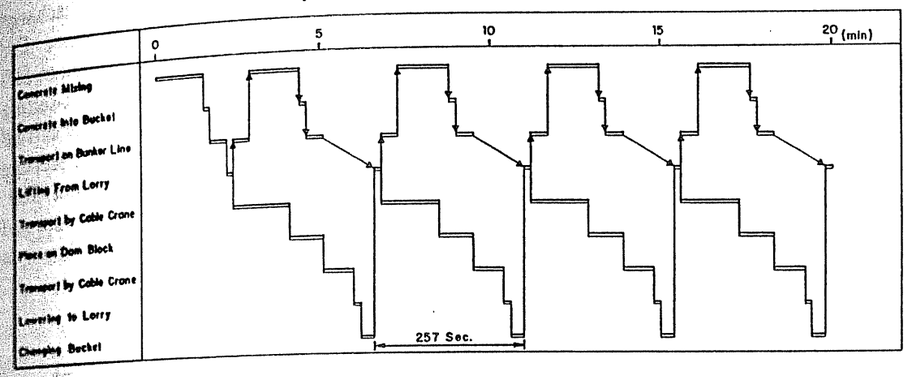
Key Date for Dam

Part of Works	Contractual Key Dates	Completion Date	Advance/ Delay	Remarks
(1) Partial completion of dam ready for dam inside penstock installation	15/03/95	Actual 15/10/94	Advance	BL5 : 15/10/94 BL6 : 20/11/94 BL7 : 30/01/95
(2) Partial completion of dam ready for intake gate installation	01/03/96	Actual 17/01/96	Advance	No.1: 17/01/96 No.2: 03/02/96 No.3: 24/01/96
(3) Partial completion of dam ready for spillway gate installation	01/01/96	Actual 14/08/95	Advance	No.1: 14/08/95 No.2: 14/08/95 No.3: 29/08/95 No.4: 28/08/95 No.5: 13/09/95
(4) Partial completion of dam ready for trashrack installation	01/03/96	Actual 17/01/96	Advance	No.1: 17/01/96 No.2: 03/02/96 No.3: 24/01/96
(5) Partial completion of dam ready for trashrake installation	01/03/96	Actual 09/02/96	Advance	09/02/96

5.2.2 Mixing and Transportation

Concrete for the dam was mixed at the dam batching plant located in the left bank. After mixing, the concrete was discharged directly into the bucket from the mixer in order to avoid segregating the concrete material, especially coarse aggregate. Afterwards, the concrete in the bucket was transported to the dam axis area by battery car then lifted up by a 13.5 cable crane to the dam concreting block. The average cycle time for all parts of the concrete placement was 257 sec as shown in the figure below. The average and the maximum concrete placing rate was recorded as 63.0 m^3/hr and 100.8 m^3/hr at BL.3 EL.67.5-EL.69.5 respectively.

Cycle Time of Dam Concreting



5.2.3 Preparation of Concrete Placement

(1) Cleaning of foundation rock surface

Prior to placement of concrete on the foundation rock, the rock surface was trimmed and cleaned up manually by using breakers, compressed-air, water jets and vacuum machine to remove all deleterious materials such as rock fragments, loose rock, debris of excavation and so on to obtain good bonding between the concrete and its foundation.

(2) Cleaning of previous concrete surface

The previous concrete surface, namely the horizontal construction joint, was cleaned up by compressed-air, water jets and vacuum machine to remove all deleterious materials such as remnant construction materials and free-lime caused by forming, reinforcing, pending and so on.

In the case of the long-term left joint, it was left as it was for one month or longer and chipping was carried out additionally on the surface.

(3) Seepage Treatment

25 locations of ground water seepages from the foundation were found during excavation of the dam and their locations are shown in the table and figure below.

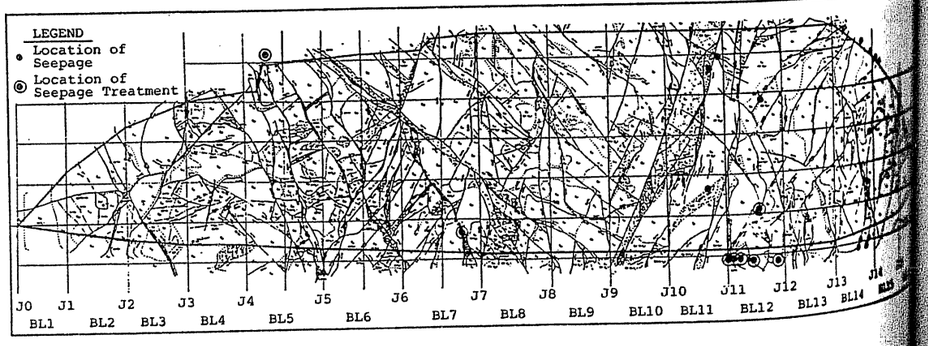
To avoid losing good bonding between the dam concrete and the foundation rock due to seepage water, the following seepage treatment was carried out depending on the discharge of the seepage.

In the case of a small discharge of seepage water, that was the greater part of the seepage treatment, a steel pipe (Ø 75 mm) or a rubber hose was installed at the seepaging site by quick-setting cement and then cement milk (c/w = 1.5) was injected into the pipe or hose with 3 kg/cm^2 pressure after placing the dam concrete upto the elevation to balance with the pressure of the seepage water.

The seepage water from the foundation of BL.7 and BL.12 showed a large discharge which was not able to be treated by the steel pipe or rubber hose mentioned above. For the seepage treatment for these large discharges of seepage water, a steel pipe of 500 mm diameter was installed at the seepaging site as a catchment well by quick-setting cement and then the dam concrete placing was carried out while dewatering was carried out through the catchment well using the drain-pump. Afterwards, when the seepage water pressure was balanced with the dam concrete covering the catchment well, cement milk (c/w=1.5) was injected to fill the well after installation of an air vent and injection pipe covered by throwing gravel into the well to fill it up as shown in Fig.2.5.7.

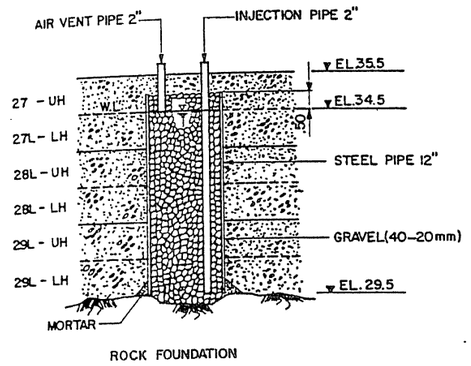
Location and Treatment for Seepage

BL No.	Location		Nos	Discharge Condition	Treatment
5	Penstock route	DA+43 m	1	small flow	steel pipe Ø 75
7	Upstream	DA -4 m	1	large flow	steel pipe Ø 500
11	Midstream	DA+7 m	1	wet	not treatment
	Downstream	DA+36-39 m	2	wet	not treatment
12	Upstream	DA-12 m	5	small flow	steel pipe Ø 75
	Midstream	DA+1 m	1	wet	no treatment
		DA+29 m	1	large flow	steel pipe Ø 500
14	Upstream	DA-9-11 m	4	wet	no treatment
	Midstream	DA+2~5 m	2	wet	not treatment
		DA+18 m	1	wet	not treatment
	Downstream	DA+38-40 m	2	wet	no treatment
15	Test adit	DA+1 m	1	wet	no treatment
		DA+16 m	1	wet	no treatment
	Downstream	DA+35-36 m	2	wet	no treatment



Location of Seepage

Fig. 2.5.7 Seepage Treatment



After completion of all preparation works, the following inspection was carried out for concrete placement.

- geological inspection
- survey work for recording topography of the final surface
- installation of water-stop, drain pipe and other embedded material
- installation of monitoring equipments and lead cables
- checking of form and reinforcement
- final cleaning condition of foundation rock and concrete surface
- dampening condition of foundation rock and concrete surface
- checking of seepage treatment and dewatering

5.2.4 Concrete Placement

(1) Lift Height

The lift height for concrete placement was adopted in principle to be 2 m (Full-Lift) for the lifts contacting with the steep-sloped foundation rock in both abutments and rising blocks except for the lifts contacting with the gentle-sloped foundation rock including river bed or having been left as it was without receiving the overlying lift for four months or longer. For the latter cases, a half lift of 1.0 m high was selected and carried out.

The concrete placement by the full lift and half lift was carried out by a 5 days cycle and 3 days cycle respectively.

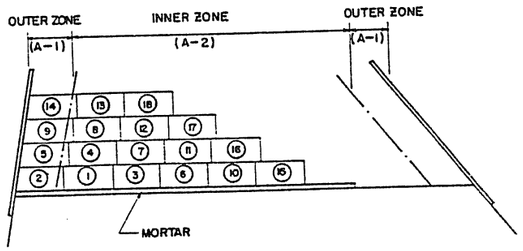
The actual number of the half lifts was 6 lifts in the river bed blocks, 3-8 lifts in the left abutment blocks contacting with the gentle-sloped foundation rock and 2 lifts in the BL.5-7 that were not receiving the overlying lift for four or seven months.

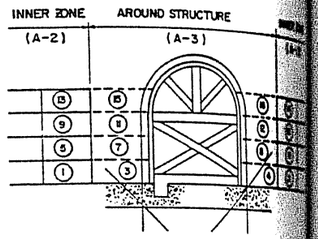
Maximum height difference between adjacent blocks of 9 lifts occurred between the dam block with penstock and its adjacent block due to erection of the penstock pipes.

(2) Placing Method

The step method was adopted to place the concrete. The concrete placing was carried out in principle from upstream side towards downstream side in 4 layers each 50 cm thick as shown below. This placing method is a usual practice having the advantage of minimising the exposure time of placed concrete and simplifying the dewatering from concrete surface and horizontal joint.

In the case of placing concrete around the openings embedded in the dam body such as the gallery and the penstock pipe, concrete was placed on both sides alternatively in order to avoid undesirable displacement and deformation caused by difference of horizontal pressure due to placing concrete.



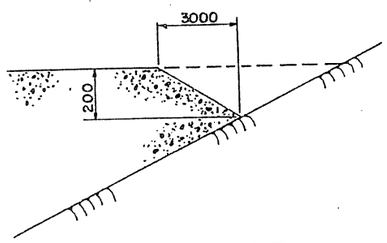


(3) Placing and Compacting

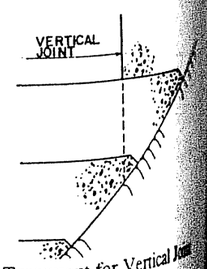
Prior to placing concrete, mortar was placed 20mm thick and 15mm thick on the foundation rock and the previous lift surface respectively to obtain good bonding between concrete and foundation rocks. Concrete was basically placed from bucket directly and it was prohibited to free-fall more than 1.5 m and to move it using compacting vibrators to assure the quality of the concrete.

Concrete was compacted with a viback, a crawler with vibrator, in the wide spaced part of the block and hand-held vibrators in the narrow spaces near the form work and the joint along the previous lift concrete.

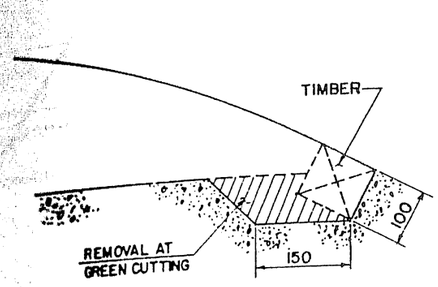
Acute angled parts in the concrete lift, namely knife edge parts, such as the upper part of the lift contacting with the gentle-sloped foundation rock or concrete form and the lower part of the lift placing between vertical contraction joint and steep-sloped foundation rock, were required to obtain sufficient thickness of the concrete mortar by following treatments as shown in the figure below because they were difficult to compact sufficiently and easy to generate some cracks due to the stress concentration.



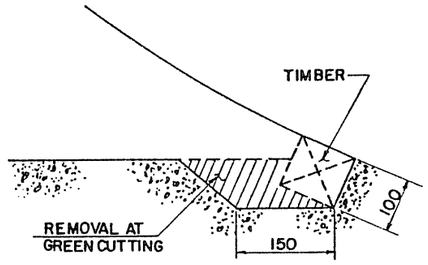
Treatment for Knife Edge
at Gentle - Sloped Foundation



Treatment for Vertical Joint
at Steep - Sloped Foundation



Treatment for Knife Edge at Top of Spillway



Treatment for Knife Edge at Bottom of Spillway

(4) Curing

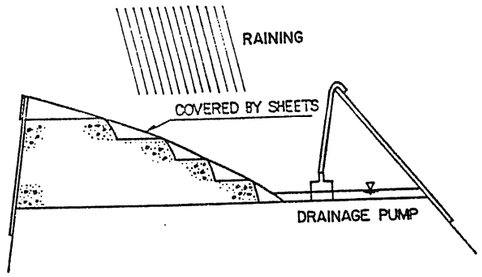
Immediately after completion of concrete placement, concrete was cured with a water pond and water sprays using perforated hose. The lift surface was kept moistened continuously until the subsequent lift was placed.

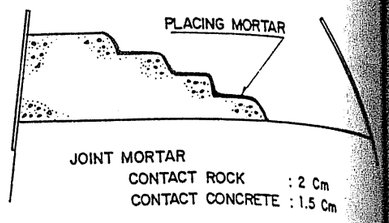
(5) Treatment of Lift Surface

Horizontal concrete surfaces (construction joints) were cleaned by high pressure water jet so as to remove laitance and loose aggregate. This operation is called a "green-cut" and its timing was decided to be done three hours after completion of concrete placing based on the field test result.

(6) Placing under Sudden Rainfall

Placing of concrete was not allowed in principle under the rainfall conditions. However, concreting was allowed to continue in the rain if the intensity was less than 4 mm/hr. Concrete placed under such conditions was protected immediately with cover sheets as shown below which prevented deterioration of the concrete surface caused by splashing of precipitation that might wash away the mortar. When sudden rainfall with severe intensity (more than 4 mm/hr) came out, concrete placing was suspended temporarily, and resumed by placing mortar over the lift surface prior to continuing concrete placing.





Cold joints occurred eleven times in the dam concrete placement when the pour in surface was left as it was because of sudden heavy rainfall and or there was trouble with placement equipment. Treatment of cold joints was carefully carried out after hardening of the concrete by means of joint treatment. Joint treatment was exercised by stripping sound concrete surfaces by chipping. Particularly irregular or sharp-edged portions were throughly removed and treated. The location and cause of each cold joint is shown in the table below.

Records of Cold Joints

BL. No.	EL. (m)	Cause
5	43.5-45.5	Heavy Rain
	69.5-71.5	Heavy Rain
6	51.5-53.5	Hoisting Wire Cut
	61.5-63.5	Heavy Rain
7	41.5-43.5	Heavy Rain
	43.5-45.5	Crane Trouble due to Lightning Strike
10	39.5-41.5	Heavy Rain
12	29.5-30.5	Heavy Rain
	43.5-45.5	Crane was used for evacuation for upstream facilities due to Flooding
13	51.5-53.5	Heavy Rain
14	32.5-33.5	Crane Trouble due to Lightning Strike

(7) Construction performance data

The summarized tables below are the main performance data such as equipment for the work and the progress rate of concrete placement.

Main Equipment for Concrete Work for Dam and Stilling Basin

Operation	Equipment	Capacity	Quantity Dam	Quantity Stilling Basin
Concrete Mixing	Dam Batching Plant	90 m^3/hour	1	
	General Batching Plant	30 m^3/hour		1
Transportation and Placing	Cable Crane	13.5 ton	1	(1)
	Shuttle Train Line	L = 60 m	1	
	Roughterrain Crane	25 ton		1
	Concrete Bucket	4.5 m^3	3	(1)
		1.0 m^3		1
	Agitator Truck	4.5 m^3		2
		3.0 m^3		2
	Concrete Pump Truck	100 m^3/hour		1
	Rolling Tower	H = 30 m		1
Compacting	Vi-Back	ϕ 130 mm x L 650 mm x 4 pcs	1	
	Hand Vibrator	ϕ 131 mm x L 660 mm	8	(2)
		ϕ 52 mm x L 366 mm	5	5
Cleaning, Green Cut and Curing	Air Compressor	18.5 m^3/min	2	
	Reservoir Tank	7 kg/cm^2	1	
	Water Supply Pipe	Steel, ϕ 6"	400 m	
		Steel, ϕ 4"	400 m	
	Water Supply Hose	Fibre Sheathed Rubber, ϕ 2"	400 m	
		Fibre Sheathed Rubber, ϕ 1"	2,000 m	
	Water Jet	Max. 210 kg/cm^2	2	
	Submersible Pump	ϕ 6"		1
		ϕ 3"	4	
		ϕ 2"	8	4
Form Work	Truck Crane	20 ton	1	
	Roughterrain Crane	25 ton		1
	Mobil Crane	4.9 ton	1	
	Crawler Crane	4.8 ton	1	
	Truck with Crane	3 ton	1	

Progress Rate of Dam Concrete Placement

Description	Dam		Stilling Basin	
	Primary	Secondary	Slab	Wall and Sill
Total Volume (m^3)	300225	1110	17455	21580
Working Days	505	114	103	233
Working months	24	12	13	18
Ave. Daily Progress (m^3/day)	595	10	169	93
Ave. Monthly Progress (m^3/month)	12509	93	1343	1199
Max. Monthly Progress (m^3/month)	27398 (Aug. 1994)	-	3118 (Aug.1994)	3560 (Apr.1995)
Fastest Placing Speed (m/hr)	100.75 (BL. 3 EL.67.5 - 69.5 V = 534 m^3)	-	58.58 (BL. E-3, EL.305 - 32.5, V=215m^3)	55.90 (BL. H-1, EL.43.5 - 45.5, V=289m^3)
Fastest Placing Speed (m^3/hr) (more than 1000 m^3)	88.93 (BL.7, EL.47.5 - 495 V = 1534 m^3)	-	-	-

5.2.5 Formwork

Concrete formwork for the dam body was "Flying-form" (cantilever type), 2m high and 3m wide standard steel form with integral strong backs and walkways.

Formwork for the inspection gallery was made of timber having 2.0m wide and 2.5m high.

For formwork of spillway chute from EL.32.5 to EL.35.5, universal columns (100x100x6x8) were used as vertical soldiers. Plywood panels were set and fixed by bolts.

The formwork for the spillway crest was prepared for concrete placement in 2 lifts (i) Lift 1: EL.65.5 to EL.67.21, and (ii) Lift 2 : EL.66.8 to EL.67.50. This sequence was planned to ensure position of the blackouts, proper profile of the "ogee" crest curve and minimising fresh concrete height differences between the pier construction joints and exposed concrete surfaces.

Formwork for the intake structure was supported by scaffolding on the floor beams and stringer. H-beam stringers (H-1000x300x15x30) were installed on the brackets in the Dambody and the temporary foundation placed on the upstream rock. Floor beams were bolted at a right angle to the stringers.

Formwork for the overhang upstream spillway structure was supported by the main support brackets which were made of H-beam (H-125x125x6.5x9).

C-bolts and anchor bolts were embedded in lifts between EL.49.5 and EL. 54.5 to fix the main support brackets and formwork. By these support brackets, scaffolding and telescopic shore support the formwork of the overhang structure.

The formworks of dam body, gallery, spillway chute, spillway crest, overhang for intake structure and overhang for spillway upstream structure are as shown in Fig. 2.5.8 to 2.5.16.

5.2.6 Temperature Control of Dam Concrete

(1) General

During the process of hardening of concrete, the rise in temperature of concrete takes place by the heat of hydration. In massive concrete such as the dam concrete, the heat of hydration of cement does not radiate rapidly and therefore the process of setting produces temperatures in the interior of the dam which are well above those in the adjacent atmosphere. After concrete has hardened, it gradually cools down to the temperature of the final stable state, (the mean ambient temperature) by heat transfer.

In this process of falling temperature, the internal and external restraints prevent the volumetric change of concrete due to changing temperature and therefore tensile stresses

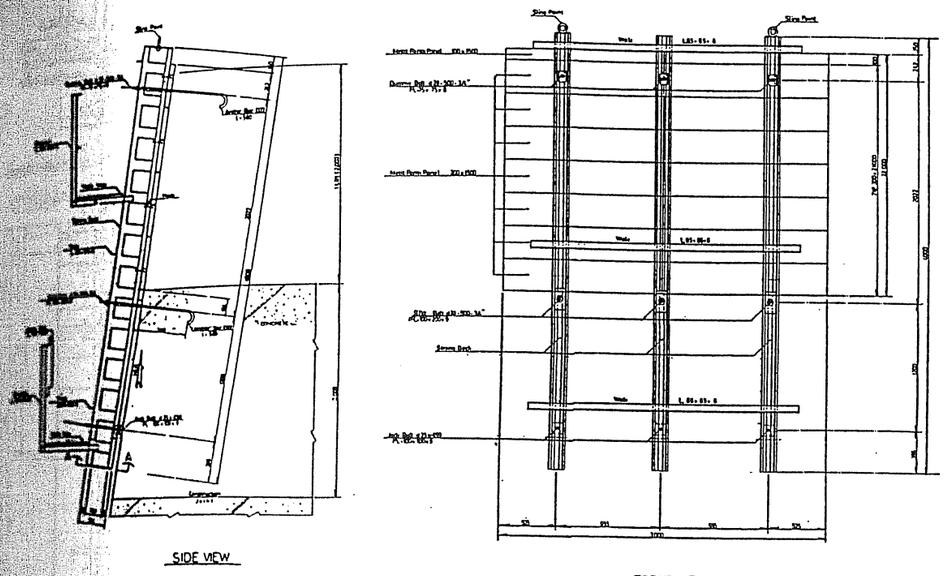


Fig. 2.5.8 Dam Upstream Form Work

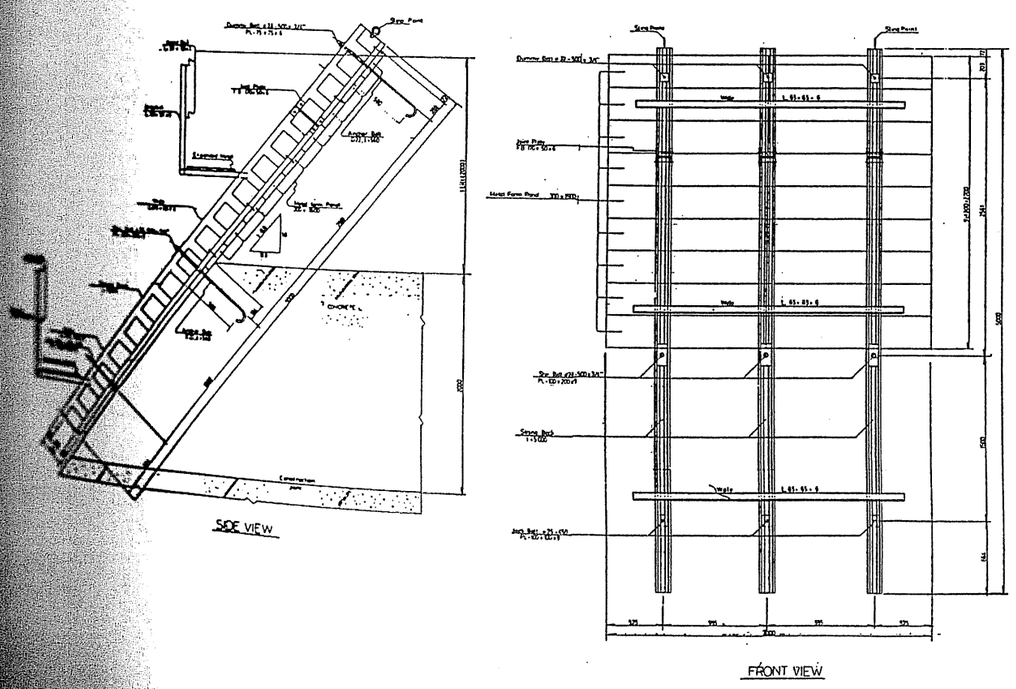


Fig. 2.5.9 Dam Downstream Form Work

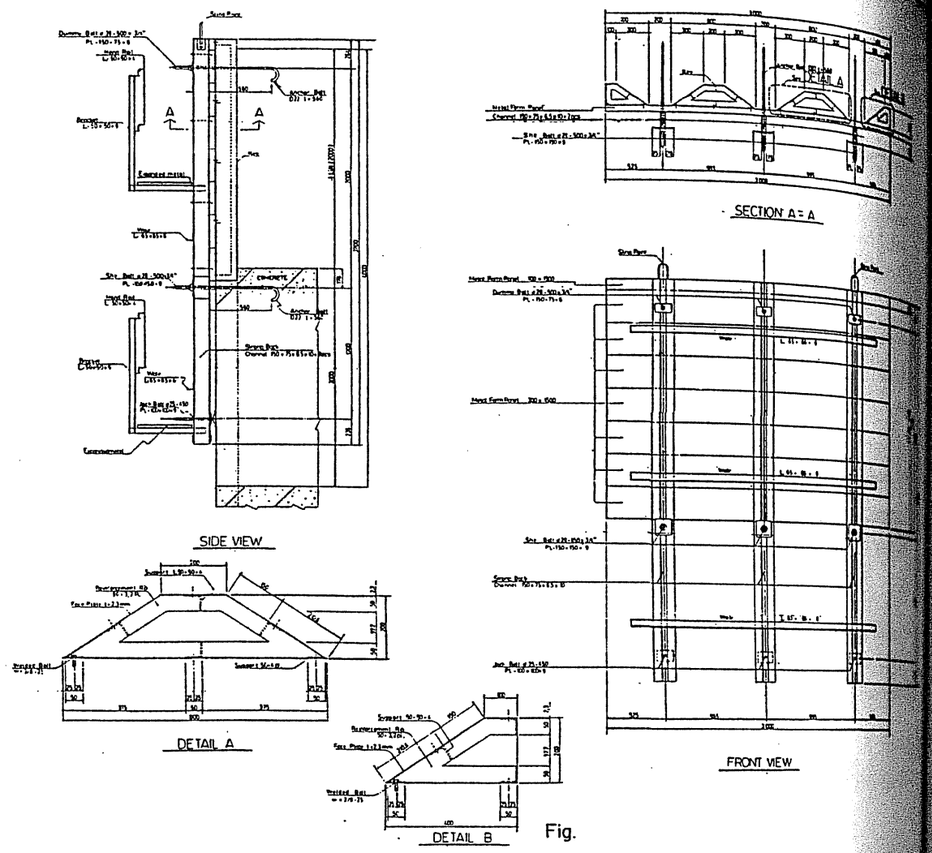


Fig. 2.5.10 Transverse Joint Form Work

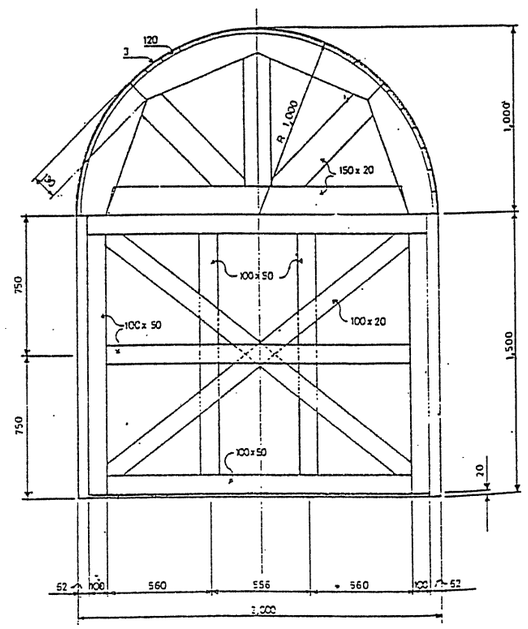


Fig. 2.5.11 Dam Gallery Form Work

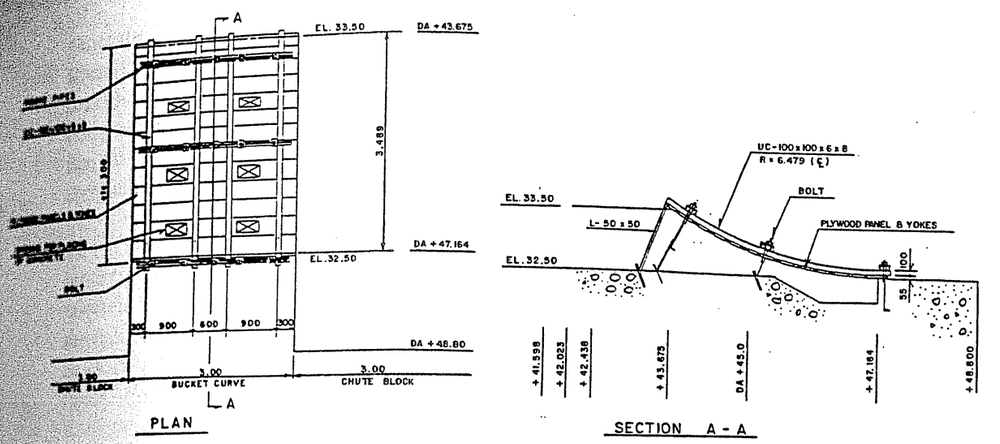


Fig. 2.5.12 Spillway Bottom Part Form Work

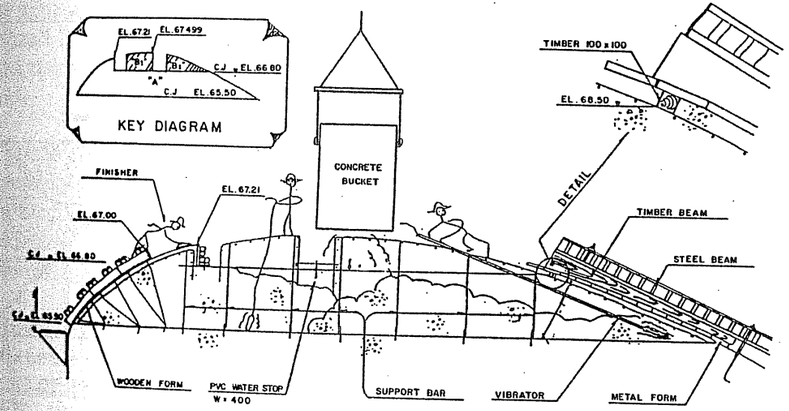


Fig. 2.5.13 Concrete Placing at Top of Spillway (EL. 65.50 - 67.21)

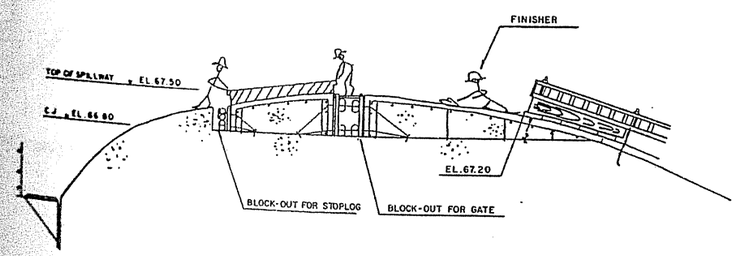
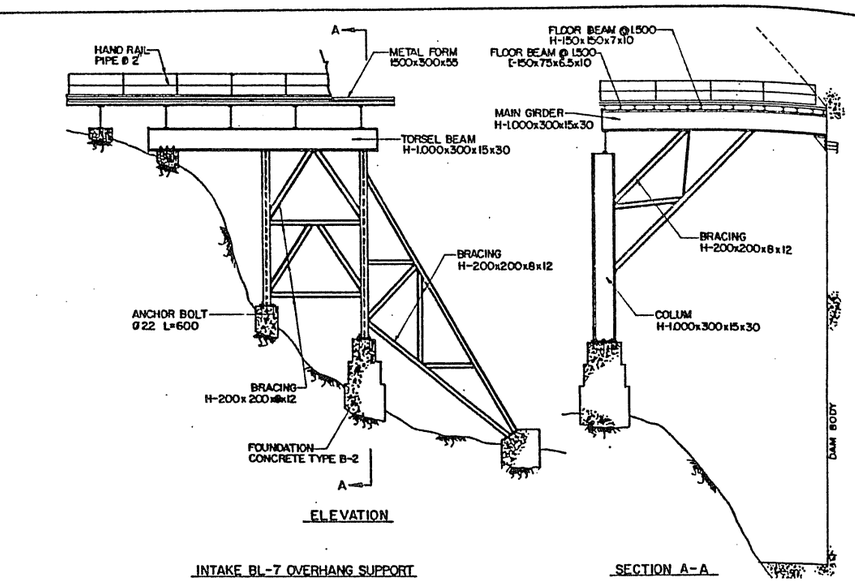
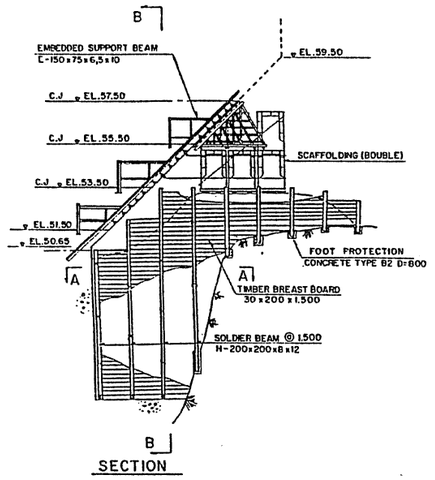
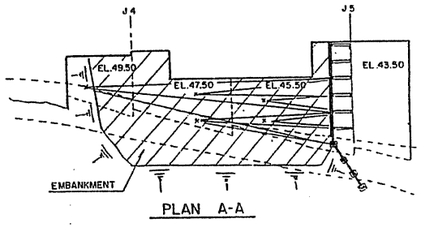


Fig. 2.5.14 Finishing Works at Top of Spillway







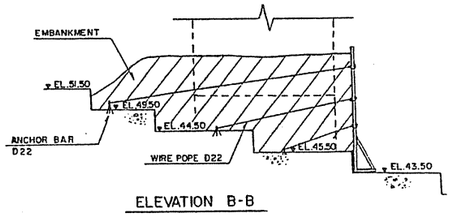


Fig. 2.5.15 Intake Overhanging Support

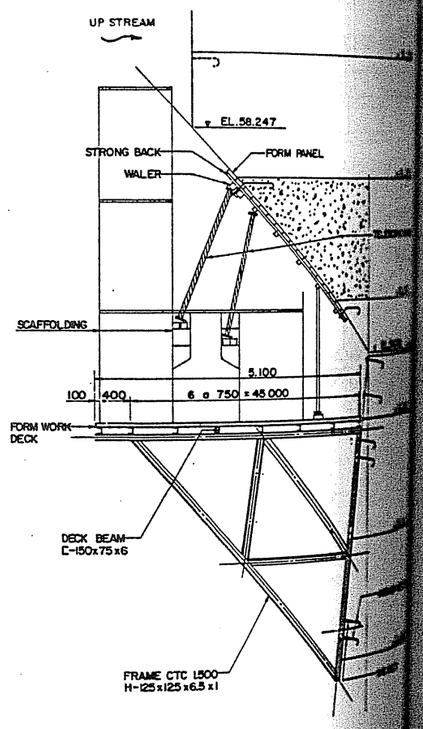


Fig. 2.5.16 Spillway Overhanging Support

generate in the concrete. When the tensile stresses develop and exceed the tensile strength of concrete, thermal cracks occur in the concrete and they cause stress concentration and discontinuities which prevent the structure from carrying loads as designed. Then the dam may lose its stability and watertightness. Therefore, the cracks due to temperature changes or steep temperature gradients should be reduced to acceptable levels by suitable temperature control measures.

The temperature regulation of the dam concrete by pre-cooling was selected as the temperature control measure for the purpose of reducing the highest temperature of concrete based on the results of heat transfer and thermal stress analyses carried out in the design stage. However, the proposal to modify the lift height to 2.0m was proposed to shorten the dam construction period for the purpose of getting margins against all key dates. For that, both finite element analyses were carried out again to confirm the temperature regulation for the dam concrete based on the actual construction conditions of the dam, mix proportion of concrete, dam concrete zoning and lift height.

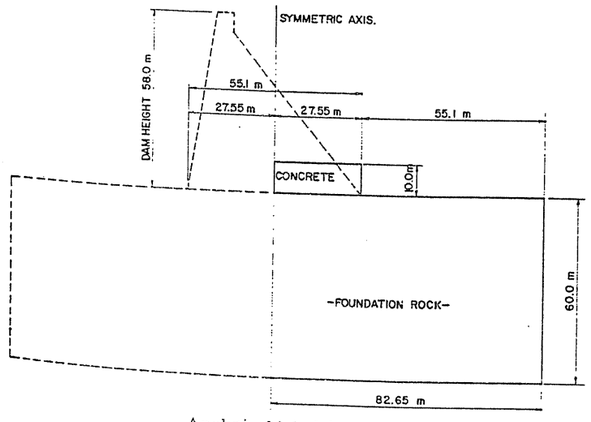
(2) Analysis Condition

(i) Analytical Model

In general, the concrete placing at the dam base portion needs most attention in the overall lift schedule, since the concrete block at this location has the maximum length and there is a large degree of restraint between the concrete block and its foundation rock.

Therefore, the dam concrete block at the dam base portion in the river bed block was selected for the analytical model, and its length was set in 27.55m as the half length of the block by taking advantage of the structural symmetry : distribution of heat and stress generated in the model are always symmetric with respect to the center line while the actual length of the concrete block was 55.1m.

The concrete model was 10m in height, which corresponded to a total of 7 lifts, (4 half lifts and 3 full lifts). Each lift was cast at an interval of 6 days. The dimension of the foundation rock model was set for three times in horizontal direction and equivalent to dam height in vertical direction ; length and height being 82.6m and 60m respectively. The analytical model employed in this analysis is shown below.

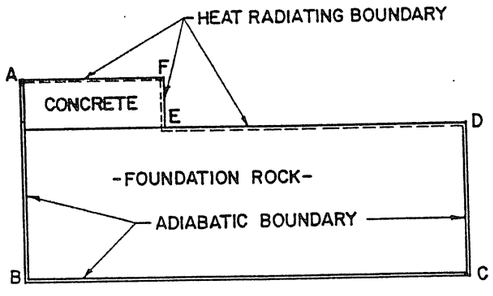


Analytical Model

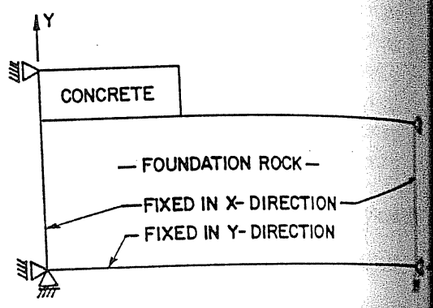
(ii) Boundary Condition

In the heat transfer analysis, the heat transfer coefficient Q, equal to 10 kcal/m².hr.°C was applied for the heat radiating boundaries, D-E, E-F, F-A and the transmission of heat was suppressed at the boundaries, A-B, B-C, C-D, as shown below.

The boundary conditions for the thermal stress analysis was shown below.



Heat Transfer Analysis



Thermal Stress Analysis

Boundary Condition

(iii) Plan for Concrete Placement

Lift No.	Lift height	Placing date	Temperature at placing	Cement content	Remarks
1	1.0 m	0th	30°C	223 kg/m³	Type A-1
2	1.0 m	7th	30°C	223 kg/m³	"
3	1.0 m	14th	30°C	223 kg/m³	"
4	1.0 m	21st	30°C	166 kg/m³	Type A-2
5	2.0 m	28th	30°C	166 kg/m³	"
6	2.0 m	35th	30°C	166 kg/m³	"
7	2.0 m	42nd	30°C	166 kg/m³	"

(iv) Material Properties

Items	Concrete A-1	Concrete A-2	Rock
Unit weight (t/m³)	2.4	2.4	2.65
Thermal Conductivity (Kcal/m.hr.°C)	2.3	2.3	2.65
Specific heat (Kcal/kg.°C)	0.3	0.3	0.2
Adiabatic temperature Rise (°C)	29.5 (1-e-0.72t)	23.3 (1-e-0.58t)	-
Temperature in final stable state (°C)	28.0	28.0	28.0
Compressive strength (Kg/cm²) : f'c(t)	f'c(t) = t / (6.2+0.93t) f'c (91)		1,000
Tensile strength (kg/cm²) : ft (t)	$ft\ (t) = 1.4\sqrt{f'c\ (t)}$		83
Elastic Modulus (kg/cm²) : E (t)	$Et = 1.1 \times 10^4 \sqrt{f'c\ (t)}$	(t ≤ 3)	30,.000
	$Et = 1.5 \times 10^4 \sqrt{f'c\ (t)}$	(t ≤ 3)	
Poison's ratio	0.17	0.17	0.25
Thermal lineal coefficient (1/°C)	10×10^{-6}	10×10^{-6}	10×10^{-6}

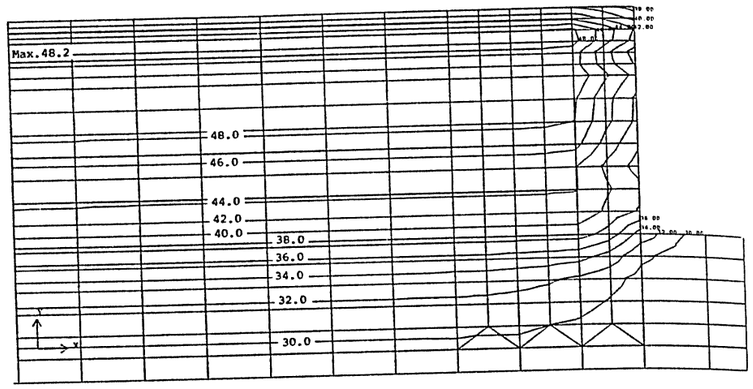
(3) Results of Analysis

(i) Heat transfer analysis

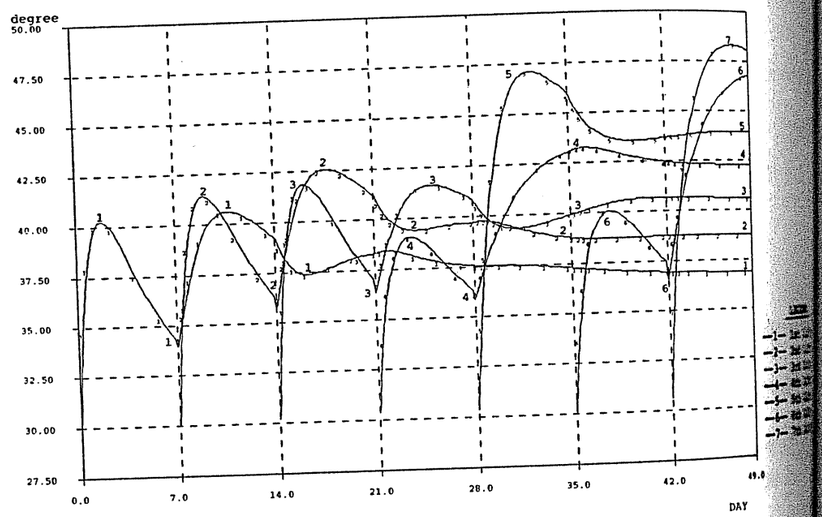
The distribution of the maximum temperature in each element in the model is show in the figure below.

The highest maximum temperature of 48.2 °C occurred in the element located at the lower part of the 7th lift near the symmetrical axis on the 46.7 day after first lift placing which was equivalent to 4.7 day after the placing of this lift . The maximum temperatures in other elements gradually decreased towards both surfaces of concrete block and foundation rock due to heat radiating and transferring condition.

The time history plot for the temperature in vertical section near the symmetrical axis is shown below.



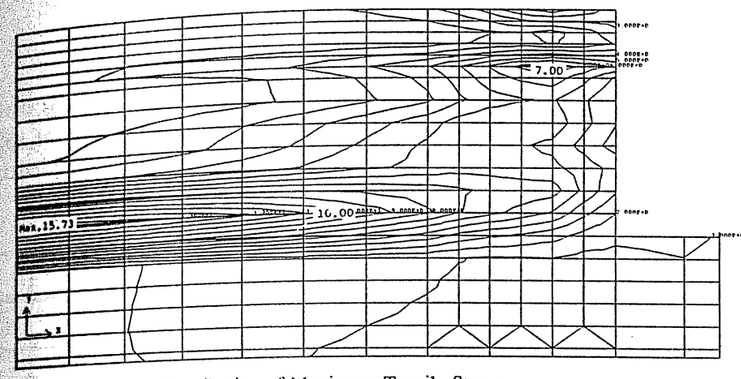
Distribution of Maximum Temperature



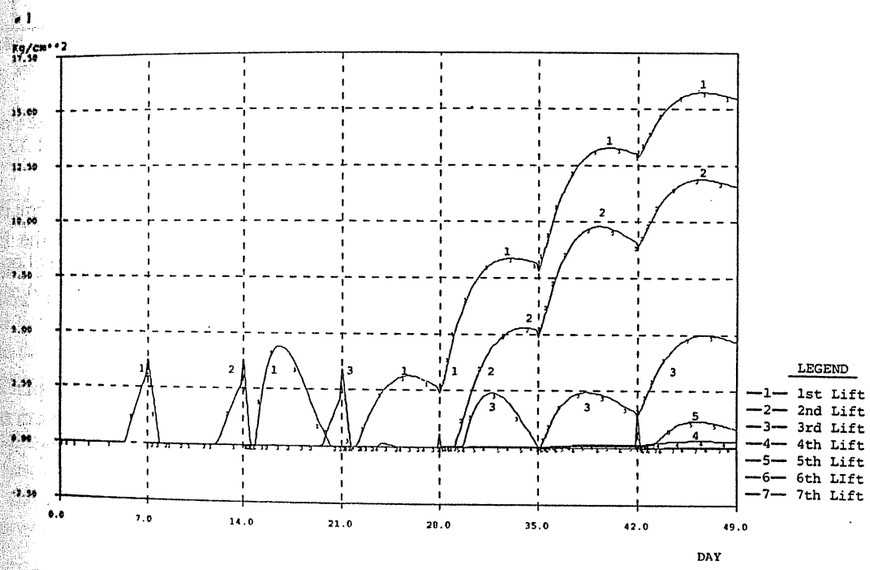
Time History of Temperature

(ii) Thermal Stress Analysis

The distribution of the maximum tensile stress in each element in the model is shown in the figure below. The largest maximum tensile stress of 15.7 kg/cm^2 occurred in an element located at 1st lift near the symmetric axis at 46.7 day after placing this lift and this stress was directed in the horizontal direction. The serial records of tensile stress at the this location are shown below.



Distribution of Maximum Tensile Stress



Time History of Tensile Stress

(iii) Evaluation

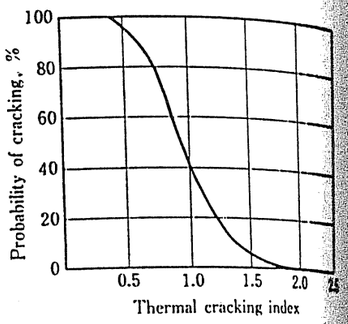
The tensile stress is developed by the degree of the concrete temperature decreasing from the maximum temperature to the temperature at final stable state. On the other hand, the tensile strength of the concrete is developed by the age of the concrete. Therefore, the potential for occurrence of the the thermal crack is generally evaluated by thermal cracking index which is defined by the following equation.

Thermal Cracking Index (T.C.I) = ft (t) / σ (t)

where ; ft (t) : Tensile strength of concrete at t day age
σ (t) : Thermal tensile stress at t day age.

The relationship between probability of cracking and thermal cracking index is shown in the figure below, which was determined based on the experiences of some experiments and actual constructions. The standard value of thermal cracking index is shown below :

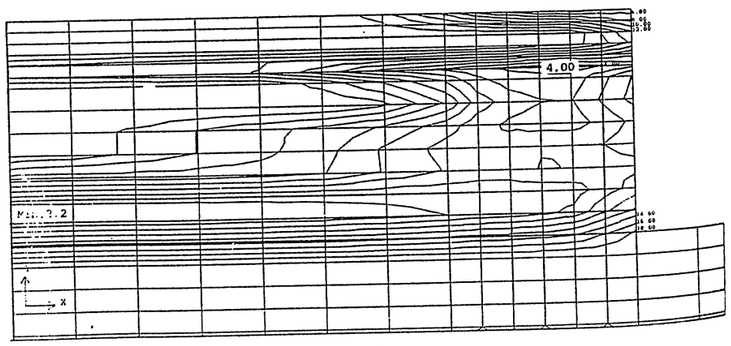
TCI > 1.5 : Cracking is prevented
1.2 < TCI < 1.4 : Width and number of cracking are to be controlled while allowing of cracking.
0.7 < TCI < 1.1 : Not applicable to the above



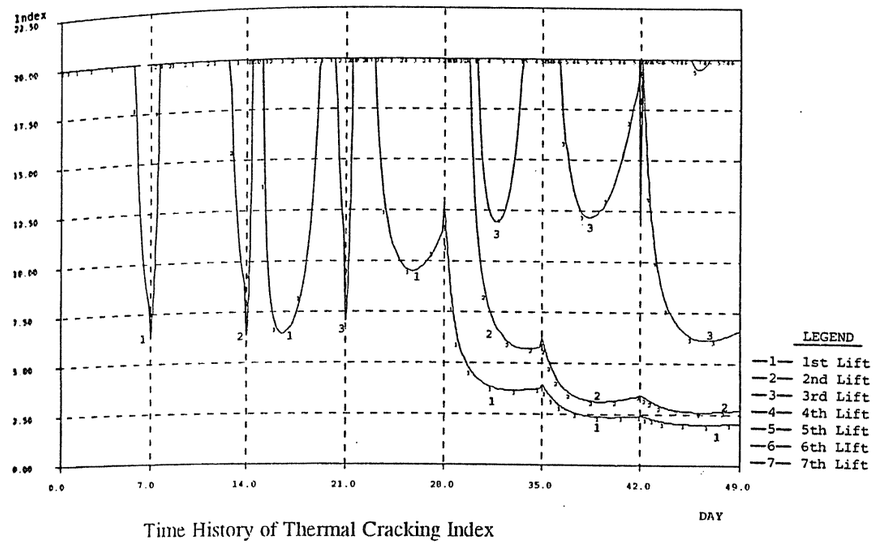
Thermal Cracking Index and Probability of Cracking

Applying the above to the result of the thermal stress analysis, the distribution of the minimum value of thermal cracking index in each element is shown in the figure below. The smallest value of thermal cracking index of 2.2 occurred in the element located at 1st lift near the symmetric axis at 46.7 day after placing this lift, and was evaluated to be sufficiently lower than 1.5.

Therefore, it was reasonably predicted from the above results that concrete placing in 2.0m lift height with a temperature below 30°C caused no detrimental problem on the curing concrete. So it was not necessary to modify the design of the dam.



Distribution of Thermal Cracking Index



Time History of Thermal Cracking Index

(4) Result of Temperature Control

(i) Temperature Control Plan

Based on the calculation results mentioned above, the temperature of the fresh concrete was specified not to exceed 30°C when placing. For that, a cooling plant was provided to cool down the river water, used as the concrete mixing water, by 25°C to 5°C to produce the concrete of temperature 27°C just after mixing in order to conform to the requirement for the temperature of concrete at placing, does not exceed 30°C.

The temperature of fresh concrete was mainly monitored and controlled at the concrete bucket just after pouring. And the temperature change of the concrete placed in a short period and over the long term was also monitored by the thermocouple inserted into each concrete lift and the thermometers embedded in the block 8 respectively.

(ii) Results of temperature control and Monitoring.

The results of temperature control for the fresh concrete and monitoring results of the temperature change in the long term are mentioned in detail in Part I.5.4. Concrete Quality Control and Part 2. Chapter 9. Dam Monitoring and Instrumentation respectively.

Monitoring for the maximum temperature

The results of the maximum temperature before upper lift was placed are shown in the table below. The value of them were lower than the predicted maximum temperature in the analysis.

Maximum temperature of each lift

Lift	EL. (m)	BL. No.	Maximum Temperature (°C)
1	29.5 - 30.5	8	41.5
		12	42.6
2	30.5 - 31.5	10	42.8
		12	41.9
		13	43.4
3	31.5 - 32.5	10	35.9
4	32.5 - 33.5	9	40.6
		11	41.2
5	33.5 - 34.5	9	38.9
		13	41.2
6	34.5 - 35.5	9	40.0

5.2.7 Treatment for Cracks in Dam

The concrete placement of the dam body was commenced on 5th March 1994 at Block 9. After completion of concrete placement, one hair line crack was found 2 days later and two cracks 5 days later in parallel to the dam axis. Block 11 was followed and some cracks were also found. These cracks appeared vertically at almost full width of the block and the width of them was between 1.0 and 1.5 mm. Based on an investigation for side view of these block, it was assumed that the depth of the cracks had penetrated through to each layer.

Based on the above the cracks were considered to be caused by an increment of thermal stress due to the external restraint because cooling and curing were carried out in accordance with the Technical Specifications.

In general, to prevent the thermal cracking, the following three methods are generally applied.

(1) Low cement concrete application
(2) Block partitioning
(3) Cooling

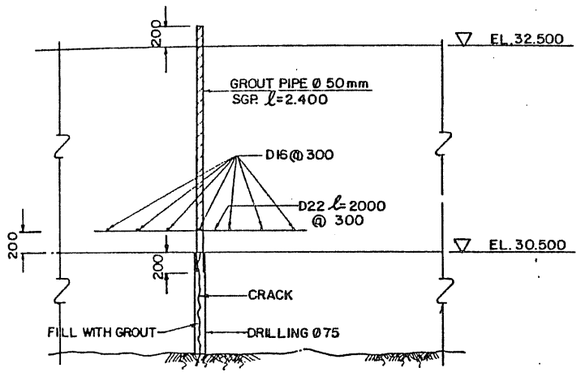
The later two countermeasures of (2) and (3) require provision of the longitudinal joint newly and pipe cooling additionally. Both methods, however, were not selected because they needed a large modification of the dam design and construction methods with and extra additional cost and extension of construction period.

Therefore, the mix proportion of this zoning concrete placed between on the foundation rock and up to 3 m high was redesigned as follows. The concrete on the foundation rock must be rich enough to ensure filling the cracks joints of the foundation rock. Then the following Type A2-R1, which contained more cement than that of concrete Type A2, was designed and placed at 3 layers from river bed. (The first lift of BL.9 and BL.11 had been placed with out concrete Type A1).

Mix Proportion

	A2-R1	A1	A2
Design strength	220 kg/cm²	330 kg/cm²	220 kg/cm²
Water cement ratio	55%	45%	61%
Cement content	184 kg/m³	223 kg/m³	166 kg/m³
Remarks	on foundation	outer	inner

To countermeasure the cracks occurring, steel bar mats as shown below were installed to prevent cracking extending into the next layer, together with grouting into the cracks so that the dam section acts monolithically and the design loads are shared by the full section of the dam body.



Crack Treatment

The final crack investigation results are shown in Table 2.5.1.

The result of temperature measurement is shown in the following table which shows that the maximum temperature rise of concrete was less than the temperature calculated (46°C) and the difference between the maximum temperature rise and the minimum temperature before placing the next layer was also smaller than the design value, 18°C.

At the left abutment blocks were built up on the gentle slope. Cracks were found at the 1st layer of BL.4, 3rd and 4th layer of BL. 6 and 3rd layer of BL. 7.

No cracks were found in all of the layers in the dam except the layers above.

It is essentially undesirable that occurrence of cracks will have a recognizable adverse influence on the dam deformation. However, from the result of monitoring during impounding, no displacement was observed at the dam crest. Therefore these cracks have not influenced on the dam deformation.

Temperature Measurement Results

Block	EL.	Temperature (°C)									Remark
		A1			A2			A3			
		Max	Min	Det	Max	Min	Det	Max	Min	Det	
6	38.5~39.5	46.2	29.3	16.9	42.4	28.0	14.4	-	-	-	Partial Crack
8	29.5~30.5	50.4	31.0	19.4	41.5	27.7	13.8	-	-	-	Crack
9	32.5~33.5	45.9	31.0	14.9	40.6	29.8	10.8	51.5	30.4	21.1	No Crack
	33.5~34.5	46.2	29.4	16.8	38.9	29.1	4.8	52.5	30.6	21.9	"
	34.5~35.5	46.7	27.9	18.8	40.0	29.7	10.3	50.2	29.0	21.2	"
10	30.5~31.5	45.5	27.5	18.0	42.8	32.7	10.1	-	-	-	No crack
	31.5~32.5	-	-	-	35.9	30.4	5.5	-	-	-	"
11	32.5~33.5	38.1	27.3	10.8	41.2	27.0	14.2	54.3	28.5	25.8	No crack
12	29.5~30.5	45.1	28.3	16.8	42.6	30.0	12.6	-	-	-	Crack
	30.5~31.5	44.8	30.0	14.8	41.9	30.3	11.6	-	-	-	No crack
13	30.5~31.5	45.1	31.1	14.0	43.4	31.0	12.4	-	-	19.8	No crack
	33.5~34.5	44.3	28.1	16.2	41.2	30.6	10.6	48.5	28.7		"

Table 2.5.1 Results of Crack Investigation

BL	EL.	Cracks	Concrete Type	Countermeasures
4	47.5	Crack	A2-R1	Reinforcement
6	35.5	No	A2-R1	-
	36.5	No	A2-R1	-
	37.5	No	A2-R1	-
	38.5	Crack	A2-R1	Reinforcement
	39.5	Crack	A2-R1	Reinforcement
7	30.5	Crack	A2-R1	Reinforcement
	31.5	-	A2-R1	-
	32.5	Crack	A2-R1	Reinforcement
8	30.5	Crack	A2-R1	Reinforcement
	31.5	Crack	A2-R1	Reinforcement
9	30.5	Crack	A1	Reinforcement
10	30.5	Crack	A2-R1	Reinforcement
	31.5	No	A2-R1	-
	32.5	Crack	A2-R1	Reinforcement
11	30.5	Crack	A1	Reinforcement
12	30.5	Crack	A2-R1	Reinforcement
13	30.5	Crack	A2-R1	Reinforcement
	31.5	No	A2-R1	-
	32.5	Crack	A2-R1	Reinforcement

Data as of 1st layer to final found cracks

5.3 Foundation Treatment

In order to obtain a stable foundation for construction of a concrete gravity dam, foundation treatment was planned to establish an effective barrier to seepage under the dam and to consolidate the foundation. Consolidation grouting required 12,360 m of drilling through foundation rock with 183 ton of cement injection that commenced in April 1994 and completed in August 1995. Curtain grouting also required 15,464 m of drilling with 505 ton of cement injection that started in August 1994 and completed in July 1996.

5.3.1 Geological Condition of Dam Site

The dam site features an asymmetrical V-shaped valley consisting of a relatively narrow gorge 80 to 85 m wide at the normal river water level, with a moderate slope of 30^0 on the left bank, and a steep cliff of 50^0 on the right bank, as shown in Fig. 2.5.17.

The whole area of the dam site is underlined by bedrock of dacitic tuff of Carboniferous to Early Permian age. There are many discontinuities of rock mass occurring such as joints, cracks, fissures and fracture zones. (See Fig. 1.2.10-1.2.12 Part 1 Chapter 2 Geology). Some of the discontinuities are filled with iron-oxide and/or quarts veins due to hydro thermal alteration. In general, the rock condition is described as hard, massive, homogeneous, sound rock, and or well-jointed dacitic tuff.

A Talus deposit covered the dam right bank and powerhouse foundations, river deposit (sand and gravel) covered the riverbed dam foundation. The thickness of those materials was around 2 to 4 m, while on the left dam abutment, residual soil of dacitic tuff formed an overburden 2 to 3 m thick.

Particular features of the geological condition of the dam foundation was recorded as follows:

(1) Left Bank Area (Refer to Fig. 1.2.10)

The breciated zone occurred in the lower part of the left bank area. They run from the downstream of Block 6 to upstream of Block 7.
Thickness of this breciated zone is 2 m wide.
The main direction of the joint in the area is east to west.
Two locations of groundwater seepage coming out of the foundation were found in Block 2 and 7 near the dam axis.

(2) Riverbed (Refer to Fig. 1.2.11 and 1.2.12)

The breciated zone was found in BL 10 and 11 with orientation of north-east to south-west, dipping towards south-east. Thickness is 2 to 8 m.
Another breciated zone was found in the upstream left bank side developed from the breciated zone in Block 7.

The main direction of the joints in this area is north-east to south-west. Groundwater seepage from the foundation were found at five locations in the upstream part of BL 12, at two locations near the dam axis of BL 11 and BL 12, and at three locations in the downstream part of BL 11 and BL 12.

(3) Right Bank

The discontinuities of rock mass are generally in the form of joints which run in three main directions.
Seepage water was found in the area from upstream to downstream of the BL 14 and BL 15.

(4) Permeability of Dam Foundation

The permeability of the dam foundation generally tended to descend with depth. Then permeability of 2 Lugeon or less was estimated at about 50 m depth in the left abutment, and more than 35 m depth in the riverbed, and more than 20 m depth in the right abutment.

The ground water at H.W.L was confirmed through drilling holes located along the dam axis and was estimated at about 50 m horizontal distance from the original ground surface in the left abutment and 30 m horizontal distance in right abutment in design stage.

The grout curtain was designed and installed to include the area between the dam body and the ground water surface. The dam site Lugeon map is shown in Fig. 2.5.17.

(5) Summary of Dam Site Geology

The foundation of the dam site is hard, massive homogeneous and well-jointed dacitic tuff. The fractured zones exist in the riverbed and the left bank.
The joints in the dam site show generally north-south and east-west direction with steep dip.
Seepage of groundwater were found in the left bank, riverbed and right bank.
Ground water level is high except low water level area in the upper left bank.

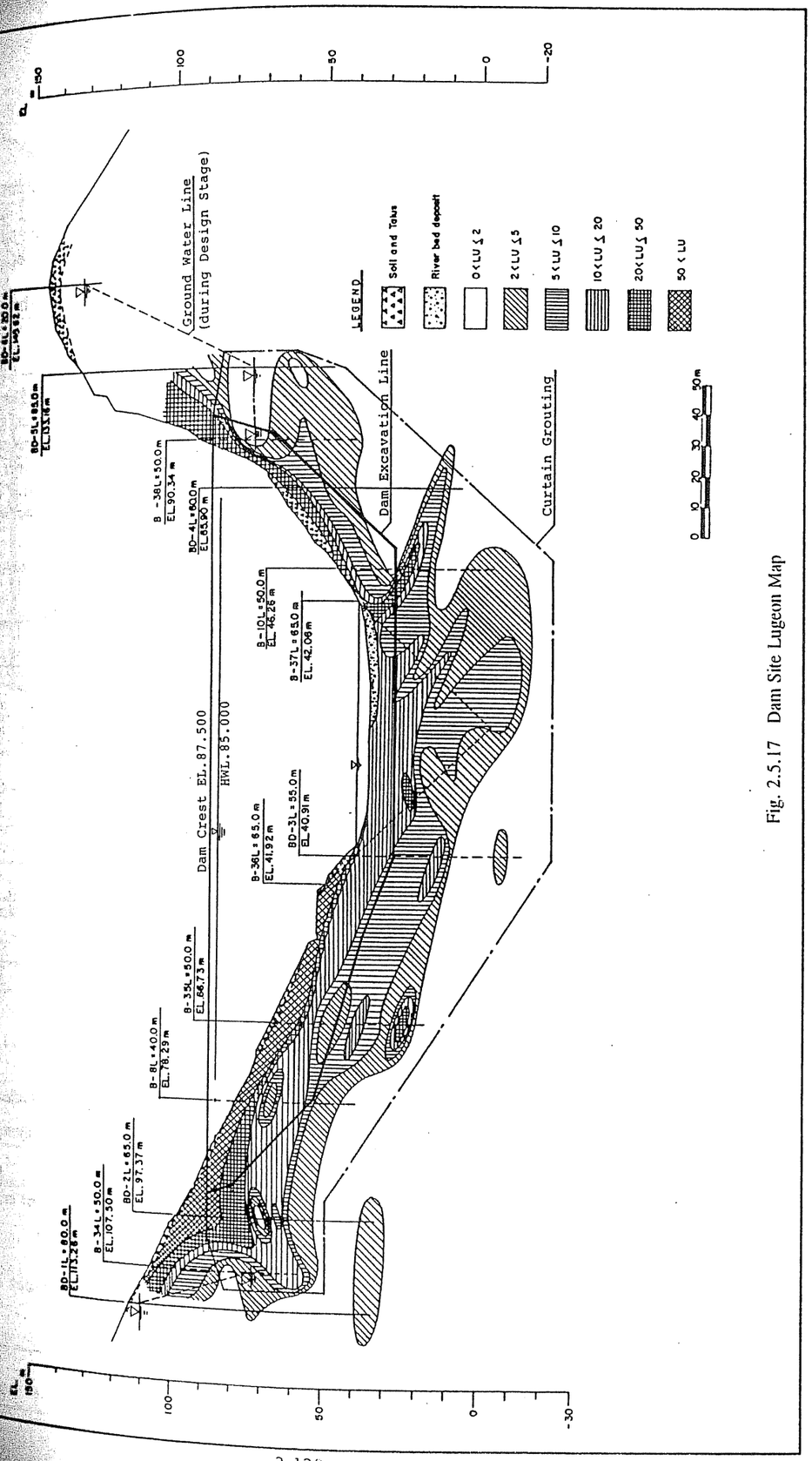


Fig. 2.5.17 Dam Site Lugeon Map

5.3.2 Consolidation Grouting

(1) Grouting Method

Consolidation grouting was carried out from the dam concrete surface after placing concrete 3 m high.
The grouting area covers from the heel to toe of the dam and holes were spaced 3 m apart. Holes were staggered on alternative lines to provide better coverage of the area.

In the riverbed area the drilling depths of holes were 10 m in rocks for the primary, 7m for secondary and tertiary holes. In both the left and right abutments, the drilling depth was 7 m for all holes as shown in Fig. 2.5.18.

The drilling direction was vertical to the concrete surface except at the higher part of both abutments where drilling direction was perpendicular to the rock surface. Grouting method was downstage for 2 stages. Check holes were performed around the stage that showed more than 10 Lu or more than 50 kg/m in cement injection. Target permeability of the final sequence of injection was 10 Lu with probability equal or exceeding less than 15%. Specification of consolidation grouting is summarized in the table below :

Specification of Consolidation Grouting

Work Item	Specification
Drilling	Percussion drilling ϕ66 mm
Drilling depth	Riverbed 10 m for primary 7 m for secondary and tertiary Both banks 7 m for primary and secondary
Water Pressure Test	2.5 kg/cm^2 for 1st stage 4.0 kg/cm^2 for 2nd stage
Grout Mix	W/C = 8, 6, 4, 2, 1 Max.3000 liter
Grouting Pressure	3 kg/cm^2 for 1st stage 5 kg/cm^2 for 2nd stage
Mix at Commencement and Adjusting of Mix	W/C: Less than 20 Lu — Batch num. / Volume; More than 20 Lu — Batch num. / Volume 8: 3 / 498ℓ / - / - 6: 3 / 570 / 3 / 570 4: 3 / 519 / 3 / 519 2: 3 / 558 / 3 / 558 1: 5 / 795 / 9 / 143
Pressure increase rate	Not to exceed 1 kg/cm^2 per minute
Injection rate	Not to exceed 6 liter/m per minute
Suspension and Resumption of Injection	- Pressure and injection rate were steady at total injection volume. - Leakage could not be stopped by caulking or adjusting the packer. For above, injection was suspended and hole was washed. At least 6 hours after suspension, water testing and grout injection resumed.
Concurrent injection	6 m apart at least horizontally.
Completion Criteria	Injection rate less than 0.2 liter/m/min.at designated grouting pressure for 30 minutes
Plugging hole	W/C = 0.5 used

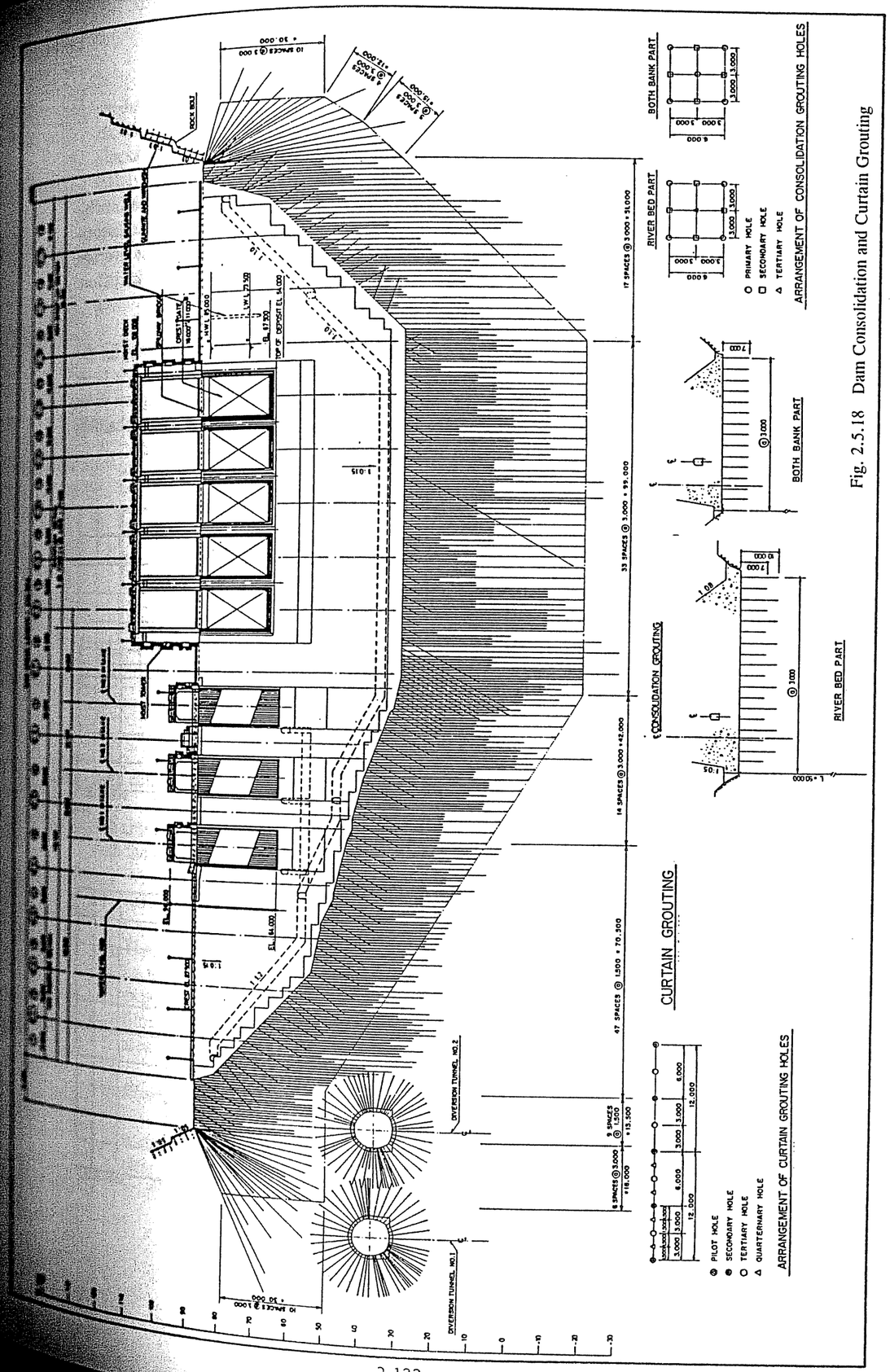


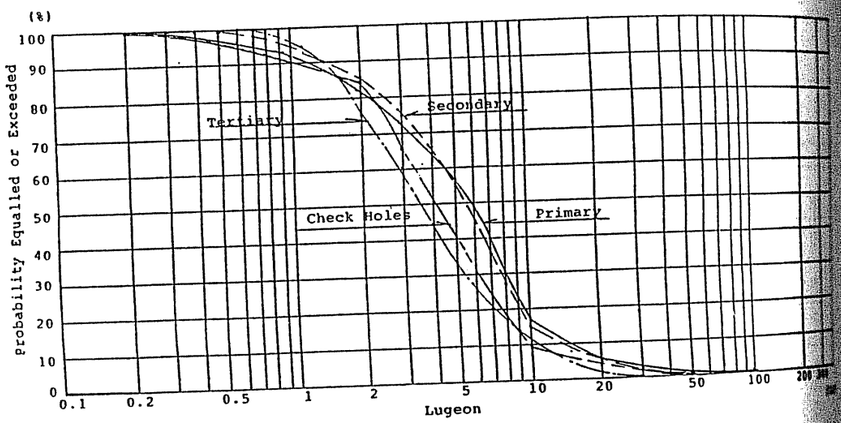
Fig. 2.5.18 Dam Consolidation and Curtain Grouting

(2) Grouting Results

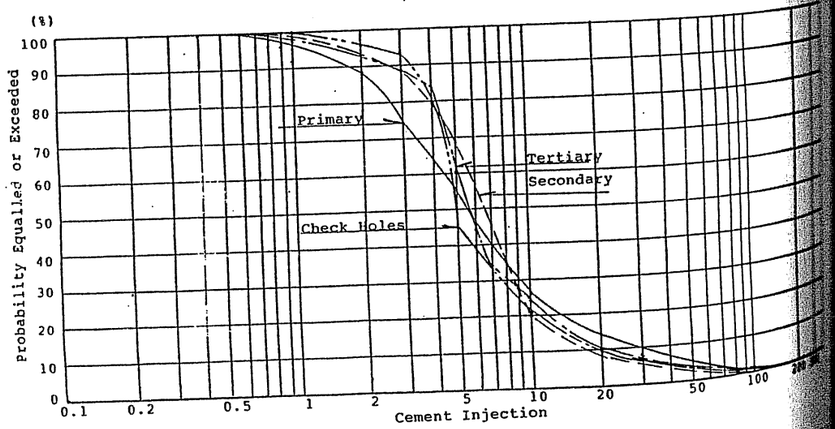
(a) Summary

Table 2.5.2 and Figure 2.5.19 show the results of Lugeon value and grout injection by blocks. In total 183 ton of cement was injected in 3209 stages in holes 12,360 m in total length. The overall average injection rate was 14.8 kg/m. In consequence, the average Lugeon value of the foundation rock was improved and decreased from 7.9 Lu before injection of primary holes to 5.2 Lu for check holes.

The following figures show Lugeon and cement injection rate probability curves for the primary, secondary, tertiary and check holes. The Lu probability curve of check holes shows the probability of 10% equaled or exceeded than the design target Lugeon value (10 Lu) is less than 15% that satisfies our design requirement. For the probability curves of cement injection rate, they decrease as grouting sequence proceeds.



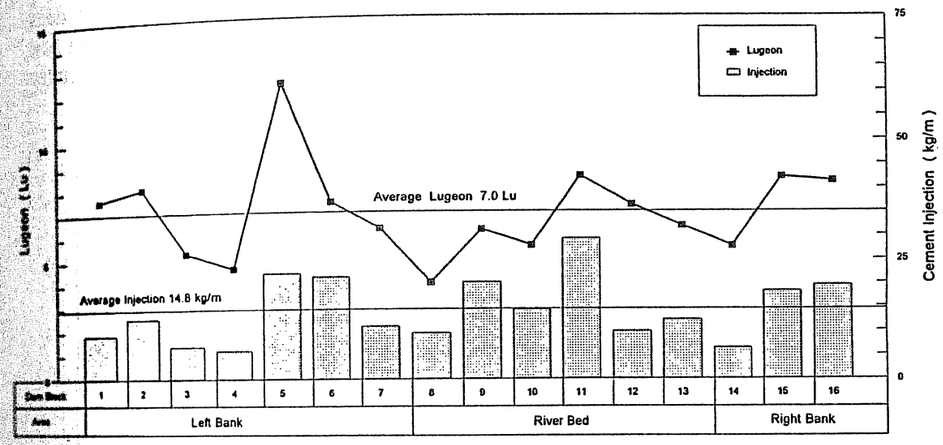
Probability Curve of Lugeon



Probability Curve of Cement Injection

On the other hand, regarding to the grouting results of each block, the following figure shows average Lugeon value and injection rate in each block. In this figure, some blocks show high Lugeon and/or high injection rate and they were considered to be mainly caused by fractured zones and/or joints located in the foundation rock of the dam block. However, these areas were also improved to the same quality as the area without factual zones and joints by consolidation grouting as mentioned below in detail.

Therefore, the overall foundation rock of the dam was improved by the consolidation grouting to satisfy our design requirement.



Lugeon and Injection in each block

(b) Both abutment and Riverbed

The change of Lugeon and injection rate in each block and average value in each area with the progress of grouting sequence is shown in Fig. 2.5.20.

(i) Left Bank

Though reduction of injection rate is not clear, Lugeon value decreases from 8.7 Lu for the primary to 5.1 Lu for the check hole. Generally, injection rate was low except Block 5 and 6 where the fractured zone exists in the downstream area. Block 5 shows very high Lugeon with 22 kg/m of injection in the downstream area. A additional excavation was carried out for removal of the fractured rock.

(ii) Riverbed

Although Lugeon is low for the primary, improvement of Lugeon value is not clear. On the other hand, injection rate decreases remarkably from 25 kg/m for the primary to less than 9 kg/m for the check holes.

In Block 9, high Lugeon and injection holes were found in the downstream area where a number of the joints intersect with each other. In Block 10 and 11, high Lugeon and injection holes were found where the major fractured zone is located in the upstream to downstream direction. In the Block 12, high Lugeon and injection rates occurred in the fractured zone intersected with the joints located in the midstream. In the Block 13, high Lugeon and cement injection holes were found in the dam axis area where seepage of the ground water exists.

(iii) Right Bank

Fig.2.5.20 indicates that improvement of Lugeon is clear, however injection does not decrease. In Block 15, high Lugeon and injection occurred near the test adit where a major joint is located. In the Block 16, high Lugeon and injection holes were found in the upper part of the block above EL.80.0.

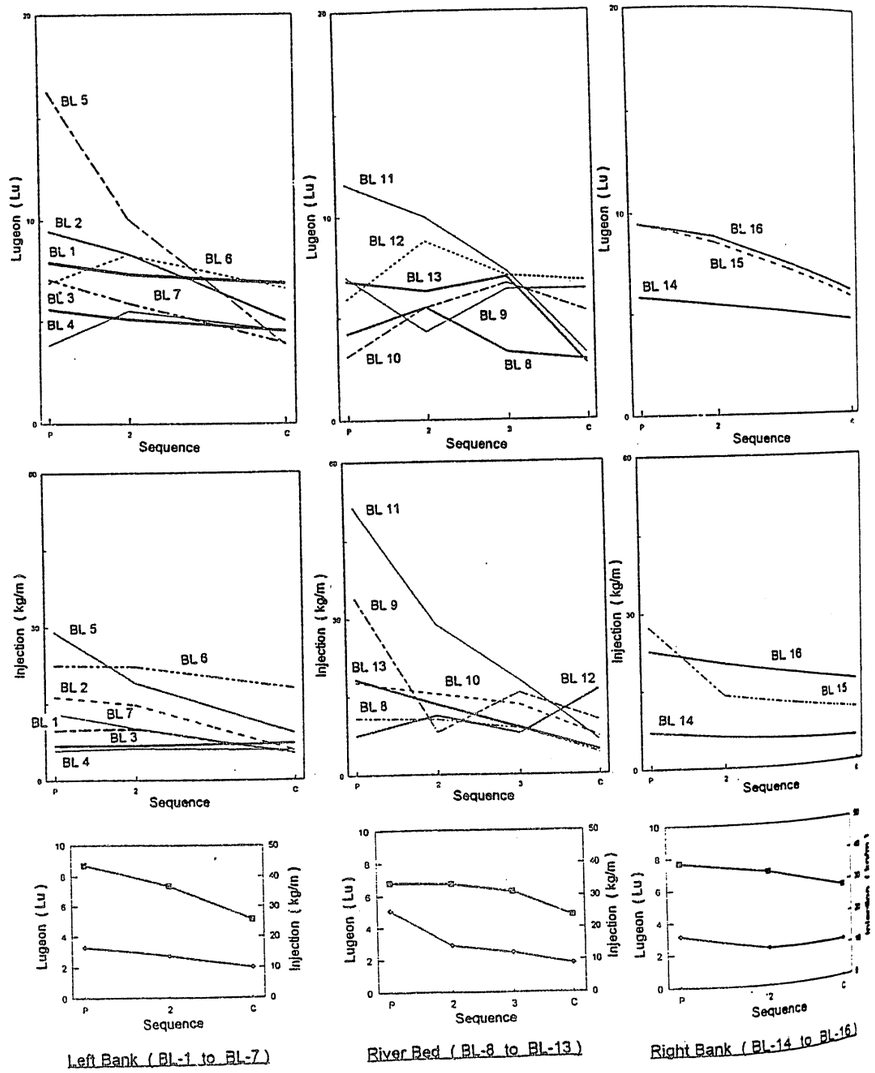


Fig. 2.5.20 The change of Lugeon and Injection Rate

Table 2.5.2 (2) Result of Consolidation Grouting
(Left Bank Zone)

Block No	Sequence of Injection	No Of Stages	Drilling Length (m) Concrete	Drilling Length (m) Rock	Aveg. Lugeon	Total Grout Injection (kg)	Aveg. Injection (kg/m)
1	Primary	24	30,20	85,00	7,94	824,79	9,70
	Secondary	22	29,80	78,00	7,34	780,89	10,01
	Check Hole	4	15,00	20,00	5,86	101,98	5,10
	Sub - total	50	75,00	183,00	7,59	1707,66	9,33
2	Primary	52	72,80	182,00	9,48	2.960,75	16,27
	Secondary	52	72,80	182,00	8,36	2.669,42	14,67
	Check Hole	29	74,30	145,00	5,00	833,28	5,75
	Sub - total	133	220,40	509,00	8,07	6463,45	12,70
3	Primary	70	103,45	248,50	5,60	1.689,92	6,80
	Secondary	72	107,25	254,80	5,10	1.731,07	6,79
	Check Hole	7	10,80	35,00	4,50	249,66	7,13
	Sub - total	149	221,50	538,30	5,30	3670,65	6,82
4	Primary	60	95,10	210,00	3,80	1.215,17	5,79
	Secondary	59	95,70	203,00	5,50	1.250,57	6,16
	Check-Hole	6	11,50	30,00	4,50	173,84	5,79
	Sub - total	125	202,30	443,00	4,64	2639,58	5,96
5	Primary	113	149,25	380,30	16,30	11.058,79	29,08
	Secondary	109	160,60	378,35	10,10	7.210,39	19,06
	Check-Hole	23	70,90	115,10	3,80	1.050,08	9,12
	Sub - total	245	380,75	873,75	12,37	19319,26	22,11
6	Primary	98	139,65	362,40	6,90	8.129,67	22,43
	Secondary	100	130,75	336,30	8,30	7.452,23	22,16
	Check-Hole	38	75,30	190,00	6,60	3.386,46	17,82
	Sub - total	236	345,70	888,70	7,44	18968,36	21,34
7	Primary	114	153,85	449,00	7,10	5.816,63	12,95
	Secondary	115	157,30	403,00	5,90	4.083,13	10,13
	Check-Hole	19	47,85	95,00	3,90	504,66	5,31
	Sub - total	248	359,00	947,10	6,30	10404,42	10,99

Table 2.5.2 (1) Result of Consolidation Grouting
(All Zones (BL-1 to BL-16)

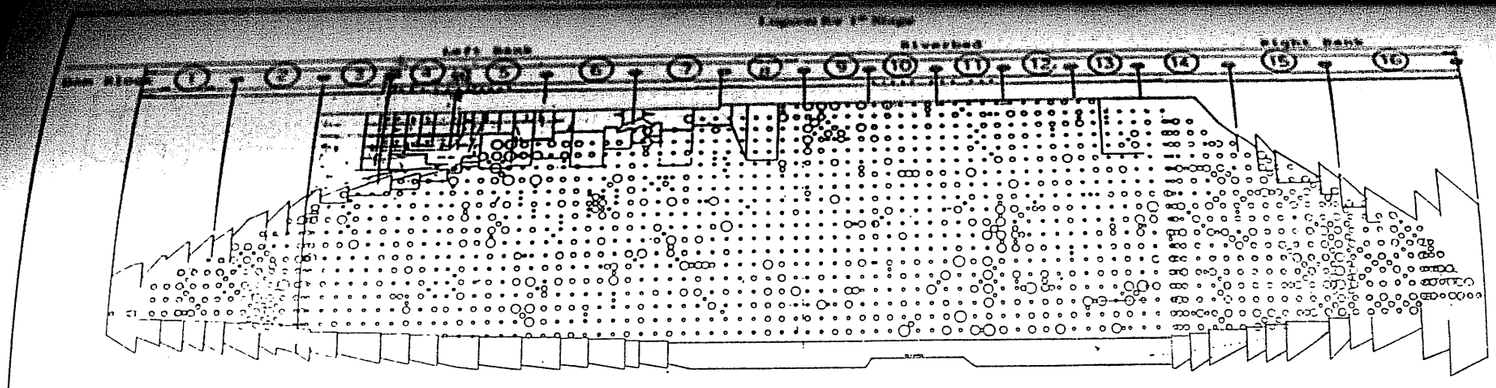
Block No	Sequence of Injection	No Of Stages	Drilling Length (m) Concrete	Drilling Length (m) Rock	Aveg. Lugeon	Total Grout Injection (kg)	Aveg. Injection (kg/m)
I. LEFT BANK ZONE							
	Primary	531	744,30	1 917,20	8,71	31695,72	16,53
	Secondary	529	754,2	1 835,55	7,37	25 177,70	13,72
	Check Hole	126	306,15	630,10	5,11	6 299,96	10,00
	Sub - total	1.186	1804,65	4.382,85	7,73	63.173,38	14,41
II. RIVER BED ZONE							
	Primary	309	445,93	1.530,07	6,79	38.717,04	25,30
	Secondary	269	393,53	946,47	6,71	13.487,04	14,25
	Tertiary	577	839,17	2.030,83	6,26	24578,10	12,10
	Check Hole	123	195,31	604,79	4,77	5.362,56	8,87
	Sub - total	1.278	1873,94	5112,16	6,34	82.145,34	16,07
III. RIGHT BANK ZONE							
	Primary	310	478,05	1.143,60	7,70	18.088,61	15,82
	Secondary	305	470,55	1.076,20	7,05	12.374,12	11,50
	Check Hole	130	413,10	645,00	5,68	7339,98	11,38
	Sub - total	745	1361,70	2.864,80	7,08	37802,71	13,20
IV. GRAND TOTAL BL-1 TO BL-16							
	Primary	1.150	1.668,28	4.590,87	7,92	88.501,37	19,28
	Secondary	1.103	1.618,28	3.858,22	7,12	51.039,46	13,23
	Tertiary	577	839,17	2.030,83	6,26	24.578,10	12,10
	Check-Hole	379	914,56	1.879,89	5,20	19002,50	10,11
	Sub - total	3209	5040,29	12.359,81	7,03	183121,43	14,82

Table 2.5.2 (3) Result of Consolidation Grouting
(River Bed Zone)

Block No.	Sequence of Injection	No. Of Stages	Drilling Length (m)		Aveg. Lugeon	Total Grout Injection (kg)	Aveg. Injection (kg/m)
			Concrete	Rock			
8	Primary	48	72,00	240,00	4,20	2.607,93	10,87
	Secondary	52	78,00	182,00	5,50	1.922,00	10,56
	Tertiary	102	153,00	357,00	3,30	3.152,02	8,83
	Check Hole	10	15,00	50,00	2,90	202,82	4,06
	Sub - total	212	318,00	829,00	4,02	7884,77	9,51
9	Primary	62	89,30	300,70	7,00	10.215,72	33,97
	Secondary	36	54,20	125,70	4,30	1.008,45	8,03
	Tertiary	96	134,05	335,95	6,40	5.274,69	15,70
	Check Hole	26	40,60	116,40	6,40	1.201,60	10,32
	Sub - total	220	318,15	878,75	6,23	17700,46	20,14
10	Primary	41	58,43	201,57	3,10	3.580,82	17,76
	Secondary	55	79,75	190,25	5,50	2.938,69	15,45
	Tertiary	97	141,81	338,19	6,70	4.538,63	13,42
	Check Hole	14	27,00	70,00	5,30	503,69	7,20
	Sub - total	207	306,99	800,01	5,57	11561,83	14,45
11	Primary	62	86,65	303,35	11,60	15.606,88	51,45
	Secondary	36	50,08	130,02	10,00	3.747,00	28,82
	Tertiary	94	137,11	332,89	7,30	6.022,38	18,09
	Check-Hole	26	41,35	130,65	3,20	840.63	6,43
	Sub - total	218	315,19	896,91	8,48	26216.89	29,23
12	Primary	40	60,00	200,00	5,90	1.491,53	7,46
	Secondary	54	81,00	189,00	8,80	2.135,21	11,30
	Tertiary	96	144,00	336,00	7,10	2.626,33	7,82
	Check-Hole	26	39,50	131,60	6,80	2.133.19	16,21
	Sub - total	216	324,50	856,60	7,27	8386,26	9.79
13	Primary	56	79,55	284,45	6,80	5.214,16	18,33
	Secondary	36	50,50	129,50	6,30	1.736,29	13,41
	Tertiary	93	129,30	330,80	7,00	2.984,05	9,26
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 2.5.2 (4) Result of Consolidation Grouting
(Right Bank Zone)

Block No.	Sequence of Injection	No. Of Stages	Drilling Length (m)		Aveg. Lugeon	Total Grout Injection (kg)	Aveg. Injection (kg/m)
			Concrete	Rock			
14	Primary	151	248,40	590,6	5,84	4.148,07	7,02
	Secondary	153	246,90	537,2	5,44	3.275,16	6,10
	Check Hole	33	119,40	165	4,68	786,44	4,77
	Sub - total	337	614,70	1.292,80	5,54	8.209,67	6,35
15	Primary	89	121,60	301,00	9,48	8.236,40	27,36
	Secondary	84	119,20	294,00	8,55	4.131,31	14,05
	Check Hole	40	112,60	195,00	5,84	2.012,42	10,32
	Sub - total	213	353,40	790,00	8,43	14.380,13	18,20
16	Primary	70	108,05	252,00	9,45	5.704,14	22,64
	Secondary	68	104,45	245,00	8,83	4.967,65	20,28
	Check Hole	57	181,10	285,00	6,14	4.541,12	15,93
	Sub - total	195	393,60	782,00	8,27	15212.91	19,45



Lugeon for 2nd Stage

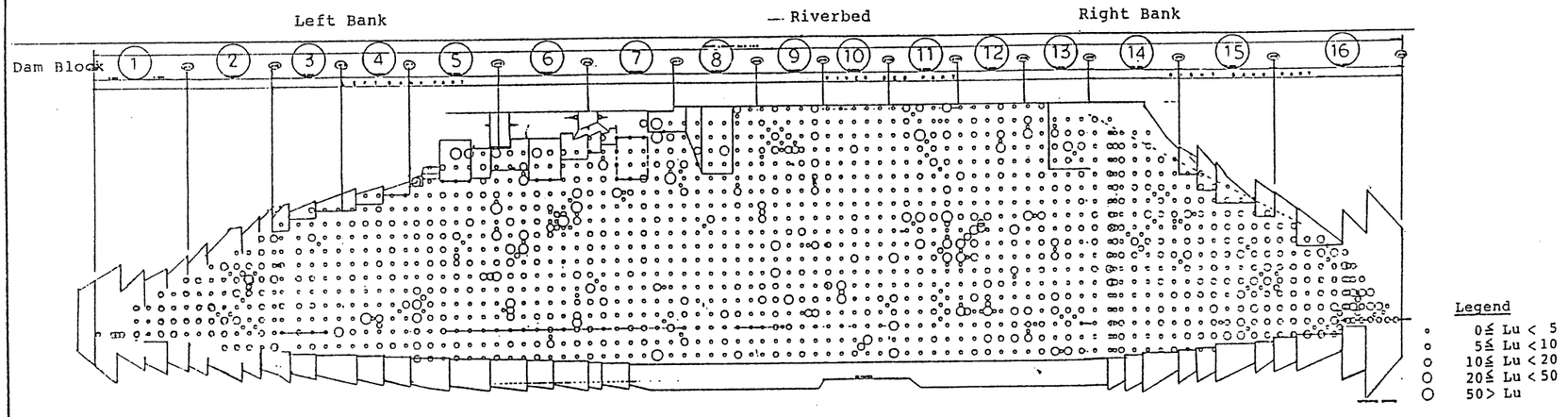
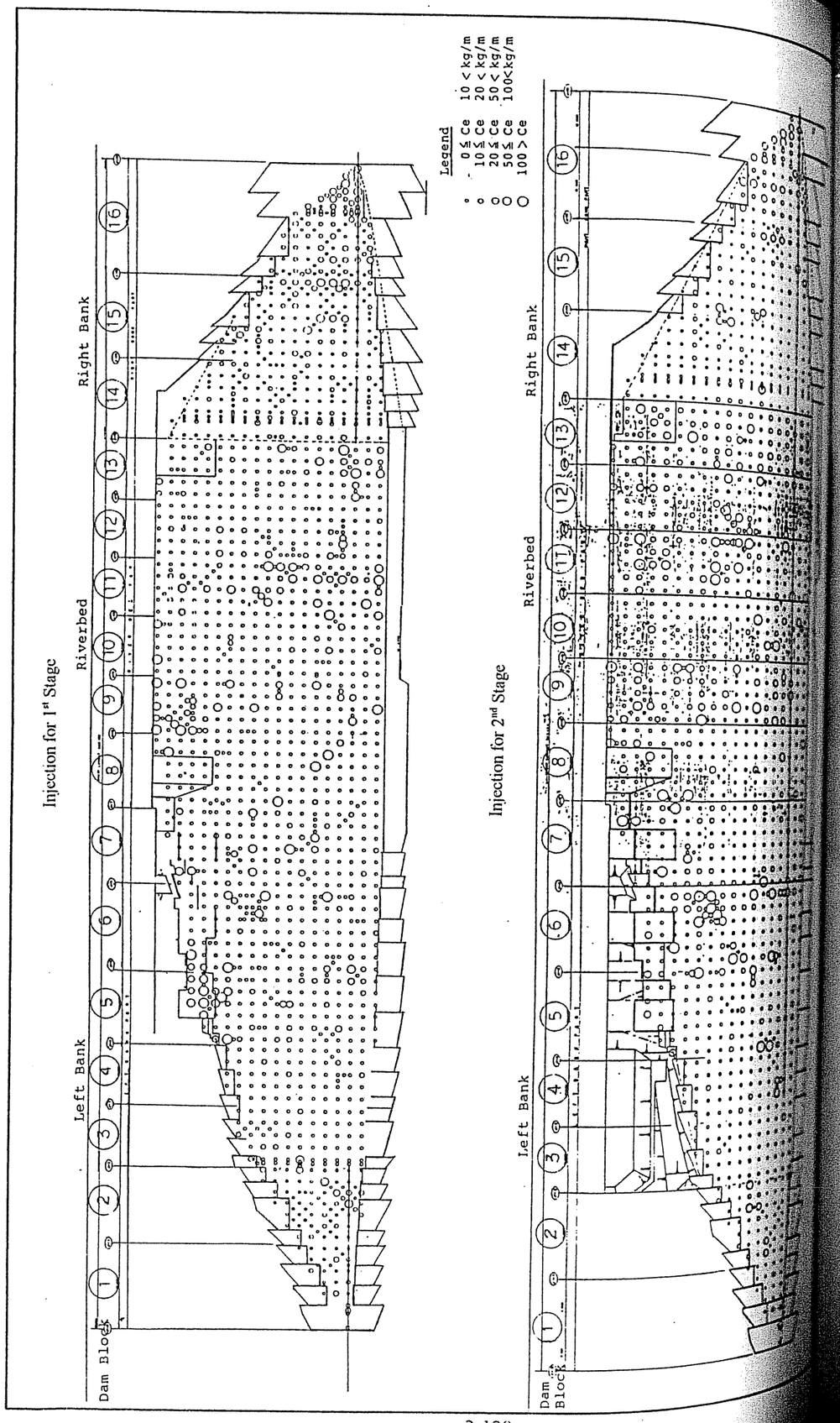


Fig. 2.5.19 (1) Results of Consolidation Grouting (Lugeon)



5.3.3 Curtain Grouting

(1) Grouting Method

A single line of curtain grouting was done from the upstream footing of the dam after placing dam concrete for 10 m above the footing, as shown in Fig.2.5.18.

Drilling depth in the rock was decided to be 50 m which covers the area having permeability more than 2-5 Lugeon and the ground water level. Grouting method for curtain grouting was split spacing with downstage method. The primary holes are located at every 12 m, and core sampled. The secondary holes are drilled at the center of the previous sequence holes. No core sampling was taken after primary holes.

Grouting was completed when the final check holes showed the probability of Lugeon equalled or exceeded 5 Lugeon is 15%. The table below shows the specification of the curtain grouting.

Specification of Curtain Grouting

Items	Specification
Drilling	Rotary Drilling ϕ66 mm for primary holes with core sampling ϕ46 mm for general holes (non primary) without core sampling
Permeability Test Water Pressure Test	for primary holes, P = 1 - 25 kg/cm^2 for general holes, P = 2 - 20 kg/cm^2
Grout Mix, Mix at commencement and adjusting of Mix.	Grout mix, mix at commencement and adjusting of mix were modified as shown in the table below in order to increase efficiency of injection for the foundation rock near by the dam.

[Original]

	Less than 10 Lu		More than 10 Lu	
W/C	Batch num.	Volume	Batch num.	Volume
10	6	618	-	-
6	3	570	3	570
4	3	519	3	519
2	3	558	3	558
1.5	3	549	3	549
1	5	795	9	1431

[Modified]

	Less than 10 Lu		More than 10 Lu	
W/C	Batch num.	Volume	Batch num.	Volume
10	6	618	-	-
8	3	498	3	498
6	3	570	3	570
4	3	519	3	519
2	3	558	3	558
1	5	795	9	1431

Items	Specification
Grouting Pressure	Grouting pressure for each stage was modified as shown in the table below in order to increase efficiency of injection for the foundation rock near by the dam. Stage No. / Grouting Pressure (kg/cm^2) Original / Modified 1: 5 / 7 2: 7 / 10 3: 10 / 15 4: 15 / 20 5: 20 / 25 6: 25 / 25 7: 25 / 25 8: 25 / 25 9: 25 / 25 10: 25 / 25 If critical pressure was apparent during water testing, the grouting pressure was to be increased to 2 kg/cm^2 above critical pressure.
Pressure increase rate	Not to exceed 1 kg/cm^2 per minute
Injection rate	Not to exceed 6 liter/m per minute
Suspension and Resumption of Injection.	- Pressure and injection rate were steady at total injection volume. - Leakage could not be stopped by caulking or adjusting the packer. For above, injection was suspended and hole was washed. At least 6 hours after suspension, water testing and grout injection resumed.
Concurrent Injection	At least 6 m apart horizontally only if there was 10 m vertical separation between injecting stages in the adjacent hole.
Criteria for additional holes.	The following three criteria were adopted to carry out additional holes : (1) If any stages continuously exceed the objective Lugeon, 2 Lu, for more than three stage. (2) If Lugeon value of any stages exceed 4 Lu. (3) If probability of equals or exceeds for the objective Lugeon does not exceed 15% after final sequence of injection.
Completion Criteria	Injection rate less than 0.2 liter/m/min. at designated grouting pressure for 30 minutes.
Plugging hole	W/C = 0.5 used

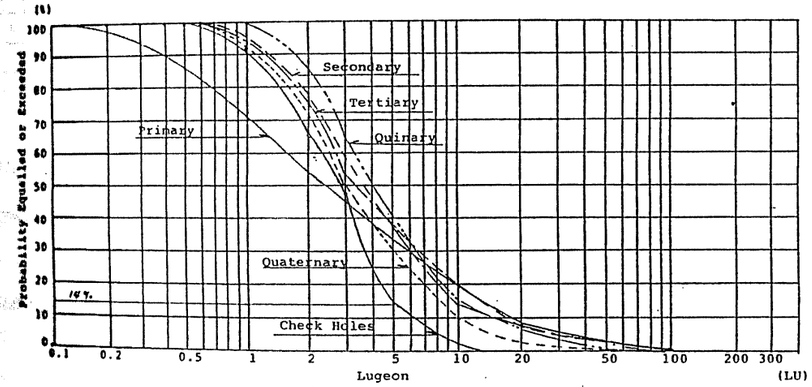
(2) Grouting Results

(a) Summary

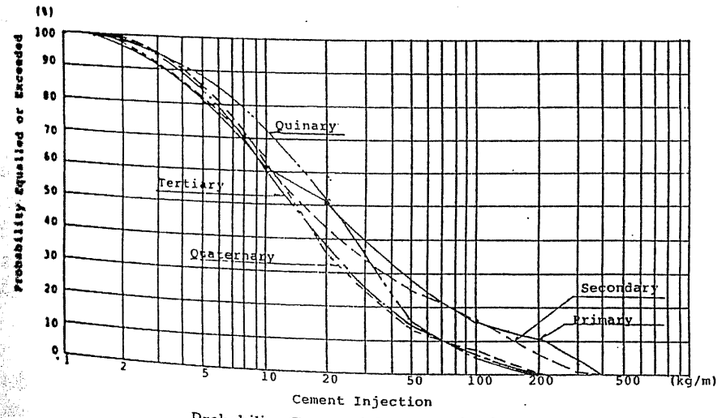
Table 2.5.3 and Figure 2.5.21 show the results of Lugeon value and grout injection by zone. In total 505.68 kg of cement was injected in 3165 stages in holes 15464 m in total length and over-all average injection rate was 32.7 kg/m. In consequence, the average Lugeon value of the foundation rocks was improved and decreased from 7.5 Lu before injection of primary holes to 3.3 Lu of check holes. Following figures indicate Lugeon and cement injection rate probability curve for the primary to check holes and Lugeon value decreased from the primary to the Quinary for the permeability over 3 Lu.

Especially Lugeon more than 5 Lu shows remarkable improvement. The probability equaled or exceeded 5 Lu is 14% that satisfies the permeability required for the curtain grouting.

Regarding to injection, injection rate decreased as grouting sequence proceeds.

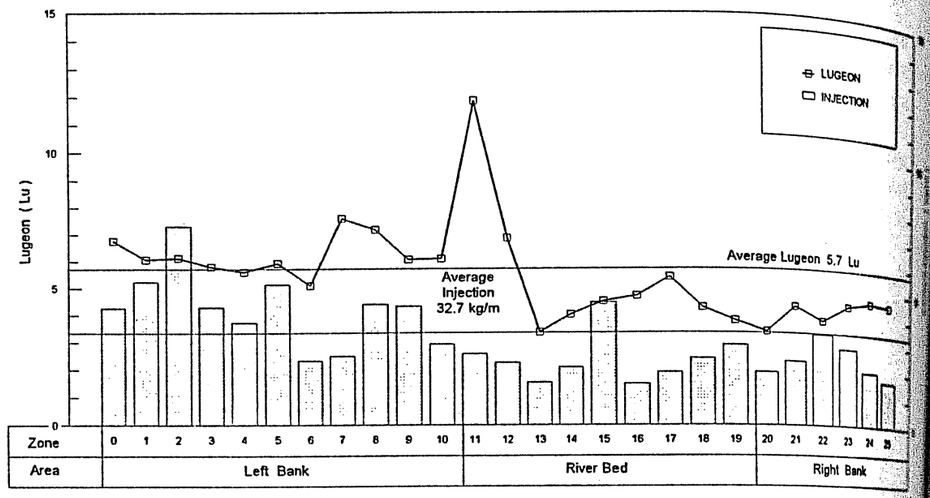


Probability Curve of Lugeon



Probability Curve of Cement Injection

On the other hand, regarding to the grouting results of each zone, the following figure shows average Lugeon value and injection rate in each zone and generally injection rate was not high in all areas except the left abutment.



Lugeon and Injection in each zone

The upper part of the foundation showed high permeability and required additional holes.

The ground water level was high in the river bed and right bank. In the left abutment the ground water level was slightly low, however grout was sufficiently injected.

(b) Both abutment and Riverbed area

The change of Lugeon and injection rate in each area with progress of the grouting sequence is shown in Fig. 2.5.22.

(i) Left Bank

Generally Lugeon and injection rate are high in primary and secondary holes, however subsequent holes shows remarkable reduction in both Lugeon and injection. The high Lugeon and injection areas were located from 20 m to 30 m below the surface. Injection rate of the primary holes in the left bank reaches 100 kg/m which decreases to 41 kg/m in the quinary holes. An average Lugeon shows 9.5 Lu in the primary holes and declines to 4.9 Lu in the quinary holes.

The additional drilling and injection below the design line was mainly done to inject high Lugeon areas at two locations ; one was above the No.1 diversion

tunnel and the other was the right hand side of the No.2 tunnel as shown in Fig.2.5.21 (1), (2). By these additional drilling and grouting, the foundation rock in these areas was thoroughly injected. As shown in Fig.2.5.21 (3), (4) the records of the additional grouting show a large number of stages exceeding 50 kg/m.

(ii) Riverbed

In the riverbed zones, permeability and injection were lower than the left bank, except the G11 zone where a number of seepage holes were found during grouting.

Referring to Fig.2.5.22, the Lugeon value decreases from the secondary hole to check holes, while injection is low and does not decrease from the secondary. The river bed area had generally low permeability in the primary holes, therefore the improvement of Lugeon was not significant.

The upper part of the foundation had high permeability, but the lower part had low permeability, as shown Fig.2.5.21 (1), (2). From the rock surface to 15 m deep, Lugeon exceeded 5 Lu while below 20 m Lugeon was almost less than 5 Lu.

In order to reduce permeability, the additional inclined holes were drilled and grouted for the upper part of the riverbed and high Lugeon area of G11, as shown in Fig.2.5.21 (3) and (4).

(iii) Right Bank

In the right bank the permeability and injection rate are generally low ; The average Lugeon is 4.2 Lu and the average injection rate is 23.6 kg/m.

Generally the surface and upper part of the right bank were high Lugeon, however in the lower part, from G20 to G22, high injection holes occurred at the low permeability hole, as shown in Fig.2.5.21 (1), (2). In the bottom part of the G21 and G22, some additional drilling and injection were required to fill the high injection holes which required injection rate from 30 to 100 kg/m. The permeability in this area was low from the primary holes, but did not decrease as shown in Fig.2.5.22. On the other hand, injection rate decreased from 36.9 kg/m for the primaries to 9.7 kg/m for the check holes.

The additional holes show low permeability ; only 3 stages out of 42 stages exceeded 5 Lugeon. Moreover injection was very low, therefore no further grouting was done.

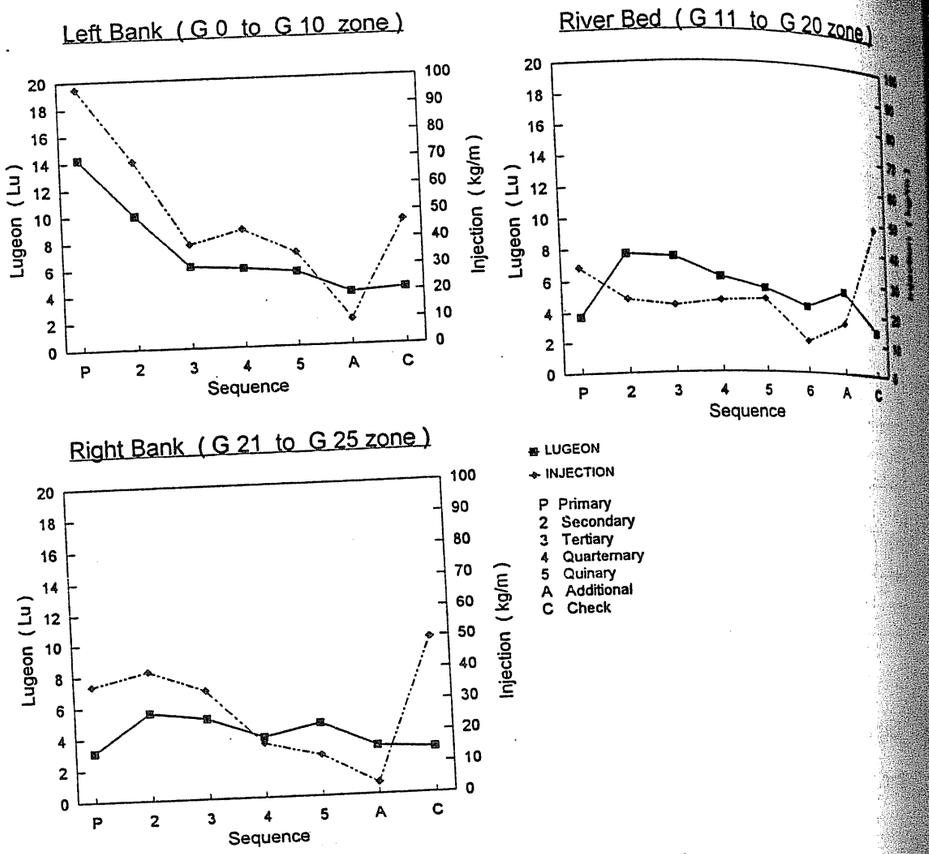
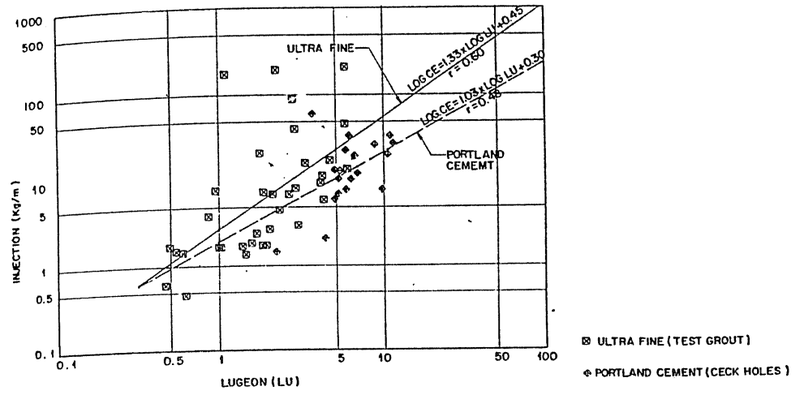


Fig. 2.5.22 Average Lugeon and Injection

(iv) Ultra Fine Cement

The results of Lugeon in the upper part of the dam foundation showed rather high permeability and indicated more additional grouting was needed. After checking the core sampling it was confirmed that fine fissures existed in the upper foundation. In order to fill the fine fissures, Ultra Fine Cement was proposed and tested in G9 in the left bank and G16 in the riverbed area.

The results of this test grouting indicated that Ultra Fine Cement was effective for injection of the fine fissures. Compared with normal Portland cement, the Ultra Fine Cement had higher grout take than normal Portland cement especially in the low Lugeon area as shown in the figure below. After the test grouting. Ultra Fine Cement was used for the quaternary, quinary and additional injections.



Result of Test Grouting for Ultra fine cement

(3) Construction performance data

The summarized tables below are the main performance data such as equipments for this work and the progress rate of drilling and grouting.

Main Equipment for Foundation Treatment Work

Operation	Equipment	Capacity	Quantity Consolidation	Quantity Curtain
Drilling	Hydraulic Crawler Drill	165 kw	1	-
	Pneumatic Crawler Drill	135 kw	2	-
	Air Compressor	18.5 m^3/min	2	-
	Reservoir Tank	7 kg/cm^2	2	-
	Boring Machine	5 - 10 kw	1	11
	Drilling Pump	40 ~ 60 ℓ/min	1	5
Grouting	Mixer & Aditator	400 ~ 600 ℓ	5	9
	Grouting Pump	105 - 120 ℓ/min	5	8

Progress Rate of Foundation Treatment

Description	Consolidation Drilling	Consolidation Grouting	Curtain Drilling	Curtain Grouting
Working Days (shifts)	508	478	762	733
No. of Equipment (man shifts)	784	2978	3559	5021
Ave. Daily Progress (m or kg/unit shift)	22	61	5	101
Max. Daily Progress (m or kg/unit shift)	89	653	14	634
Ave. Monthly Progress (m or kg/unit month)	1124	4274	244	5046
Max. Monthly Progress (m or kg/unit month)	2136	12357	356	11348

Table 2.5.3 (1) Result of Curtain Grouting
(All Zone Zone 0 to Zone 25)

ZONE	SEQUENCE OF INJECTION	NO. OF STAGES	ILLING LENGTH CONC.	ROCK	AVERAGE LUGEON	AVERAGE INJ (kg/m)	CEMENT INJ (kg)
LEFT BANK ZONE G0 - G10	PRIMARY	93.00	27.15	486.60	14.24	97.44	47,415.40
	SECONDARY	96.00	29.80	475.40	10.01	70.13	33,337.93
	TERTIARY	183.00	59.20	916.80	6.15	39.06	35,807.31
	QUATERNARY	345.00	132.40	1,707.70	5.91	44.09	75,286.79
	QUINARY	456.00	249.70	2,531.80	5.56	35.04	88,712.80
	ADDITIONAL	117.00	100.10	576.00	3.91	9.33	5,374.23
	CHECK HOLE	61.00	19.40	154.00	4.18	46.41	7,147.37
	TOTAL	1,351 00	617.75	6,848.30	6.20	42.80	293,081.83
RIVER BED ZONE G11 - G19	PRIMARY	92.00	36.70	459.00	3.71	34.32	15,754.44
	SECONDARY	93.00	36.50	465.00	7.71	24.19	11,246.30
	TERTIARY	181.00	74.50	904.20	7.49	22.07	19,954.08
	QUATERNARY	260.00	152.50	1,305.00	6.19	23.47	30,622.00
	QUINARY	361.00	292.40	1,800.00	5.47	23.96	43,136.56
	SEXTARY	8.00	4.80	40.00	4.37	10.45	418.19
	ADDITIONAL	101.00	229.60	495.00	5.42	16.76	8,297.17
	CHECK HOLE	104.00	50.50	200.00	2.92	48.42	9,683.95
	TOTAL	1,200.00	877.50	5,668.20	5.74	24.54	139,112.69
RIGHT BANK ZONE G20 - G25	PRIMARY	65.00	13.20	329.10	3.12	36.89	12,141.67
	SECONDARY	68.00	19.50	344.20	5.58	41.12	14,154.76
	TERTIARY	121.00	45.40	619.30	5.12	34.64	21,453.49
	QUATERNARY	199.00	99.90	987.10	3.83	17.18	16,954.44
		88.00	140.40	443.00	4.63	12.90	5,715.26
	ADDITIONAL	32.00	56.00	185.00	3.09	3.57	659.70
	CHECK HOLE	41.00	11.80	40.00	2.89	49.47	1,978.65
	TOTAL	614.00	386.20	2,947.70	4.22	22.69	66,891.26
TOTAL G0 - G25	PRIMARY	250.00	77.05	1,274.70	7.47	59.08	75,311.51
	SECONDARY	257.00	85.80	1,284.60	8.00	45.73	58,738.99
	TERTIARY	485.00	179.10	2,440.30	6.39	31.64	77,214.88
	QUATERNARY	804.00	384.80	3,999.80	5.49	30.72	122,863.23
	QUINARY	905.00	682.50	4,774.80	5.43	28.81	137,564.62
	SEXTARY	8.00	4.80	40.00	4.37	10.45	418.19
	ADDITIONAL	250.00	385.70	1,256.00	4.42	11.41	14,331.10
	CHECK HOLE	206.00	81.70	394.00	3.29	47.74	18,809.97
	TOTAL	3,165.00	1,881.45	15,464.20	5.74	32.67	505,252.49

Table 2.5.3 (2) Result of Curtain Grouting
(Left Bank Zone)

ZONE	SEQUENCE OF INJECTION	NO. OF STAGES	DRILLING LENGTH CONC.	ROCK	AVERAGE LUGEON	AVERAGE INJ (kg/m)	CEMENT INJ (kg)
G - 0	PRIMARY	14.00	3.65	89.90	7.82	135.59	12,189.12
	SECONDARY	15.00	4.00	75.70	7.31	44.44	3,364 01
	TERTIARY	28.00	8.00	141.00	6.05	22.98	3,240.46
	QUATERNARY	56.00	16.00	271.00	6.77	33.50	9,077.58
	QUINARY	24.00	8.00	115.50	6.64	15.12	1,746.22
	ADDITIONAL	-	-	-	-	-	-
	CHECK HOLE	-	-	-	-	-	-
	SUB TOTAL	137.00	39.65	693.10	6.77	42.73	29,617.39
G - 1	PRIMARY	7.00	2.00	35.00	14.13	229.98	8,049.27
	SECONDARY	7.00	2.00	33.70	10.99	157.48	5,306.93
	TERTIARY	15.00	4.00	67 40	7.44	72.97	4,918.17
	QUATERNARY	29.00	8.00	134.00	5.24	41.88	5,612.48
	QUINARY	46.00	16.00	225.00	4.77	16.94	3,810.88
	ADDITIONAL	9.00	-	51.00	5.12	13.91	709.65
	CHECK HOLE	7.00	-	17.00	3.97	67.76	1,151.85
	SUB TOTAL	120.00	32.00	563.10	6.08	52.49	29,559.23
G - 2	PRIMARY	7.00	2.00	34.50	23.34	284.22	9,805.62
	SECONDARY	11.00	3.00	42.30	7.14	225.67	9,545.94
	TERTIARY	18.00	7.10	87 60	7.06	110.91	9,715.63
	QUATERNARY	34.00	16.00	167 80	6.17	74.63	12,404.02
	QUINARY	73.00	31.70	281.30	3.97	24.03	6,711.80
	ADDITIONAL	12.00	11.40	60.00	4.85	17.26	1,035.57
	CHECK HOLE	8.00	-	17.00	5.00	71.71	1,219.10
	SUB TOTAL	163.00	71.20	690.50	6.13	73.05	50,437.68
G - 3	PRIMARY	9.00	2.00	46.00	7.52	92.10	4,236.68
	SECONDARY	9.00	3.00	47.70	10.32	53.84	2,568.14
	TERTIARY	18.00	6.50	89.00	6.37	51.77	4,607.39
	QUATERNARY	40.00	16.10	191.90	4.99	53.18	10,471.55
	QUINARY	38.00	32.10	335.00	5.41	55.52	10,548.59
	ADDITIONAL	9.00	11.00	45.00	4.71	8.12	365.27
	CHECK HOLE	5.00	1.50	15.00	3.24	17.77	266.60
	SUB TOTAL	128.00	72.20	769.60	5.80	42.96	33,064.22
G - 4	PRIMARY	8.00	3.40	42.00	17.02	50.04	2,101.62
	SECONDARY	8.00	3.50	39.70	3.88	63.82	2,533.72
	TERTIARY	12.00	3.30	61.00	4.94	54.15	3,303.42
	QUATERNARY	25.00	12.00	129.60	4.00	81 39	10,548.59
	QUINARY	38.00	28.00	190.00	5.41	1.79	339.94
	ADDITIONAL	12.00	10.40	60 00	3.82	10.23	613.62
	CHECK HOLE	5.00	2.50	15 00	4 89	40 91	613 62
	SUB TOTAL	108.00	63.10	537.30	5.61	37.32	20,054.53
G - 5	PRIMARY	5.00	2.50	25 30	11.51	57.43	1,452.87
	SECONDARY	7.00	2.30	37 30	12.08	55.97	2,087.60
	TERTIARY	12.00	4.90	64.60	5.33	18.87	1,270.67
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 2.5.3 (3) Result of Curtain Grouting
(Left Bank Zone)

ZONE	SEQUENCE OF INJECTION	NO. OF STAGES	DRILLING LENGTH CONC.	ROCK	AVERAGE LUGEON	AVERAGE INJ (kg/m)	CEMENT INJ (kg)
G - 6	PRIMARY	7.00	2.00	34.30	51.40	68.84	2,361 13
	SECONDARY	8.00	1.50	31.50	4.83	37.12	1,169.26
	TERTIARY	12.00	5.00	62.90	5.04	11.81	742.78
	QUATERNARY	26.00	10.10	135.90	5 50	17.44	2,370.54
	QUINARY	40.00	21 80	200 00	5.13	23.82	4,764 55
	ADDITIONAL	9 00	8 00	45 00	4 81	8.72	392 62
	CHECK HOLE	5 00	1 80	15 00	3.99	30.84	462.60
	SUB TOTAL	105.00	50.20	524.60	5.11	23.38	12,263.48
G - 7	PRIMARY	7 00	1 20	33.50	8.36	27 67	927 10
	SECONDARY	7 00	3 00	35.40	30.77	48.26	1,708.52
	TERTIARY	16 00	3 50	81.20	6.05	14.89	1,209.01
	QUATERNARY	29.00	8 70	147.90	7.73	29.87	4,417.52
	QUINARY	47 00	16 30	235.00	5.13	23.52	5,527.20
	ADDITIONAL	12.00	8.00	45.00	5.61	13.42	604.05
	CHECK HOLE	6.00	1.40	15.00	5.80	32.34	485.05
	SUB TOTAL	124.00	42.10	593.00	7.58	25.09	14,878.45
G - 8	PRIMARY	8.00	2.00	38.10	9.14	35.76	1,362.64
	SECONDARY	8.00	1.50	40.30	15.02	88.46	3,564.82
	TERTIARY	16.00	5.10	80.50	10.34	45.81	3,687.41
	QUATERNARY	22.00	10.30	110.00	5.64	45.90	5,048.60
	QUINARY	4.00	21.80	220.00	6.29	45.47	10,003.63
	ADDITIONAL	9.00	10.30	45.00	3.78	5.57	250.68
	CHECK HOLE	6.00	2.00	15.00	2.86	18.33	274 95
	SUB TOTAL	73.00	53.00	548.90	7.18	44.07	24,192.73
G - 9	PRIMARY	9.00	3.20	47.30	14.04	74.24	3,511.60
	SECONDARY	9.00	3.00	44.20	8.79	23.52	1,039.66
	TERTIARY	17.00	5.60	86.90	3.62	16.20	1,407.55
	QUATERNARY	29.00	12.60	145.00	6.53	51.43	7,457 82
	QUINARY	54.00	23.30	270.00	6.87	63.05	17,023.16
	ADDITIONAL	27.00	20.30	135.00	2.09	6.23	840.55
	CHECK HOLE	7.00	3.50	15.00	5.28	65.21	978.15
	SUB TOTAL	152.00	71.50	743.40	6.08	43.39	32,258.49
G - 10	PRIMARY	12.00	3.20	60.70	6.10	23.36	1,417.75
	SECONDARY	9.00	3.00	47.60	4.82	9.44	449.33
	TERTIARY	18.00	6.60	94.70	4.95	18.00	1,704.82
	QUATERNARY	29.00	12.00	145.00	6.34	36.55	5,299.35
	QUINARY	48.00	28.10	240.00	7.84	36.07	8,657 16
	ADDITIONAL	9.00	12.00	45.00	2.42	3.55	159 76
	CHECK HOLE	8.00	3.50	15.00	3.30	96.15	1,442.20
	SUB TOTAL	133.00	68.40	648.00	6.11	29.52	19,130.37
	TOTAL	1,351.00	617.75	6,848.30	5.03	42.80	293,081.83

Table 2.5.3 (4) Result of Curtain Grouting
(River Bed Zone)

ZONE	SEQUENCE OF INJECTION	NO. OF STAGES	DRILLING LENGTH CONC.	ROCK	AVERAGE LUGEON	AVERAGE INJ (kg/m)	CEMENT INJ (kg)
G - 11	PRIMARY	10.00	3.30	49.00	2.51	18.38	900.68
	SECONDARY	10.00	3.00	50.00	21.13	26.75	1,337.31
	TERTIARY	20.00	7 10	99.20	17 81	39 66	3,933.88
	QUATERNARY	35.00	12.50	175.00	19 48	37.42	6,549 21
	QUINARY	60.00	28.10	295 00	9 48	24 73	7,294 57
	ADDITIONAL	28 00	28 50	135 00	5 31	8 93	1,205.51
	CHECK HOLE	8 00	9 00	65 00	4 58	19 22	1,249 55
	SUB TOTAL	171.00	91.50	868.20	11.86	25.86	22,470.71
G - 12	PRIMARY	12.00	4 20	60 00	9 72	68 91	4,134.46
	SECONDARY	12.00	4.50	60 00	3.29	8.78	526.95
	TERTIARY	21 00	8 90	105.00	16.84	28.16	2,956.69
	QUATERNARY	36.00	19.30	180 00	4.14	30.01	5,401 17
	QUINARY	32.00	26.90	160.00	2.47	5.33	852.10
	ADDITIONAL	35.00	32.50	175.00	8.66	15.80	2,765.19
	CHECK HOLE	9.00	5.00	20.00	3.95	28.90	577.95
	SUB TOTAL	157.00	101.30	760.00	6.86	22.65	17,214.51
G - 13	PRIMARY	10.00	4.60	50 00	1.38	23.13	1,156.74
	SECONDARY	10.00	4.40	50 00	3.88	13.08	653.81
	TERTIARY	20.00	8.50	100.00	3.43	16.15	1,615.15
	QUATERNARY	34.00	17.70	170.00	3.45	11.31	1,922.16
	QUINARY	56.00	34.30	280.00	3.87	17.56	4,917.42
	ADDITIONAL	3.00	16.50	15.00	2.11	1.83	27.51
	CHECK HOLE	9.00	5.50	25.00	2.08	15.19	379.75
	SUB TOTAL	142.00	91.50	690.00	3.38	15.47	10,672.54
G - 14	PRIMARY	10.00	4.00	50.00	3.35	46.64	2,331.87
	SECONDARY	10.00	4.20	50.00	2.54	7.12	356.24
	TERTIARY	20.00	8.60	100.00	2.93	12.61	1,260.87
	QUATERNARY	16.00	17.20	80.00	4.21	9.81	784.70
	QUINARY	28.00	32.80	140.00	6.14	22.65	3,170.92
	ADDITIONAL	3.00	16.50	15.00	1.78	1.89	28.34
	CHECK HOLE	12.00	5.00	10.00	3.06	144.53	1,445.25
	SUB TOTAL	99.00	88.30	445.00	4.03	21.07	9,378.19
G - 15	PRIMARY	10.00	3.90	50.00	1.58	69.03	3,451.26
	SECONDARY	10.00	4.20	50.00	6.17	114.22	5,711.15
	TERTIARY	20.00	8.90	100.00	4.63	33.49	3,348.55
	QUATERNARY	25.00	18.00	125.00	4 59	28.17	3,521.53
	QUINARY	31.00	35.90	155.00	5.86	34.21	5,302.42
	ADDITIONAL	3.00	16.50	15.00	2.67	3.58	53.74
	CHECK HOLE	12.00	5.50	5.00	2.27	199.92	999.60
	SUB TOTAL	111.00	92.90	500.00	4.52	44.78	22,388.25
G - 16	PRIMARY	10.00	4.60	50.00	6.11	32.34	1,617.01
	SECONDARY	11.00	4.00	55.00	3.08	7.72	424.56
	TERTIARY	20.00	7.90	100.00	9.72	20.44	2,043.54
	QUATERNARY	30.00	16.80	155.00	4.55	19.39	3,004.98
	QUINARY	46.00	33.00	230.00	4.28	20.79	4,780.82
	SEXTARY	8.00	4.80	40.00	4.37	10.45	418.19
	ADDITIONAL	19.00	61 30	85.00	2.84	48.30	4,105.12
	CHECK HOLE	12.00	5.50	5.00	2.22	104.09	520.45
	SUB TOTAL	156.00	137.90	720.00	4.73	14.90	16,914.67

Table 2.5.3 (5) Result of Curtain Grouting
(River Bed Zone)

ZONE	SEQUENCE OF INJECTION	NO OF STAGES	DRILLING LENGTH		AVERAGE LUGEON	AVERAGE INJ (kg/m)	CEMENT INJ (kg)
			CONC.	ROCK			
G - 17	PRIMARY	10.00	3 90	50.00	3.04	9.65	482.35
	SECONDARY	10.00	4.30	50.00	22.16	13 97	698.27
	TERTIARY	20.00	8.70	100.00	4.07	17.12	1,711.57
	QUATERNARY	32.00	17 40	160.00	4.57	21 18	3,389.40
	QUINARY	42.00	36.10	210.00	4.27	21.80	4,577.84
	ADDITIONAL	3.00	16.50	15.00	2.39	2.39	35 89
	CHECK HOLE	11.00	4.50	55.00	2.49	25.64	1,410.10
	SUB TOTAL	**128.00**	**91.40**	**640.00**	**5.42**	**19.23**	**12,305.42**
G - 18	PRIMARY	10.00	4.20	50.00	1.73	6.18	308.93
	SECONDARY	10.00	4.20	50.00	4.99	11.57	578 32
	TERTIARY	20.00	8.30	100.00	4.22	12.78	1,277.70
	QUATERNARY	29.00	17.50	145.00	3.97	22.82	3,308.65
	QUINARY	36.00	35.00	180.00	5.82	34.47	6,204.23
	ADDITIONAL	3.00	15.50	15.00	1.58	1.92	28.77
	CHECK HOLE	12.00	5.50	10.00	2.93	162.39	1,623.85
	SUB TOTAL	**120.00**	**90.20**	**550.00**	**4.30**	**24.24**	**13,330.45**
G - 19	PRIMARY	10.00	4.00	50.00	2.78	27.42	1,371.14
	SECONDARY	10.00	3.70	50.00	3.51	19.19	959.69
	TERTIARY	20.00	7.60	100.00	3.33	18.06	1,806.13
	QUATERNARY	23.00	16.10	115.00	3.51	23.83	2,740.20
	QUINARY	30.00	30.30	150.00	5.73	40.24	6,036.24
	ADDITIONAL	4.00	25.80	25.00	2.43	1.88	47.10
	CHECK HOLE	19.00	5.00	5.00	2.67	295.49	1,477.45
	SUB TOTAL	**116.00**	**92.50**	**495.00**	**3.82**	**29.17**	**14,437.95**
	TOTAL	1,200.00	877.50	5,668.20	4.19	22.87	139,112.69

Table 2.5.3 (6) Result of Curtain Grouting
(Right Bank Zone)

ZONE	SEQUENCE OF INJECTION	NO. OF STAGES	DRILLING LENGTH		AVERAGE LUGEON	AVERAGE INJ (kg/m)	CEMENT INJ (kg)
			CONC.	ROCK			
G - 20	PRIMARY	10.00	3.50	50.00	2.73	19.53	976.43
	SECONDARY	9.00	4.40	46.80	3.71	26.28	1,230.08
	TERTIARY	19.00	9.90	96.60	3.96	23.58	2,277.69
	QUATERNARY	18.00	22.70	93.50	3.16	10.26	959.09
	QUINARY	6.00	20.20	30.00	5.90	8.45	253.42
	ADDITIONAL	4.00	8.40	20.00	2.07	2.88	57.57
	CHECK HOLE	12.00	5.00	10.00	2.50	92.80	927.95
	SUB TOTAL	**78.00**	**74.10**	**346.90**	**3.42**	**19.26**	**6,682.23**
G - 21	PRIMARY	9.00	2.60	46.00	1.16	17.52	805 91
	SECONDARY	12.00	4.50	61.00	6.72	51.97	3,170.42
	TERTIARY	24.00	8.50	117.40	4.65	42.21	4,954.87
	QUATERNARY	27.00	21.00	132.00	4.18	9.89	1,305.27
	QUINARY	21.00	40.20	105.00	5.13	13.22	1,387.77
	ADDITIONAL	6.00	16.70	40.00	3.43	3.70	147.93
	CHECK HOLE	7.00	2.80	10.00	2.72	23.13	231.25
	SUB TOTAL	**106.00**	**96.30**	**511.40**	**4.37**	**23.47**	**12,003.42**
G - 22	PRIMARY	13.00	1.70	66.60	2.14	22.36	1,489.45
	SECONDARY	12.00	3.50	61.70	5.90	65.06	4,014.23
	TERTIARY	21.00	7.30	107.50	5.18	62.62	6,731.24
	QUATERNARY	40.00	17 80	193.20	3.29	23 12	4,467 43
	QUINARY	12.00	30.00	60 00	4.55	9 49	569.65
	ADDITIONAL	6.00	10.20	30.00	2.33	2.99	89.79
	CHECK HOLE	7.00	2.00	10.00	2.86	28.67	286.65
	SUB TOTAL	**111.00**	**72.50**	**529.00**	**3.85**	**33.36**	**17,648.44**
G - 23	PRIMARY	12.00	2.00	61.00	2.32	54.63	3,332.45
	SECONDARY	11.00	2.00	57.50	7.03	66.15	3,803.37
	TERTIARY	21.00	11.20	110.00	7.03	39.73	4,370.77
	QUATERNARY		21.40	218.50	3.99	31.09	6,792.78
	QUINARY		50.00	248.00	4.28	14.13	3,504.42
	ADDITIONAL		20.70	95.00	3.51	3.84	364.41
	CHECK HOLE		2.00	10.00	3.29	53.28	532.80
	SUB TOTAL		**109.30**	**800.00**	**4.42**	**28.38**	**22,701.00**
G - 24	PRIMARY		1.70	54.50	4.87	55.81	3,041.79
	SECONDARY		1.70	57.20	6.49	22.57	1,290.90
	TERTIARY		3.40	94.30	3.77	11 84	1,116.38
	QUATERNARY		6.80	186.90	4.39	12.88	2,407.10
	QUINARY		-	-	-	-	-
	ADDITIONAL		-	-	-	-	-
	CHECK HOLE		-	-	-	-	-
	SUB TOTAL		**13.60**	**392.90**	**4.60**	**20.00**	**7,856.17**
G - 25	PRIMARY		1.70	51 00	5 60	48.93	2,495 64
	SECONDARY		3.40	60 00	3.52	10.76	645.76
	TERTIARY		5.10	93.50	6.18	21 42	2,002.54
	QUATERNARY		10.20	163.00	3.69	6.27	1,022.77
	QUINARY		-	-	-	-	-
	BEATANY		-	-	-	-	-

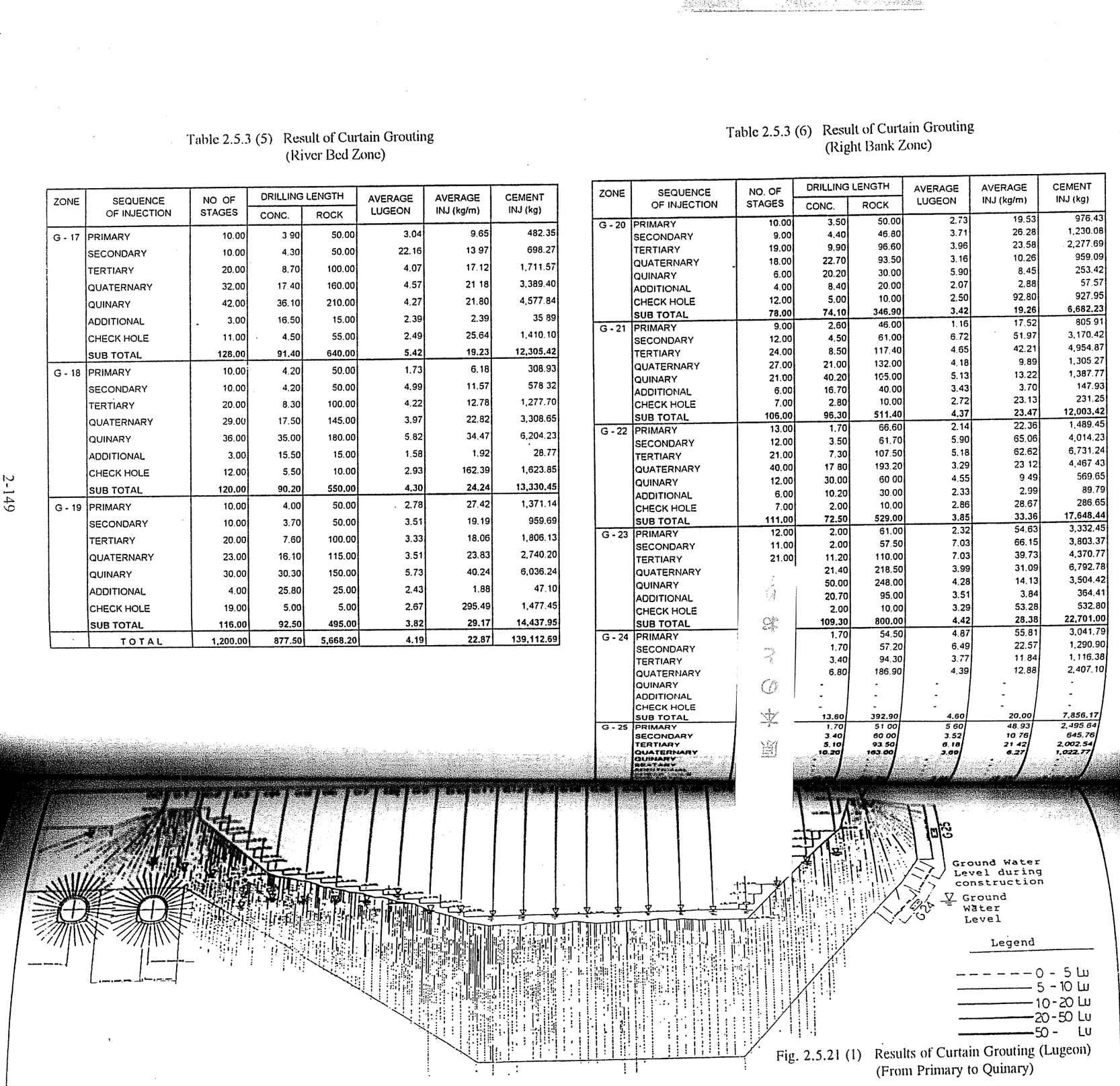


Fig. 2.5.21 (1) Results of Curtain Grouting (Lugeon)
(From Primary to Quinary)

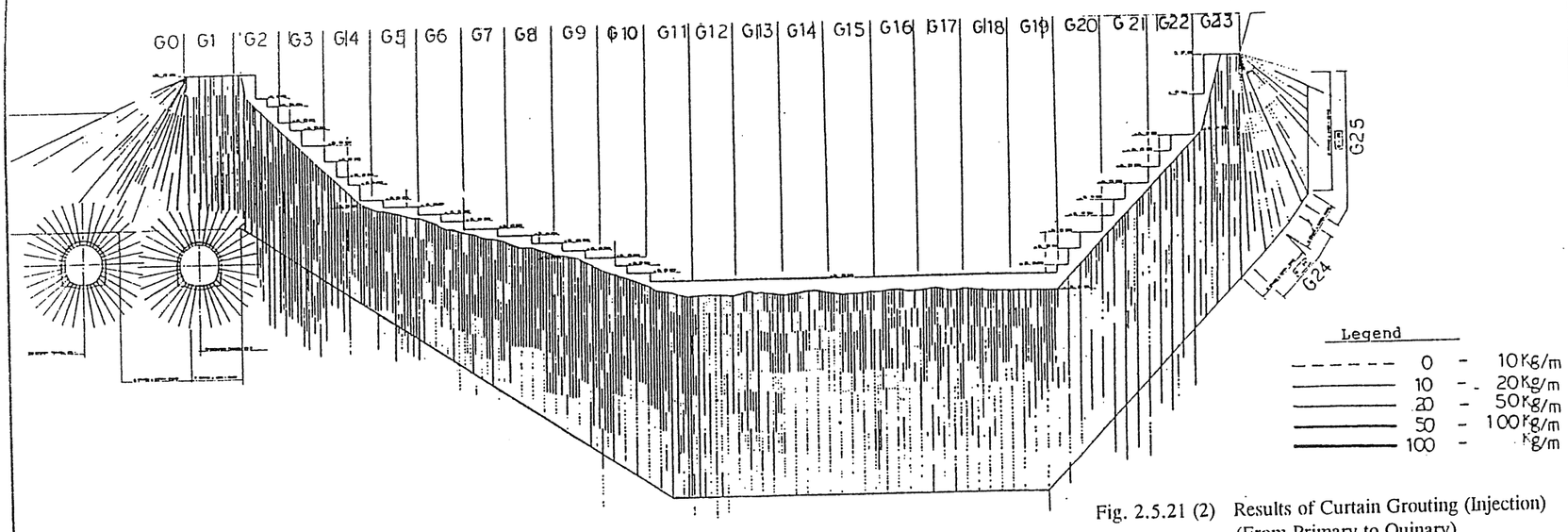


Fig. 2.5.21 (2) Results of Curtain Grouting (Injection)
(From Primary to Quinary)

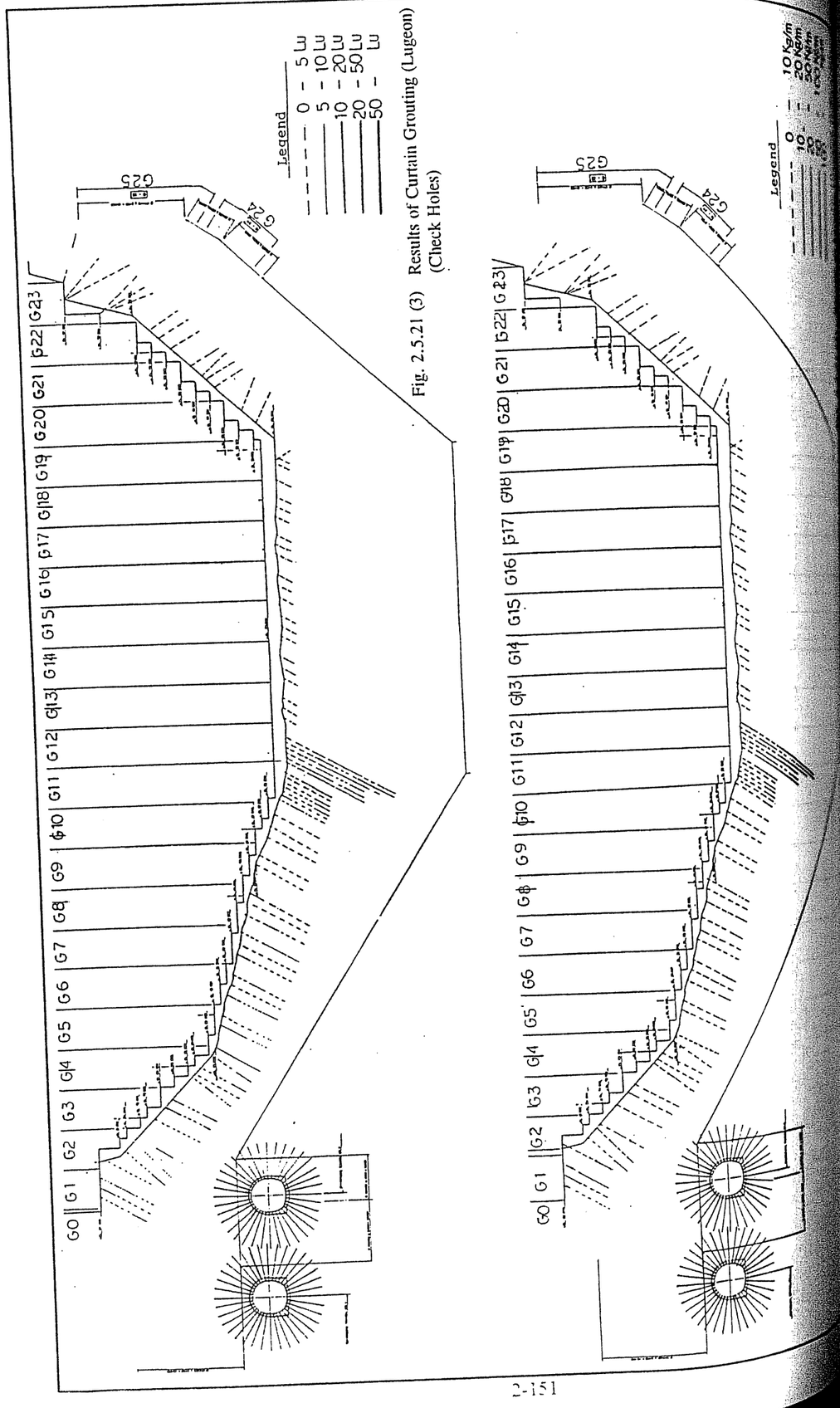
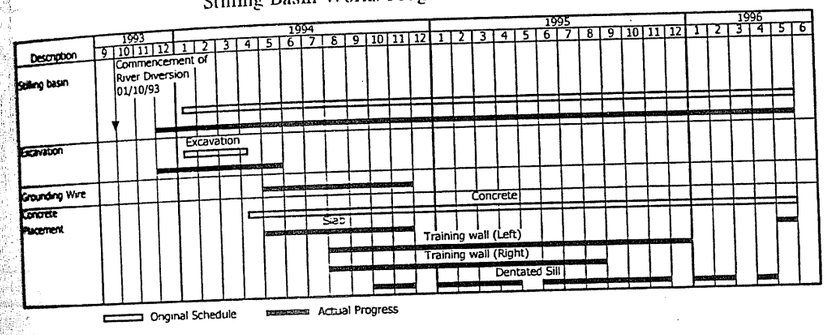


Fig. 2.5.21 (3) Results of Curtain Grouting (Lugeon) (Check Holes)

5.4 Stilling Basin

Excavation of the stilling basin commenced in December 1993 about two months after the original riverflow was diverted into the diversion tunnels on the 1st of October 1993. The excavation took nearly six months and was completed the next year in May 1994. Soon after completion of the excavation, concrete placement of the stilling basin slab started and completed 7 months later in November 1995 except one block remained till March 1996 because it was used for the temporary drainage sump pit. The work schedule is shown below.

Stilling Basin Works Progress



5.4.1 Excavation

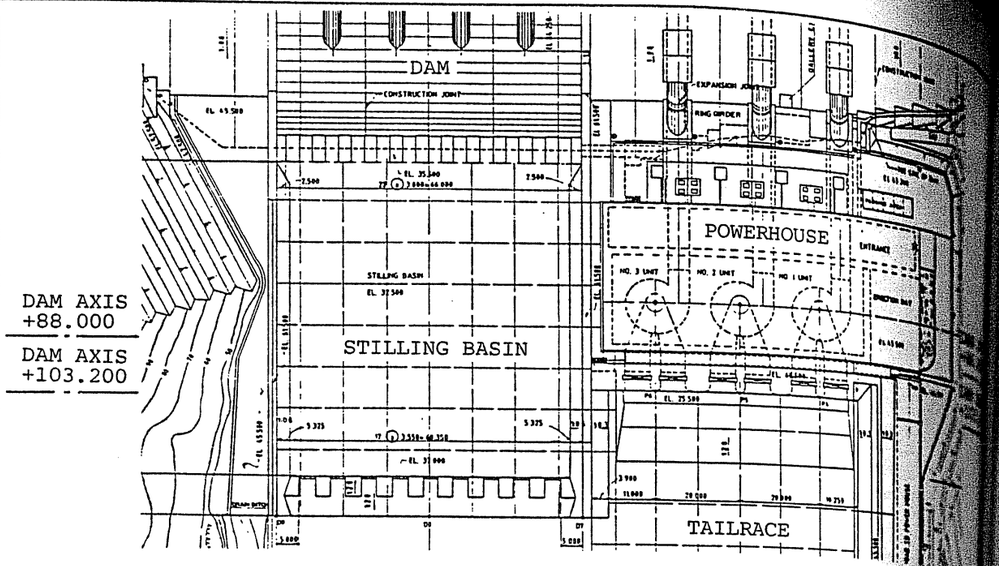
Since sand and gravel were found at the design elevation of EL. 30.5 in the river bed area where foundation rock is characterized by slightly weathered dacitic tuff, the foundation of the stilling basin was excavated further to EL.27.3. The additional excavation below the design line reached 38,000m^3 approximately and backfilled with concrete. The additional excavation area is shown in Fig. 2.5.23. The total volume of excavation was 105,722 m^3 including the additional excavation.

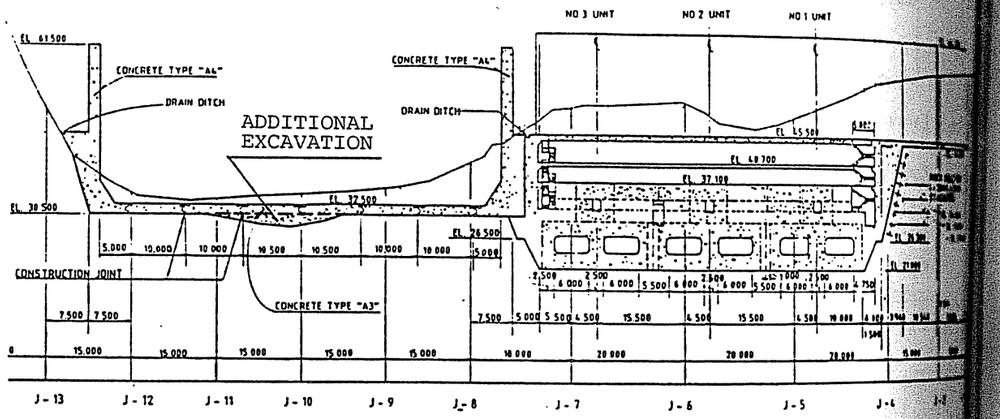
The main equipment used for excavation works, were bulldozer (32 ton), excavator (1.3 m^3), wheel loader (2.4 m^3), dump truck (11,8 ton), hydraulic breaker and crawler drill.

After completion of the excavation work in the work in the stilling basin, split steel pipes and grounding wires were installed on the exposed rock surface by Lot I contractor and Lot IIIB contractor respectively.

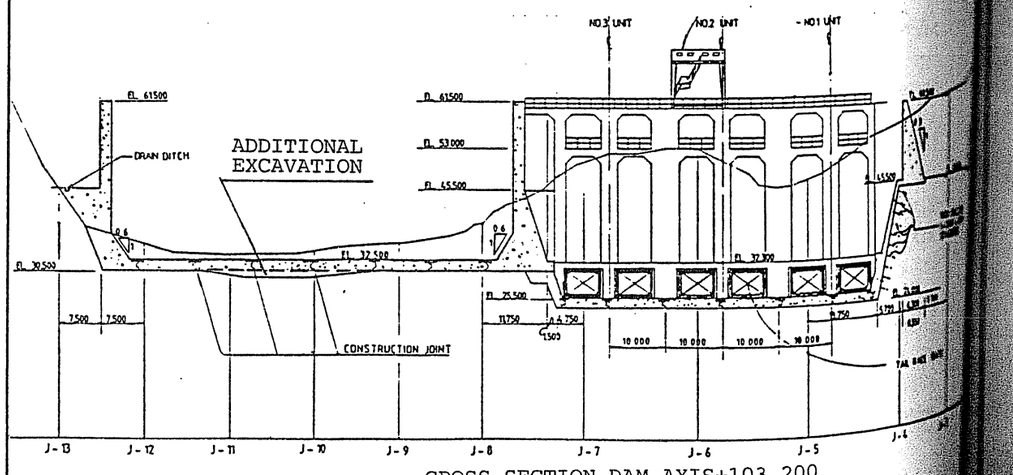
The split steel pipe (Ø 250) was set to reduce the uplift pressure acting below the slab concrete in the stilling basin as shown in Fig. 2.5.24. The layout of the split pipe was planned at a square of 10 m x 10 m in the original design, but the installation layout was modified to collect the water from lower part of the foundation taking into consideration the final surface configuration.

The grounding wire was set on the rock surface to obtain low enough electric resistivity for the electric equipment. The grounding wire was laid on the entire area of the stilling basin slab concrete and connected to the grid in the powerhouse and tailrace areas.



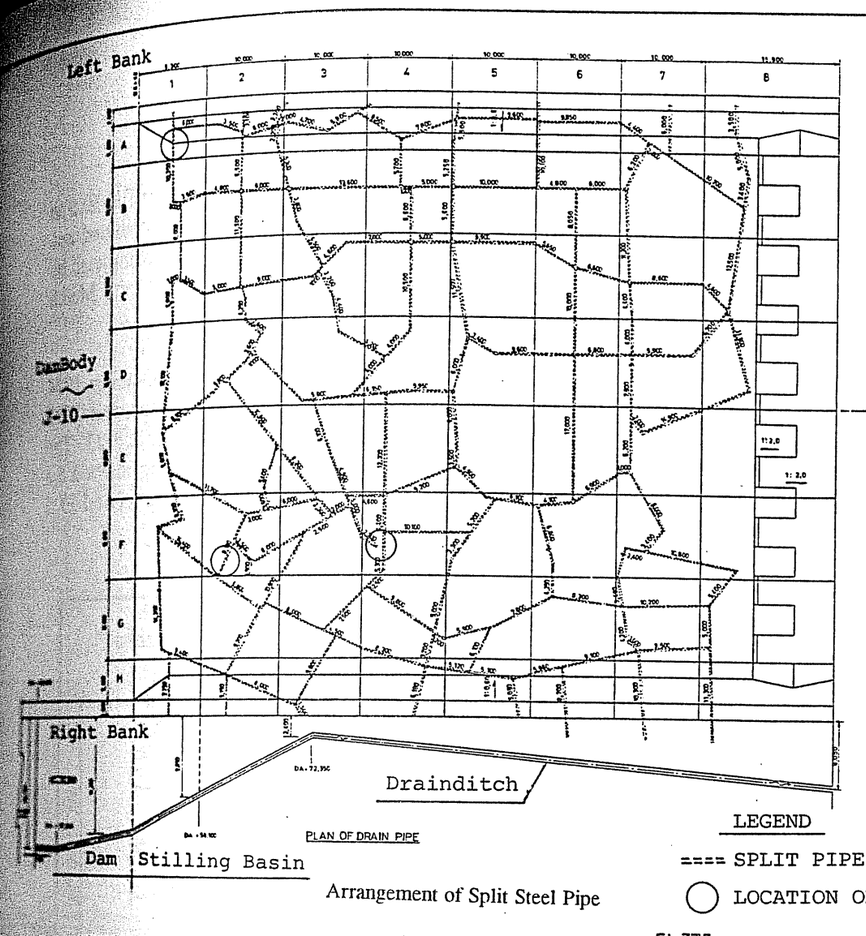


CROSS SECTION DAM AXIS+88.800

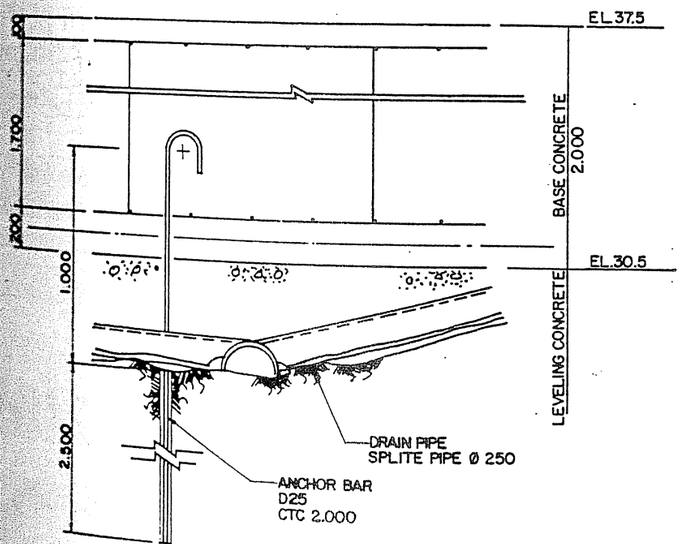


CROSS SECTION DAM AXIS+103.200

Fig. 2.5.23 Additional Excavation Area in Stilling Basin



Arrangement of Split Steel Pipe



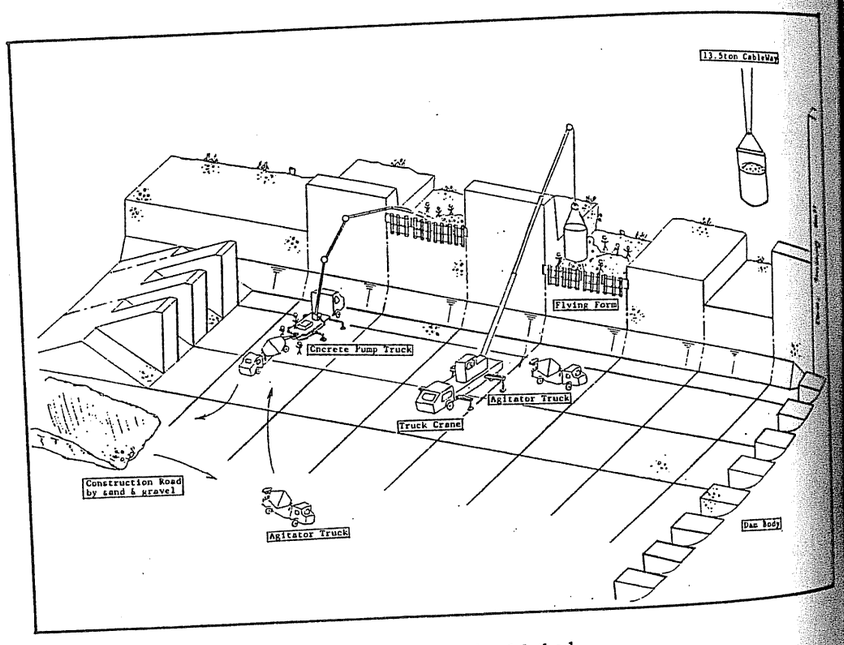
NOTE: GROUNDING RODS AND WIRES ARE TO BE SIPPLIED AND INSTALLED BY OTHERS

Fig. 2.5.24 Installation of Split Steel Pipe

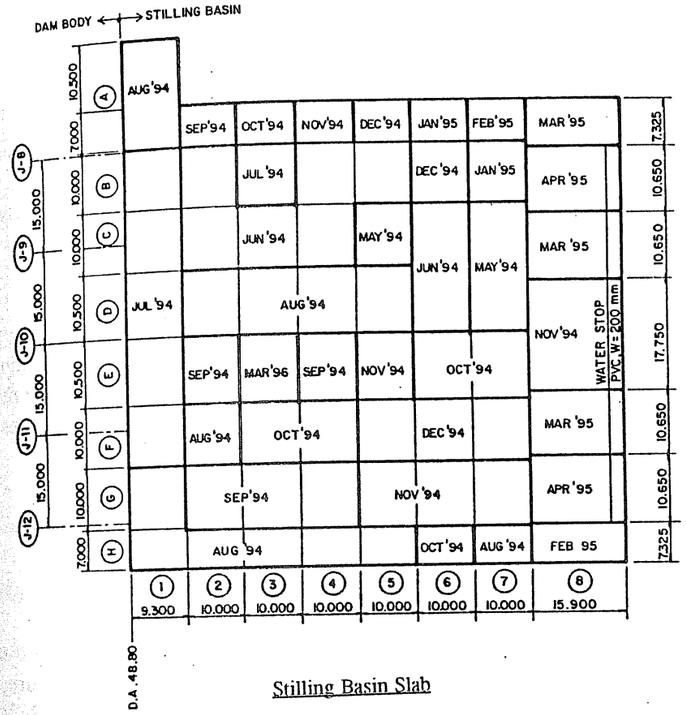
5.4.2 Concrete Placement

Backfill concrete was placed from EL. 27.3 up to the design elevation of EL.30.5 to improve the foundation where the weathered rock and gravel were distributed and accordingly removed. Concrete placement for the stilling basin slab was completed by November 1995 at the dentated sill foundation, but one block remained until March 1996 for the purpose of collecting the seepage in the foundation.

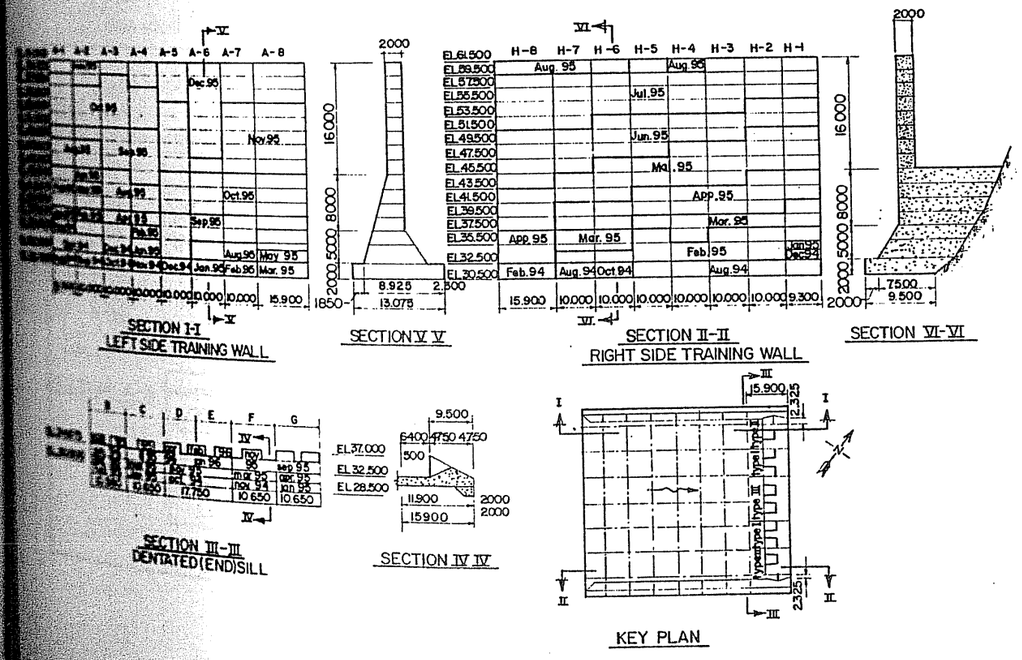
Concrete works for the training wall and dentated sill commenced in October 1994 after partly completing the stilling basin slab and were completed in December 1995 for the training wall and April 1996 for the dentated sill as shown Fig. 2.5.25. The total concrete volume of the stilling basin was 39,035 m^3. Concrete placing method for training walls was chosen using concrete pump, bucket and crane, and bucket and 13.5 ton cableway as shown below.



Concrete Placing Method



Stilling Basin Slab



Training Wall and Dentated Sill

Fig. 2.5.25 Stilling Basin Concrete Monthly Progress

5.5 Spillway Gate Hoist Deck

5.5.1 Concreting

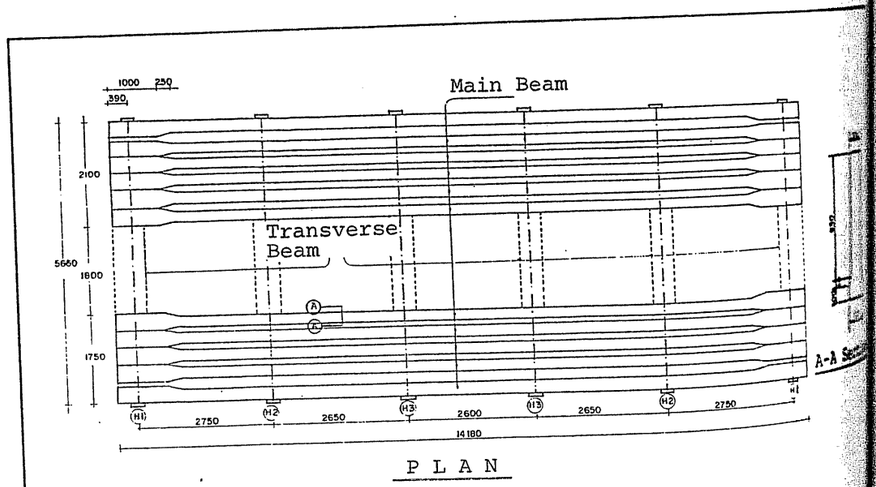
The hoist deck above the spillway gates was installed at the top of the spillway structure for the gate winches with auxiliary equipment. The hoist deck slabs consist of prestressed concrete slab structures with transverse beams and precast concrete panels. The main beam is 14.18 m long and 100 cm deep, as shown in the figure below.

PC beam concreting was commenced on 29th November 1995 and completed by 20th January 1996 for the 55 pieces of PC beams. The PC beams were produced at the upstream job site of the stilling basin. Concrete type was PC-R1 and the result of the compressive strength test at 28 days age were more than 500 kg/cm^2 , as shown in Fig.2.5.26.

Mix Proportion and Compressive Strength of PC-R1

σ_{ck} (Kg/cm^2)	G_{max} (mm)	Slump (cm)	W/C (%)	S/a (%)
500	20	18-24	35	43

After the PC beams were installed at the top of the spillway structure, concrete placement of the transverse beams commenced on 17th February 1995 and were completed by 15th March 1996 for the (6) x (5 span).



PC Beam of Spillway Hoist Deck

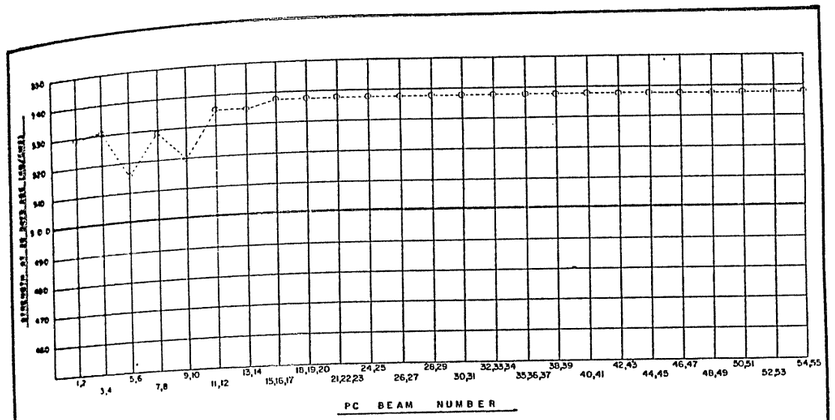


Fig. 2.5.26 PC - R1 Compressive Strength at 28 Days for Hoist Deck

5.5.2 Stressing

(1) PC Main Beam

Stressing work was carried out after the PC beam concrete age became more than 7 days to achieve required minimum concrete strength 400 kg/cm^2. Freyssinet k-200 prestressing jack (RAM area: 318.0m^2, Jack Friction : 3%, Jack Length : 637mm) was used for stressing the upper and lower tendons.

The upper and lower tendons have the following specifications ;

Modulus of Elastity	: 195 KN/mm^2
Area	: 98.71 mm^2
Ultimate Tensile Strength	: 184 KN (190 kg/mm^2)
Yield Strength	: 166 KN (171 kg/mm^2)
Max.force at Anchorage	: 0.7 x 190 = (133 kg/mm^2)

The stressing of the upper and lower tendon required the following pressure and elongation.

Stressing of Tendon

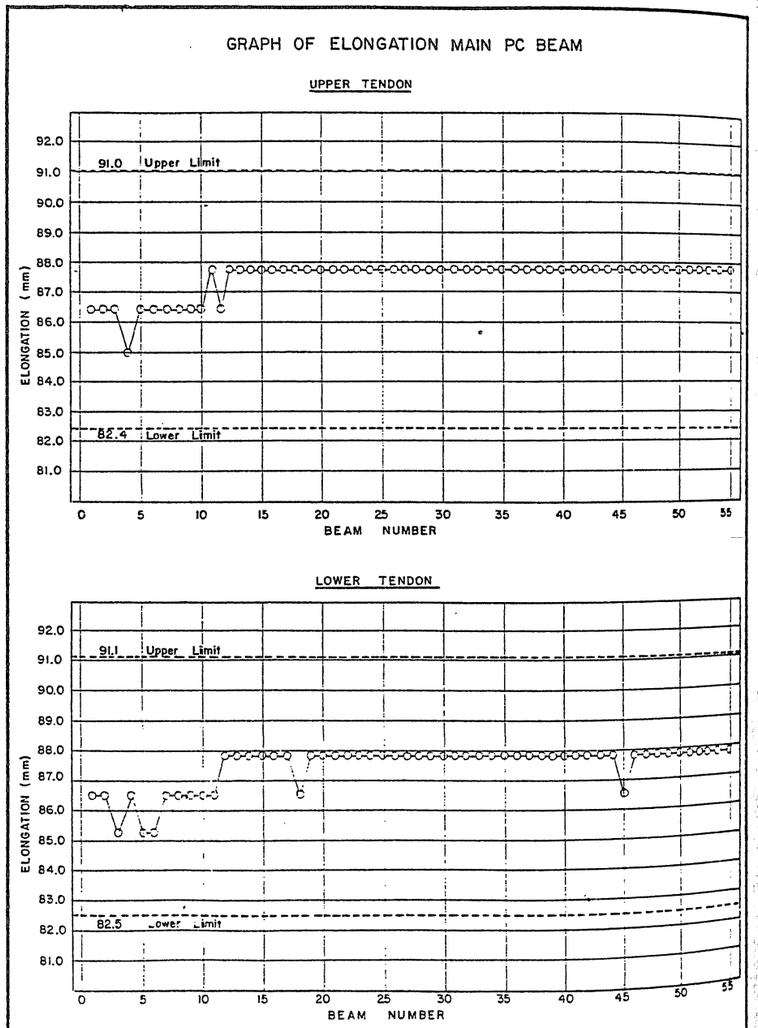
Tendon	Required Jacking per Strand (kN)	Number of Strand	Required pressure (MPa)	Elongation (mm)
Upper	129.33	7	29.3229	86.7
Lower	128.45	7	29.1049	86.8

Note : Required pressure includes 3 % of jack friction.

Tolerance of the tendon elongation

Tendon	Design Elongation (100 %)	Max. Tolerable Extension (105 %)	Min. Tolerable Extension (95%)
Upper Tendon	86.7mm	91.0mm	82.4mm
Lower Tendon	86.8mm	91.1mm	82.5mm

The result of the stressing tendon for PC beam of the Spillway gate hoist is shown in the figure below which indicates that stressing of the tendons satisfied the design requirement.

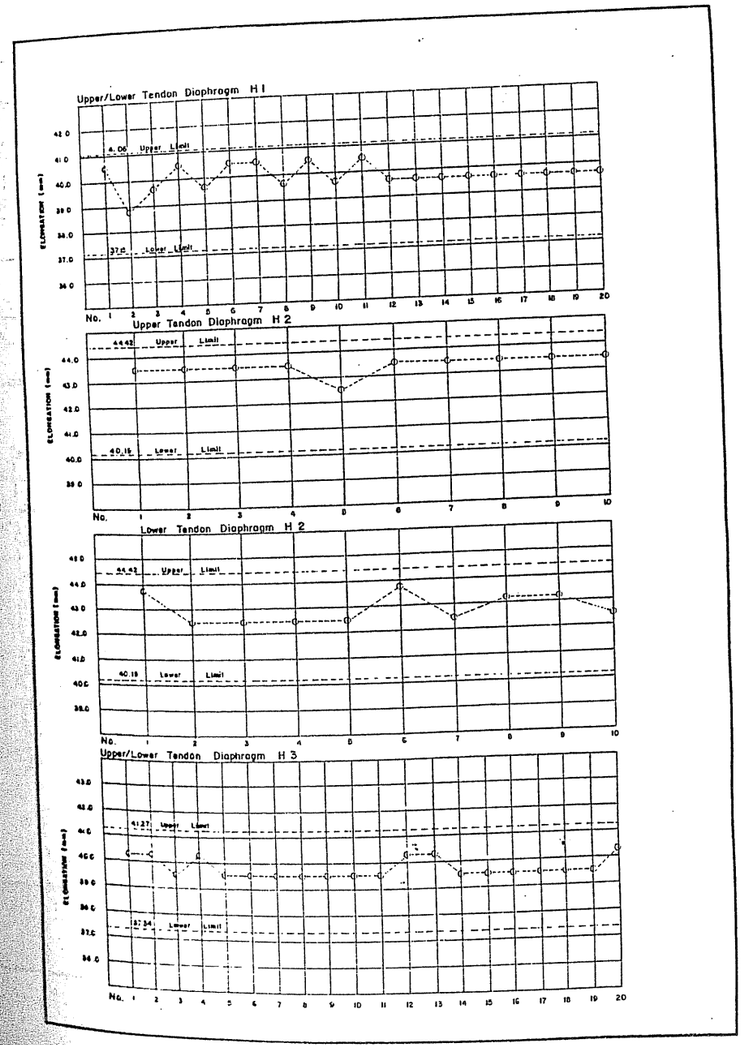


Result of Stressing for Hoist Deck Main Beam

(2) Transverse Beam

Transverse duct sheaths were set between the PC beams. After concreting, the transverse beams were stressed using the same method for the PC main beam. The result of stressing is shown in the figure below which shows that stressing satisfies the requirement.

Tendon	Design Elongation (100%)	Max. Tolerable Extension (105%)	Min. Tolerable Extension (95%)
H1 Upper and Lower	39.11	41.06	37.15
H2 Upper and Lower	42.305	44.42	40.19
H3 Upper and Lower	39.31	41.27	37.34



Result of Stressing for Hoist Deck Transverse Beam

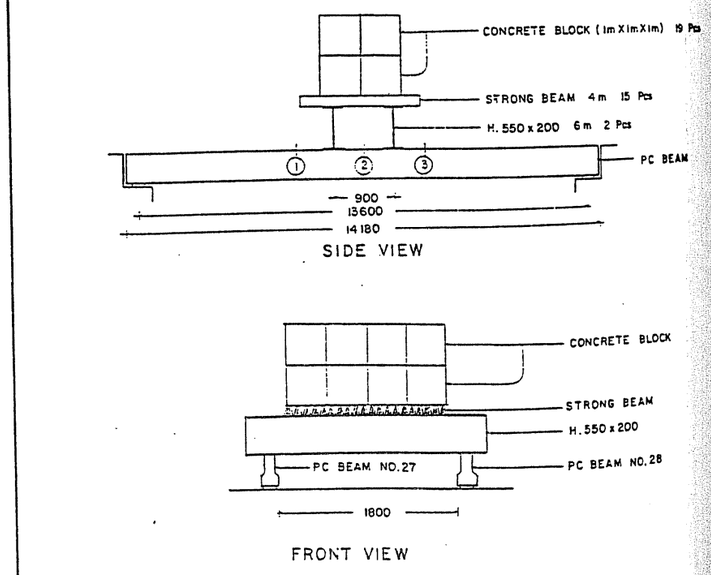
5.5.3 Loading Test

The loading test was carried out after erection into the position on 13th February 199 for PC beam No.27 and No.28. The beam length is 14.180 m with span 13.600 m. The self weight is 0.60 t/m and working moment 83.08 t.m.

Concrete block (1m x 1m x 1m) 19 pieces, H-Beam (H-550 x 200) 2 pieces and Stee support 15 pieces were used as the test load. The total load for testing was 45.295 ton. Therefore, 1 PC beam load was 22.6475 ton. The PC beams were loaded for 24 hours and by visual check no crack was found. Deflection of the beams were recorded as shown in the Table below.

Load Test

PC BEAM	Static Deflection (mm)					
	Beam No.1			Beam No.2		
Position	1	2	3	1	2	3
Before Loading	0	0	0	0	0	0
After Loading	21	24	21	17	21	16
After 1 -day	24	24	23	19	21	17
Visual Checking	No crack			No crack		



Loading Test Method for Hoist Deck Main Beam

5.6 Spillway Bridge

5.6.1 Concreting

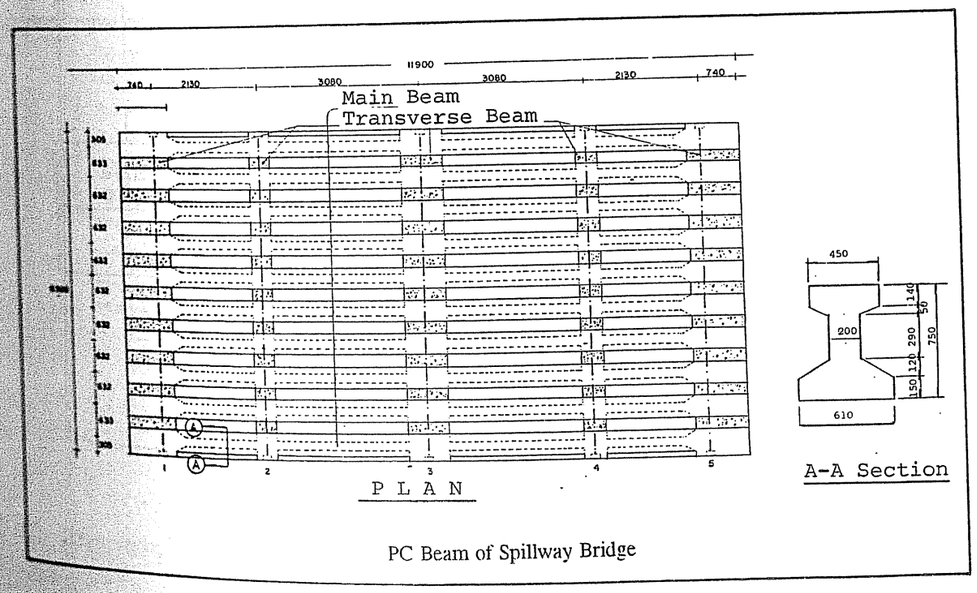
The spillway bridges were constructed at the dam crest EL.87.5 over the spillway chute. The structure of spillway bridges is a prestressed concrete slab which has dimension of 6.3m wide and 11.9m long as shown in the figure below.

PC beam concreting was commenced on 8th August 1995 and completed by 17 th November 1995 for the 50 pieces of PC beam. PC beams were fabricated at the upstream job site of the Stilling Basin. Concrete type was PC-R1 and the result of compressive strength test at 28 days age satisfies the required strength of 500 kg/cm^2 as shown in Fig. 2.5.27.

Concrete Mix Proportion of PC-R1

σ_{ck} (Kg/cm^2)	G_{max} (mm)	Slump (cm)	W/C (%)	S/a (%)
500	20	18-24	35	43

After the PC main beams were installed, concrete placement of the transverse beams commenced on 27th October 1995 and completed on 11th December 1995 for (5) x (5 span).



PC Beam of Spillway Bridge

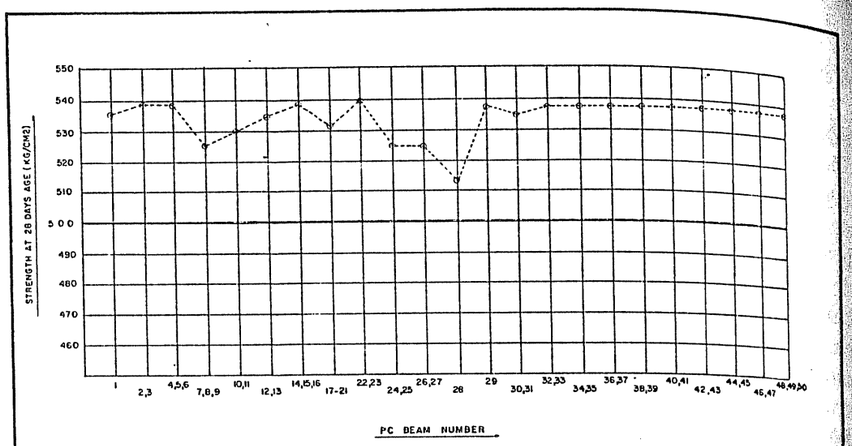


Fig. 2.5.27 PC - R1 Compressive Strength at 28 Days for Spillway Bridge

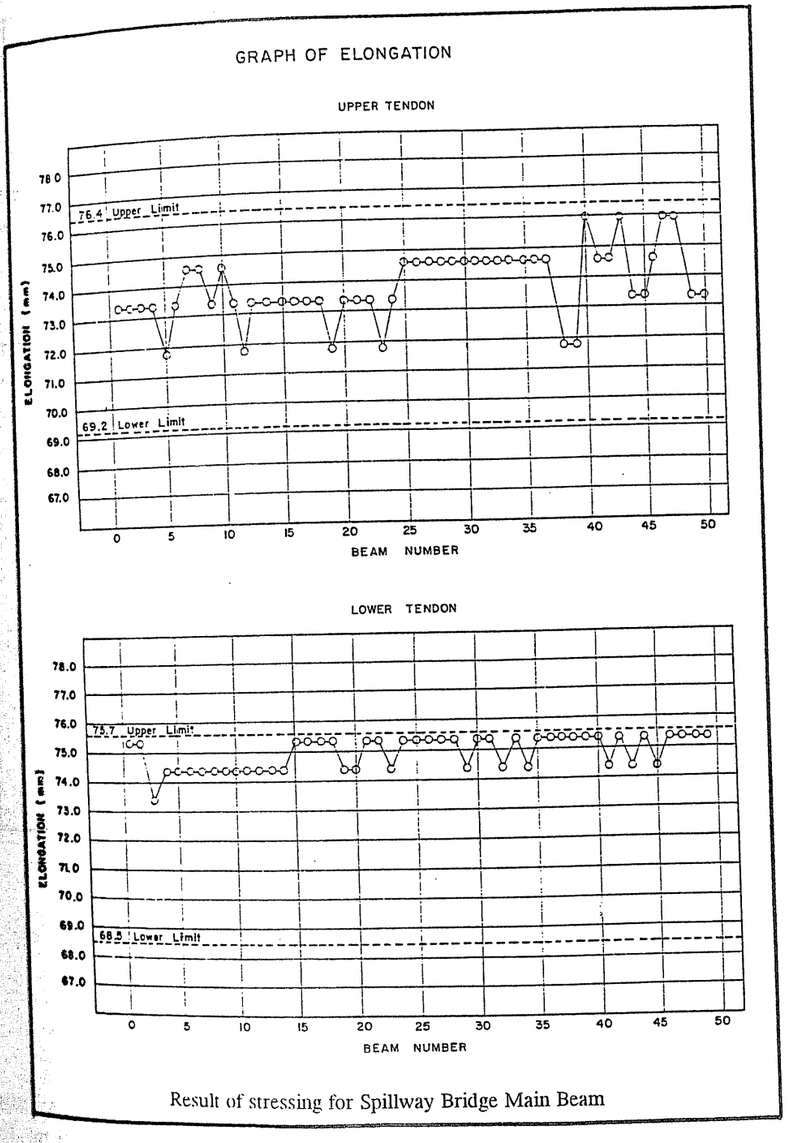
5.6.2 Stressing

(1) PC Main Beam

Stressing work commenced after concrete age of the main beams became more than 7 days. When the PC Beam achieved required transfer strength, 400 kg/cm², the strands were cut and grouted with mortar. The result of stressing the tendon for PC beam of Spillway Bridge, is shown in the figure below which shows that stressing satisfies the requirement. The required elongation for prestressing for the main beam is shown below.

Required Elongation for Prestressing of Spillway Bridge Main Beam

Item	Design Elongation (100%)	Max. tolerable Extension (105%)	Min. Tolerable Extension (95%)
Upper Tendon	72.8	76.4	69.2
Lower Tendon	72.1	75.7	68.5



Result of stressing for Spillway Bridge Main Beam

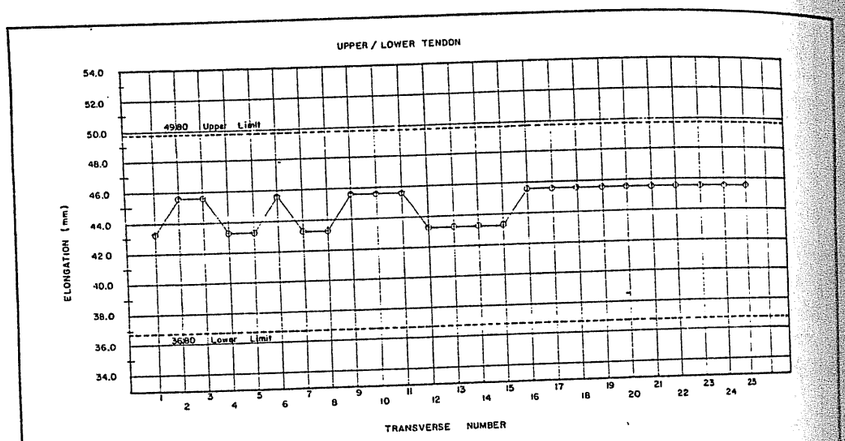
(2) Transverse Beam

Transverse duct sheaths were set between the PC Main beam. The required elongation for prestressing for the transverse beam is shown below.

Required Elongation for Prestressing of Spillway Bridge Transverse Beam

Item	Design Elongation (100%)	Max. Tolerable Extension (115%)	Min. Tolerable Extension (85%)
Upper and Lower Tendon	43.3	49.8	36.8

After concreting, tendons for the transverse beam were stressed using the same method for the PC main beam and the result is shown in the figure below which shows that stressing satisfies the design requirement.



Result of Stressing for Spillway Bridge Transverse Beam

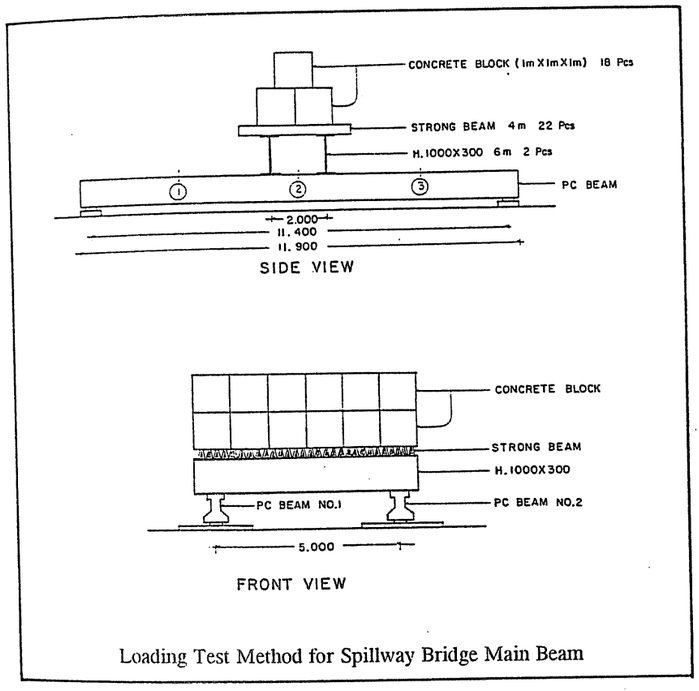
5.6.3 Loading Test

The PC beam loading test was carried out before erection on 12 to 13th October 1[illegible] for PC Beam No.1 and No.2. The length is 11.900 m with span 11.400 m. Th[illegible] weight is 0.69 t/m and the working moment 67.17 t.m.

Concreting block (1m x 1m x 1m) 18 pieces, H Beam (H-1000 x 300) 2 pieces and Steel support 22 pieces were used as the test load. Total load for testing was 46.8[illegible] ton. Therefore, 1 PC Beam load was 23.325 ton. PC Beam were loaded for 24 hour[illegible] and by visual check no crack was found. Deflection of the beams were recorded a[illegible] shown in the table below.

Result of Loading Test

PC BEAM	Static Deflection (mm)					
	Beam No.1			Beam 45 No.2		
Position	1	2	3	1	2	3
Before Loading	0	0	0	0	0	0
After Loading	0	11	0	0	12	0
After 1 -day	0	11	0	0	12	0
Visual Checking	No crack			No crack		



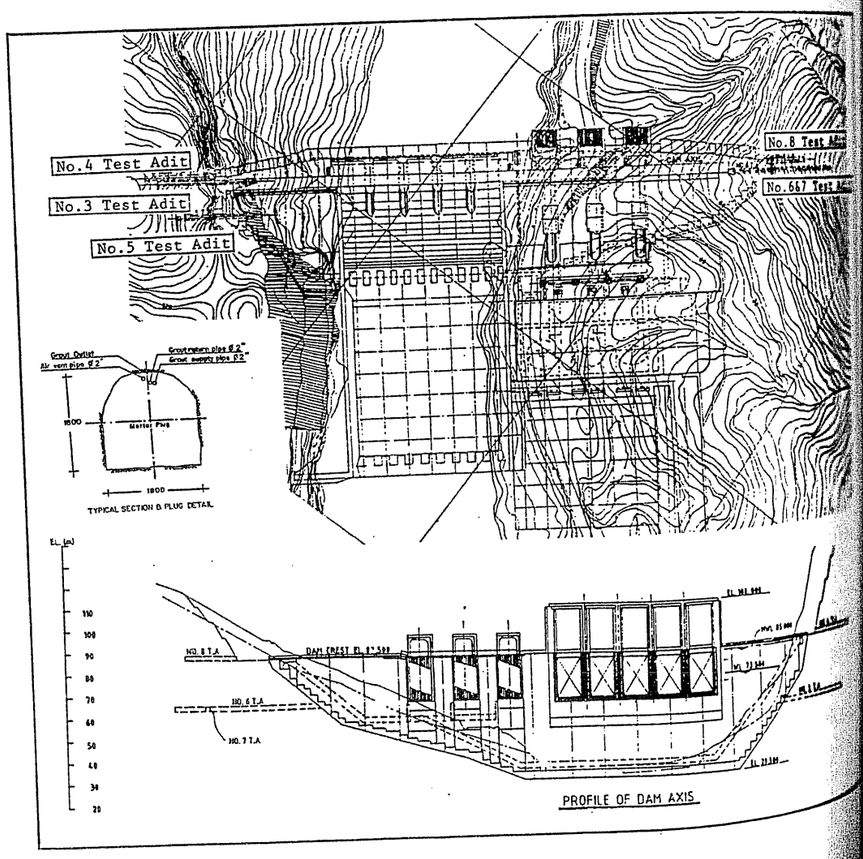
Loading Test Method for Spillway Bridge Main Beam

5.7 Plugging of Test Adit

There are eight test adits in the dam site area, which had been used for geological investigation including in-situ rock test during the feasibility study (1982-1984) and the detailed design stage (1987-1988). The test adits near the dam are shown in the figure below. Among them, four adits in the dam body were required to be plugged with concrete and grout. The total length of plug work was 183 m which was backfilled with 618 m^3 of concrete and 120.7 m^3 of grout.

Location of Test Adits and Plug Length

No.	Location	EL. (m)	Plug Length (m)
1	Upstream right bank	62.410	No. Plug
2	Upstream left bank	66.780	No. Plug
3	Dam axis right bank	54.160	46.000
4	Dam axis right bank	87.500	No Plug
5	Dam axis right bank	53.850	36.000
6	Dam axis left bank	61.030	66.193
7	Dam axis left bank	61.030	34.830
8	Dam axis left bank	87.290	No Plug
-	-	-	183.023 (Total)



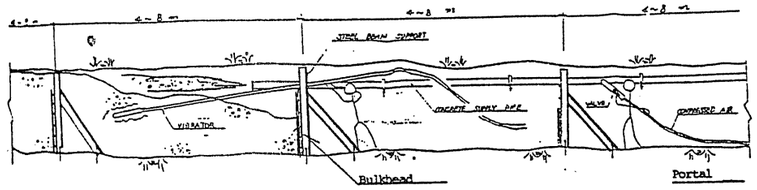
(i) Concrete Placement

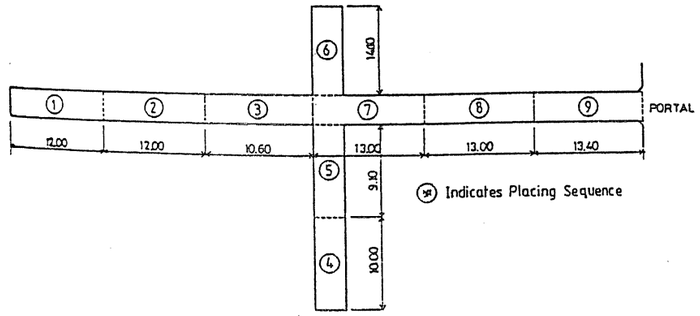
In order to backfill the adits with concrete, the adits were divided into several blocks at intervals of 4 to 15 m bulkheads as shown in below. An adit was backfilled from the face to the portal, leaving bulkheads in concrete. The bulkhead made of steel beams was installed firmly to support the concrete pressure during concrete placement. Concrete Pumping was boosted with compressed air (6 kg/cm^2) to ensure good filling.

Plugging the test adits were carried out after (i) cleaning and removal of debris and loose material in the adits, (ii) joint survey of Engineer and Contractor for records of the actual dimension of the test adits. After placing concrete, water was supplied through the grout pipe to flow into the top of concrete to assist in cooling and to flush the grout pipe.

Concrete Plugging of Test Adit

No.	Location	EL.	Plug Length (m)	Concrete Volume (m^3)	Grout (m^3)	Progress of Work
3	Dam axis right bank	54.160	46.0	172.0	27.9	12/94 - 1/95
5	Dam axis right bank	53.850	36.0	114.0	11.0	12/94 - 1/95
6&7	Dam axis left bank	61.030	101.0	332.0	81.8	11/93 - 9/94





Concrete Placing Method and Sequence

(2) Grouting Work

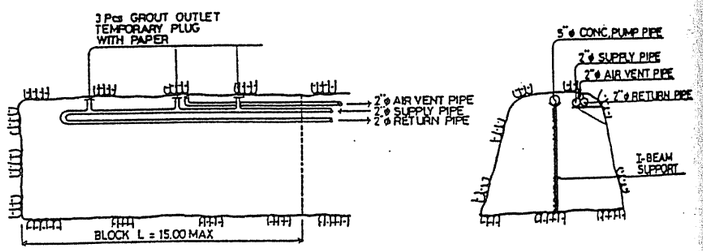
Backfill grouting was carried out to fill gaps between the plug concrete and the crown of the adit.

Grouting was placed two weeks after placing concrete when the temperature of concrete was reduced below 30 °C. Before placing concrete, a set of supply and return pipes (Ø 5 cm) and an air vent (Ø 5 cm) were installed near the crown of the adit, as shown below. Actual procedure of the grouting was as follows :

(a) Flush the grouting circuit with water under pressure.
(b) When water shows at the return pipe, shut the return pipe to expel the paper plugs in grout outlets.
(c) Flush the air vent pipe with water.
(d) Commence grouting with cement/water mixture; using 300 liters.
(e) Inject mortar grout via the grout pipe, with return pipe open.
(f) When mortar grout shows at return pipe, shut the return pipe and continue grout injection with pressure less than 5kg/cm^2.
(g) Grouting is continued until the injection rate decreases to zero under the maximum pressure.

Mix proportion of Grout by Volume

Grout Mix Details	Cement	Sand	Water
Grout	1	-	4
Mortar	1	2	Field Adjustment



Grout Pipe Arrangement

5.1 Instrumentation for Dam and Others

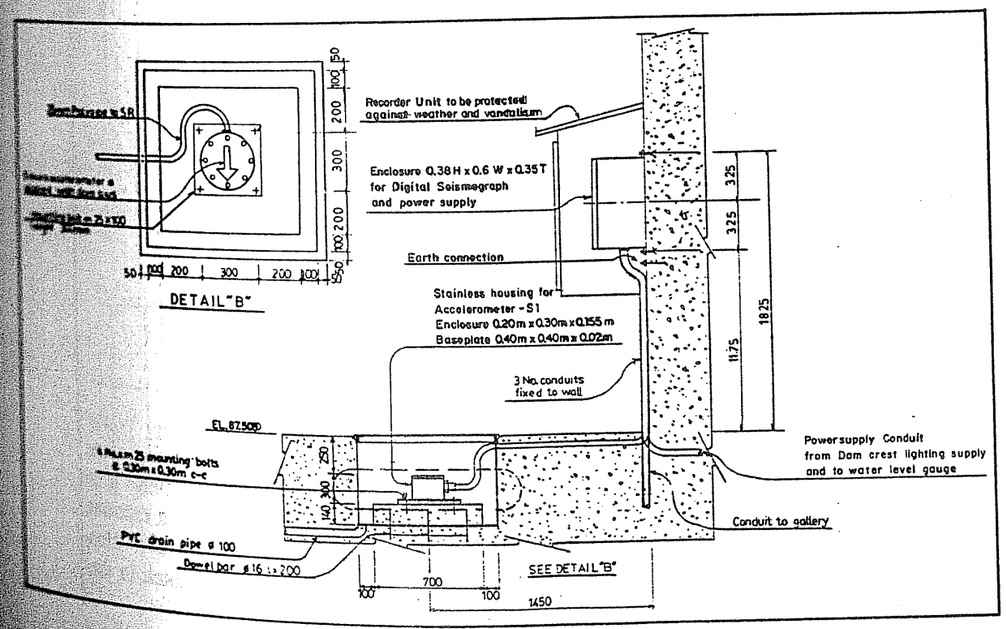
Monitoring instrument was installed in the dam, powerhouse and tailrace to record observation items such as, joint opening, temperature, pore pressure, leakage and water level of the reservoir and tailrace.

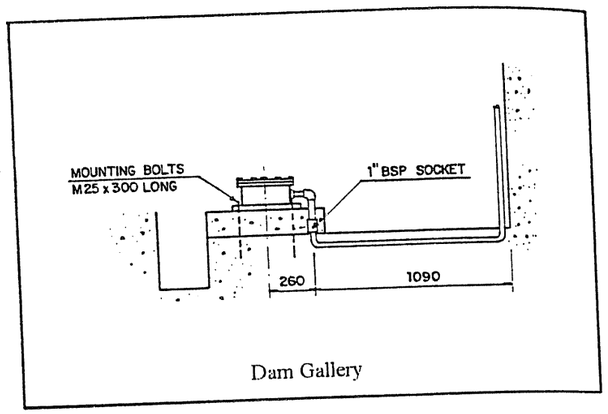
These data were monitored to check the dam behavior during construction and subsequently after start of impounding. The monitoring is being carried out by PLN since the start of powerstation commercial operation. The monitoring instruments, measurement items, the data recorded and their evaluation are mentioned in Part 2 Chapter 9 Dam Monitoring and Instrumentation. Monitoring and instruments are listed in Table 2.5.4.

Details of instrument installation are as follows.

(i) Seismograph

The detailed installation of Seismograph is shown in the figures below. The system comprises a 6-channel digital recorder (Kelunji, Seismology Research Center, Australia) with two external triaxial accelerometer transducers (Sprengnether Instruments Inc., HSA-3, USA). The Kelunji digital recorder incorporates 2 MB data storage, clock, batteries, software for operation and data analysis. The digital recorder and one triaxial transduce were installed at the crest, and the other transducer installed in the gallery. An Apple Note Book Computer with software will be used for data retrieval and analysis.

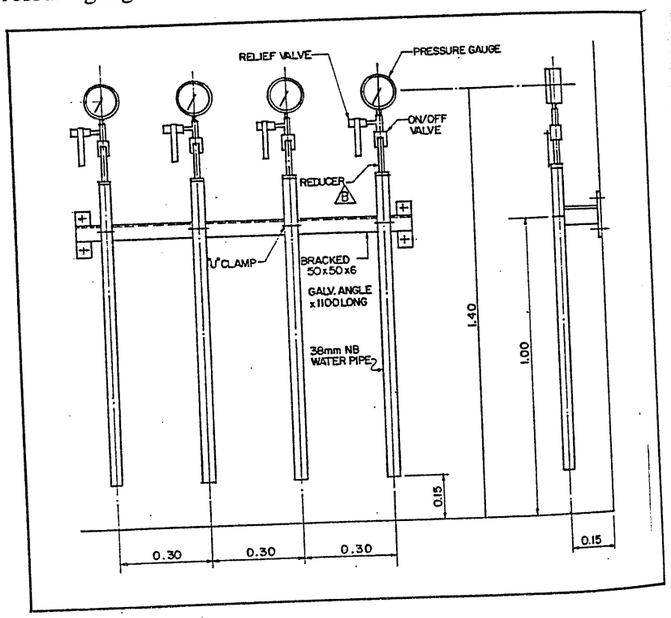




Dam Gallery

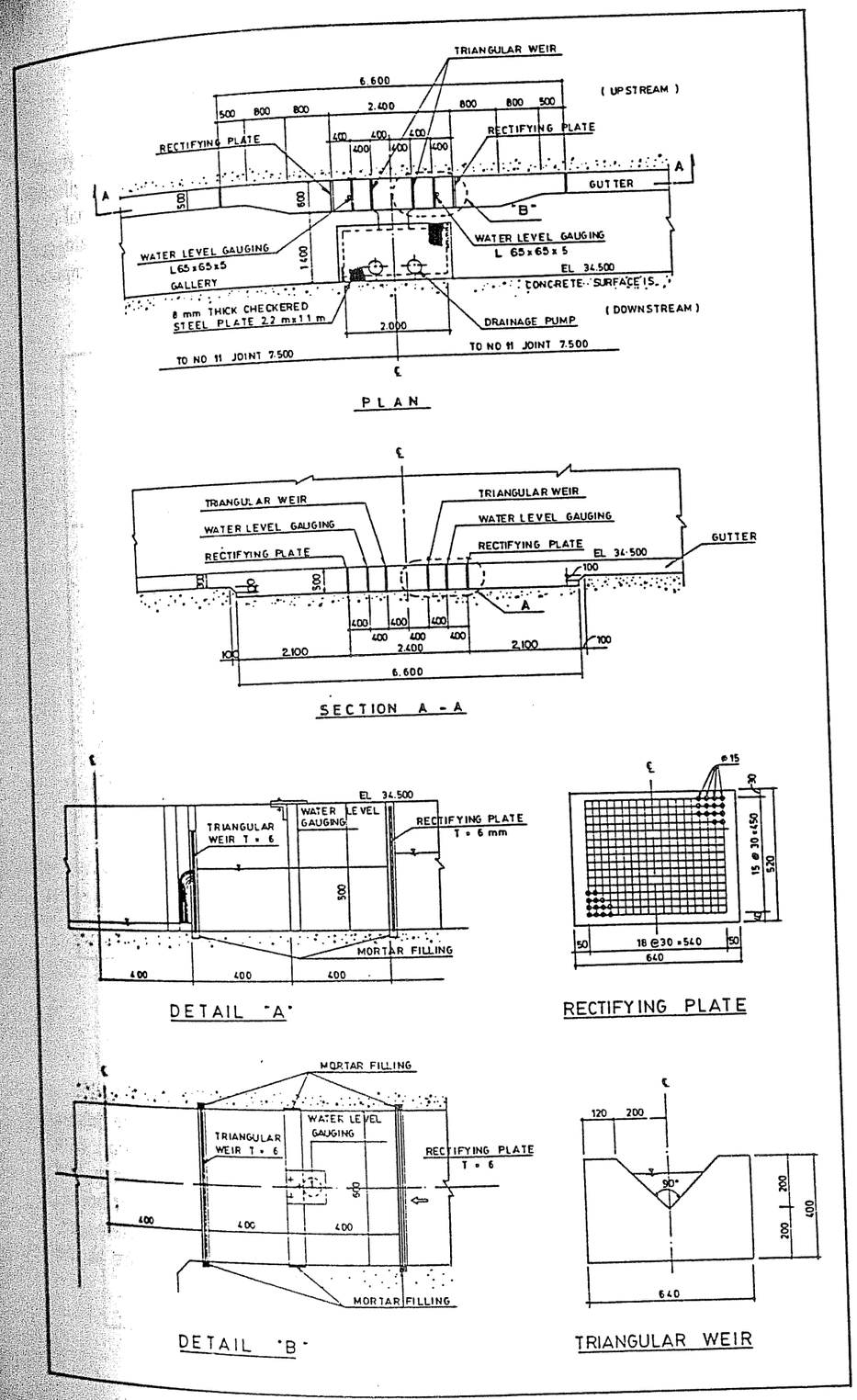
(ii) Uplift Meter

The detailed installation of uplift meter is shown in the figure below. Four (4) sets of uplift meter were embedded in the dam block J7+9.00 to obtain uplift pressure acting on the dam foundation. The uplift meter consists of Bourbon tube pressure gauge with brass valves and fittings.



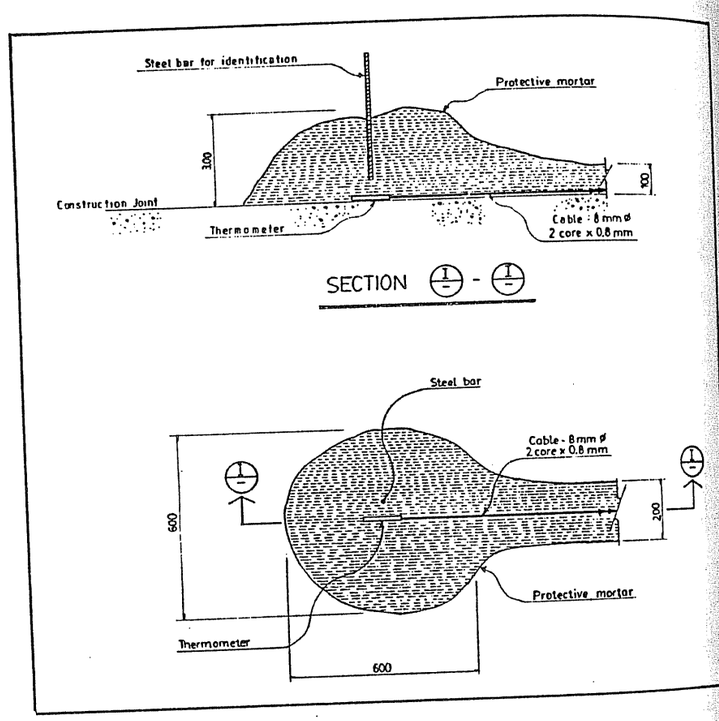
(iii) Leakage Meter

Detailed installation of leakage meter is shown in the figure below. Water leakage monitoring equipment consists of a stainless steel weir, rectifying plate and a level measuring instrument. The level measuring instrument is a precision pressure transducer mounted in the weir, and connected to a readout unit mounted in a waterproof housing in the dam gallery. Data is also transmitted via cable to the central control room. Readings can be taken in the control room using a personal computer.



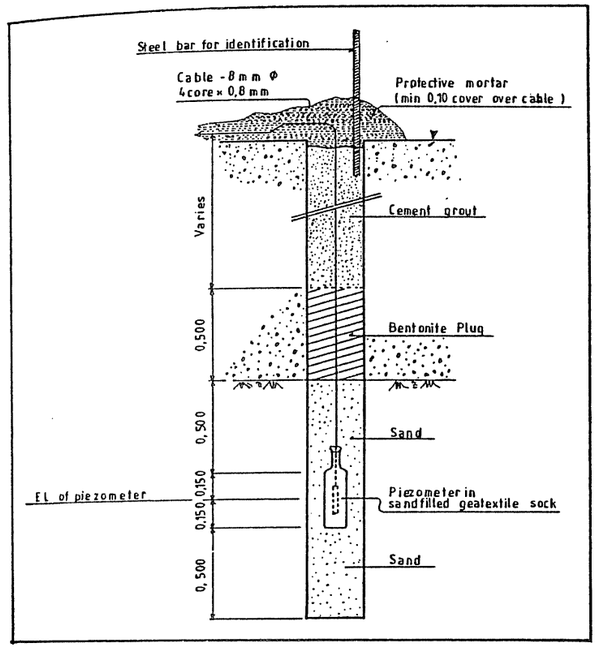
(iv) Thermometer

The detailed installation procedure of the thermometer is shown in the figure below. Thermometer type temperature sensors are used. These are placed directly in the concrete, and connected by cable to a waterproof terminal box in the gallery. Cables run directly from terminal boxes to the datalogger located in the central control room at the power house. A personal computer is used to record and display readings.



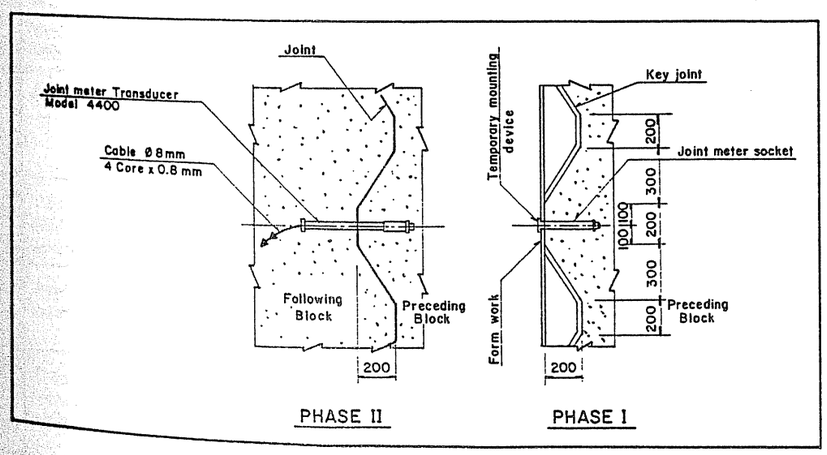
(v) Pore Pressure Meter (Piezometer)

The detailed installation procedure of piezometer is shown in the figure below. Geokon vibrating wire type piezometers, Model 4500S, 250 psi range with 0.1% accuracy are connected by cable to the terminal boxes in the gallery, and then by cable to the datalogger at the central control room at the power house. Piezometers are read using a personal computer linked to the datalogger. Piezometers are fitted with thermistors permitting the temperature at the piezometer location to be monitored as well.



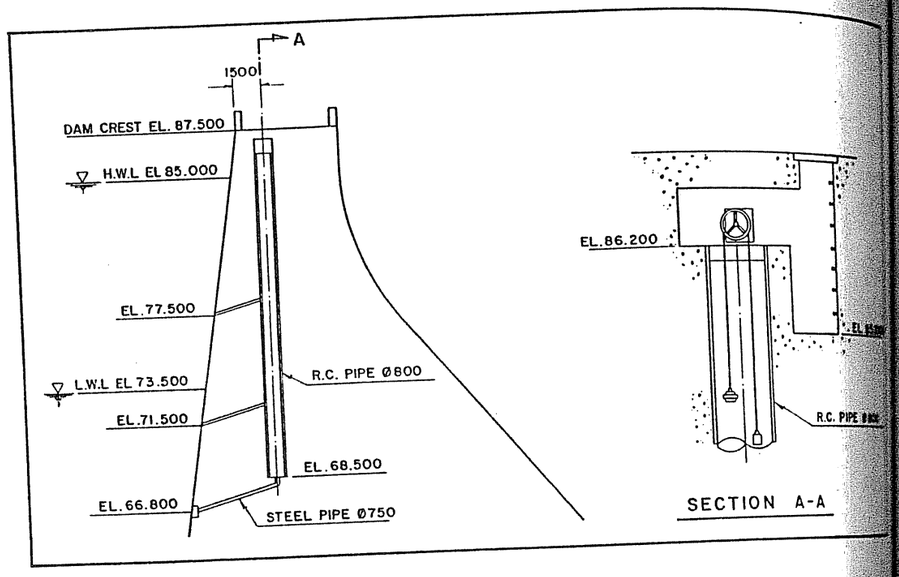
(vi) Joint meter

The detailed installation procedure of joint meter is shown in the figure below. Geokon vibrating wire type joint meters, Model 4400, with cable connections to the terminal box in the gallery, and then by cable to the datalogger at the central control room. Joint meters are also fitted with thermistors for temperature monitoring. Both joint movement and temperature are read using a personal computer.



(vii) Water Level Gauge

The Unidata Water Level Instrument Model 6509 is used for both Dam and Tailrace. The system comprises a float type transducer, connected by cable to the central control room. The data is retrieved and read using a personal computer.



(viii) Datalogger

The CR10 is a fully programmable datalogger/controller with memory and a battery backed clock in a small rugged, sealed module. The datalogger is manufactured by Campbell Scientific , Inc.

Table 2.5.4 Equipment List of Dam Instrumentation

Instrument and Item	Item or Module or Sub-Assembly	Model No. or Material	Quantity	Remarks	Location of Installation
Seismograph	Independent Unit comprising. - accelerometers - microprocessor - time clock - RAM memory	Teledyne Geotech A-800 Accelocorder II	2 Nos.	Includes stainless steel housing, modems, all cabling and Table 1.5.9-1 EQUIPMENT LIST OF DAM INSTRUMENT ATION set-up software.	Dam Gallery and Crest
Uplift Meter	Pressure gauge with 2 valves, a T-junction and reducer.	Bourbon tube pressure gauge with brass valves and fittings.	4 Nos.	Connects to 38mm dia.pipe.	Foundation in Dam Block No.8.
Leakage Measurement Device	Stainless steel weir and rectifying plates, pressure transducer, datalogger with digital display.	Druck PDCR 830 transducer and Starlogger Model 6004-2	2 Nos.	Includes modems and all cabling to the control room.	Dam gallery
Thermometer	Thermistor temperature sensor connected to cable.	Geosystems TP02	15 Nos.	Direct burial in dam wall. Includes all cabling.	Dam Block No.8.
Pore Pressure meter	Vibrating wire piezometer connected to cable.	Geokon 4500S	9 Nos.	Includes all cabling to control room.	Foundation in Dam Block No.5, 8 and 13.
Joint Meter	Vibrating wire jointmeter connected to cable.	Geokon 4400	20 Nos.	Includes all cabling to control room.	Joint 7 and 8
Water Level Meter	Float type water level instrument with datalogger.	Unidata 6509-11 and Starlogger Model 6004-2	2 Nos.	Includes modems and all cabling.	Dam Block No.14 and Tailrace.
Water Level Rod	Engraved scales.		2 No.	Mounted on U/S wall of dam, between EL. 63 and EL. 87.	Dam Block No.4 and Tailrace.
Portable Readout	Vibrating wire readout unit (Direct sealing)	Geokon GK 403	1 No.	Includes battery charger.	Powerhouse central control room.
Portable Computer	Apple Notebook computer.	Apple powerbook	1 No.		Powerhouse central control room.
Data Logger	Data logger and associated components.	Campbell scientific CR-10 Data logger and IBM PC.	1 No.	Includes all cabling.	Powerhouse central control room.

5.9 Reservoir Impounding

(1) The Reason of Carrying Out Impounding During Construction

The construction of the dam and associated works has proceeded in accordance with the contract schedule since the excavation of the dam foundation started in February 1993. The overall schedule of main dam structures were eventually shortened by some months because the river diversion was completed ahead of schedule chiefly due to smooth excavation work of the tunnel.

As of the month of December 1996, the dam, stilling basin, spillway gates and switchyard were substantially completed. The associated works of powerhouse, intake and outlet gates were completed on time in February 1997. The installation of electrical and mechanical plant was proceeding on schedule with more than 80% of components delivered to site and the erection of turbine 75% complete and generator 60% complete.

The relocation of national road had caught up the schedule. The two major bridges were opened to construction traffic on 27th December 1996 and the new road was opened to traffic in mid February following the joint inspection with PU and Department of Transport. The transmission line was also under construction between the dam site and Payakumbuh and between the dam site and Pekanbaru. These works were scheduled to be completed in June 1997 and July 1997 respectively.

In consideration of the works progress mentioned above and the timing of the lowering and closing the gate of diversion tunnel in the wet season, the start date of impounding was set at the beginning of March 1997.

(2) Closing of Diversion Tunnel

The method of the tunnel gate closure was prepared according to the requirement of the Technical Specifications that the closing water elevation shall be at 41.500m and the relevant Contract Drawing. After completion of lowering of each gate assembly, a temporary cofferdam was constructed at the outlet then the concrete plug was placed in the tunnels. The comparison was done for lowering of gate assemblies for two (2) tunnels simultaneous or at different times.

Simultaneous lowering of gate assemblies for 2 tunnels

(i) It will be very difficult to lower the gate assemblies for the 2 tunnels at the same lowering speed. There will be differences between each other in the friction between grooves and the gate assemblies and in the hydraulic pressure acting on the gate assemblies. If the four gate assemblies are lowered simultaneously, high hydraulic pressure will act on each of the four gate assemblies when they are being lowered. Therefore, in order for all four gate assemblies to be thoroughly lowered simultaneously, deliberate additional work to provide the additional weight on either of the gate assemblies would become necessary.

(ii) Lowering of gate assemblies for 2 tunnels at different time

In this case, water will be allowed to flow through a tunnel when the other is closed. The two gate assemblies will be first lowered with less hydraulic pressure than in Case (1) above. Then, the remaining two gate assemblies will be lowered in the remaining tunnel at a different time with high hydraulic pressure. However, attention will be more concentrated on providing additional weight on the remaining two gate assemblies than on the four gate assemblies as in Case (I) above.

Therefore, the two gate assemblies for one tunnel in case (ii) can be lowered under more favourable conditions than in case (I) and the remaining gate assemblies can be lowered under more concentrated conditions. It was considered that the case (ii) would have an advantage over the case (I) and finally this method was adopted.

The sill elevation of the Tunnel No. 2 is lower than that of the Tunnel No. 1 by 1.5m. In order to prevent deposit from accumulating at the inlet and in the grooves, the Tunnel No. 2 was first closed. Then the Tunnel No. 1 gate assemblies was lowered.

Lowering of tunnel No. 2 gate assemblies

Before lowering each gate assembly, possible deposit accumulating in the grooves was removed by using compressed air. Driftwood was also removed. Each 36 ton gate assembly was lowered by using a 80 ton crawler crane. Both gate assemblies were lowered one by one by this method. The 80 ton crawler crane has a 50 ton lifting capacity. Therefore, the gate assembly could be lifted up and down by the crawler crane.

Lowering of tunnel No. 1 gate assemblies

Both gate assemblies were lowered simultaneously. For lifting the gate assemblies, two gantry cranes were equipped with a steel framed structure set on the deck of the diversion tunnel inlet structure. One gantry crane lifted one gate assembly. Each gate assembly was lifted by separate gantry crane. Each gantry crane equipment had capacity of approximately 83 ton.

The steel frame structure with which the gantry cranes were equipped was built on concrete foundation and the bottom thereof was embedded in concrete for precautionary measures against floods.

The gate assemblies were lifted by using motor winches. Which equipment is more suitable for lifting was further studied and determined taking account of the actual site and river flow conditions. Each of the gate assemblies was lifted by 2 tackles each of which has 5 pulleys. The required pulling capacity for one gate assembly was approximately 4 ton. When the gate assembly was lifted by a winch, the capacity of the winch was at least 37 kW.

In order to maintain the safe position of the lifted gate assemblies against an unexpected increase in river flows, the gate assemblies (H10 m x W 5.8 m) were inserted half into the opening in the deck of the inlet structure and fixed by stoppers. When the river flow unexpectedly increases, the hydraulic pressure thereof would be dispersed to the gate assembly and to the steel structure through the stoppers. To secure the smooth lowering of the gate assemblies, the grooves were checked and foreign materials therein were removed before lowering the gate assemblies.

(3) Impounding Plan and Actual Results

The original schedule was to start impounding on March 1st 1997. However according to the site condition impounding actually started 11 days behind the plan on March 12, 1997. The Impounding Ceremony was held attended by many people. A wide area was selected and beautifully arranged for the Ceremony on the left bank of the Kampar Kanan River. At the sound of the siren turned on by the Governor of Riau Province and the Governor of West Sumatra, the gates of No. 2 Diversion Tunnel were closed by the Contractor at 9 o'clock A.M. Then the gates of No. 1 were lowered as described earlier.

The reservoir water level rose rapidly at first at the rate of 1.4m per day because of the narrow river canyon and reached spillway fixed crest level EL. 67.50 at the end of the first month, March 31st 1997. On the day of March 31th, discharge to the downstream area was started by opening one spillway gate, three gates on the following day and all five gates 5 days later. The actual rate of water level rising 1.4 m/day up to spillway fixed crest level was smaller than that of the original estimate of 2.0 m/day.

It was decided by the Employer that the reservoir water level should be controlled to keep water level fixed at spillway crest level EL. 67.50 for the time being until it was certain all environmental matters on the upstream side were satisfactorily completed to such a stage that impounding could continue. All 5 gates were left fully open, in order to let inflow discharge flow down to downstream and to maintain the water level as close as possible to the fixed crest level of EL. 67.50.

On May 18th 1997, impounding resumed, the gates were closed and water level started to rise again. From the end of April to the beginning of May, there was a flood with estimated peak flow with 900 m^3/s and the water level rose very fast at the rate of 0.45m per day in this period. After then, however, water level rose very slowly at 0.05m per day. The water level was not expected to rise so fast after June because the dry season had started and also the reservoir area was being widely extended.

On July 5th 1997, water level reached NWL EL. 80.60 which is the minimum required for start of commissioning test of turbine and generator. The test was to start on August of 1997 but actually it was delayed until October 1997. Reservoir water level was rising very slowly about 0.01 m/day in August and reached at around EL. 82.5 in the end of September.

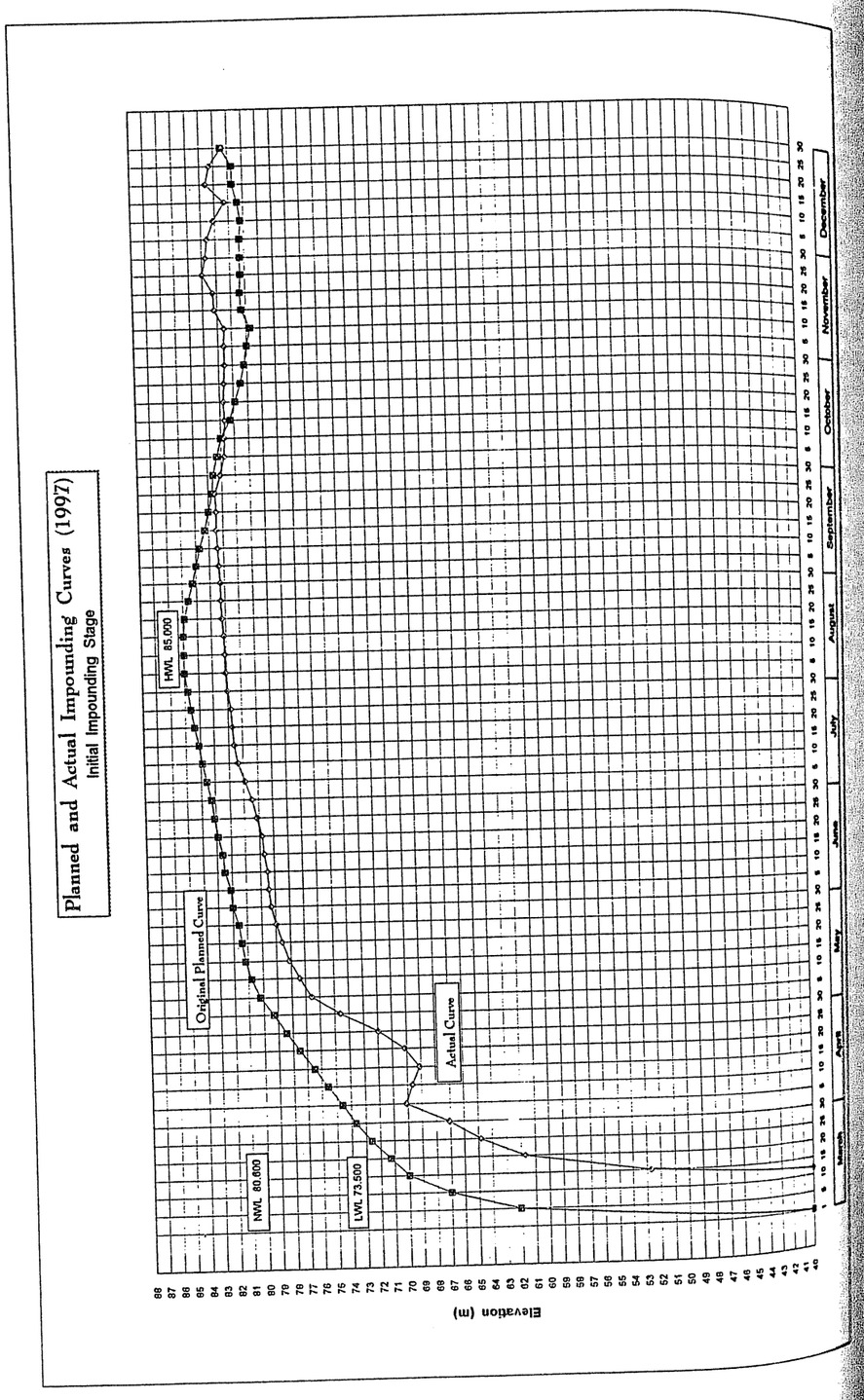
The reservoir water level did not reach HWL 85.0 in the month of August that was originally expected because the start date of impounding was delayed nearly two weeks and there was an intermediate stop of impounding for two weeks in April.

It was judged from the inflow data in this season that there was no possibility of the water level reaching HWL 85.0 in August or September. Therefore, as precaution for safety for the initial impounding of the reservoir, the lowering of water level had to start in September so that the reservoir water level could be lowered to EL. 80.6 (NWL) before start of rainy season, that is November 1997.

September 26th 1997, Gate No. 3 and Gate No. 5 were opened with the opening degree of 0.5m continuously. On September 27th, the gates were opened to 0.8m. The discharge through the spillway was roughly estimated based on the latest information of turbine and generator commissioning and the planned operation schedule in the near future. In consideration of the commissioning schedule, the reservoir water level was kept higher to some extent than EL. 80.6 (NWL) of original plan.

As Kotapanjang reservoir capacity is very large and the fluctuation of the reservoir water level is very small, it was not necessary to check the velocity of ascent and descent of water level. It is recommended by Ministry of Construction in Japan that the ascent and descent velocity of water level should be less than 1 m/day, with 0.5 m/day more desirable for storage capacity larger than $10x10^6$ m^3. Ascent of reservoir water level was planned with 0.18 m/day from spillway crest level EL 67.5 to HWL 85.0 but the actual ascent was only 0.12 m/day.

During the initial impounding, observations on measurement instruments were carried out in order to inspect the safety of the dam facilities of Kotapanjang Dam and confirm the function of the dam control facilities and generating system, and also collect the data for the future control of the dam facilities.



Chapter 6 POWERHOUSE

6.1 General

The powerhouse is located in the left bank immediately downstream area of the dam and has three 38MW turbine and generator units. In the powerhouse area, the waterway system consists of the penstock, turbine, draft tube and tailrace for each unit of generators, as shown in Fig. 2.6.1.

The powerhouse building is divided into two major functions; (i) the powerhouse building for turbines and generators and (ii) control building for cubicles and control rooms. Construction work commenced in July 1993 for excavation and was completed in October 1996 for the third stage concrete placement for embedment of the spiral casing and turbine and generator pit. The progress is shown in Fig. 2.6.2.

Fig. 2.6.1 Powerhouse Traverse Section

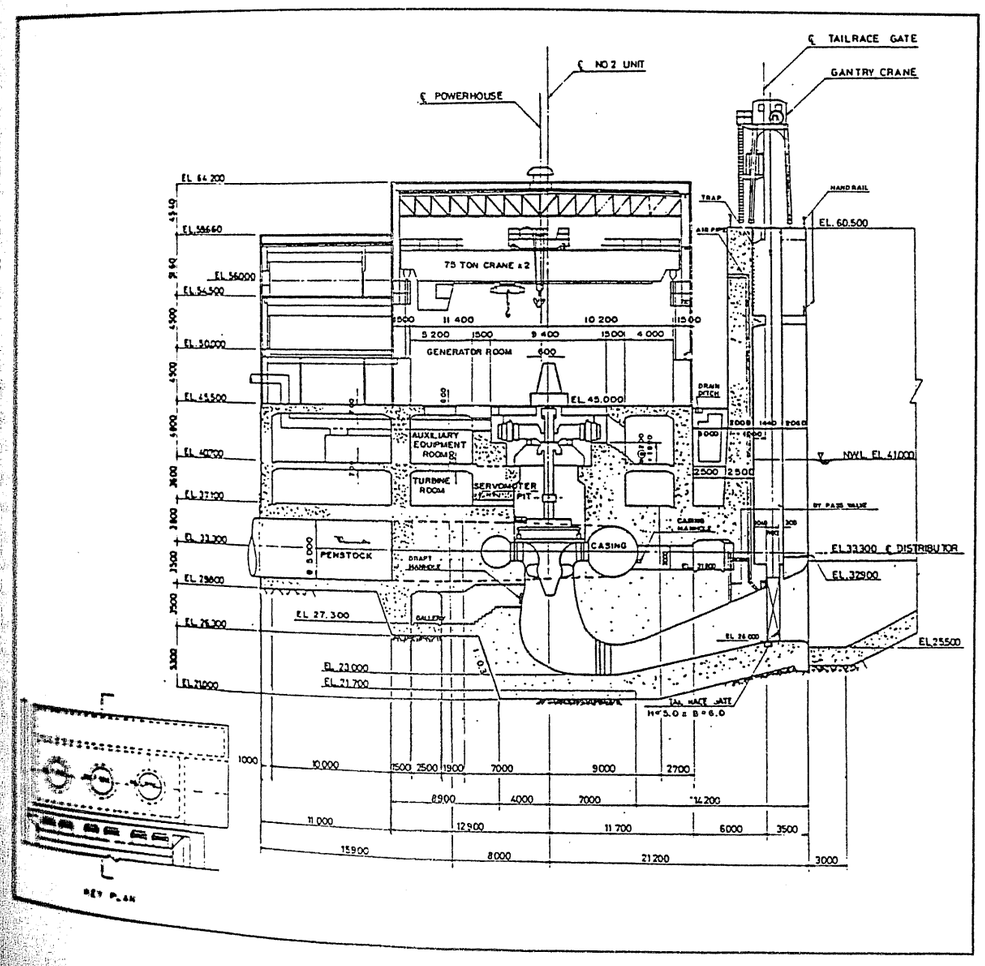
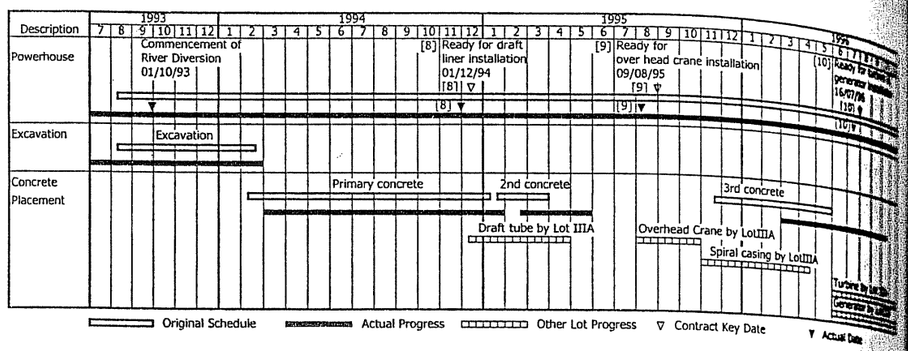


Fig. 2.6.2 Progress of Powerhouse Works

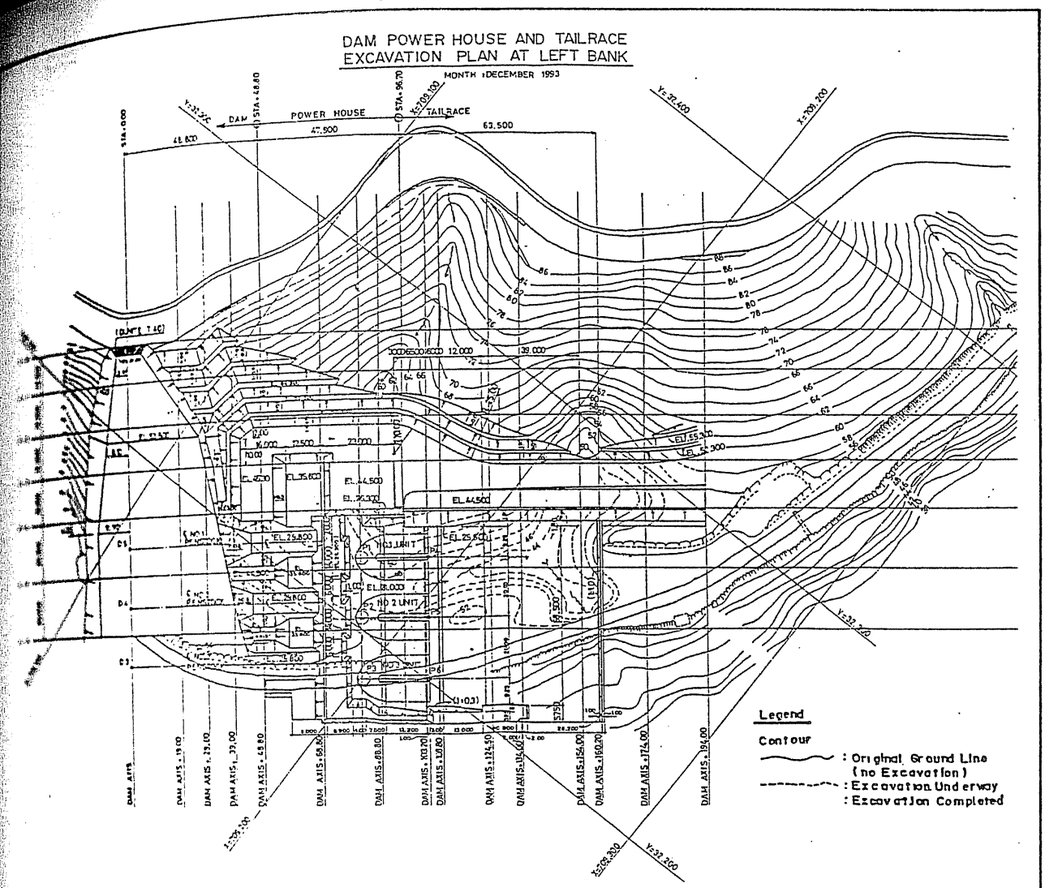


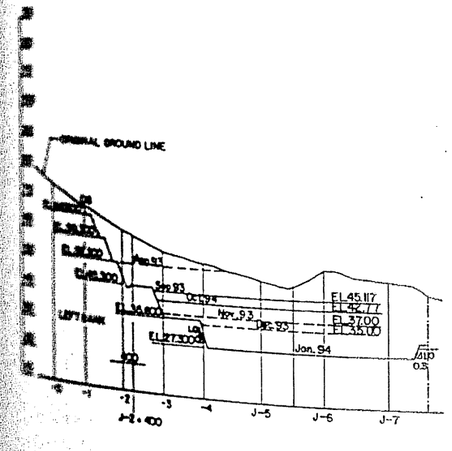
6.2 Excavation

In July 1993, the excavation of the powerhouse commenced at EL 88.0 downstream of the dam in the left bank. By the end of September 1993, excavation work reached the generator room elevation at EL 45.3, as shown in Fig. 2.6.3. After the riverflow was diverted in October 1993, excavation resumed and was completed to the lowest part of the powerhouse foundation at EL.21.0 by the end of February 1994. The total volume of the powerhouse excavation is 106,226m^3 approximately. The monthly progress is shown in Fig. 2.6.3 and the construction equipment used for powerhouse excavation are listed below.

Main Equipment for Powerhouse Excavation

Operation	Equipment	Capacity	Quantity
Drilling	Hydraulic Crawler Drill	165 kw	1
	Pneumatic Crawler Drill	135 kw	1
	Air Compressor	18.5 m^3/min	1
Excavation	Bulldozer with Ripper	45 ton	1
		32 ton	1
	Excavator	1.3 m^3	1
Trimming	Hydraulic Breaker	1,500 kg	1
Loading	Wheel Loader	3.5 m^3	1
		2.4 m^3	1
	Dozer Shovel	2.2 m^3	1
Hauling	Dump Truck	11 ton	4
		8 ton	20
Spreading and Leveling	Bulldozer	26 ton	2
(Disposal Area)	Vibration Roller	10 ton	1
Final Treatment	Hand Breaker	20 kg	2
	Pick Hammer	7.5 kg	3
	Air Compressor	18.5 m^3/min	1
	Reservoir Tank	3 m^3, 7 kg/cm^2	1





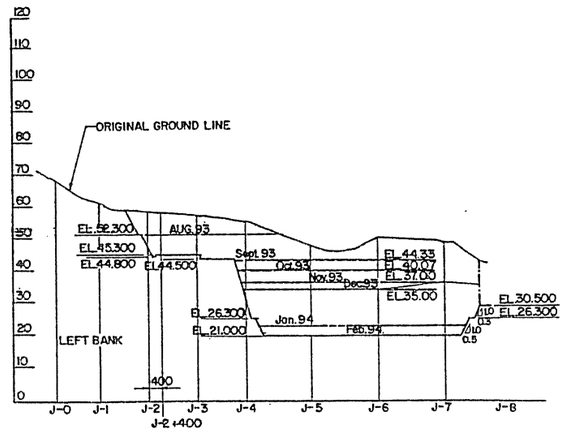


Fig. 2.6.3 Powerhouse Excavation Monthly Progress

6.3 Concrete Placement

Concrete for the powerhouse was placed in three stages; the first stage concrete for the main structures of the powerhouse and control buildings, the second stage concrete for the embedment of the draft tube and the third stage concrete for embedment of the spiral casing and turbine and generator pit. The concrete progress is shown in Fig.2.6.4.

(1) First Stage Concrete

The first stage concrete placement commenced on 1 March 1994, and was completed 23 January 1995. During placement of the first stage concrete, pipes and materials were installed for the following items :

- Grounding wire
- Lower part of the penstock
- Drain pipes for draft tube, spiral case and head cover
- Sump pit connection pipes
- Cooling water supply and return pipes
- Spiral casing support plates and block outs
- Draft tube plinth and embedded plates

Before commencement of concrete works for the powerhouse substructure, grounding wire was installed on the exposed foundation rock by Lot IIIB Contractor. Installation of grounding wire also continued to the upper part of the powerhouse structure to supply terminal points to various electric equipment, as shown in Fig. 2.6.5.

In the upper part of the control building, various block outs for the cables were installed in the floors and walls in the cubicle, battery and inverter rooms, as shown in Table 2.6.1. Partition walls between the control building and the generator room has block outs for the main bus and other cables, as shown in Fig. 2.6.6.

After completion of the first stage concrete, and the draft tube installation, the steel structure erection work commenced in February 1995 in order to meet the key date for the overhead crane installation. The total volume of the primary concrete was 19,250 m³ with reinforcement of 679 ton.

The contractual key date was set ready for draft tube liner installation by Lot IIIA Turbine Contractor. The actual hand over from Lot I Civil Works Contractor to Lot III A is shown in the table below.

Key Date for First Stage Concrete

Part of Works	Contractual Key Date	Completion Date	Remarks
Powerhouse sub-structure concrete ready for draft tube liner installation	15/12/94	Actual 01/12/94	No. 3 : 01/12/94 No. 2 : 17/01/95 No. 1 : 15/02/95

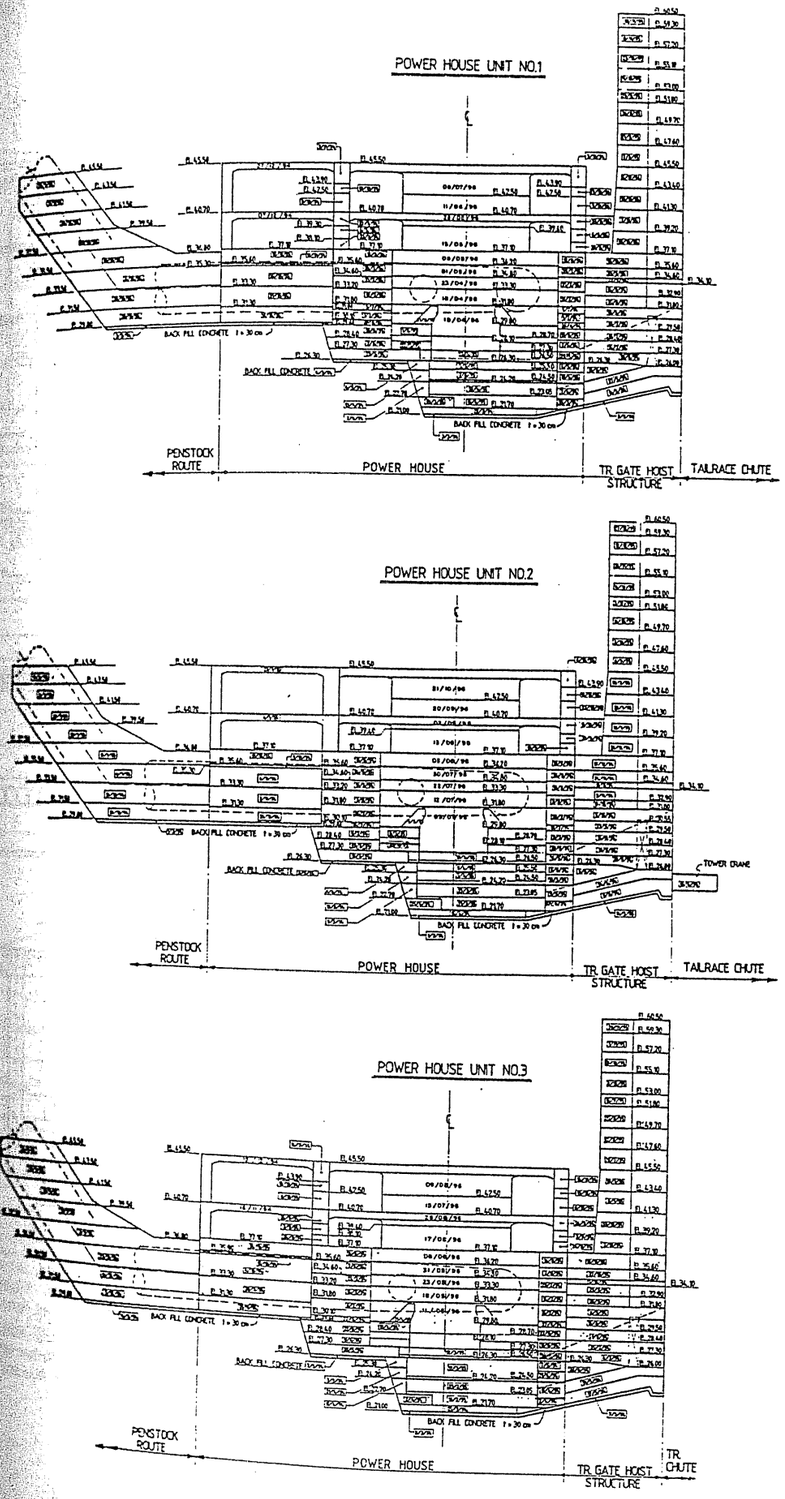


Fig. 2.6.4 Monthly Progress of Powerhouse Concrete

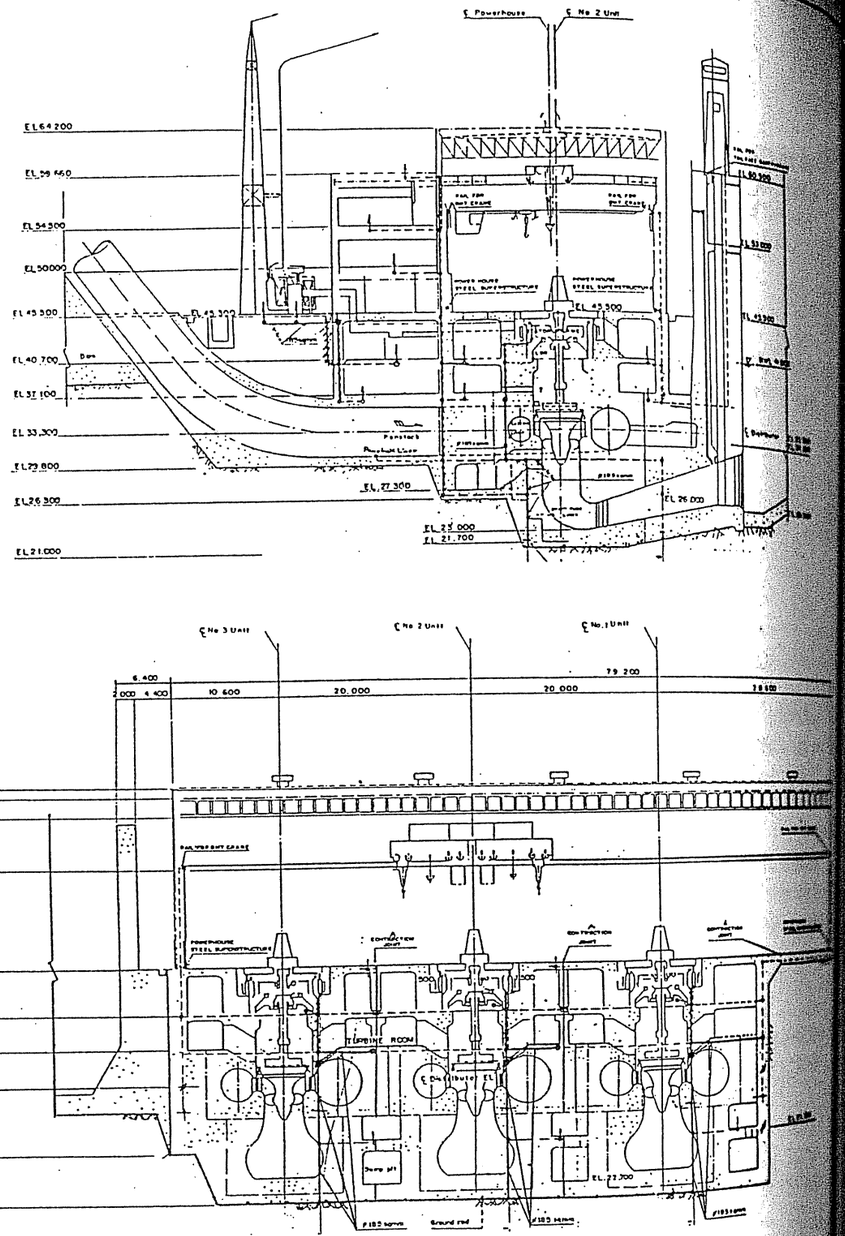
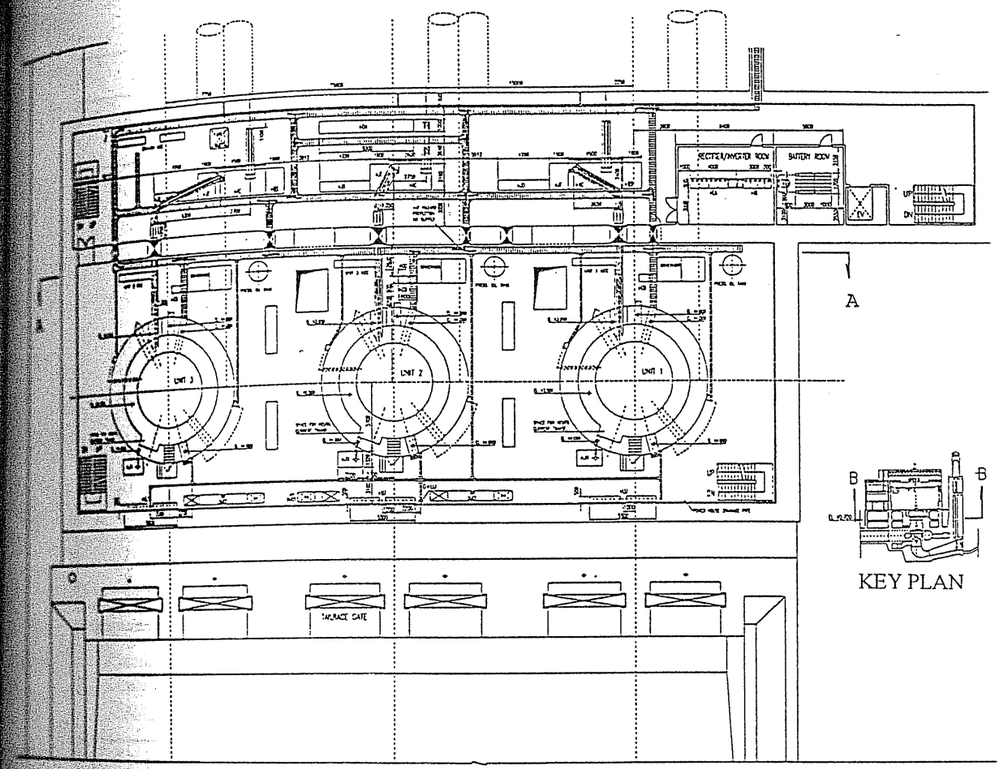


Fig. 2.6.5 Installation of Grounding Wire



B-B PLAN at EL. 40.700

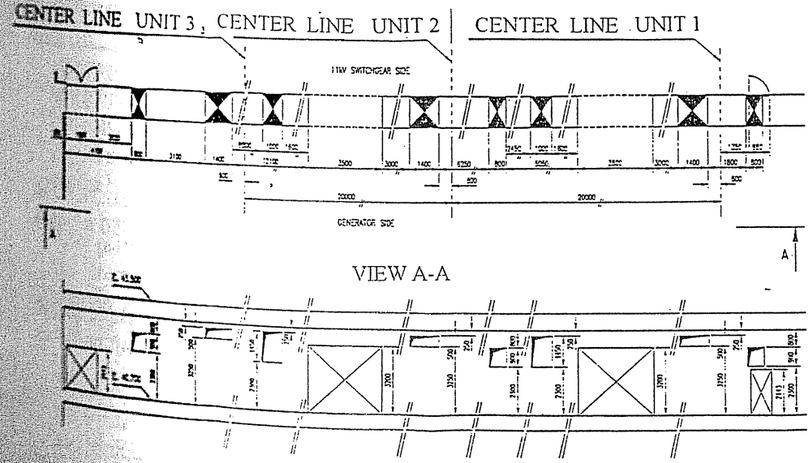


Fig. 2.6.6 Block out in First Stage Concrete
(Wall between Generator Room and Control Building)

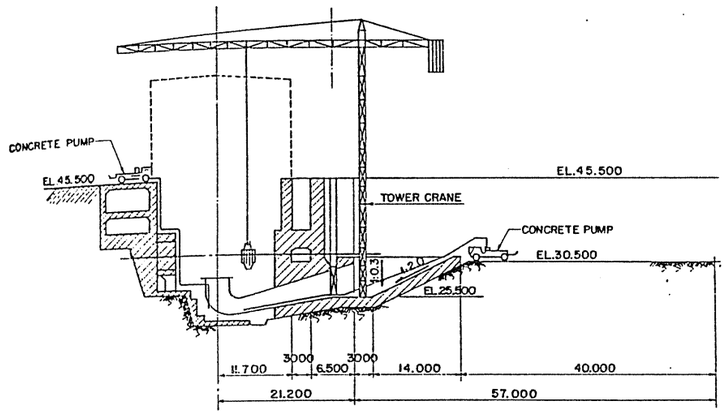
Table 2.6.1 Embedded Materials and Blockouts in Powerhouse

Schedule	Lot IIIA Contractor	Lot IIIB Contractor
Excavation		Grounding wire installation on final finishing rock
First stage concrete EL. 21.7 to EL. 37.1	Cooling water pipe Draft tube drain pipe Sump pit connection pipe Sump pit drain pipe Draft tube drain pipe valve spindle Sump pit grating block out Draft manhole gallery Spiral casing plinth Sole plates for spiral casing supports	
EL. 37.1 to EL. 45.5		Cut outs for wall and floor in control building. Battery room Inverter room Cubicle room Diesel engine generator Cable treatment room Communication equipment room
Second stage concrete	Draft tube drain pipe Sump pit drain pipe Cooling water pipe Gallery for draft manhole	
Third stage concrete	Winter Kennedy pipe Spiral drain pipe Stairs for turbine pit Servo motor anchor pit	Block outs around barrel Upper and lower brackets Stairs for generator room
Concrete for Architectural works		Block out in Wain Scott wall

(2) Second Stage Concrete

The second stage concrete was placed for embedment of the draft tube up EL.29.80 as shown in Fig. 2.6.7. The total quantity of the second stage concrete was 3,980 m^3 with reinforcement of 163 ton.

Concrete was transported to placing sites by concrete pump on the tailrace chute slab at EL.30.50, in the control building at EL.45.5, or by the tower crane. The layout is shown below.



Layout of Concrete Placing Equipment

Second Stage Concrete Schedule

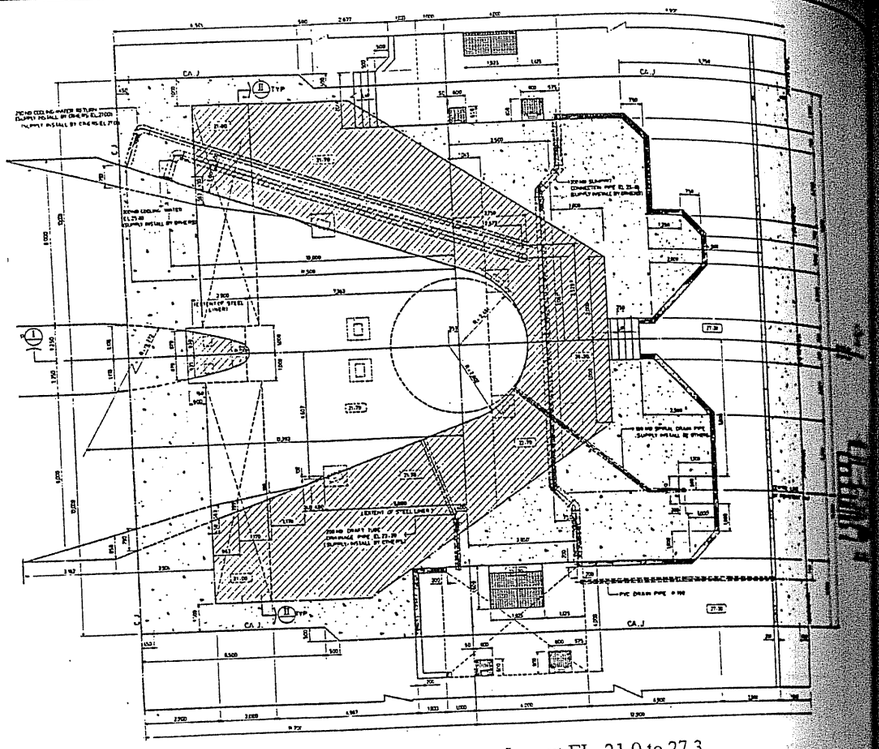
Date	Unit 1	Unit 2	Units 3
Commencement Date	15/4/95	21/3/95	26/2/95
Completion Date	21/6/95	5/6/95	24/4/95
Duration (months)	2.2	1.5	2.1

During this period of concrete work, the draft tube drain pipes, sump pit connection pipe, spiral casing drain pipe and cooling water pipes were embedded in the second stage concrete. The concrete pedestals for the spiral casing were also placed at EL.29.8 to support the spiral casing. These layouts are shown in Fig.2.6.8.

Although Lot I contractor proposed the first lift concrete at EL. 23.50 to ensure concrete filling under the draft tube, the first lift concrete was placed up to EL. 23.05, 5 cm above the bottom of the draft tube to avoid excessive buoyancy during concrete placement, as shown in Fig.2.6.7. Concrete filling under the draft tube was confirmed through openings on the bottom of the draft tube.

After placement of the secondary concrete, contact grouting was done to fill voids under the draft tube. The grout (W/C = 1) was injected by a hand pump at the maximum pressure 3 kg/cm^2. The total quantity of grout injection was at Unit 1 1805 liter, Unit 2 761 liter and Unit 3 837 liter respectively.

The steel structure for the powerhouse building work was erected from Unit-3 towards the erection bay. A tower crane was installed in the tailrace chute area for transportation and loading of material in the powerhouse and tailrace concrete works. The tower crane was post height 43m, jib length 50m, hoist capacity 150 t.m and Manufacture JIANGLE.JL-150.



Second Stage Concrete Layout EL. 21.0 to 27.3

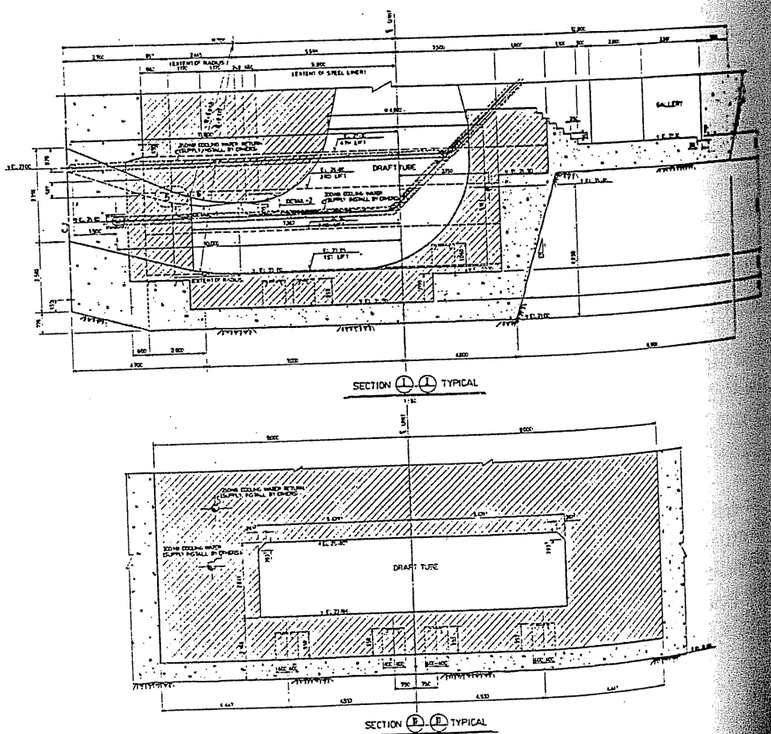
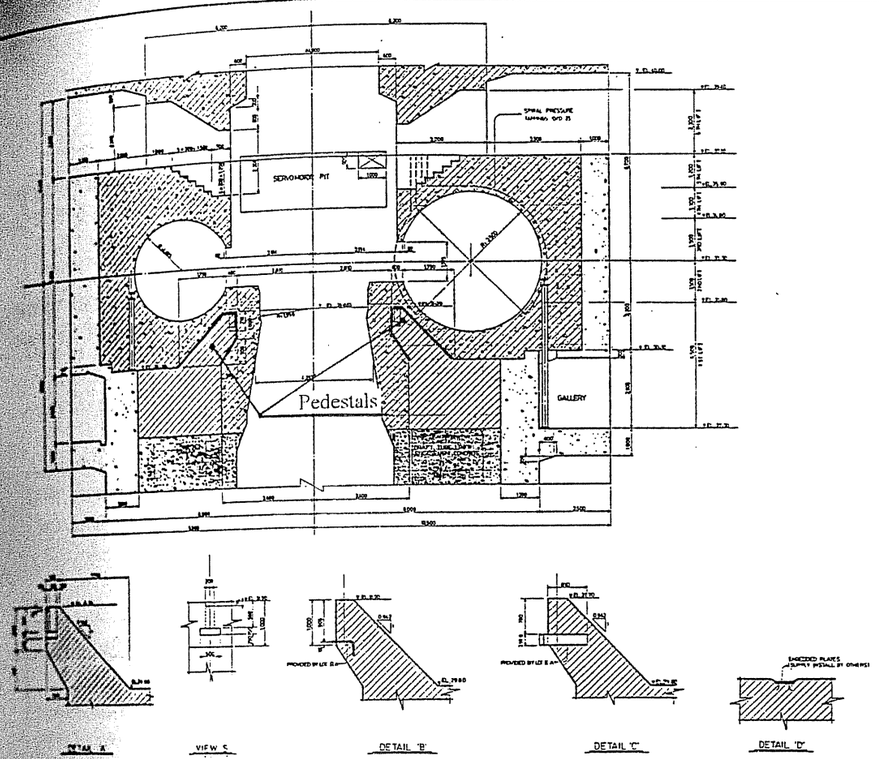


Fig. 2.6.7 Second Stage Concrete Around Draft Tube



Pedestal Concrete

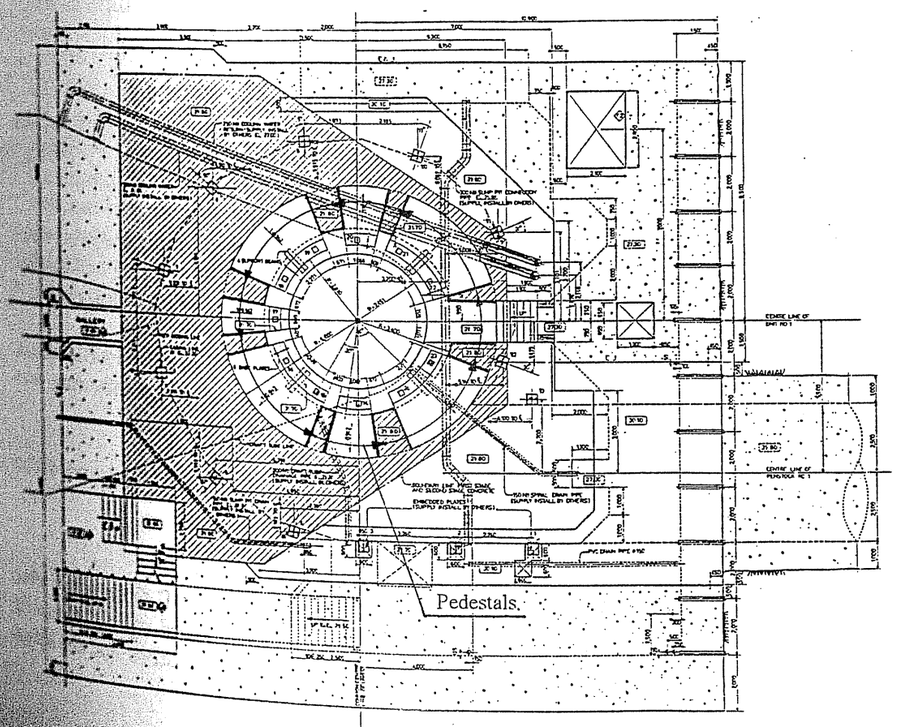


Fig. 2.6.8 Pedestals for Spiral Casing at EL. 29.80

(3) Third Stage Concrete

After completion of the second stage concrete up to the pedestals of the spiral casing at EL.31.8, concrete work was suspended for 10 months for installation of the overhead crane and spiral casing.

The third stage concrete commenced in April 1996 at Unit 1. The sequence of the concrete placement was from Unit 1, Unit 3 to Unit 2. The actual schedule of concrete placement for each unit is shown in the table below.

Progress of Third Stage Concrete

Date	Unit 1	Unit 2	Units 3
Commencement	10/4/96	3/7/96	11/5/96
Completion	8/7/96	21/10/96	8/8/96
Duration (months)	2.9	3.7	2.9

During the third stage concrete, the following embedded materials and block outs were installed, as shown in Fig. 2.6.9 and Fig. 2.6.10.

- Spiral casing including upper draft and runner chamber
- Penstock at closure of spiral casing
- Winter Kennedy Tappings
- Drain pipes for spiral casing and sumpit
- Cooling water pipes
- Block outs at EL.37.1 ; servomotor, ventilation duct, and oil pit
- At EL.40.5, the lower brackets cut outs
- At. EL.40.7, MCC Cable Tray Cut outs. Actuator pipe works cut outs and cut outs for air/oil receiver.
- At EL.42.5, the upper brackets cut outs

Reinforcement bars around the upper draft tube had to be set in the narrow space between the concrete and the draft tube liner. After coordination between the civil and turbine contractors, the reinforcement bars were installed before erection of the spiral casing.

Installation of the stay ring and runner chamber required high precision of location, therefore during placement of concrete, position and elevation of the stay ring was monitored by survey instruments to measure displacement.

During concrete placement for the third stage the penstock and the spiral casing were also connected and closed at CP20 and CP21, as shown in Fig.2.6.9.
The duration of the work is shown in the table below.

Closure of the penstock and spiral casing was carried out by Lot II Contractor after completion of concreting up to EL. 36.2, 40 cm above the spiral casing, to avoid displacement of the spiral casing by the closure work between the penstock and the spiral casing.

Progress of Penstock and Spiral Casing Closure Work

	Unit 1	Unit 2	Units 3
Commencement	29/04/96	18/06/96	25/05/96
Completion	11/06/96	02/09/96	08/07/96
Duration (months)	1.4	2.5	1.5

Due to delays in the installation of the spiral casing by Lot IIIA contractor, it was concluded at the 10th coordination meeting that the key date for powerhouse sub-structure ready for turbine and generator installation be revised from 1 June 1996 to 1 August 1996.

Although Lot I contractor commenced embedding the No. 1 spiral casing on 10 April 1997, some two months later than the original schedule, the contractor completed the No. 1 unit concrete works on 16 July, 1996 as shown in the table below.

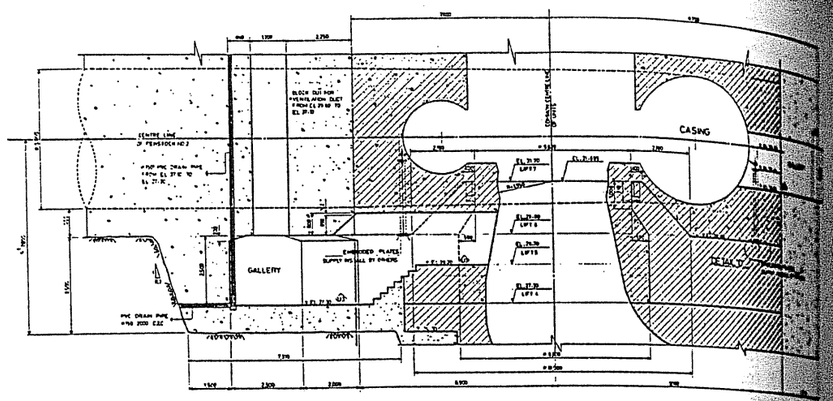
Key Date for Third Stage Concrete

Part of Works	Contractual Key Date	Completion Date	Remarks
Powerhouse sub-structure concrete ready for turbine and generator installation	01/08/96	Actual 16/07/96	No. 3 : 16/07/96 No. 2 : 16/08/96 No. 1 : 27/10/96

The total quantity for the third stage concrete work was 7,990 m^3 and reinforcement 524 ton.

(a) Lift-1 Concrete Placement

Figure 2.6.11 shows lift-1 concrete (EL.29.8-31.8) supply pipes for surrounding area of the upper draft and runner chamber. After the narrow section around the runner chamber was completed the lower part of the spiral casing was embedded in the concrete by using the pipe layout shown in Fig.2.6.12. Concrete was also injected to fill in the voids under the stay ring.



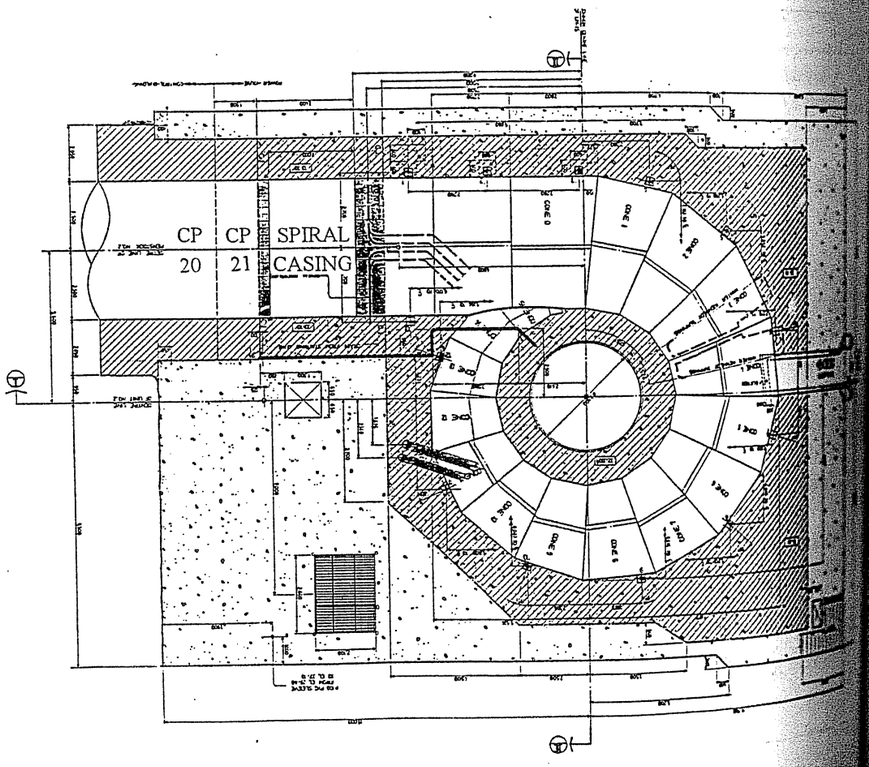
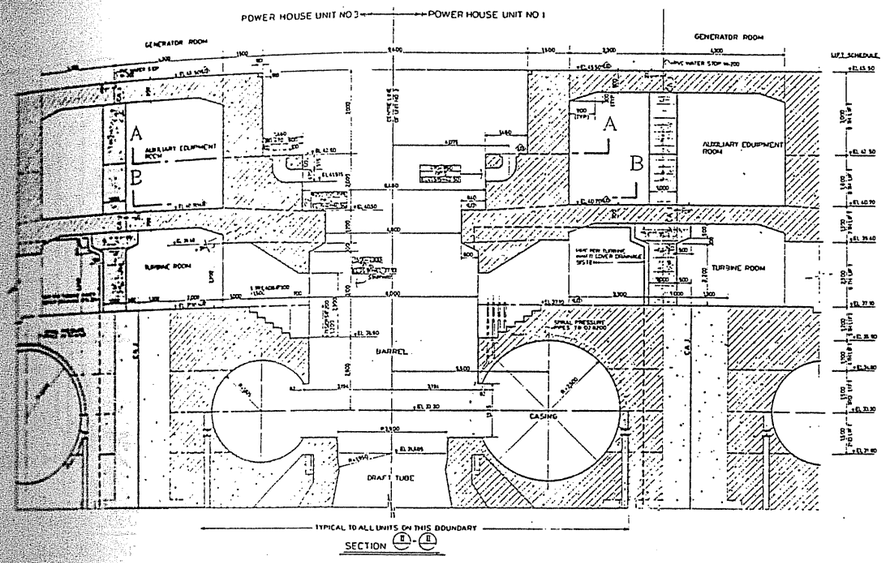
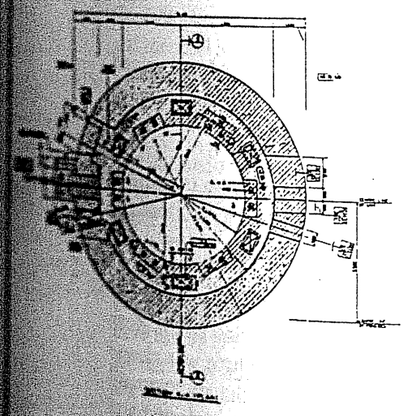


Fig. 2.6.9 Third Stage Concrete for Spiral Casing and Slab





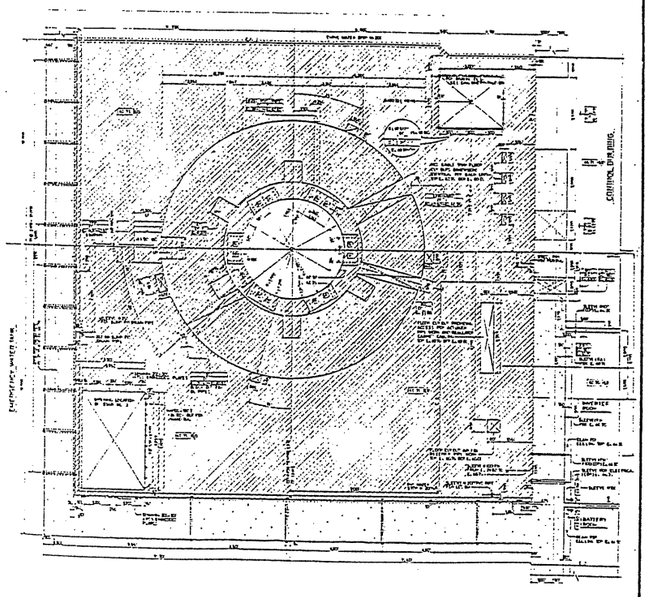


Fig. 2.6.10 Third Stage Concrete Around Generator Support

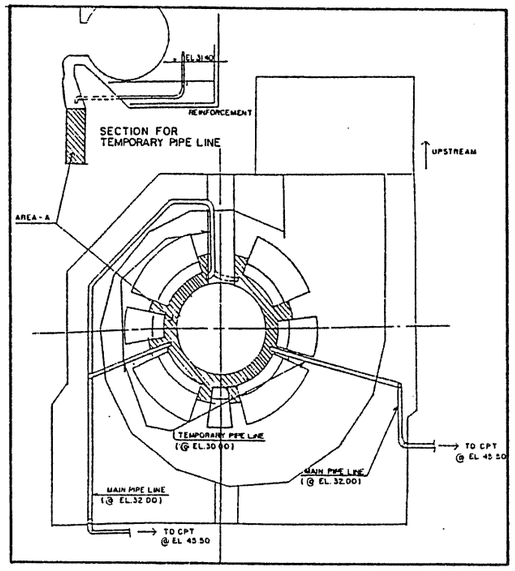


Fig. 2.6.11 Concrete Pipe Line Lift-1

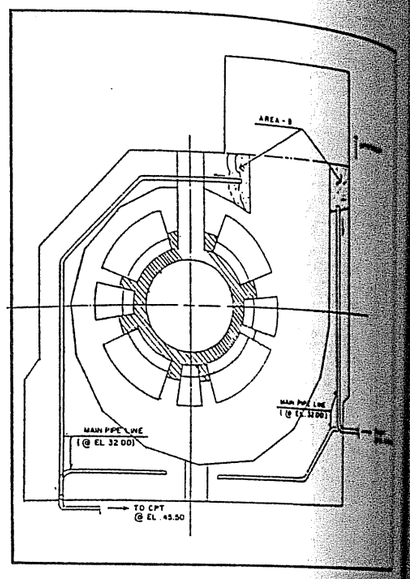


Fig. 2.6.12 Concrete Pipe Line Around Spiral Casing

(b) Mortar Filling

Mortar filling commenced after the 2nd lift concrete (EL.33.30) had hardened and gained enough strength to hold the embedded pipes during the mortar filling operation. Embedded pipes, shown in Fig.2.6.13, were solely used for mortar filling.

Mortar (MB4) was used to fill voids under the stay ring instead of concrete to prevent excessive pressure generating, since mortar is flowable and able to come out from the holes provided on the stay ring and bottom ring.

A concrete pump was used for injecting mortar, starting from the embedded steel pipe No.1. During injection, close monitoring on the hole provided on stay ring and bottom ring was done as shown in Fig.2.6.14. Once the mortar had come out, the hole was closed one by one with threaded plug (provided by Lot IIIA) until it reached the approximate location of embedded steel pipe No.2, then the pipeline connection was transferred and started injection on steel pipe No.2.

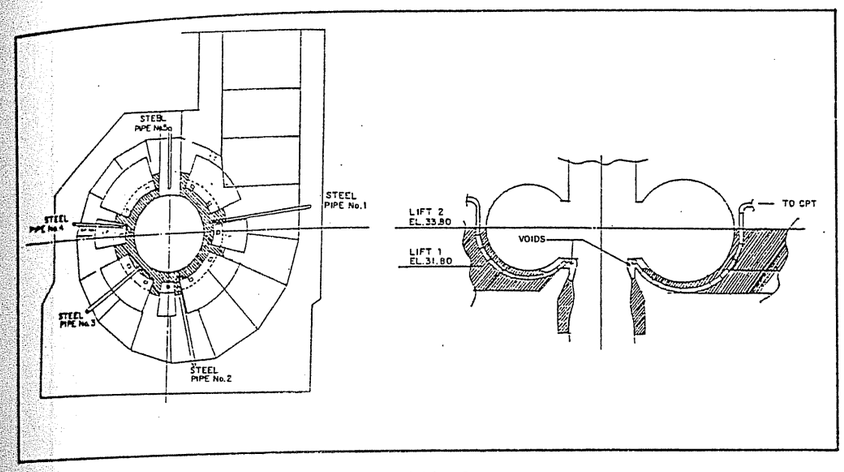


Fig. 2.6.13 Embedded Steel Pipe Layout

This procedure was repeated up to embedded steel pipe No.3. For injecting mortar on steel pipe No.5, a slow pumping operation of the concrete pump was implemented, grout holes on the stay ring and bottom ring approximately 1 meter on both sides were opened before the start of pumping, then they were closed one by one when mortar came out. The total quantity of mortar filling was of Unit 1 26.0m^3, Unit 2 39.0m^3, Unit 3 37.0m^3.

(c) Contact Grouting

Contact grouting was carried out to ensure the continuous bondage between the concrete and the steel lining of the spiral. Grouting commenced 7 days after mortar filling. Cement-water ratio of the grout material was cement-water (1:1 to 1:3). A manual pump was used to inject the grout with maximum pressure of 3 kg/cm^2.

The procedure of injecting grout is (i) replace one threaded plug with supply pipe threaded to fit in the provided grout holes on staying or bottom ring, (ii) open other plugs which are near to the supply pipe and start injecting grout through the supply pipe by hand pump. Then latter close each hole when mortar comes out, (iii) continue injection until the pressure reaches 3 kg/cm^2 and is stable for about 10 minutes (or as required by the Engineer), (iv) close the supply pipe as shown on detail (Fig.2.6.14), before disconnecting from supply hose of the pump, (v) transfer the supply hose to the next hole where mortar did not come out using a new supply pipe and (vi) repeat procedure until all grout holes are closed.

The total quantity of the contact grouting for the spiral casing of Unit 1 was 1736 liter, Unit 2 500 liter and Unit 3 527 liter.

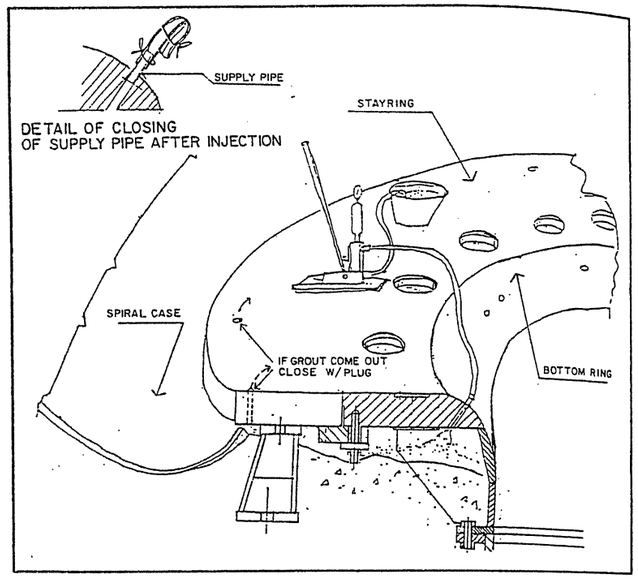


Fig. 2.6.14 Contact Grouting on Staying and Spiral Casing

6.4 Construction Performance Data

The summarized tables below are the main performance data such as summary of the excavation works and concrete placement, and main equipment for concrete placement.

Monthly Progress and Progress Rate of Excavation

Monthly Progress (m^3)			Progress Rate	
1993	Jul	5390	Total Volume (m^3)	106226
	Aug	7790		
	Sep	14680	Total Volume of Rock (m^3)	90224
	Oct	18870		
	Nov	24360	Number of Shift	509
	Dec	17370		
1994	Jan	9780	Ave. Progress Rate (m^3/shift)	209
	Feb	5290		
	Mar	2300	Total Explosive (kg)	17140
	Apr	300		
	May	50	Explosive Usage (kg/m^3. Rock)	0.19
	Jun	46		

Monthly Progress and Progress Rate of Concrete Placement

		1st Stage	2nd Stage	3rd Stage	Others
1994	Feb	124			
	Mar	991			
	Apr	1423			
	May	1165			
	Jun	2330			
	Jul	3349			
	Aug	1845			175
	Sep	1273	9		67
	Oct	1862	0		623
	Nov	1465	8		0
	Dec	1878	9		43
1995	Jan	991	8		77
	Feb	517	407		
	Mar	0	1069		
	Apr	24	1262		
	May		826		
	Jun		382		
	Jul				
	Aug				
	Sep				37
	Oct				118
	Nov	2			76
	Dec				96
1996	Jan				35
	Feb				
	Mar	5			
	Apr			834	
	May			2170	
	Jun			1089	
	Jul			1903	
	Aug			804	
	Sep			707	
	Oct			483	
Total		19250	3980	7990	1347
Working Days		154	38	44	19
Ave. Daily Progress (m^3/day)		125	105	182	71
Ave. Monthly Progress (m^3/month)		1203	284	615	135

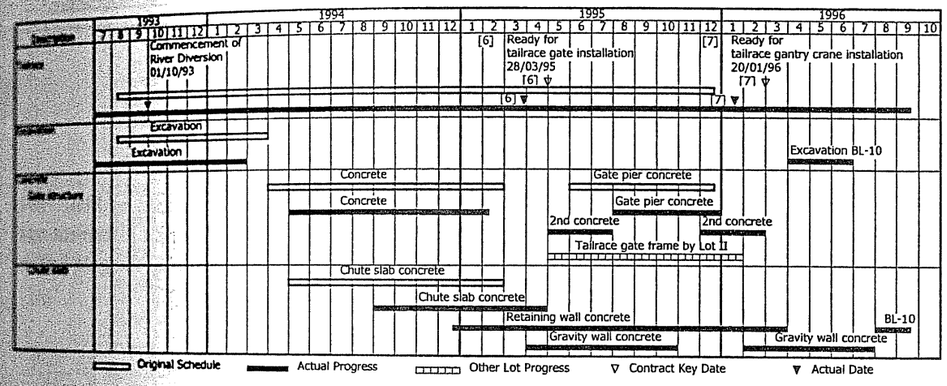
Main Equipment for Concrete Work for Powerhouse

Operation	Equipment	Capacity	Quantity
Concrete Mixing	General Batching Plant	30 m^3/hr	1
Transportation and Placing	Agitator Truck	4.5 m^3	1
		3.0 m^3	5
	Concrete Pump Truck	100 m^3/hr	2
	Crawler Crane	80 ton	1
	Concrete Bucket	1.0 m^3	1
Compacting	Hand Vibrator	ϕ 52 mm x L 366 mm	20
Cleaning, Green Cut and Curing	Air Compressor	18.5 m^3/min	2
	Reservoir Tank	7 kg/cm^2	1
	Water Supply Hose	Fibre Sheathed Rubber, ϕ 2"	100 m
		Fibre Sheathed Rubber, ϕ 1"	500 m
	Submersible Pump	ϕ 2"	4
Formwork	Tower Crane	150 ton. m	1
	Truck Crane	20 ton	1
	Roughterrain Crane	25 ton	1
	Truck with Crane	3 ton	1

Chapter 7 TAILRACE

7.1 General

The tailrace consists of the gate hoist structures, chute slab, retaining wall and gravity type retaining wall in the left bank. The gate hoist structures are located in the immediate downstream area of the powerhouse to install the gates at the outlet of the draft tubes. The chute slab covers the bottom of the tailrace waterway in the downstream area of the draft tube outlets. The retaining wall was installed to protect the left bank slope from EL.24.5 to EL.45.5. The gravity type retaining wall was constructed above the retaining wall to protect the powerhouse access road from the water level more than EL.45.5 during flood. The progress of tailrace work is shown below .



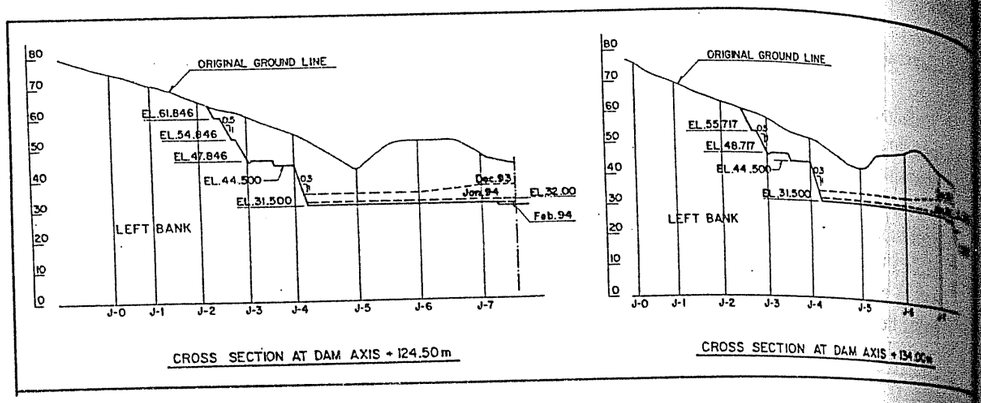
Progress of Tailrace Civil Works

7.2 Excavation

The first stage of excavation work for the tailrace was carried out concurrently with the powerhouse excavation, and completed down to EL.24.0 by February 1994.
During this first stage, excavation was completed for the gate hoist structures, chute slabs and the retaining wall below EL.45.5 as far as retaining wall BL. 10A

After relocation of the temporary access road which was located in the downstream cofferdam, the remaining area for the retaining wall and gravity type retaining wall was excavated in the downstream of Block No. 10A from April to June 1996.
The excavation monthly progress is shown in Fig.2.7.1.

Fig. 2.7.1 Tailrace Excavation Monthly Progress



7.3 Concrete Placement

(1) Gate Hoist Structure

Primary concrete placement for the gate hoist structures commenced in May 199 following the first stage of the powerhouse concrete. The primary concrete placement was suspended when the concrete reached EL.45.5 to allow for installation of the gate frames and the secondary concrete for the gate guide frames as shown in Fig.2.7.2.

Progress of Tailrace Gate Structure

Items	Unit 1	Units 2	Unit 3
Commencement Date	03/05/94	16/05/94	07/05/94
Completed up to EL.45.5	06/12/94	10/02/95	18/01/95
Resume	07/08/95	06/09/95	28/07/95
Completion up to EL.60.5	30/11/95	27/12/95	20/11/95

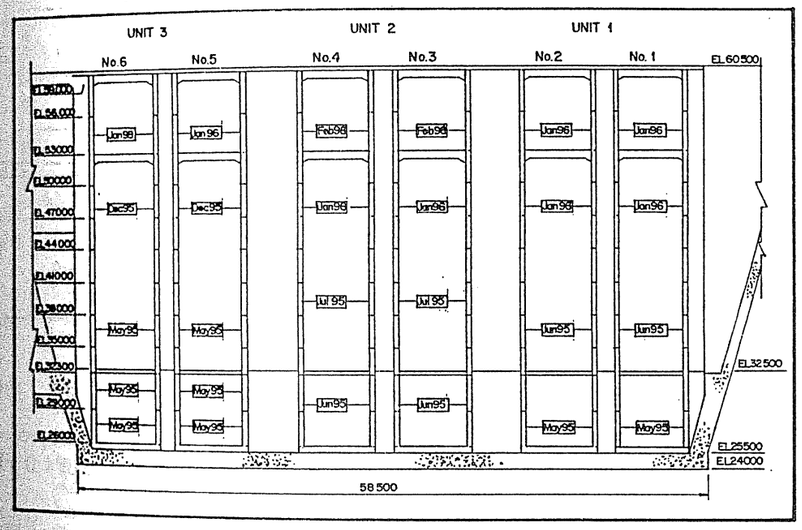
The secondary concrete for the gate frames and guide rails was placed in two periods from May to July 1995 up to EL. 45.5 and from December 1995 to February 1996 up to the crest at EL. 60.5.

Lot I Civil Works contractor's key dates were set ready for tailrace gate installation and gantry crane installation. The actual hand over dates from Lot I Civil Work contractor to Lot II Metal Works contractor are shown in the table below.

Key Date for Tailrace Works

Part of Works	Contractual Key Date	Completion Date	Remarks
Tailrace works ready for tailrace gate installation	01/05/95	Actual 28/03/95	No. 3 : 28/03/95 No. 2 : 20/04/95 No. 1 : 20/04/95
Tailrace works ready for tailrace gantry crane installation	01/03/96	Actual 20/01/96	20/01/96

Fig. 2.7.2 Tailrace Secondary Concrete Monthly Progress

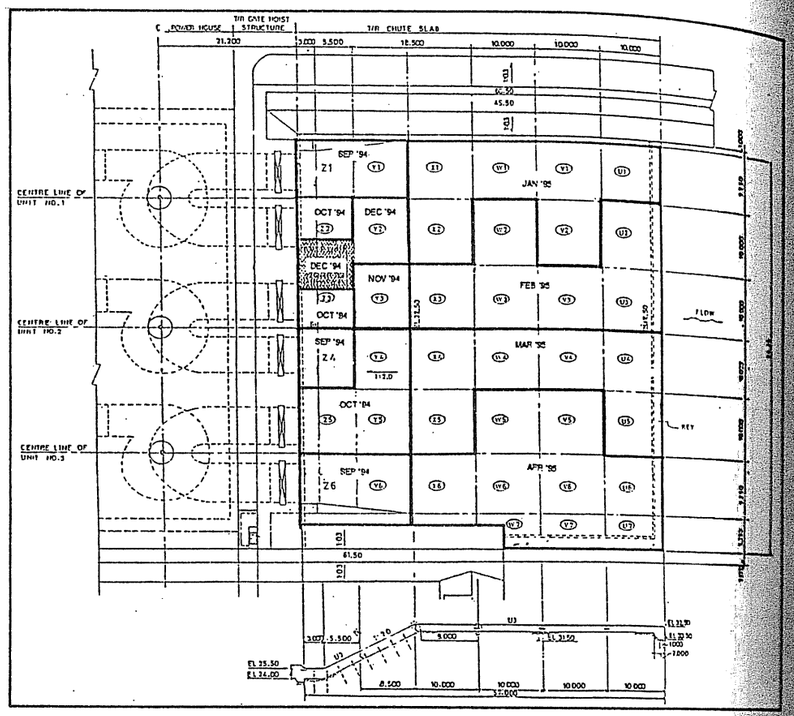


(2) Chute Slab

The chute slab was located in the downstream waterway of the tailrace. In the 1:2.0 slope area of the tailrace chute, the chute slab is 9.5m wide and 10m long with 1.1m thick concrete. In the horizontal area, the blocks are 10m wide and 10m long with 1.0m thick. Concrete placement commenced from the outlet of the tailrace gates EL.25.5 in September 1994.

The total volume 5,116 m^3 for the 39 blocks of the chute slabs were placed by April 1995 as shown in Fig.2.7.3.

Fig. 2.7.3 Tailrace Chute Slab Concrete Monthly Progress



(3) Retaining Wall

The retaining wall was built in the left bank waterway of the tailrace to protect the excavated rock surface. The wall has 1.0m thickness with surface slope of 1:0.3 below EL.45.5 and 1:1 between EL.45.5 and EL.54.6 as shown in Fig.2.7.4.

Construction of the wall commenced from the bottom of the wall at EL. 24.50 in December 1994 after completion of the gate structure up to the EL. 45.5. In April 1995 the second phase excavation was necessary from BL 11 to BL 14 where the cofferdam was partly removed in the downstream area. In the downstream area between BL 11 and BL 14 from the top of the retaining wall at EL. 45.5 to the foot of the gravity wall, a concrete slab was additionally installed to protect the slope surface from flood water. The whole structure was completed in September 1996. The total quantity of the concrete work was 3,881 m^3 with 43.7 ton of reinforcement.

(4) Gravity Retaining Wall

The gravity type retaining wall was constructed for 148.8 m along the powerhouse access road to protect the access road from floods. The width of the top concrete is 1.0m with 1:0.3 slope in the powerhouse access road side and vertical in the river side, as shown in Fig. 2.7.5. The structure comprises concrete with no reinforcement installed. The total volume of the concrete was 6,519 m^3.

Foundation rock was geologically suitable for construction of the gravity wall except the downstream area where additional excavation was required for 50 cm to reach D-CL class rock foundation. After excavation, concrete was placed block by block from BL 1 in the powerhouse entrance area to BL 14 in the downstream area. Concrete pump used for placement of 9 lifts of concrete from EL. 44.5 to EL. 60.5.

In the access road side, the gravity wall was backfilled with excavated material for construction of the access road to the powerhouse. During backfill a gravel layer was placed behind the wall to drain the ground water from the left bank.

7.4 Construction Performance Data

The summarized tables below are the main performance data such as the monthly progress, progress rate and main equipment for excavation and concrete placement works for the tailrace.

Monthly Progress and Progress Rate of Excavation

Monthly Progress (m^3)			Progress Rate	
1993	Jul	3690	Total Volume (m^3)	177,847
	Aug	6420		
	Sep	9630	Total Volume of Rock (m^3)	114,837
	Oct	22160		
	Nov	22100	Number of Shift	664
	Dec	23500		
1994	Jan	21700	Ave. Progress (m^3/shift)	268
	Feb	20100		
	Mar	18600	Total Explosive (kg)	19550
	Apr	14700		
	May	9400	Explosive Usage (kg/m^3. Rock)	0.17
	Jun	3300		
1996	Oct	200		
	Nov	400		
	Dec	300		
1997	Jan	300		
	Feb	300		
	Mar	400		
	Apr	300		
	Oct	347		

Monthly Progress and Progress Rate of Concrete Placement

		Tailrace Gate Structure		Chute Slab	Retaining Wall	Gravity Wall	Others
		1st stage	2nd stage				
1994	Apr	3		62			
	May	1013					
	Jun	715					
	Jul	657					
	Aug	761					
	Sep	1089		673			
	Oct	831		682			
	Nov	769		133	24		56
	Dec	803		161	131		0
1995	Jan	607		746	261		0
	Feb	147		732	446		67
	Mar	0		951	459		39
	Apr	0		916	258	184	0
	May	0	65		79	933	0
	Jun	0	83		67	1021	272
	Jul	129	17		167	999	0
	Aug	695	0		174	723	284
	Sep	393	0		169	274	0
	Oct	856	0		125	106	147
	Nov	930	0		112	0	144
	Dec	262	12	60	15	0	95
1996	Jan		69		36	0	40
	Feb		21		138	402	24
	Mar		0		22	633	
	Apr		22		0	354	
	May				0	0	
	Jun				0	629	
	Jul				169	261	
	Aug				579		
	Sep				439		
	Oct				11		
Total		10660	289	5116	3881	6519	1168
Working Days		101	53	57	34	119	26
Ave. Daily Progress (m^3/day)		106	5	90	114	55	45
Ave. Monthly Progress (m^3/day)		627	41	512	185	543	106

Main Equipment for Tailrace Excavation

Operation	Equipment	Capacity	Quantity
Drilling	Hydraulic Crawler Drill	165 kw	0
	Pneumatic Crawler Drill	135 kw	2
	Air Compressor	18.5 m^3/min	2
Excavation	Bulldozer with Ripper	45 ton	1
		32 ton	2
	Excavator	1.3 m^3	1
	Hydraulic Breaker	1,500 kg	1
Trimming	Wheel Loader	3.5 m^3	1
Loading		2.4 m^3	1
	Dozer Shovel	2.2 m^3	1
Hauling	Dump Truck	11 ton	4
		8 ton	28
Spreading and Leveling	Bulldozer	26 ton	2
(Disposal Area)	Vibration Roller	10 ton	1
Final Treatment	Hand Breaker	20 kg	1
	Pick Hammer	7.5 kg	2
	Air Compressor	18.5 m^3/min	1
	Reservoir Tank	3 m3, 7 kg/cm^2	0.5

Main Equipment for Concrete Work for Tailrace

Operation	Equipment	Capacity	Quantity
Concrete Mixing	General Batching Plant	30 m^3/hr	1
Transportation and Placing	Agitator Truck	4.5 m^3	1
		3.0 m^3	5
	Concrete Pump Truck	100 m^3/hr	2
	Crawler Crane	80 ton	1
	Concrete Bucket	1.0 m^3	1
Compacting	Hand Vibrator	ϕ 52 mm x L 366 mm	20
Cleaning, Green Cut and Curing	Air Compressor	18.5 m^3/min	2
	Reservoir Tank	7 kg/cm^2	1
	Water Supply Hose	Fibre Sheathed Rubber, ϕ 2"	100 m
		Fibre Sheathed Rubber, ϕ 1"	500 m
	Submersible Pump	ϕ 2"	4
Formwork	Tower Crane	150 ton. m	1
	Truck Crane	20 ton	1
	Roughterrain Crane	25 ton	1
	Truck with Crane	3 ton	1

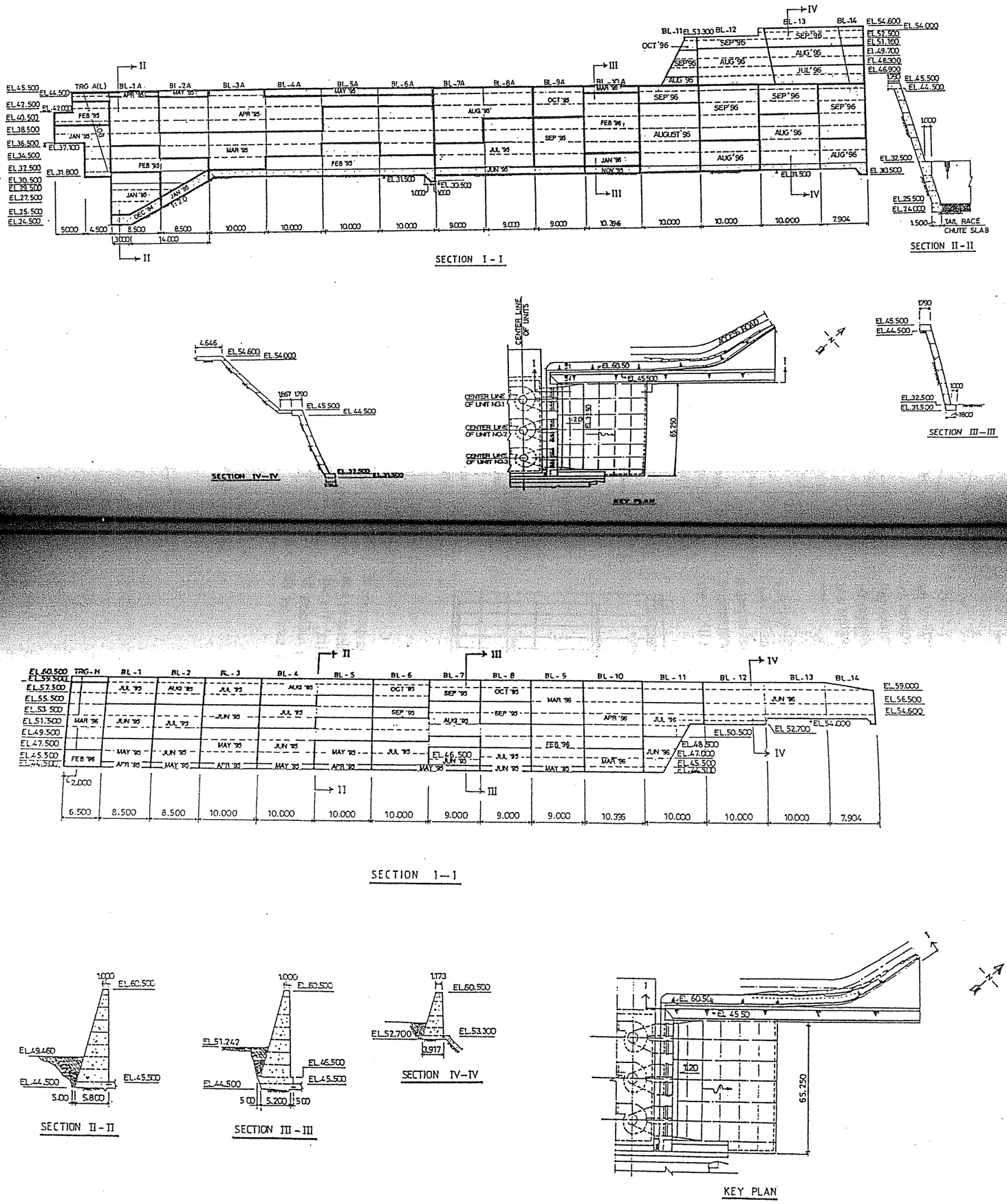


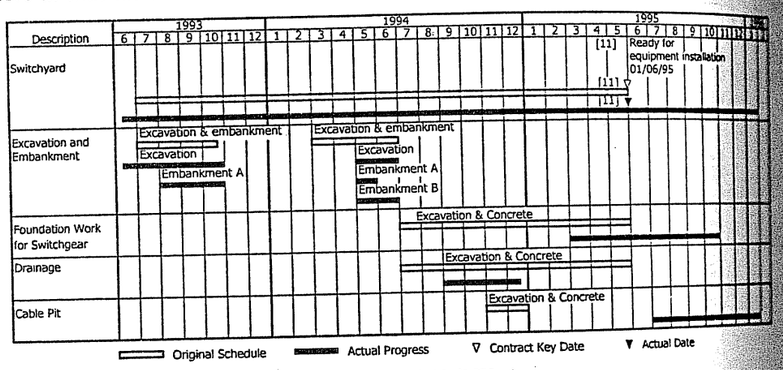
Fig. 2.7.5 Tailrace Gravity Wall Concrete Monthly Progress

Chapter 8 SWITCHYARD

8.1 General

The switchyard is located in the left bank mountains, about 500m north of the powerhouse, and occupies an area of approximately 15800 m^2 that is required for installation of 150 kVswitch-gear, various cable pits and a relay house. The main civil works for the switchyard were excavation and embankment for the site preparation, the switch-gear foundation, the drainage and cable trench works. The work progress is as shown below.

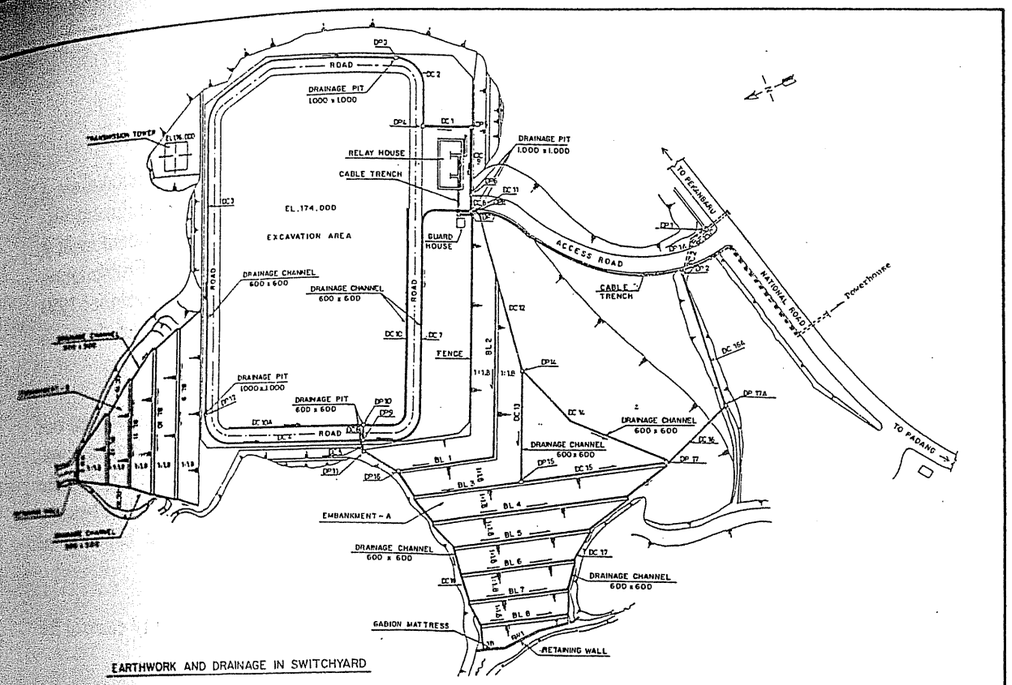
The contractual hand-over key date from Lot I Civil Work to Lot IIIC-1 Switchyard Generating Equipment was set on the 1st of June, 1995 for "switchyard foundation ready for equipment installation". On the date of this key date, the switchyard was inspected by the Engineer and the condition of the site was acceptable for hand-over as required under the conditions of Lot I Civil Work Contract. Access for Contractor Lot IIIC-1 was available at that time.



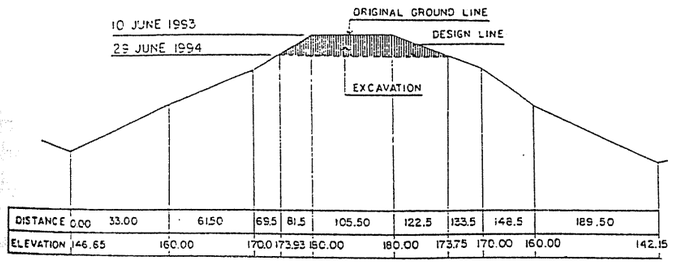
Progress of Switchyard Civil Works

8.2 Excavation

The top of the hill near the national road was excavated to EL. 174.0 for preparation of the switchyard area. It started in the middle of June 1993 and was completed by June 1994 as shown in Fig. 2.8.1. The excavation volume amounted to 124,831m^3. The rock condition of the switchyard area proved to be good after excavation sufficient for construction of the switchgear foundations. Excavated material was used for the embankment area in the switchyard.



EARTHWORK AND DRAINAGE IN SWITCHYARD



DISTANCE	0.00	33.00	61.50	69.5	81.5	105.50	122.5	133.5	148.5	189.50
ELEVATION	146.65	160.00	170.0	173.93	160.00	180.00	173.75	170.00	160.00	142.15

RECORD of EXCAVATION IN RELATION TO THE ELEVATION

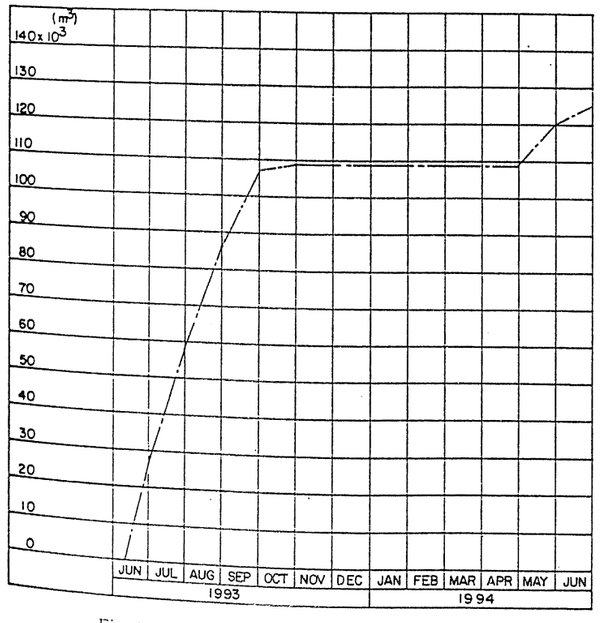


Fig. 2.8.1 Switchyard Excavation Progress

8.3 Embankment

Embankment work in the switchyard consists of two embankment areas, Embankment A and Embankment B, as shown in Fig.2.8.2. The total volume of fill embankment was 95,661 m^3, 70% for Embankment A and 30% for Embankment B. The Embankment A was commenced on 06th August 1993 and completed on 10th May 1994, and Embankment B was commenced on 13th April and finished on 11th June 1994. From November 1993 to April 1994, embankment work was suspended during the rainy season as shown in Fig.2.8.3.

Prior to commencement of embankment work, site tests for embankment were carried out to determine criteria for compaction method and moisture control. The trial embankment was compacted with D7G (22.48 ton) Bulldozer for various passes on the test site in the switchyard area. Density and moisture were measured by Radio Isotope test. Plate bearing test was also carried out after compaction to estimate loads bearing capacity of embankment.

Based on the results of the test, the following criteria was adopted for quality control for embankment to obtain 90% of the maximum dry density of 1.642 g/cm^3.

- Thickness of layer before compaction shall be 50cm.
- Number of passes for compaction shall be 4 times by D7G bulldozer.
- Moisture content shall be within 8% and 28% range. (Fig.2.8.4)

The result of quality control for density and moisture content in the embankment areas are shown in Fig. 2.8.5 and 2.8.6. Both figures indicate that embankment work satisfies the required criteria, 90 % of the minimum dry density and moisture content within 8 % and 28 %.

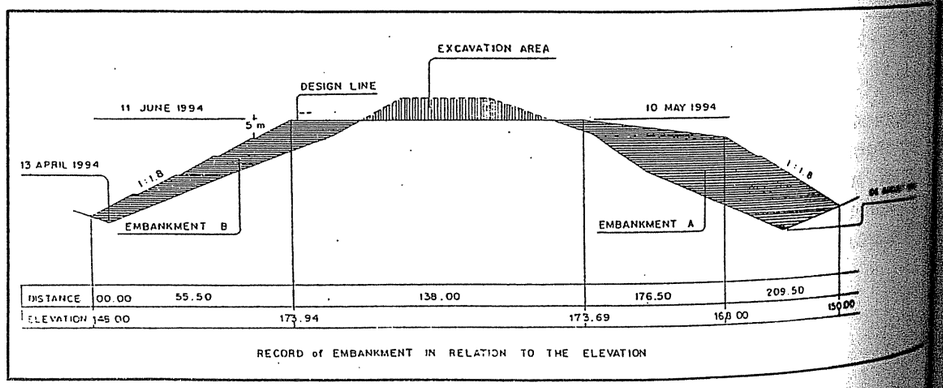


Fig. 2.8.2 Embankment Records

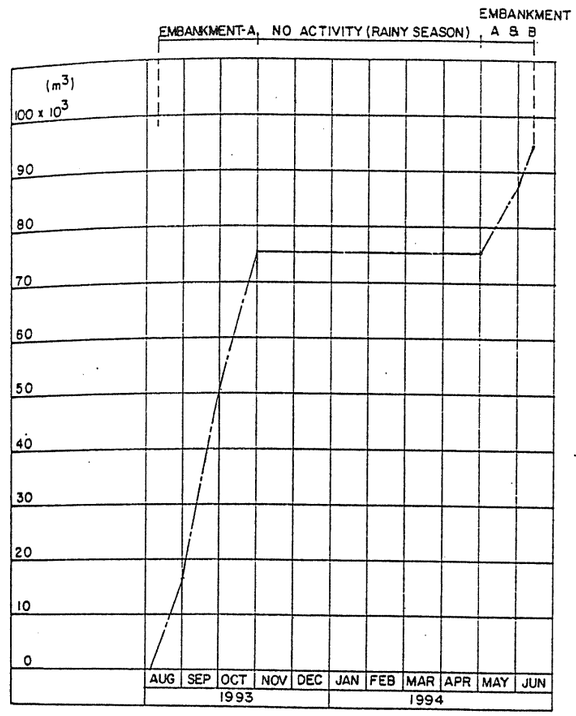


Fig. 2.8.3 Switchyard Embankment Progress

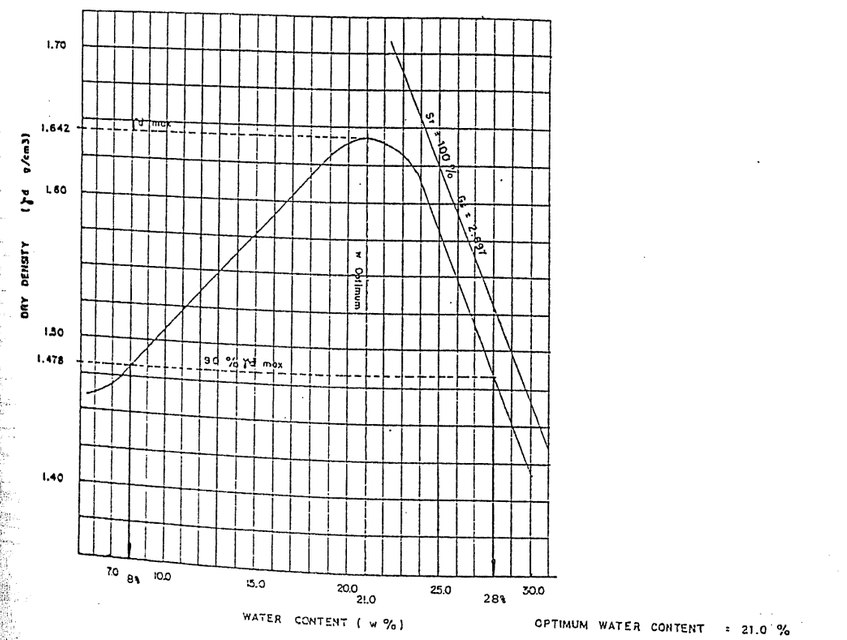
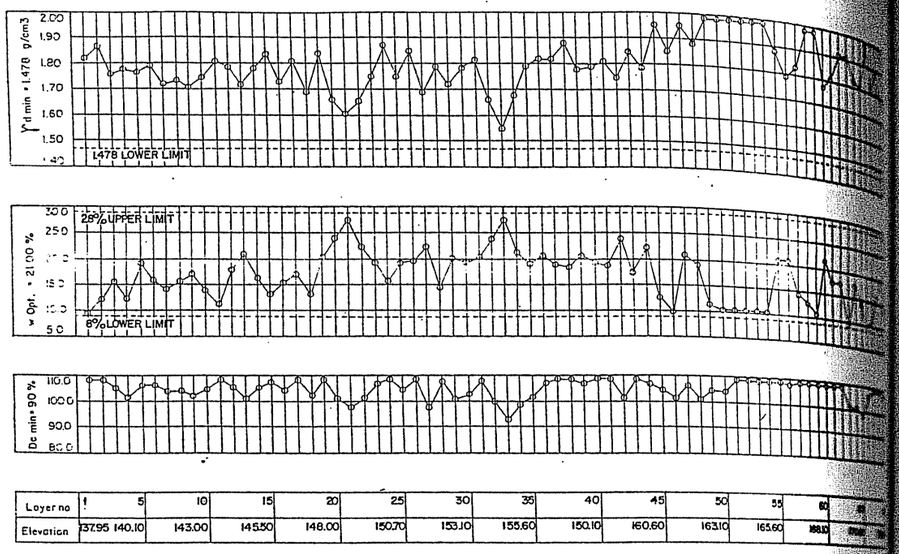
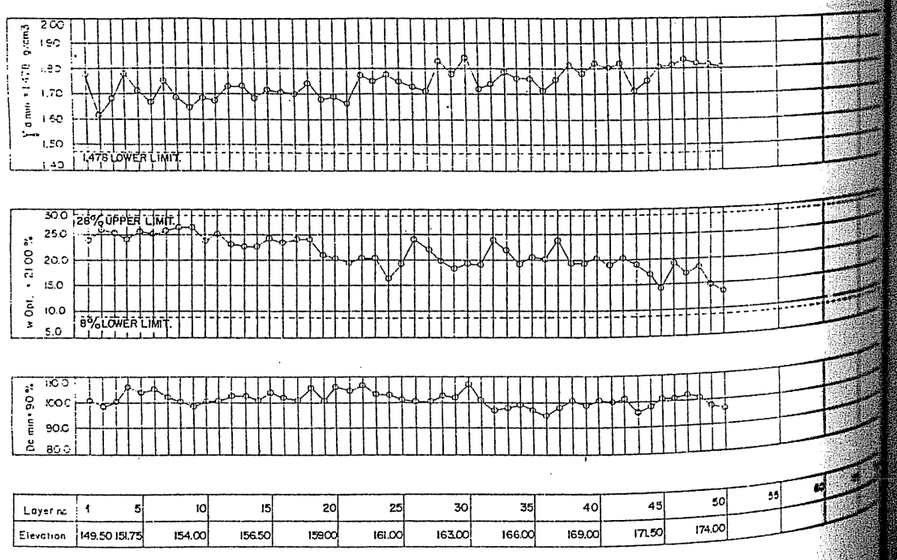


Fig. 2.8.4 Result of Compaction Test



Layer no	1	5	10	15	20	25	30	35	40	45	50	55	60
Elevation	137.95	140.10	143.00	145.50	148.00	150.70	153.10	155.60	158.10	160.60	163.10	165.60	168.10

Fig. 2.8.5 Quality Control of Switchyard Embankment : A



Layer no	1	5	10	15	20	25	30	35	40	45	50	55	60
Elevation	149.50	151.75	154.00	156.50	159.00	161.00	163.00	166.00	169.00	171.50	174.00		

Fig. 2.8.6 Quality Control of Switchyard Embankment : B

8.4 Concrete Works

(1) Drainage

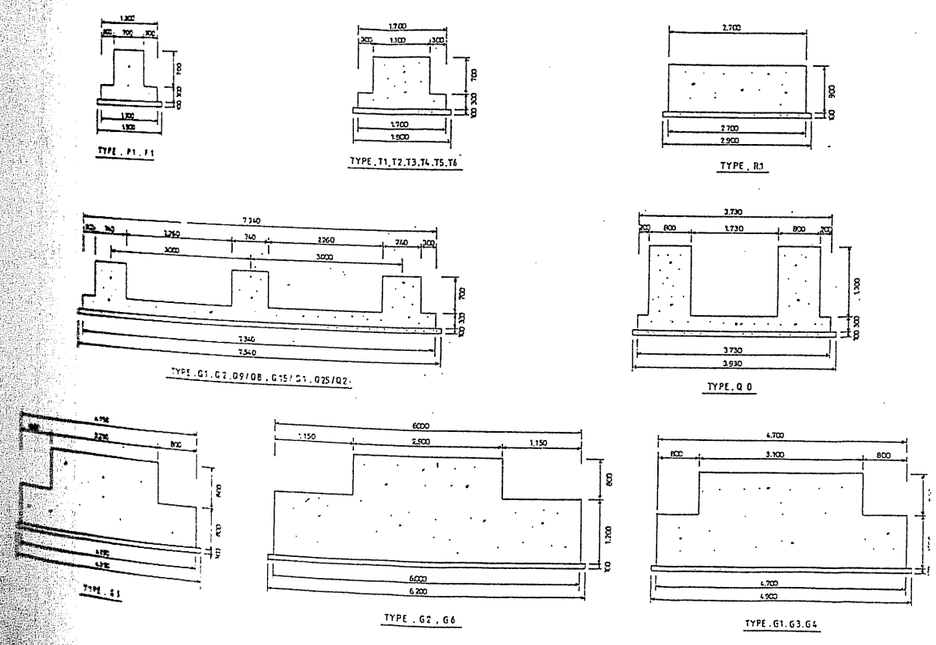
The drainage were mainly constructed in the embankment area and switchgear area. The drainage for embankment were installed along the breams and edge of embankment to protect surface slope of the embankment area, as shown in Fig.2.8.7.

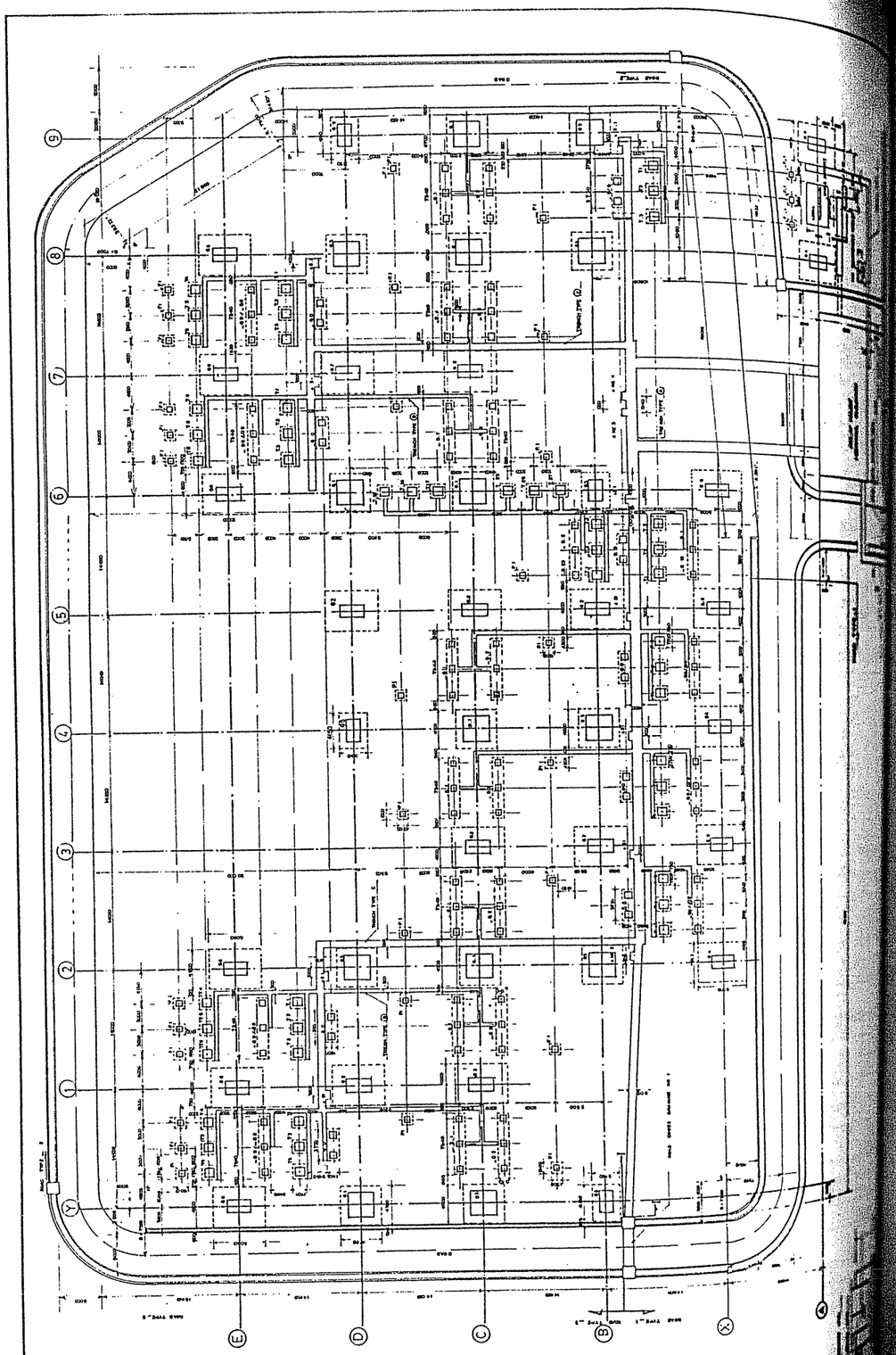
The drainage for the switchgear area was installed along the plant road to drain surface water. Construction commenced in September 1994 and was completed in December 1994. The total quantity of concrete was 616m^3 with 10 ton of reinforcement bars.

(2) Foundation Work

A total 167 foundations were built of various sizes for gantries and switchgear in the switchyard as shown below. During concrete work, anchor bolts for gantries were cast into the top of the foundations. Position and elevation of the bolts were measured from the grid lines from bench marks specially installed in the switchyard to ensure accuracy of installation. The grounding wire was also embedded in the foundation concrete for grounding work of the switchgear. After completion of concrete work, the foundations were backfilled with excavated material and compacted by mechanical tampar to obtain firm foundation for the cable trench.

Concrete placement commenced in March 1995 and all foundation work was completed in October 1995. The total volume of concrete was 1,777 m^3 with 43.9 ton of reinforcement bars.





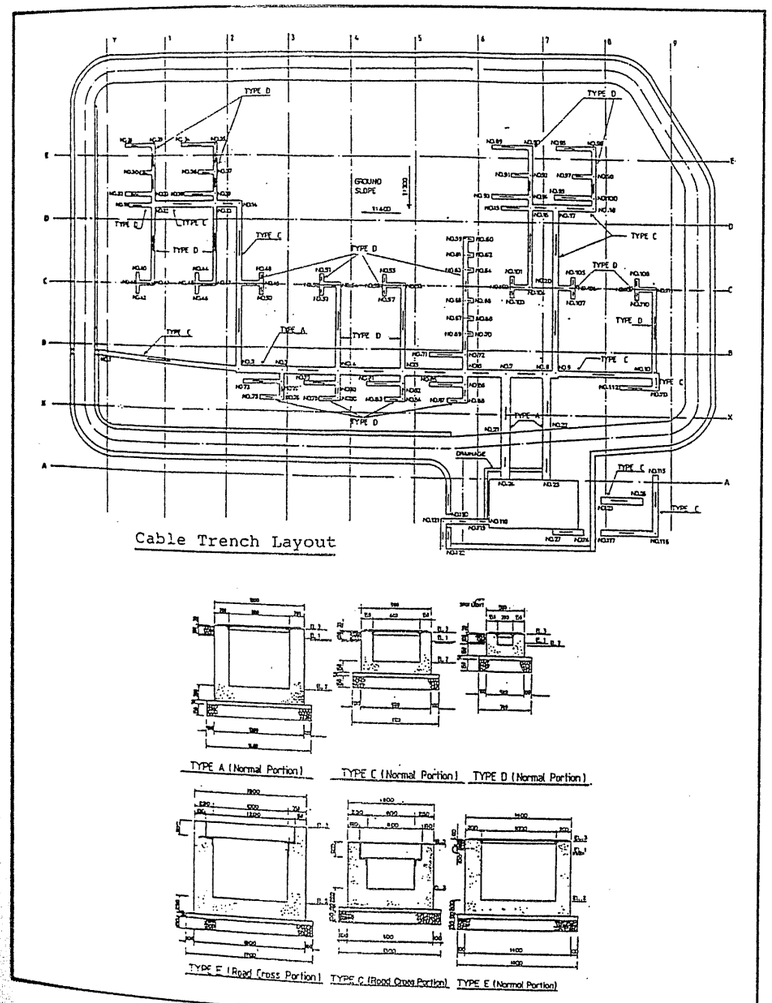
2-217

(3) Cable Trench

A cable trench was installed for various control cables from the switchgear to the relay house as shown below. Six different types of cable trench were constructed after completion of the foundation work. The covers of the cable trench were made of steel for normal portion and concrete cover for road crossing portion, both of which were set after completion of cable laying work by the Lot IIIC-1 contractor.

Concrete work for the cable trench commenced in July 1995 and was completed in January 1996.

The total quantity of concrete was 241 m^3 with 11.7 ton of reinforcement bars.



Switchyard Cable Trench

8.5 Pavement Work

Pavement work in the switchyard consists of the access road from existing national road to the entrance of the switchyard for 114 m and the plant road around the switchyard area for 418m. Excavation of sub-base commenced in May 1997. After completion of excavation, a sub-grade layer of 20 -30 cm thick was placed and compacted by a vibrating roller.

Following completion of the sub-grade, an asphalt treated base 4 cm thick and Hot Rolled Sheet in 3 cm thick were placed by an asphalt finisher and compacted by a tandem roller and a tire roller to obtain smooth surface of pavement.

8.6 Construction Performance Data

The summarized tables below are the main performance data such as the monthly progress and progress rate for excavation and embankment works, and main equipment for civil works.

Monthly Progress and Progress Rate of Excavation and Embankment

		Excavation		Embankment
		Common	Rock	
1993	May	4560	0	0
	Jun	16570	6420	3490
	Jul	11690	19920	13060
	Aug	11250	16260	33810
	Sep	7540	13210	25310
	Oct	340	1310	5310
1994	May	2530	8260	6810
	Jun	1153	3818	7871
Total Volume (m^3)		55633	69198	95661
Working Days		108	94	84
Ave. Daily Progress (m^3/day)		515	736	1139
Ave. Monthly Progress (m^3/month)		6954	9885	13666

Main Equipment for Switchyard Civil Work

Operation	Equipment	Capacity	Quantity
Earth Work			
Excavation	Bulldozer with Ripper	32 ton	2
	Bulldozer	26 ton	1
	Excavator	0.7 m^3	1
	Hydraulic Rock Breaker	1,500 kg	1
Embankment	Bulldozer	26 ton	2
	Excavator	0.7 m^3	1
	Vibration Roller	10 ton	1
Loading	Excavator	0.7 m^3	1
Hauling	Dump Truck	8 ton	8
Concrete Work			
Mixing	Batching Plant	30 m^3/hr	1
Transportation	Agitator Truck	3 m^3	3
Pouring	Excavator	0.7 m^3	1
Compaction	Hand Vibrator	ϕ 52 mm x L 366 mm	5

Chapter 9 DAM MONITORING AND INSTRUMENTATION

9.1 General

The Kotapanjang dam was designed to be safe against all of the external and internal loadings considered to act on the body and foundation in the design state. Therefore, it was necessary to monitor and control the construction program to ensure there was no significant departure from the design assumptions and the structural safety of the dam.

For this reason, monitoring on the behavior of the dam and its foundation was carried out. The aim was naturally to obtain information which may provide continuing assurance of the structural safety and water tightness of the dam, and identify any possible malfunctioning of the structure or its components, so that it may receive timely attention. Performance data also provides a basis for examining the validity of the design assumptions and needs to modify the analytical techniques in the future.

9.2 Plan of Measurement System

(1) Measurement items

The measurement instruments in the dam were selected for the following purposes.

Purpose	Items	Instruments
Assurance of structural safety of dam.	- Leakage - Uplift pressure	- Leakage meter - Pore pressure meter - Uplift meter - Water pressure meter
Control on construction program.	- Deformation - Temperature of dam concrete - Opening of vertical joints	- Collimation surveying - Thermometer - Joint meter
Improvement of design techniques in future.	- Uplift pressure - Deformation - Temperature of dam concrete - Seismic acceleration	- Seismograph

(2) Gauging section

In this dam, the dam body is divided into three structural parts, the non-overflow part, the overflow part and the part with penstock. One section at each of the three parts was selected as the gauging section for monitoring during and after impounding. J7+9m (BL.8), J12+7.5m (BL.13) and J4+10m (BL.5) respectively as shown in Fig. 2.9.1. The section J7+9m (BL.8) was also selected as the gauging section of monitoring during the construction period because this section is in the non-overflow section with maximum height in the dam.

(3) Measurement plan and locations

The measurement plan and instruments in the dam are shown in Table 2.9.1 and their locations are shown in Fig. 2.9.1, which were determined in accordance with experience of conventional concrete dams.

Table 2.9.1 Measurement Plan

Measurement Item	Measurement Instruments	Number	Location of Installation
Leakage	Leakage meter (Triangular weir and pressure gauge)	2 set	Gallery
Uplift pressure	Pore pressure meter	9 pcs	Foundation of BL.5, 8 and 13
	Uplift meter	4 set	Foundation of BL.8
	Water pressure meter	14 set	Drain hole in the gallery
Deformation of dam	Surveying	1 line	Dam crest
Concrete temperate	Thermometer	15 set	BL.8
Opening of vertical joint	Joint meter	20 set	Joint 7 and 8
Seismic acceleration	Seismograph (Three components)	2 set	Gallery and the dam crest of BL.13

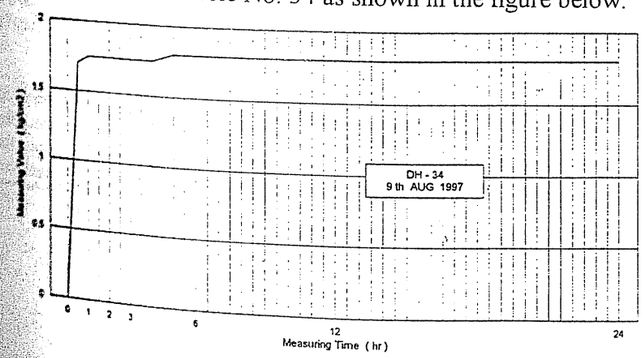
(i) Leakage

Two (2) sets of automatic leakage meters were installed in the gallery in order to measure the total leakage of the left and right abutments of the dam. In addition, measurements of leakage at each drain hole and joint drain pipe are being carried out manually.

(ii) Uplift pressure

Three (3) pore pressure meters were embedded in the rock foundation at each gauging section and in the rock foundation at J7+9m, four (4) sets of uplift meters were embedded additionally in order to obtain more precise information about the uplift pressure acting on the dam base.

In addition, measurement of the uplift pressure using water pressure meters attached at drain holes are being carried out manually. The measurement for the uplift pressure by the water pressure meter is carried out at two hours after closing the gauge cock. This is based on the measuring result between uplift pressure and closing time at drain hole No. 34 as shown in the figure below.



(iii) Deformation

Regarding to the measurement of deformation in a concrete gravity dam, measurement by plumb line is more efficient. But there is no suitable dam block in the Kotapanjang dam to install the plumb line because all of large suitably sized concrete blocks have large openings for spillway or penstock. Therefore, in this dam, deformation of the dam is measured by collimation survey using targets and instrument piers installed at the dam crest and both abutments of the dam.

(iv) Temperature of dam concrete

Five (5) thermometers were embedded on one gauge line at three different elevations in the section of J7+9m.

(v) Opening of vertical joint

Three (3) joint meters were embedded on one gauge line at three different elevations in the section of joint 7 and joint 8.

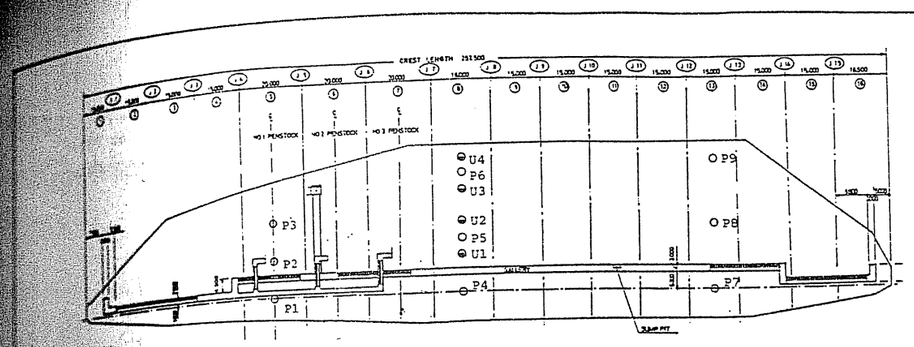
(vi) Seismic acceleration

Two (2) sets of seismographs were set on the dam crest and the floor in the dam gallery of BL. 13.

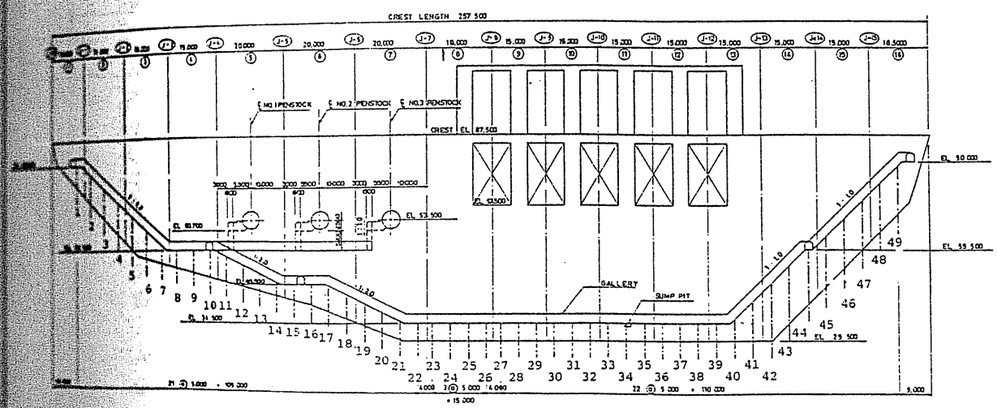
(d) Measuring Interval

The basic measuring intervals during and after impounding and routine monitoring periods thereafter are shown in the following table. The measuring periods are divided into three, 1st period during impounding , 2nd period after initial impounding when the behavior of dam becomes stable and 3rd period after the 2nd period.

Measurement Item	Measuring Interval			
	During and After Construction	During Impounding	After Impounding (Plan)	
		1st Period	2nd Period	3rd Period
Leakage	-	once a day	once a week	once a month
Uplift pressure	once a day (only pore pressure meters)	once a day	once a month	once three months
Deformation	-	once a week	once a month	once three months
Temperature of concrete	once a day	once a day	once a month	once three months
Opening of vertical joint	once a day	once a day	once a month	once three months
Seismic acceleration	-	each occasion	each occasion	each occasion



PLAN



PROFILE (UPSTREAM ELEVATION)

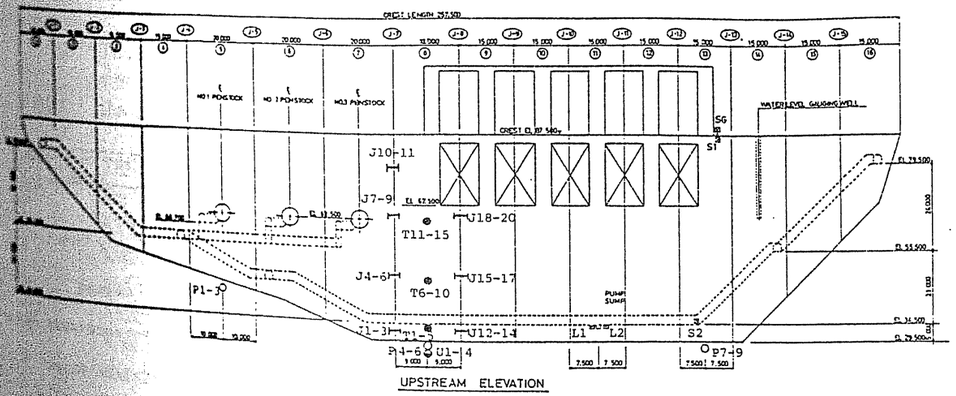


Fig. 2.9.1 (1) Monitoring Points of Dam

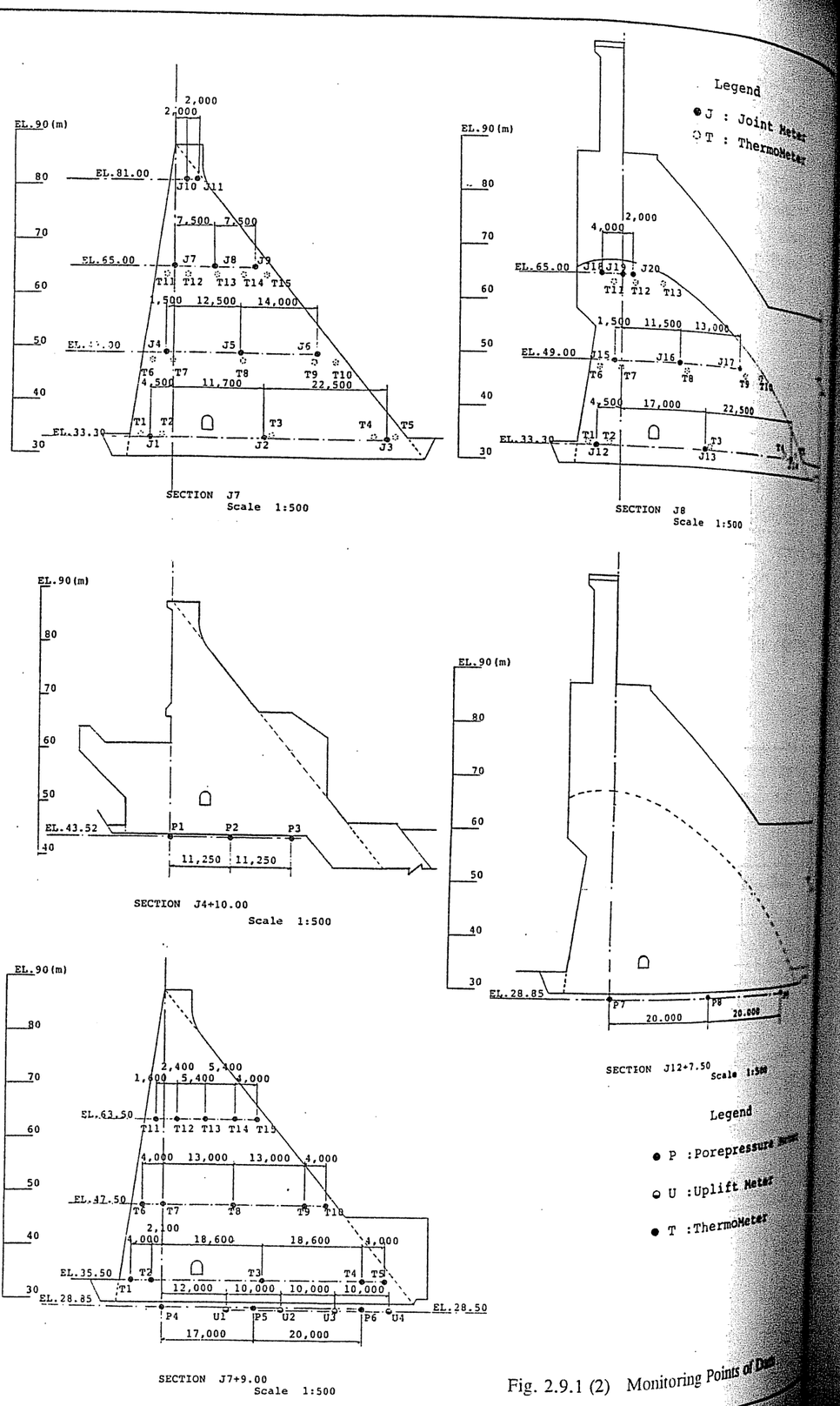


Fig. 2.9.1 (2) Monitoring Points of Dam

9.3 Monitoring Results and Evaluation

(1) Temperature

(i) Temperature Control in Kotapanjang Dam

During the process of hardening of the concrete, the rise in temperature of concrete takes place by heat of hydration. In massive concrete as in the dam, the heat of hydration of cement does not radiate rapidly. After the concrete has hardened, it gradually cools down to the mean ambient temperature due to heat transfer. In this process of falling temperature, the internal and external restraints prevent contraction and displacement of the concrete therefore generating tensile stresses in the concrete.

Cracks occur in the concrete when the tensile stresses develop and exceed the tensile strength of concrete. If cracks due to falling temperature occur in the dam body, the dam may lose its stability and water tightness.

Therefore, concrete placing was carried out based on temperature regulation which might reduce the heat build up during the hydration process thereby reducing the contraction and tensile cracks developing. Monitoring of the temperature distribution in the dam concrete is necessary to check and examine the validity of the design assumptions and modifying the analysis techniques.

At the Kotapanjang dam, from the results of the thermal stress analysis, the temperature control of the dam concrete was regulated such that the temperature of concrete during placing should be lower than 30^0C and the maximum temperature of dam concrete will not exceed 48^0C.

(ii) Monitoring Results and Evaluation

The measurement records of thermometer T6-T10 are shown in Fig. 2.9.2, which were set at EL.47.50 in Block 8. The distributions of dam concrete temperature through the construction period to the impounding period are shown in Fig. 2.9.3.

The temperature of dam concrete was high soon after concrete was placed due to the heat hydration of cement and, then descended steadily gradually becoming 27-33^0C by the end of February 1996. The temperature of concrete placed at outer and/or higher elevations decreased more rapidly than that placed at the inner and/or lower elevations because of the difference in the distance of heat transfer. Maximum temperatures just after placing the concrete reached about 49^0C which was quite high, however, there was no serious symptom like large size cracks in the dam concrete.

Since the beginning of March 1996, the temperature of the outer concrete, mainly on the downstream side of the dam fluctuated between 30-33^{0}C. This was considered to be caused by heating by solar radiation, while the temperature of the inner concrete descended gradually and continuously until it reached 30-31^{0}C by the beginning of March 1997.

After start of impounding, the temperature of upstream side of the dam concrete started to descend and reached about 25.5 ^{0}C by the end of February 1998. This was caused by the cooling down by the reservoir water, while the temperature of the downstream side and inner part of the dam concrete showed a seasonal fluctuation in the range 30-33^{0}C and steady descend respectively.

Judging from the monitoring result of the temperature of the dam concrete, the temperature is steadily cooling down to the final stable temperature of the dam. Seasonal fluctuation of the temperature is very small and therefore the small thermal stresses occurring in the dam concrete have no influence on dam stability.

(2) Opening of Vertical Joint

The measurement records of joint meter J4-J6 and J15-J17 are shown in Fig. 2.9.4, which were set at EL. 49.50 in joint 7 and joint 8.

The movement at vertical joints during and after construction were considered to be caused by the change of the dam concrete temperature. The maximum opening from the joint meter mentioned above is less than 1.0mm which is smaller than 2.7mm estimated in the design calculation.

(3) Leakage

(i) Leakage from drain hole

The serial records of the total leakage from the drain holes and of the leakage from each drain hole are shown in Fig. 2.9.5 and 2.9.6 respectively. The relation between reservoir water level and the total leakage from the drain holes is shown in Fig. 2.9.7.

- Before impounding

Before the commencement of impounding, drain hole No. 22 (BL.7), No. 23 (BL.8), No. 32 (BL.10), No.33, No. 35 (BL.11), No. 36, No. 38 (BL.12) and No. 39 (BL.13) showed some amount of leakage and the maximum leakage for this period was recorded 3.3 ℓ/min at drain hole No. 39. These leakage measurements before initial impounding were considered to be mainly caused by natural ground water pressure which was observed during the construction period.

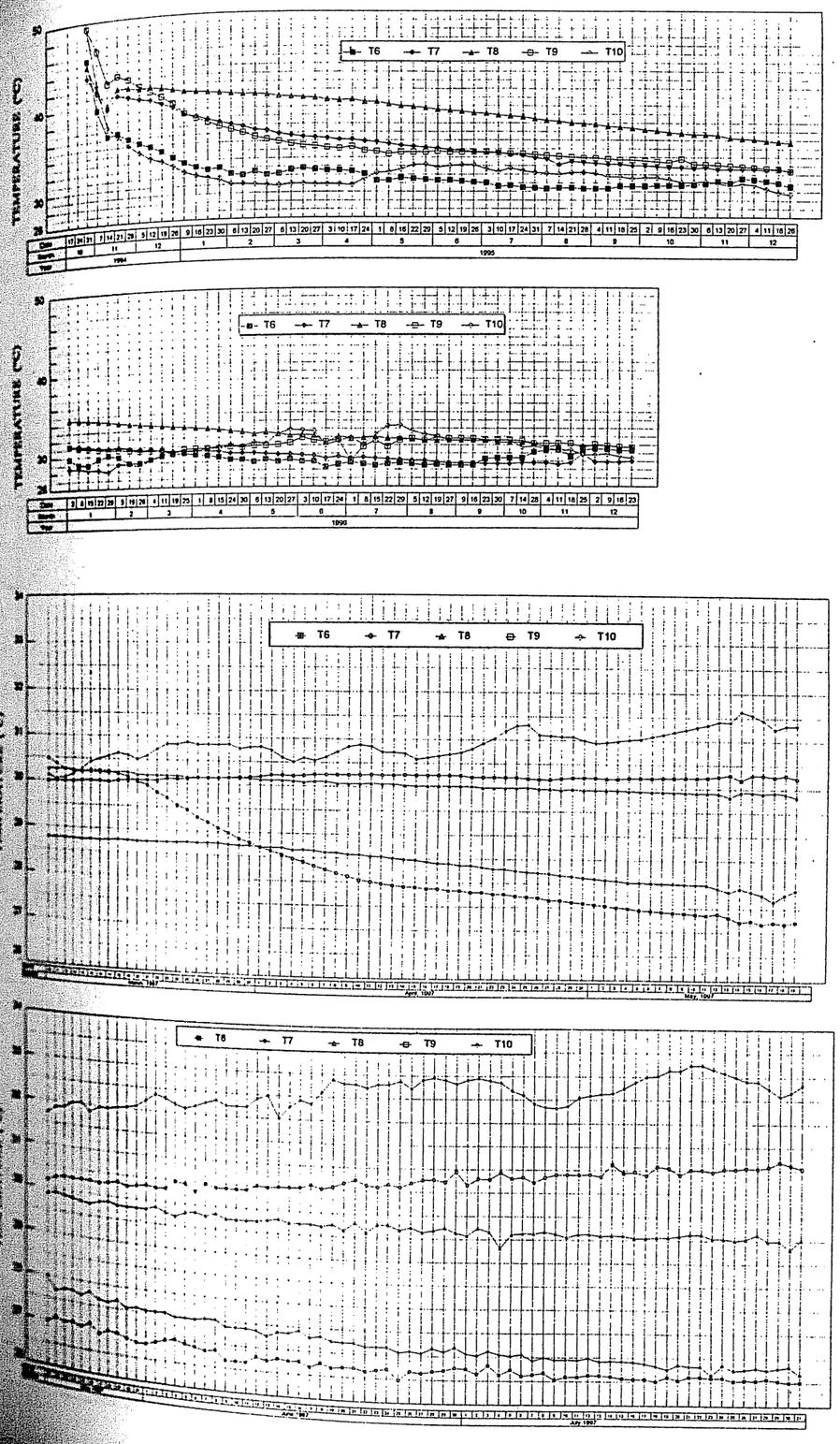
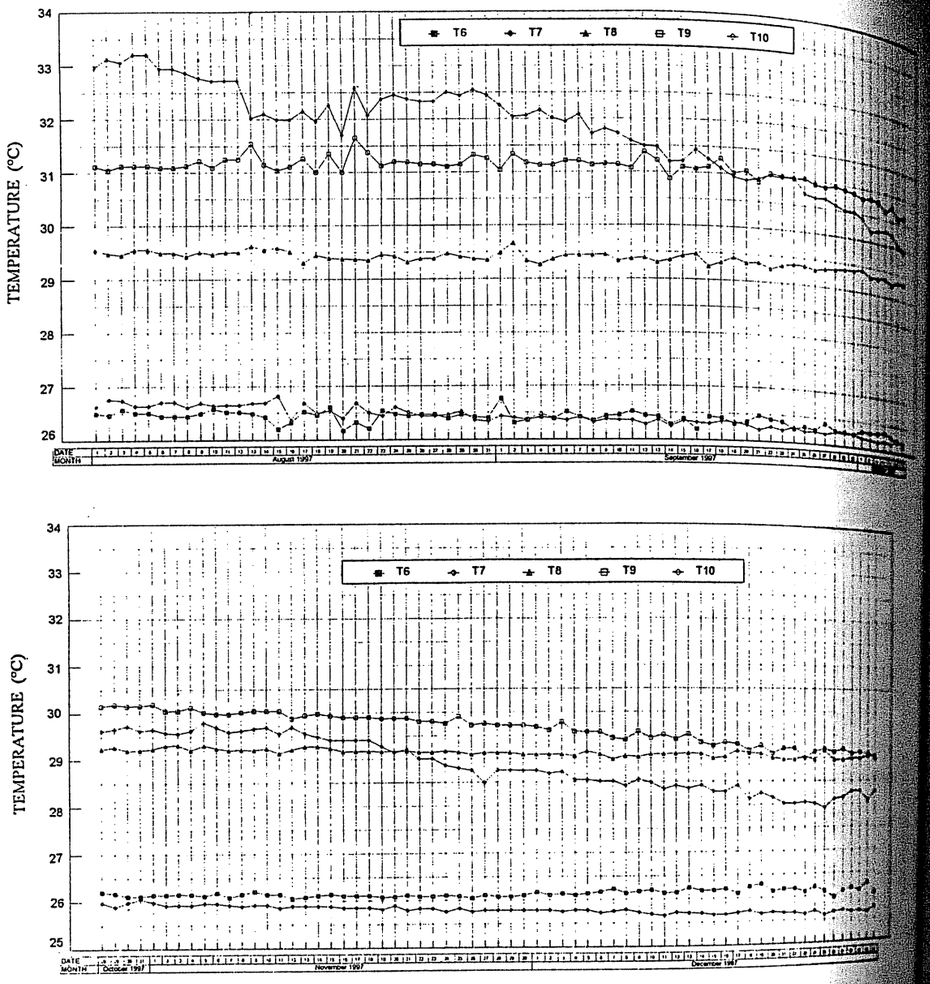


Fig. 2.9.2 (1) Measurement Records of Thermometers



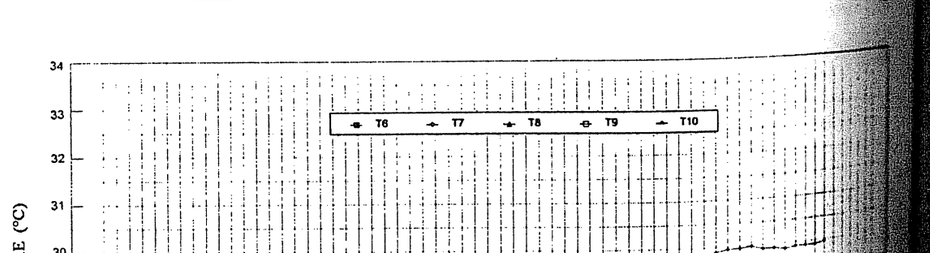
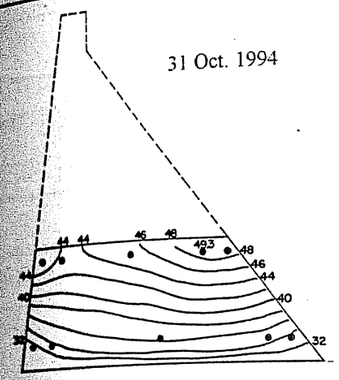
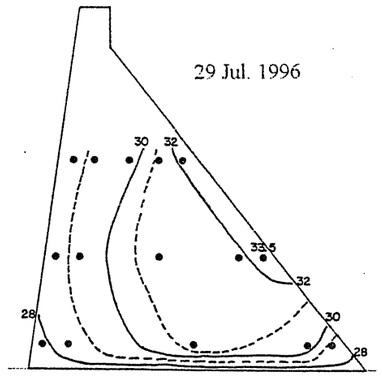
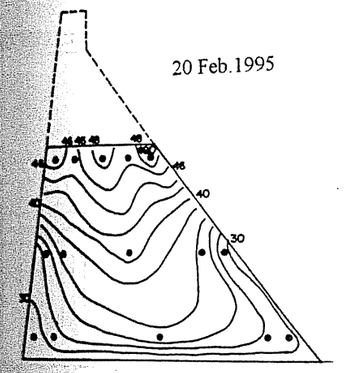
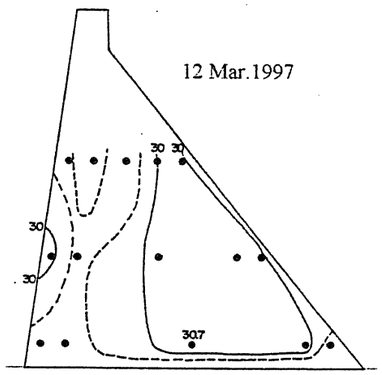


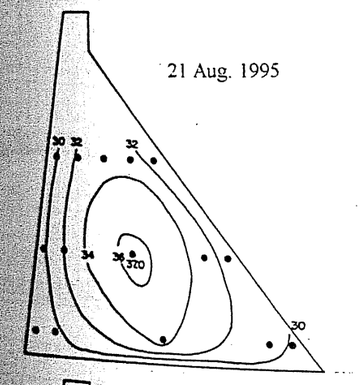
Fig. 2.9.2 (2) Measurement Records of Thermometers

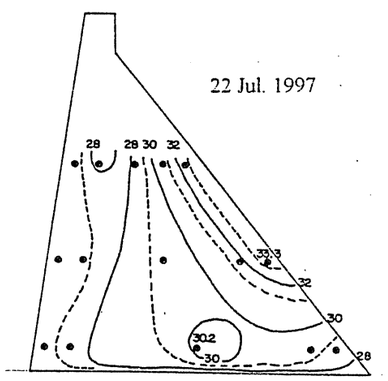


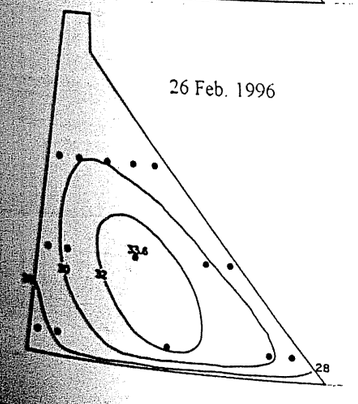












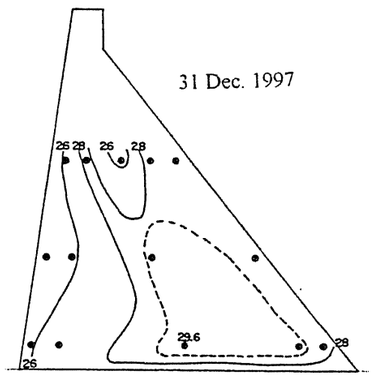
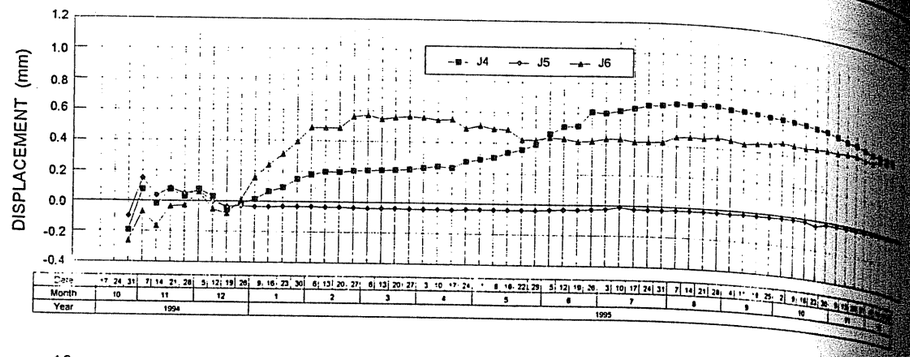
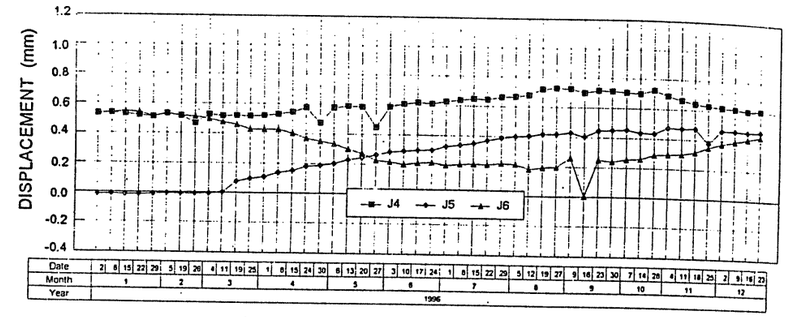
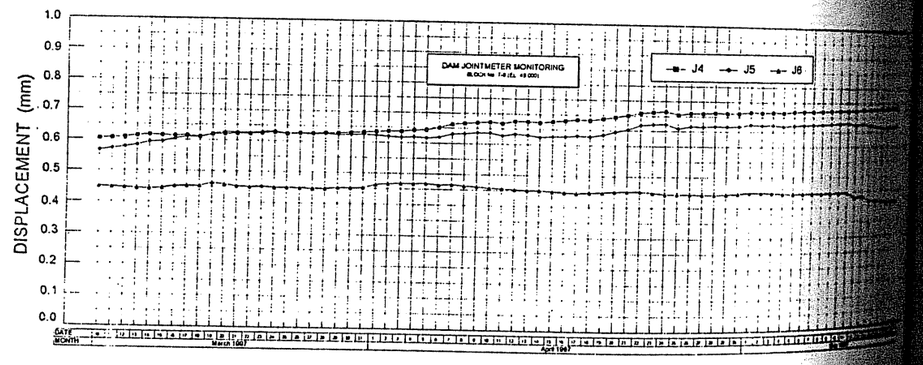


Fig. 2.9.3 Temperature Distribution of Dam Concrete







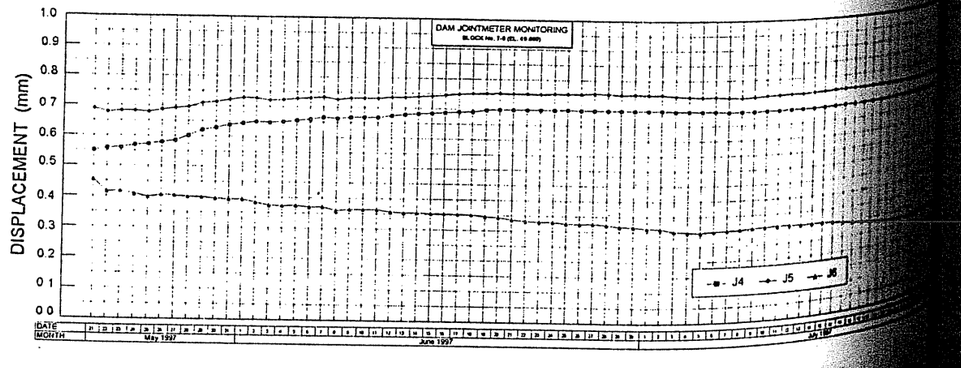
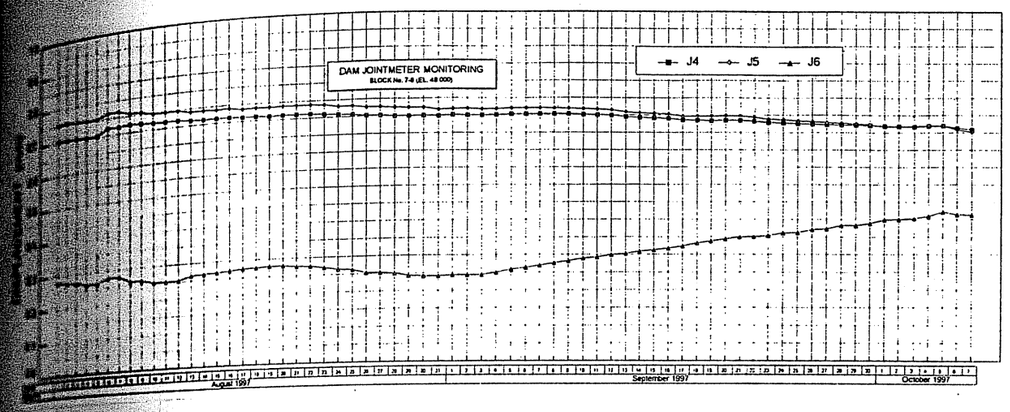
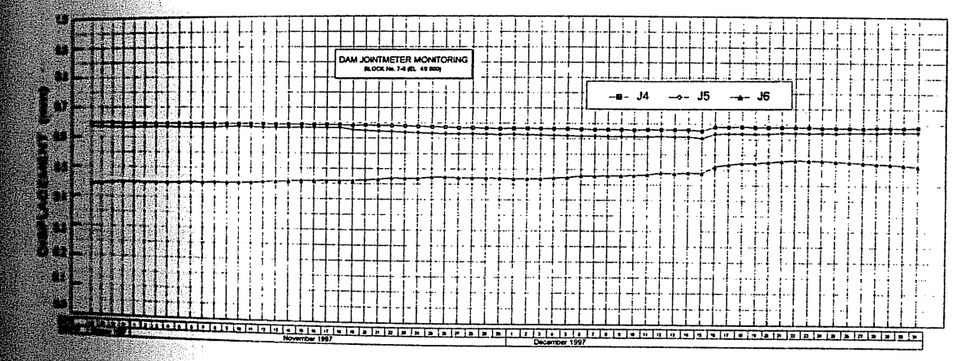


Fig. 2.9.4 (1) Measurement Records of Joint meters





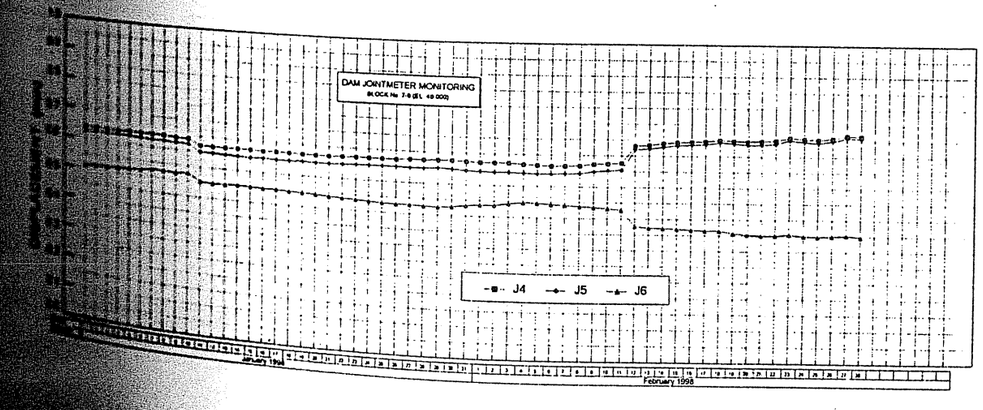
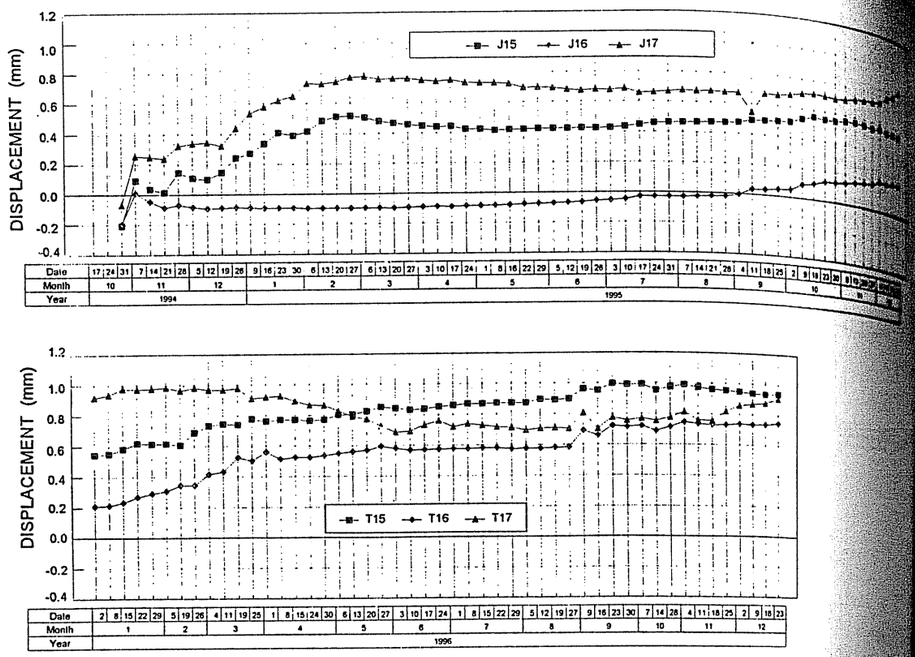


Fig. 2.9.4 (2) Measurement Records of Joint meters



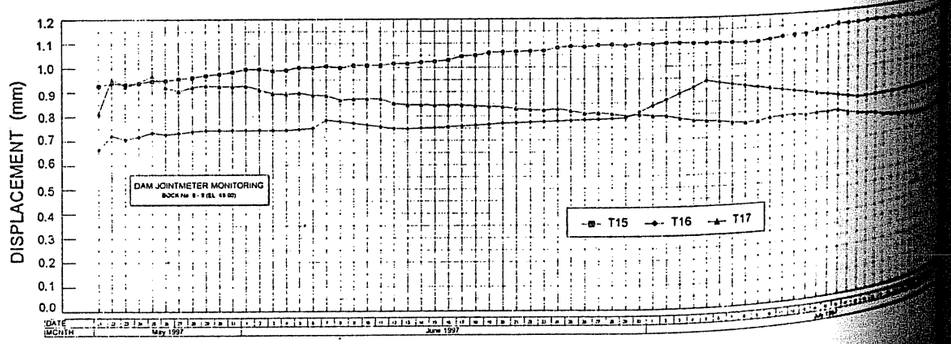
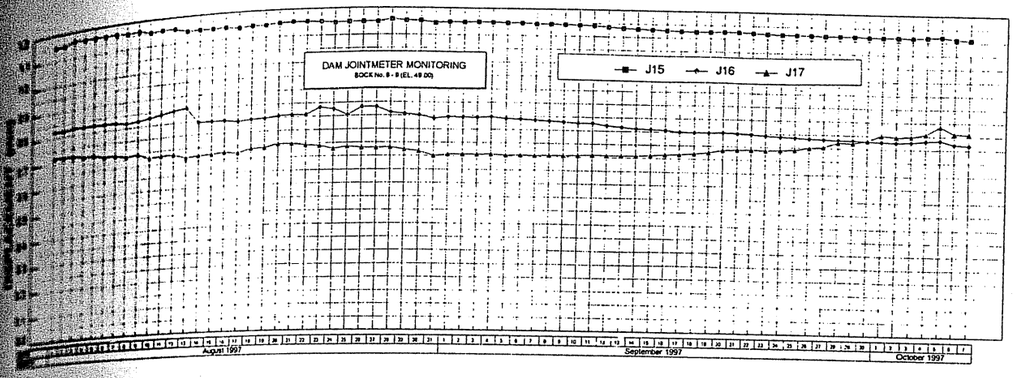
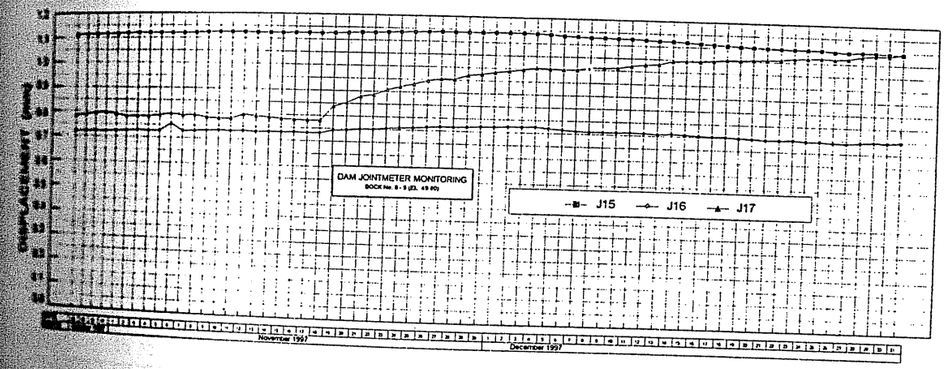


Fig. 2.9.4 (3) Measurement Records of Joint meters





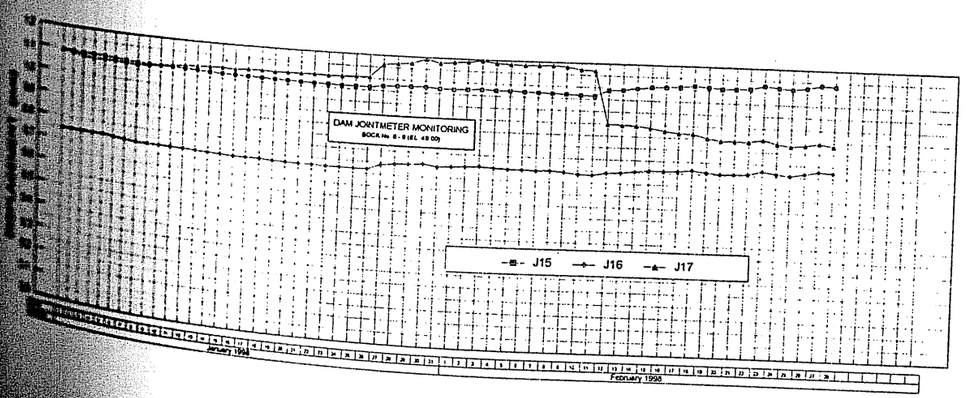


Fig. 2.9.4 (4) Measurement Records of Joint meters

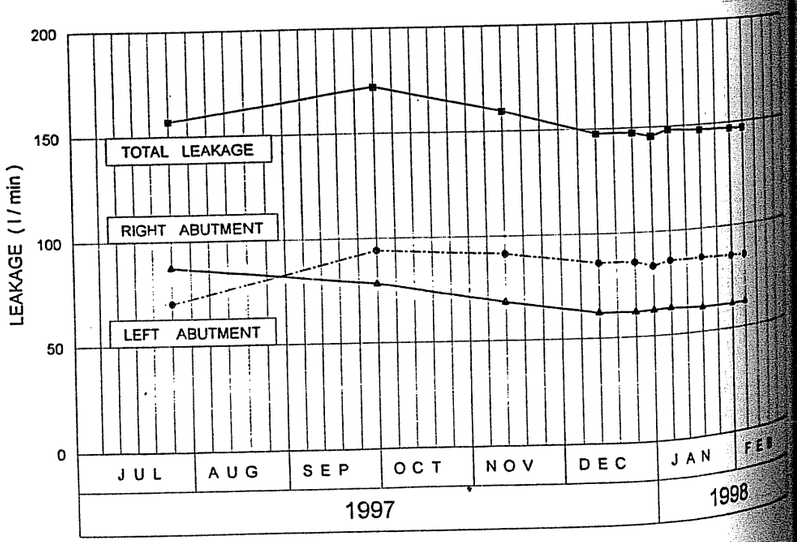
- During impounding

In general the total leakage from the drain holes fluctuated in proportion to the reservoir water level and tail water level and reached 191 ℓ/min with the reservoir water level at EL. 82.29 on 31 August 1997. Afterwards, the total leakage gradually decreased regardless of the reservoir and tail water levels and reduced to 154 ℓ/min with the reservoir water level at EL. 80.42 on 27 February 1998.

The historical change of the total leakage with the reservoir water level at EL. 82.0 is shown in the figure below. Looking at this historical record, the total leakage from the drain holes, in general, decreased as time passed except 1st October 1997. The gradual decrease with time was very likely caused by plugging up of the cracks and voids in the foundation rock.

On the other hand, the increase of the total leakage on 1st October 1997, was mainly caused by increase of leakage from the drain holes located at the dam left abutment, while leakage from the drain holes located at the right abutment decreased. From the serial records of each drain hole, the leakage from the drain hole at the left abutment started increasing gradually regardless of the reservoir water level stabilizing on 11 August 1997 and continued increasing until the beginning of October 1997.

This gradual increase of the leakage from this drain hole at the dam left abutment was considered to be caused by ground water level rising in the dam left abutment due to the installation of consolidation and curtain grouting around the diversion tunnel plug concrete. The ground water level in the dam left abutment was expected to be lowed by the diversion tunnel providing the drainage. Afterwards, the leakage from the dam left abutment became stable, while the leakage from the dam right abutment continued decreasing gradually.



Historical Change of Total Leakage at RWL. 82.0

The maximum leakage in all drain holes was recorded 28.5 ℓ/min at drain hole No. 12 (BL.5, J4+17.5m) with reservoir water level at EL. 82.31 on 3rd September 1997 and afterwards the leakage from the drain hole No. 12 decreased gradually. This recorded value was smaller than 100 ℓ/min which was considered the appropriate limit on this project for judging the necessity for taking countermeasures.

All leakage water color was transparent and clear. Therefore, it is considered that there is no problem from the leakage from all drain holes at time of writing this report.

(ii) Leakage from joint drain pipe

The serial records of the total leakage from the joint drain pipes and of leakage from each joint drain pipe are shown in Fig. 2.9.5 and 2.9.8 respectively. The relation between the reservoir water level and the total leakage from joint drain pipes is shown in Fig. 2.9.7.

- Before impounding

No leakage was measured at the joint drain pipes before impounding.

- During impounding

In general, the total leakage from the joint drain pipes increased in proportion to the reservoir water level rising and reached 87 ℓ/min with the reservoir water level at EL. 79.50 on 9th June 1997, which was also the maximum value of the total leakage during impounding.

In this period, however, rapid fluctuation of the total leakage occurred several times. Since this fluctuation and amount of total leakage was mainly caused by leakage from joint drain pipe No. 9 and No. 10, some soil mixed sawdust was thrown in to the reservoir just upstream of spillway gate No. 3 on 10th June 1997, in order to plug the leakage routes by soil and sawdust and decrease the leakage from the joint drain pipes.

As a result, the total leakage decreased rapidly to 39 ℓ/min with the same reservoir water level and then continued to decrease gradually and reached around 20 ℓ/min in the middle of July 1997. Afterwards, the total leakage increased gradually in proportion to the reservoir and tail water level rising, and reached 33 ℓ/min with reservoir water level at EL. 81.85 on 17th December 1997.

On 18th December 1997, the total leakage increased rapidly to 37 ℓ/min and afterwards, stabilized again. This rapid increase occurred at joint drain pipe No.9.

The total leakage from the joint drain pipes and the leakage from each joint drain pipe, (especially No. 9 and No. 10) had reached a stable value in this time.

Moreover, there is no detrimental influence on the stability of the dam and its foundation by the leakage from joint drain pipes as long as the uplift pressure at the block in question is smaller than the design value.

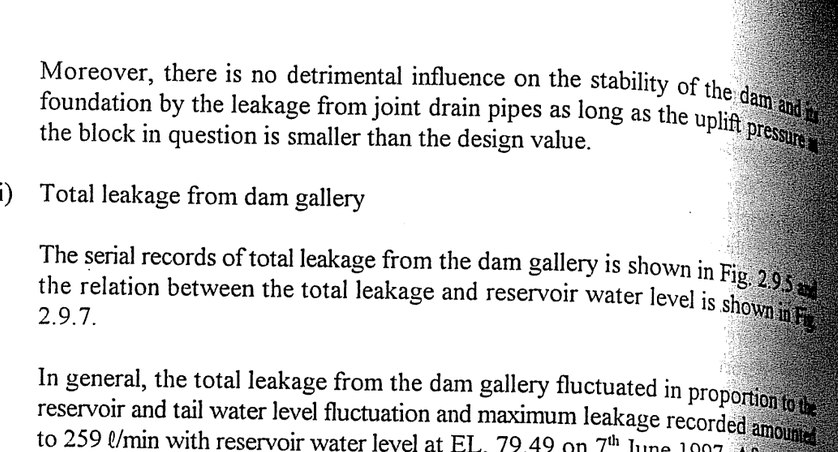
(iii) Total leakage from dam gallery

The serial records of total leakage from the dam gallery is shown in Fig. 2.9.5 and the relation between the total leakage and reservoir water level is shown in Fig. 2.9.7.

In general, the total leakage from the dam gallery fluctuated in proportion to the reservoir and tail water level fluctuation and maximum leakage recorded amounted to 259 ℓ/min with reservoir water level at EL. 79.49 on 7th June 1997. Afterwards, the total leakage decreased gradually regardless of the reservoir and tail water level fluctuation and reached 190 ℓ/min with reservoir water level at EL. 80.42 on 27th February 1998.

The maximum leakage and leakage on 27th February 1998 was compared with other project experiences in Japan, in the following figure. The both leakages of this dam were no different to other projects. Moreover, the total leakage from the dam gallery was less than that of 500 ℓ/min calculated by the seepage analysis and was within the capacity of drainage pump of 800 ℓ/min installed in the dam gallery.

Therefore, it is considered that there is no problem concerning the total leakage at time of writing this report.



Comparison with other projects experiences for leakage

(4) Uplift pressure

The serial records of pore pressure meters, uplift meters and water pressure meters attached drain holes are shown in Fig. 2.9.9, 2.9.10 and 2.9.11 respectively. And the distribution of uplift pressure acting on each gauging section and at each drain hole are shown in Fig. 2.9.12 and 2.9.13 respectively.

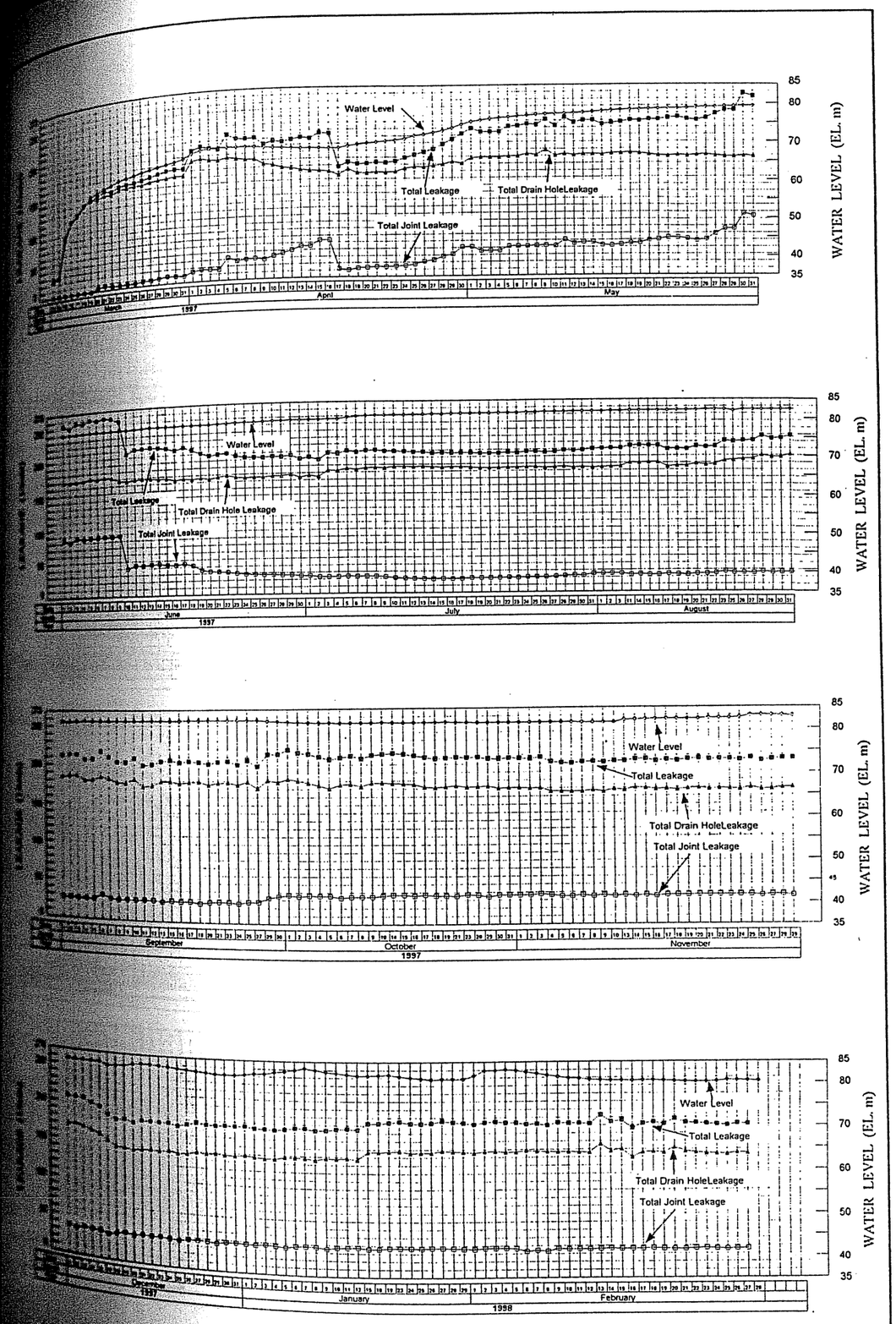


Fig. 2.9.5 Measurement Records of Total Leakage

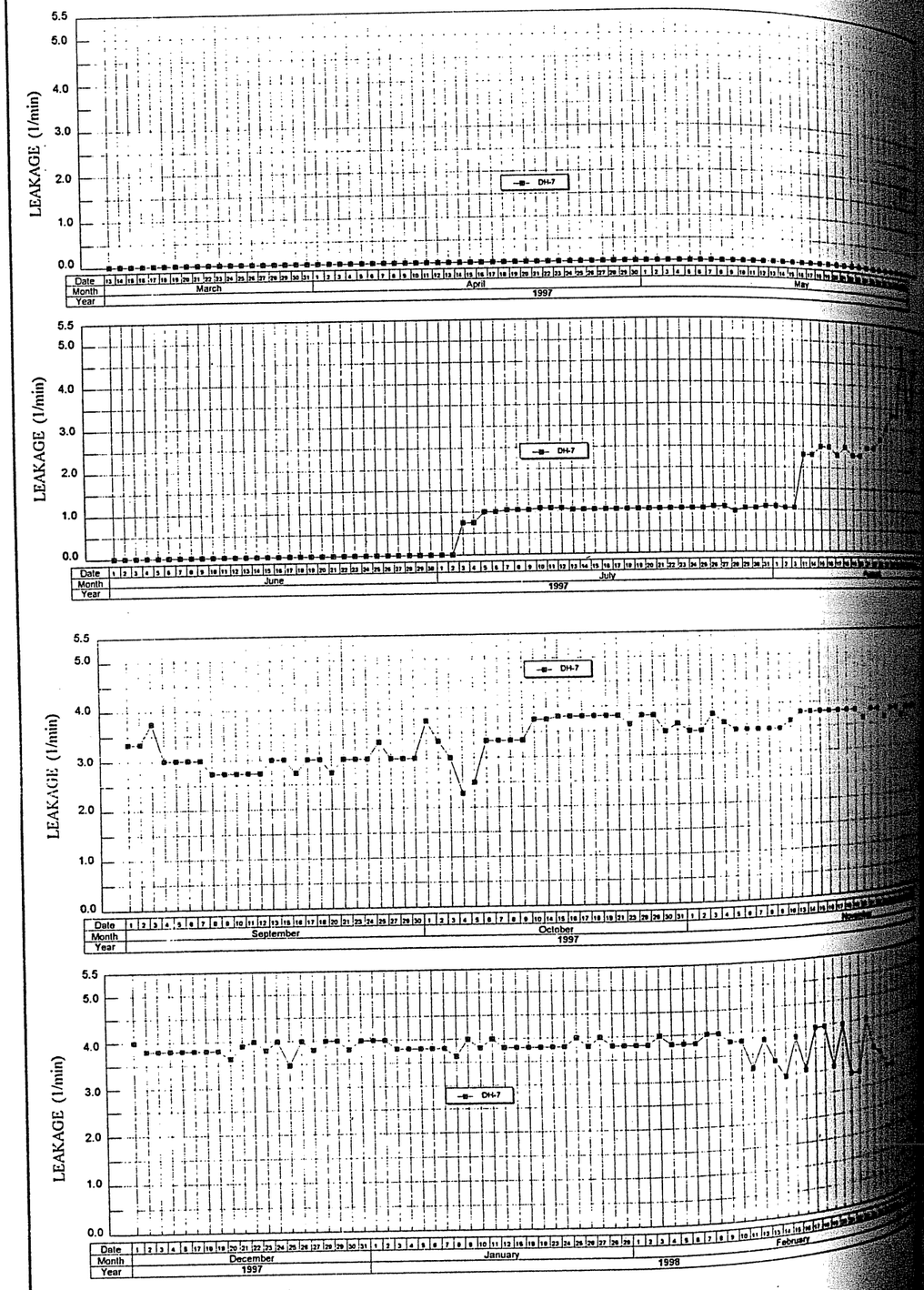


Fig. 2.9.6 (1) Measurement Records of Leakage from Drainhole

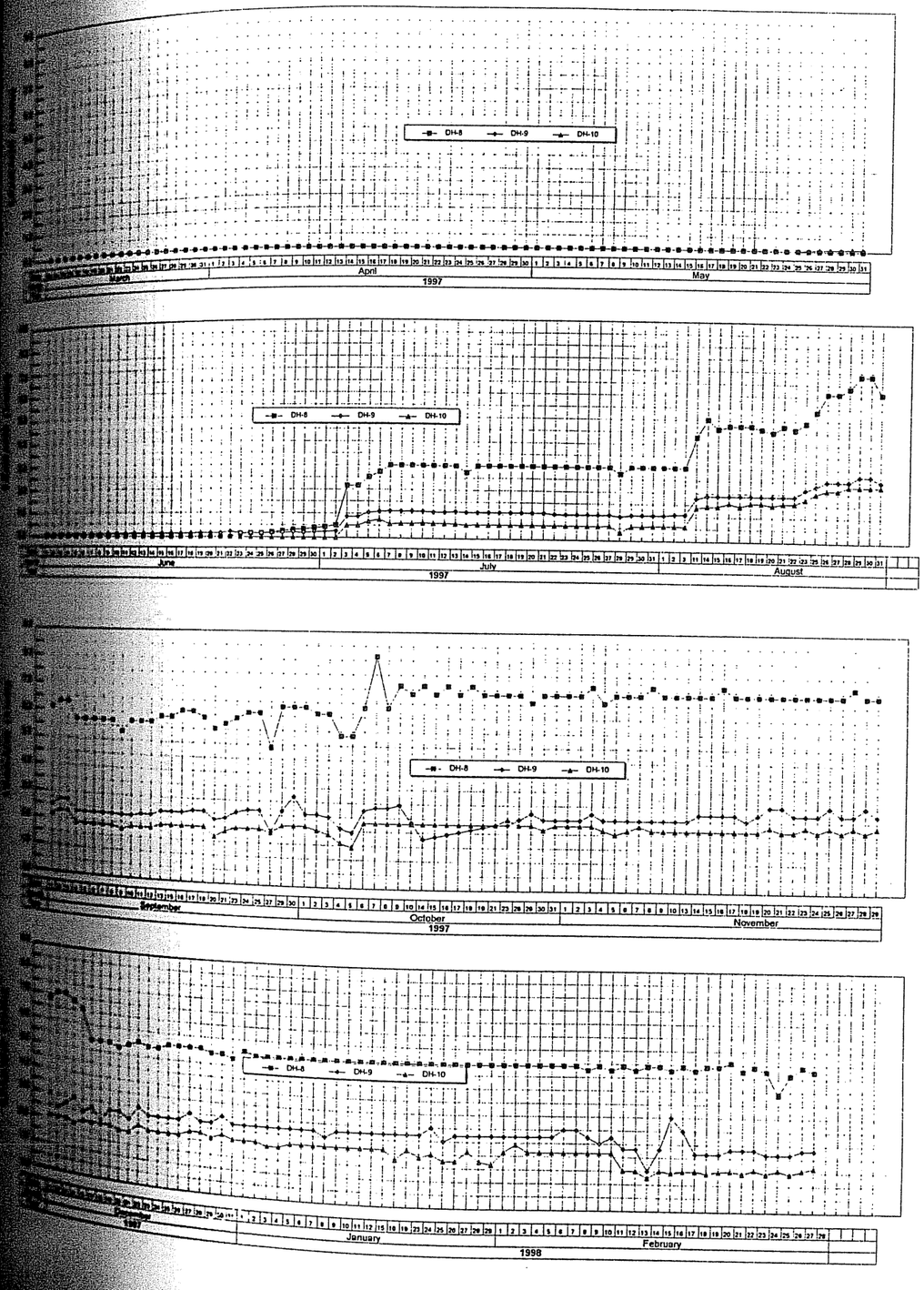
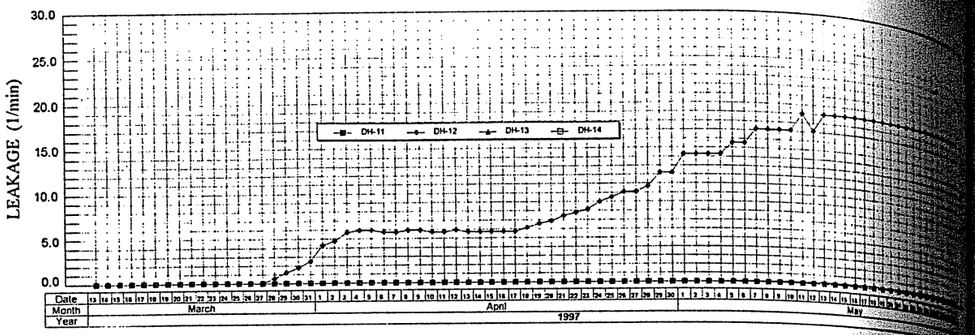
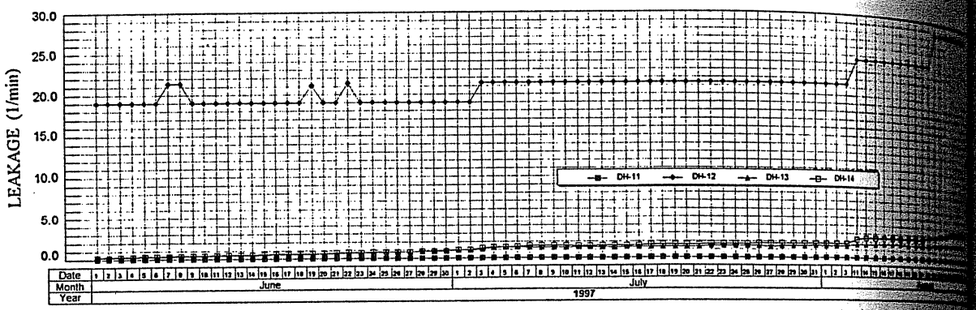
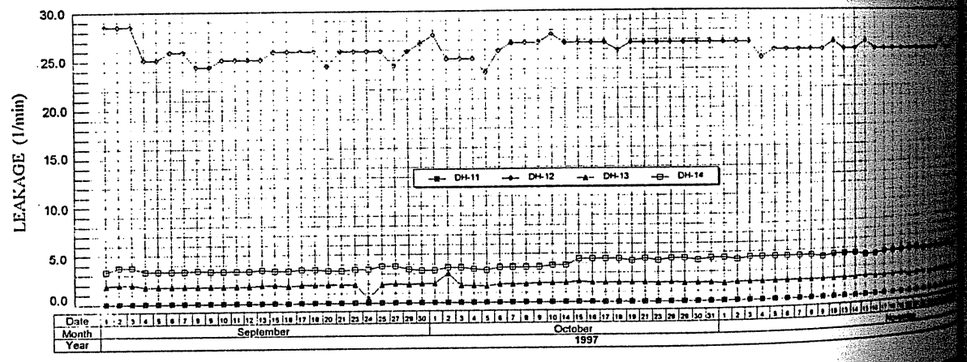


Fig. 2.9.6 (2) Measurement Records of Leakage from Drainhole







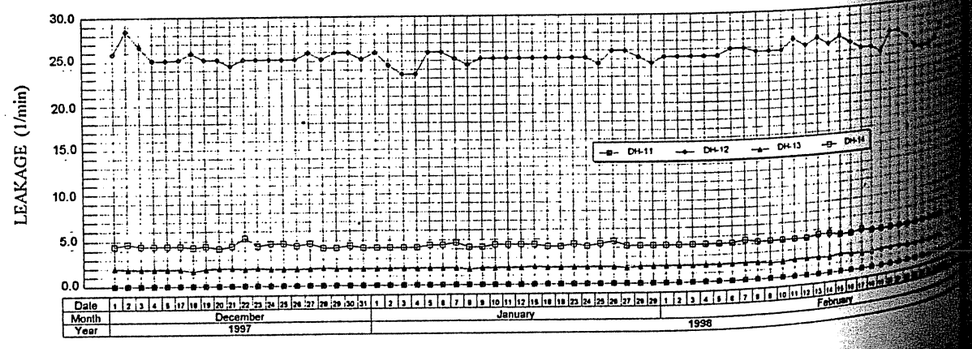
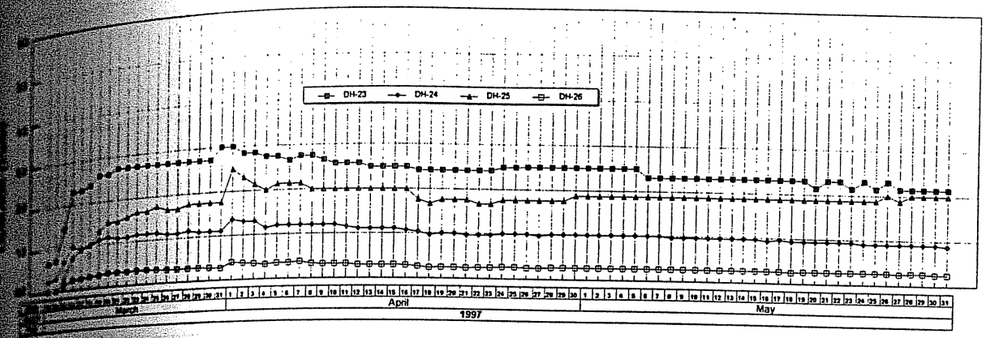
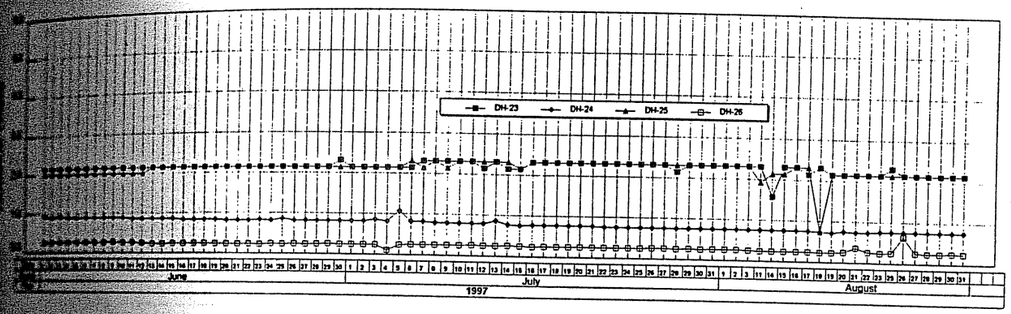


Fig. 2.9.6 (3) Measurement Records of Leakage from Drainhole





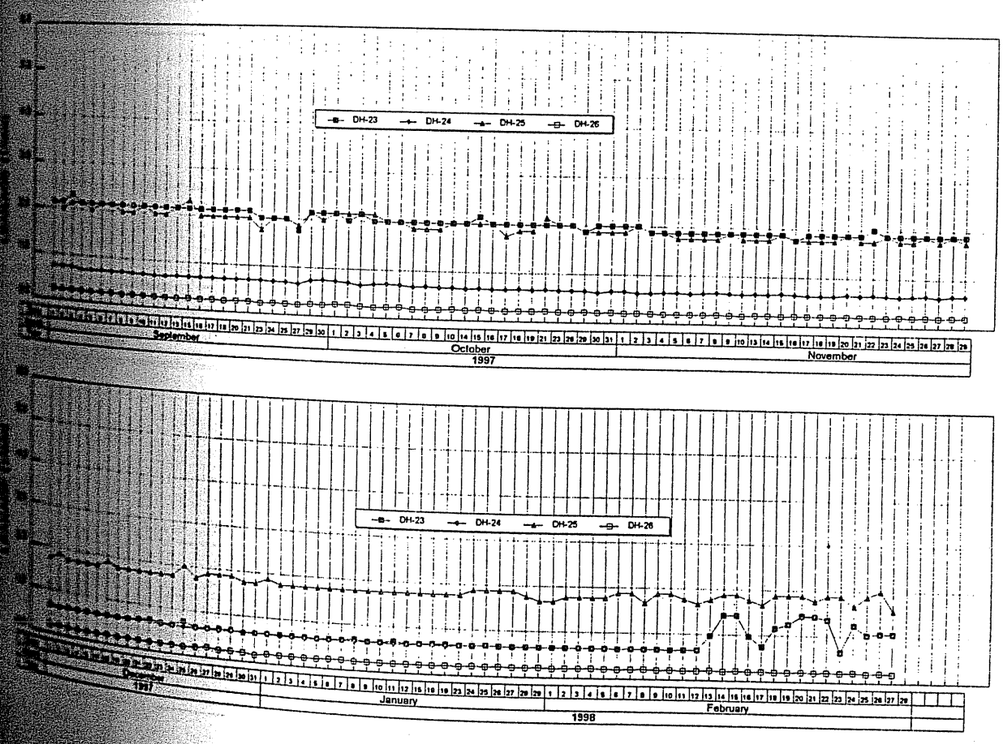


Fig. 2.9.6 (4) Measurement Records of Leakage from Drainhole

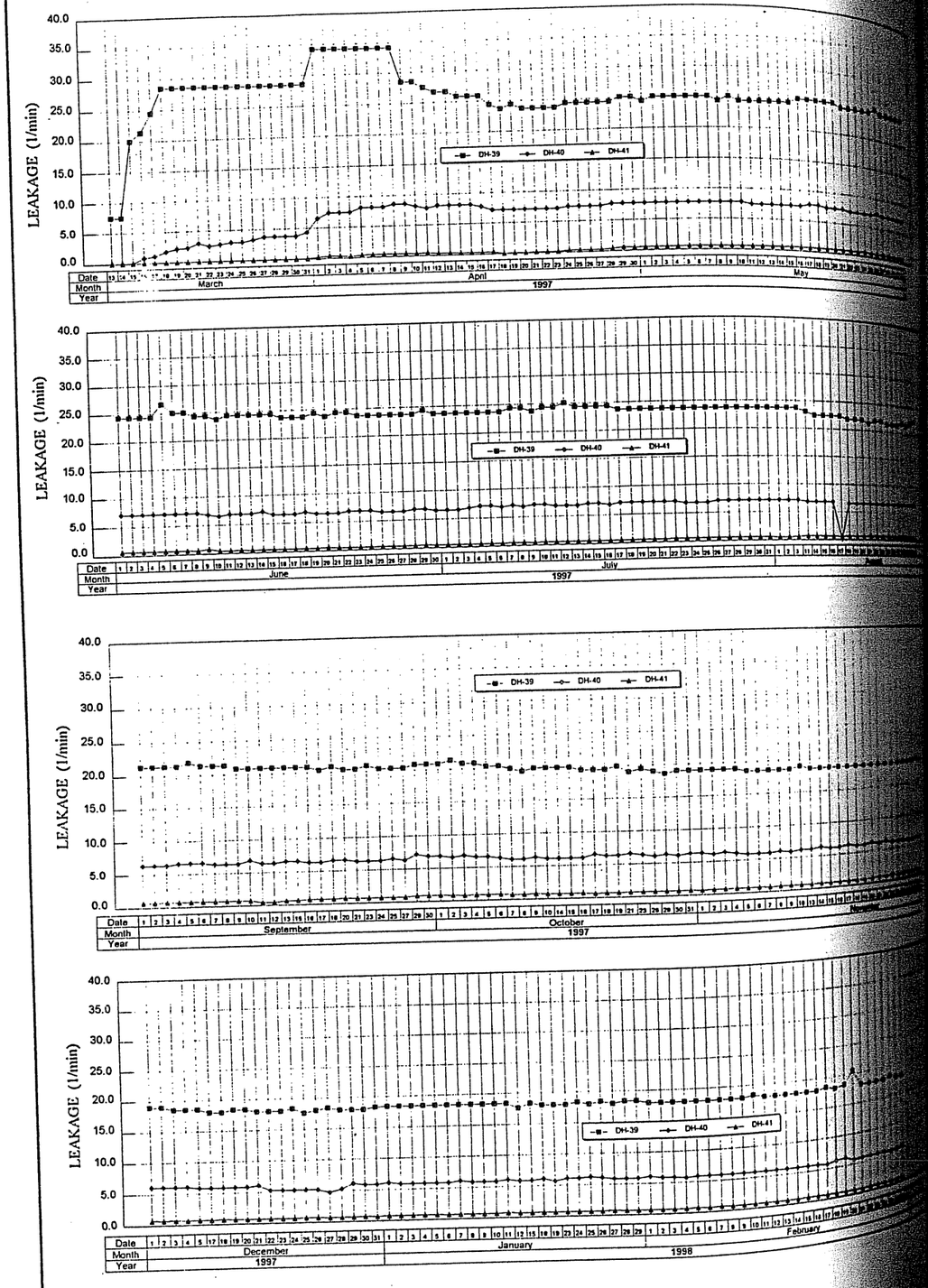


Fig. 2.9.6 (5) Measurement Records of Leakage from Drainhole

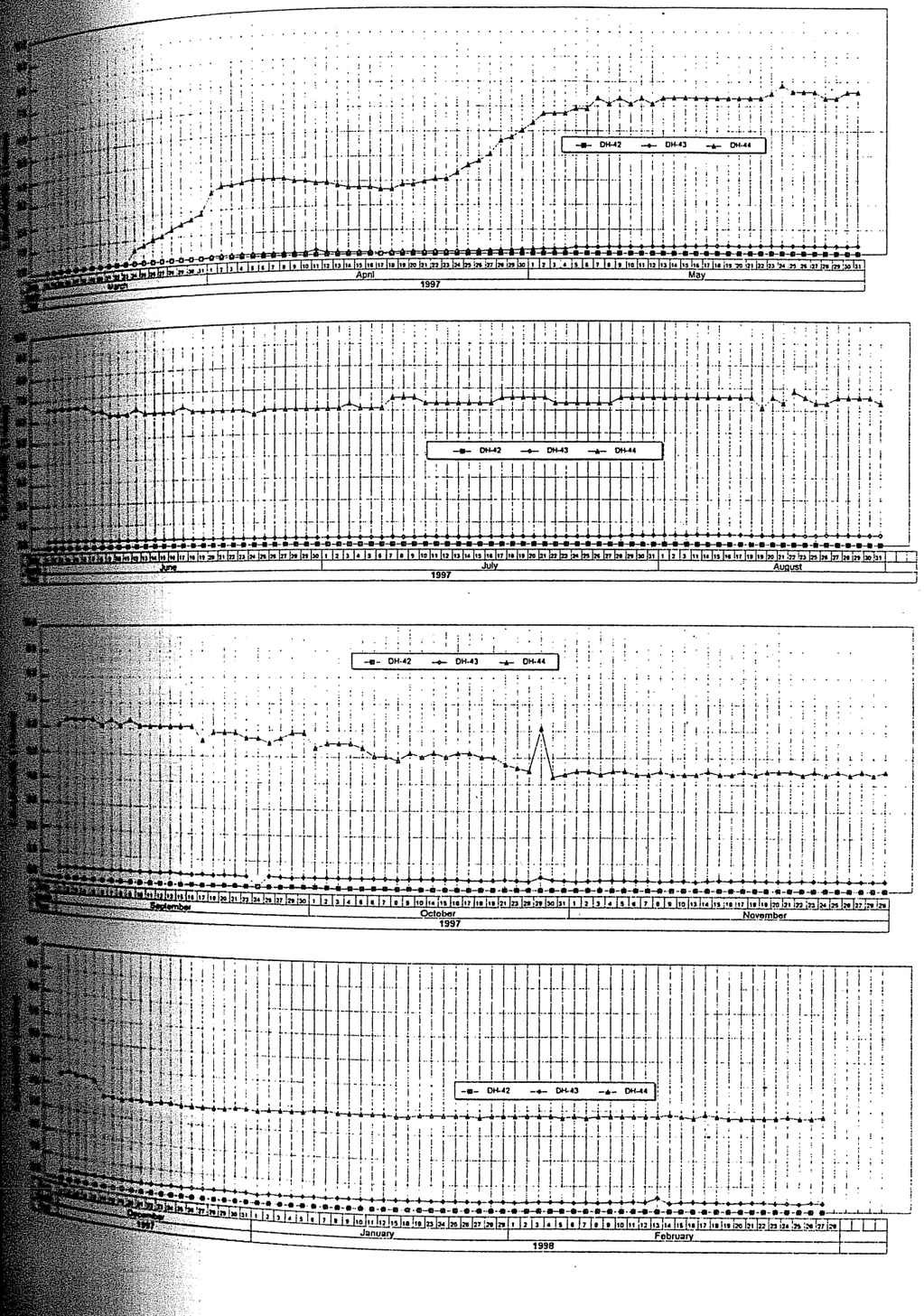


Fig. 2.9.6 (6) Measurement Records of Leakage from Drainhole

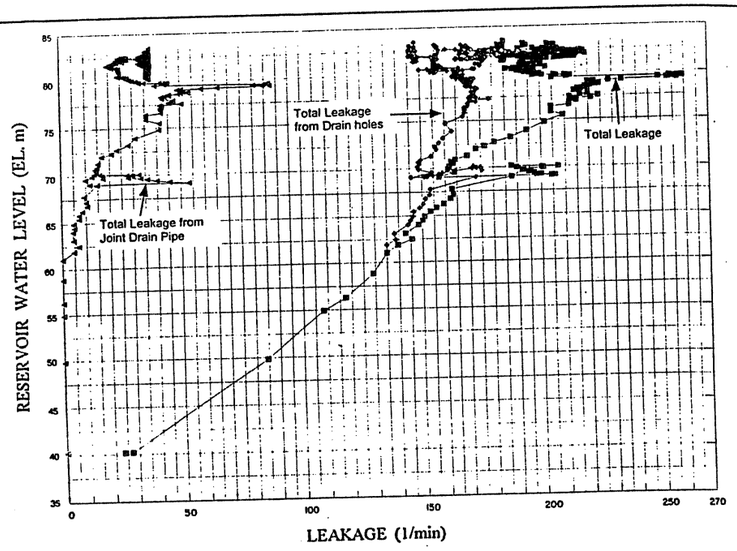


Fig. 2.9.7 The Relation between Reservoir Water Level and Leakage

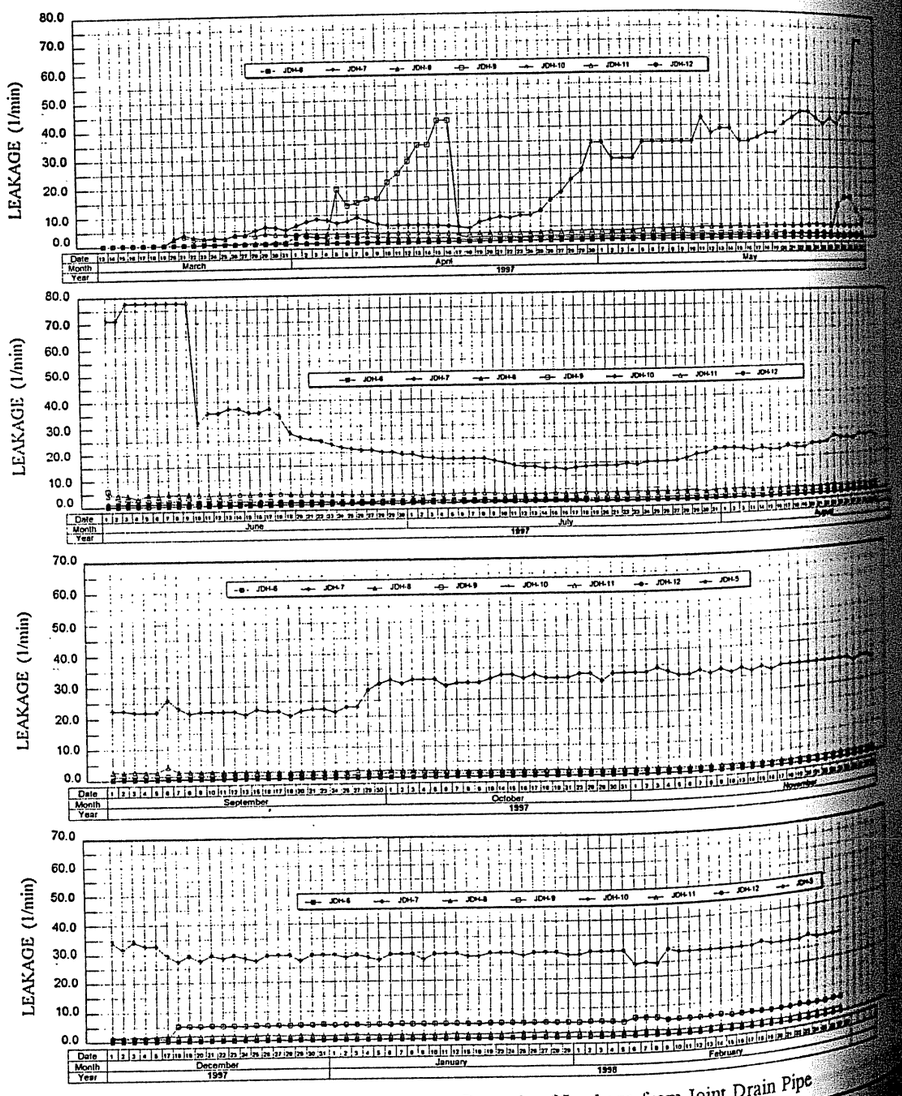


Fig. 2.9.8 Measurement Records of Leakage from Joint Drain Pipe

(i) Before impounding

Generally pore pressure meters showed a constant very low pressure, about 0.55 kg/cm^2 except during the flood periods. And there was no difference in the pressure between upstream to downstream.

On the other hand, at the flood periods which occurred in the beginning of February 1995 and February 1996, pore pressure meters recorded the high water pressure and the pressure decreased according to the distance from the upstream face. Then after the flood periods, the pressure stabilized again

(ii) During impounding

- J4+10.0m (Intake section)

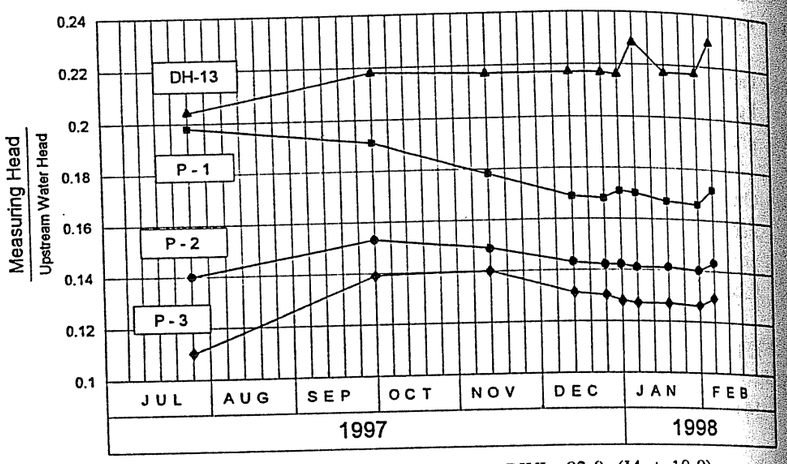
In general, the pore pressure meters P1-P3 readings fluctuated in proportion to the reservoir water level fluctuation and reached maximum values as shown in the table below. Afterwards, they gradually decreased or were stable regardless of the reservoir water level fluctuation and reached 0.62 kg/cm^2, 0.53 kg/cm^2 and 0.48 kg/cm^2 respectively with the reservoir water level at EL.80.42 on 27th February 1998.

	Maximum Value (kg/cm^2)	RWL (m)	Date
P-1	0.82	82.28	28/08/97
P-2	0.60	81.92	07/10/97
P-3	0.55	82.05	28/10/97

With regard to the actual distribution of uplift pressures acting on this section, the uplift pressures decreased in proportion to the distance from the upstream face and the ratio of the measuring head to upstream water head were within the design values, especially pore pressure meter P-1 set at the most upstream side was conspicuously smaller than the others. This is considered to be caused by the effectiveness for water-tightness of the grout curtain of the dam.

The historical change of the uplift pressure with reservoir water level at EL. 82m is shown in the figure below. From the comparison of historical change, the uplift pressures acting on the intake section decreased generally or were stable as time passed except on 1st October 1997. This tendency to gradual decrease or stabilize as time passed is judged to be the normal condition.

The increase of the uplift pressure on 1st October 1997 was considered to be mainly caused by the ground water level rising in the dam left abutment due to the completion of the consolidation and curtain grouting around the diversion tunnel plug concrete.



Historical Change of Uplift Pressure at RWL. 82.0 (J4 + 10.0)

This rising of the ground water level at the dam left abutment was clarified by the fluctuation of the water level in drain holes located in the dam left abutment as shown in Fig. 2.9.14, that shows the relation between the water level in each drain hole and the reservoir water level.

The water level in the drain hole No. 3 located in the dam left abutment descended in proportion to the reservoir water level drawing down up to 4th October. Around 6th October, the water level in the drain hole increased rapidly regardless of the reservoir water level still drawing down and reached a maximum value at the end of October, while the water level in the drain holes No. 47 and No. 48 located in the right abutment fluctuated in proportion to the fluctuation of the reservoir water level. Afterwards, the water level in the drain holes at the left abutment became stable.

Therefore, the increase in uplift pressure in the left abutment on 1st October was caused by the ground water level rising in the whole dam left abutment. This rising ground water level was considered to be caused by completion of the grouting works around the diversion tunnel plug concrete which stopped draining of water from the upstream of the plug concrete. And it can be also concluded that the diversion tunnel plug concrete with grout curtain are functioning properly.

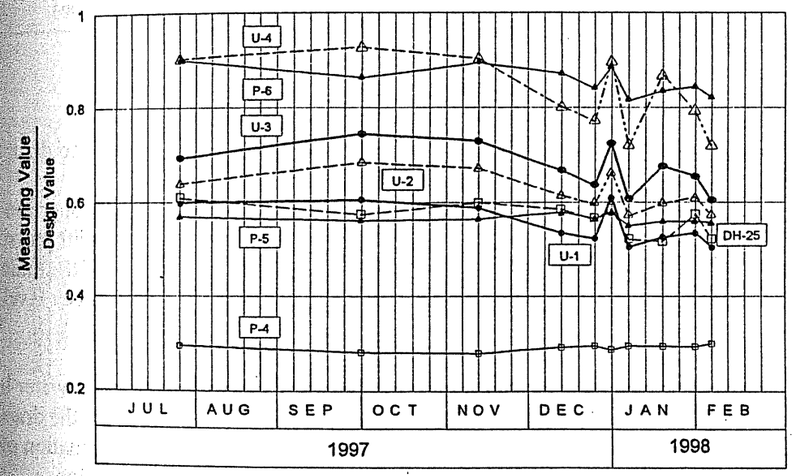
- J7+9.0m (Non-overflow section)

In general, the reading of pore pressure meters P4-P6 and uplift meters U1-U4 fluctuated in proportion to the fluctuation of the reservoir water level and the full water level reaching 0.93 kg/cm^2, 0.92 kg/cm^2, 1.07 kg/cm^2, 1.01 kg/cm^2, 1.03 kg/cm^2, 0.97 kg/cm^2 and 1.01 kg/cm^2 respectively with reservoir water level at EL. 80.42m on 27th February 1998.

With regard to the actual distribution of the uplift pressures acting on this section, uplift pressure from upstream side P-4 to downstream side U-3 showed almost the same value with no large fluctuation except P-6 and U-4 located at the toe of the dam body. Pore pressure meter P-6 and uplift meter U-4 had a tendency to indicate larger uplift pressures than other meters.

From the distribution of the ratio of measuring value to design value, the ratio of each meter descended in proportion to the reservoir water level rising and is within the design value.

The historical change of the uplift pressures with reservoir water level at EL. 82m is shown in the figure below and showed that the uplift pressures acting on this section generally decreased or stabilized as time passed. This tendency to gradually decrease or stabilize as time passed is judged to be a normal condition.



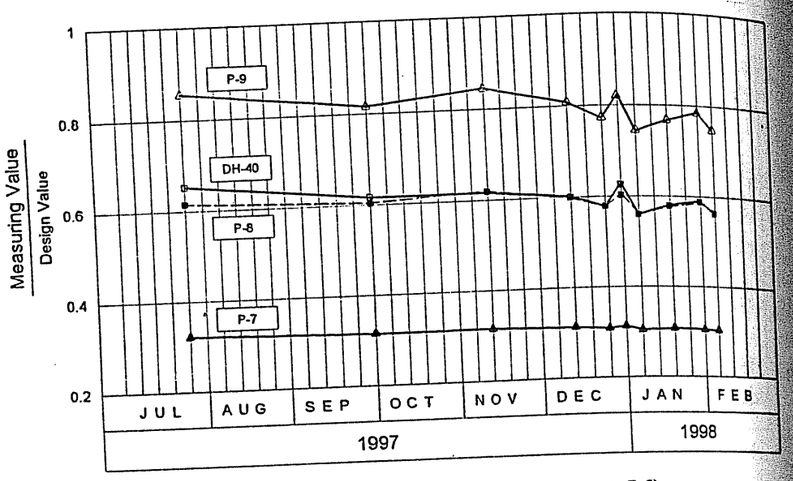
Historical Change of Uplift Pressure at RWL. 82.0 (J7 + 9.0)

- J12-+7.5m (Overflow section)

In general, the reading of pore pressure meters P7-P9 showed the same tendency of uplift pressure as the pore pressure meters P4-P6 and reached 1.00 kg/cm^2, 0.96 kg/cm^2 and 0.97 kg/cm^2 respectively with the reservoir water level at EL. 80.42 on 27th February 1998.

The actual distribution and the historical change of uplift pressures acting on this section also had the same tendency as those of the non-overflow section (J7+7.5m).

Therefore, the condition of uplift pressures acting on this section is also judged to be a normal condition.



Historical Change of Uplift Pressure at RWL. 82.0 (J12 + 7.5)

- Other dam sections (Records of water pressure meters attached to drain holes)

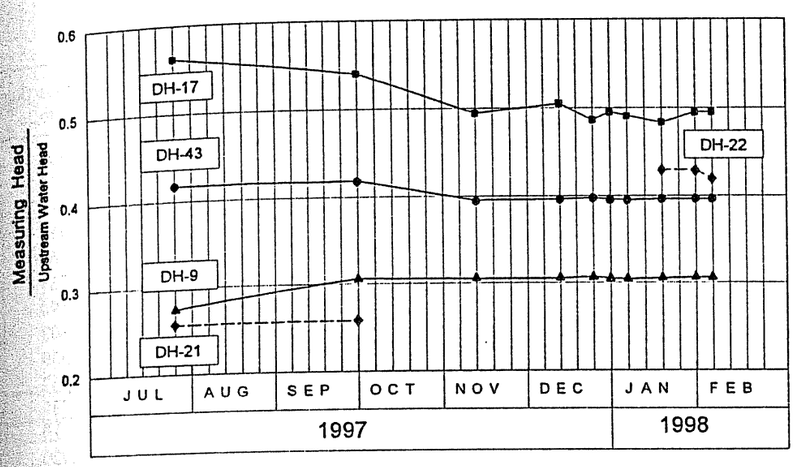
In generally, uplift pressures at each drain hole fluctuated in proportion to the reservoir water level fluctuation. The actual distribution of uplift pressures at each drain hole shows that the uplift pressures at almost all of the drain holes have a tendency to decrease in proportion to the reservoir water level rising and are within the design condition.

From the historical change of uplift pressures with the reservoir water level at EL. 82m as shown in the flowing figures, uplift pressures at all the drain holes gradually decreased or stabilized as time passed. Then the fluctuation of uplift pressure at all the drain holes is judged to be a normal condition.

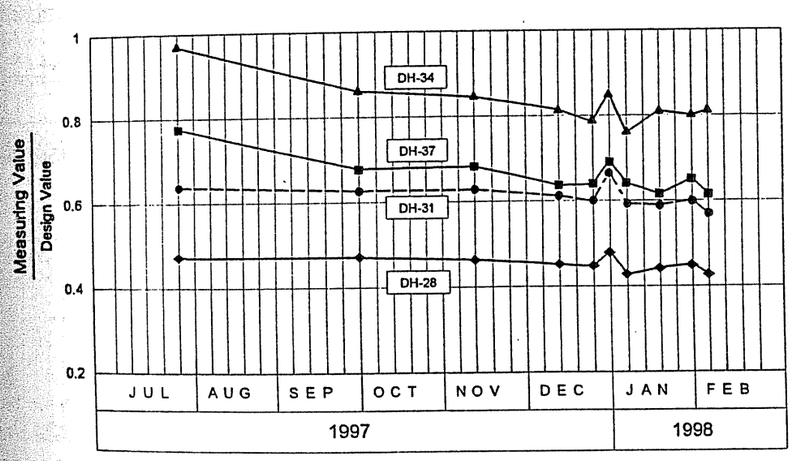
With regard to the section with larger uplift pressures than the design value, the stability of dam blocks were confirmed to be assured on the condition that actual uplift pressure was acting on each dam block respectively as shown in the table below.

And the leakage from drain holes indicated larger uplift pressure showed to be a normal condition in respect to the quantity and quality.

As mentioned above, it can be concluded that the uplift pressures acting on the dam show a good performance with the reservoir impounding.



Non-over Flow Section



Over Flow Section

Historical Change of Uplift Pressure at RWL. 82.0

Actual Uplift Pressure at HWL

BL No.	Instrument No.	Uplift pressure (kg/cm^2) Design	Ratio	Actual	BL No	Instrument No.	Uplift pressure (kg/cm^2) Design	Ratio	Actual
1	DH-1	0.133	-	(- 0.580)	8	U-3	1.865	0.597	1.113
2	DH-3	0.406	0.705	0.286		P-6	1.766	0.810	1.430
3	DH-6	0.646	1.379	0.891		U-4	1.667	0.685	1.142
4	DH-9	0.722	1.462	1.056	9	DH-28	2.030	0.411	0.834
5	P-1	2.275	0.314	0.714	10	DH-31	2.030	0.562	1.141
	DH-13	0.941	1.044	0.982	11	DH-34	2.030	0.731	1.484
	P-2	0.804	0.845	0.679	12	DH-37	2.030	0.594	1.206
	P-3	0.493	1.452	0.716	13	P-7	3.508	0.303	1.063
6	DH-17	1.444	1.560	2.253		DH-40	2.030	0.539	1.094
7	DH-22	2.186	1.028	2.247		P-8	1.746	0.552	0.964
8	P-4	3.708	0.294	1.090		P-9	1.332	0.730	0.972
	DH-25	2.374	0.512	1.215	14	DH-43	1.530	1.119	1.712
	U-1	2.261	0.496	1.121	15	DH-46	0.710	1.926	1.367
	P-5	2.162	0.553	1.196	16	DH-49	0.405	1.037	0.420
	U-2	2.063	0.555	1.145					

Note : Ratio = Measuring Value/Design Value at RWL. 83.46 on 7th January 1998 (BL. 4-14) or at EL. 81.66 on 24th October 1997 (BL. 1-3, BL. 15-16)

(5) Displacement

The actual displacement of the dam crest was almost 0 cm and therefore the movement of the dam is considered to be negligible.

(6) Seismic acceleration

There was no record of the earthquake motion in this period.

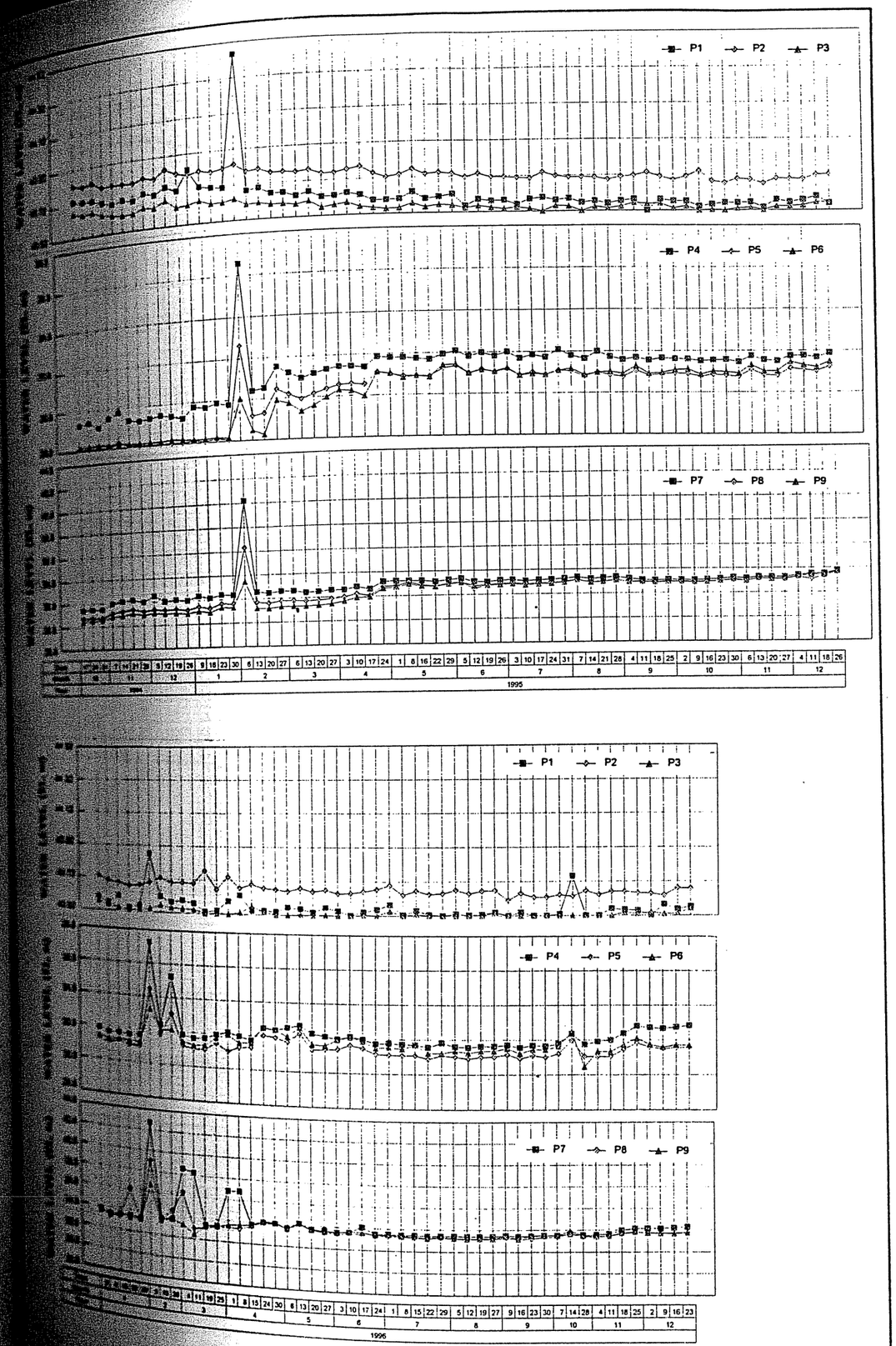
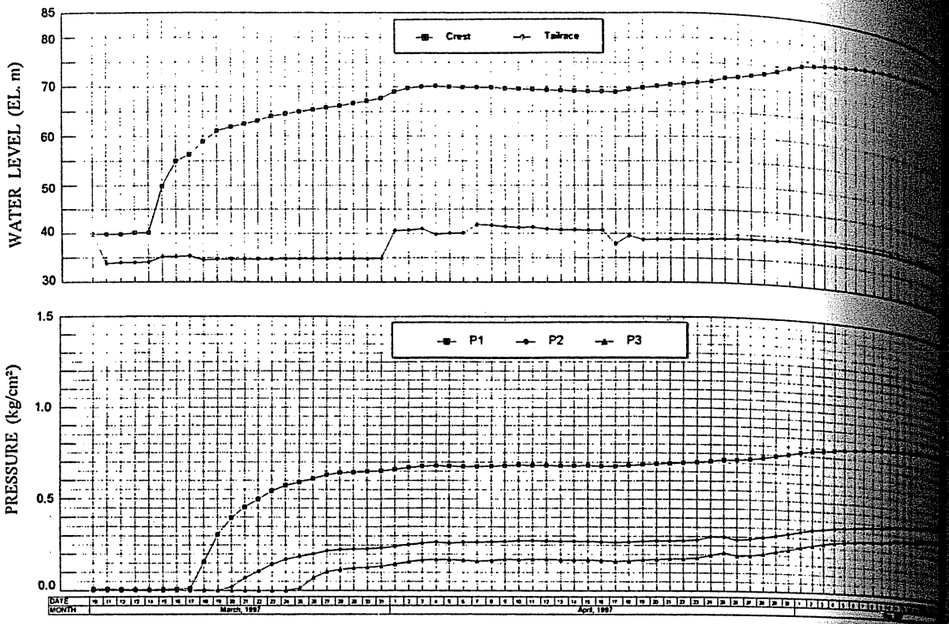
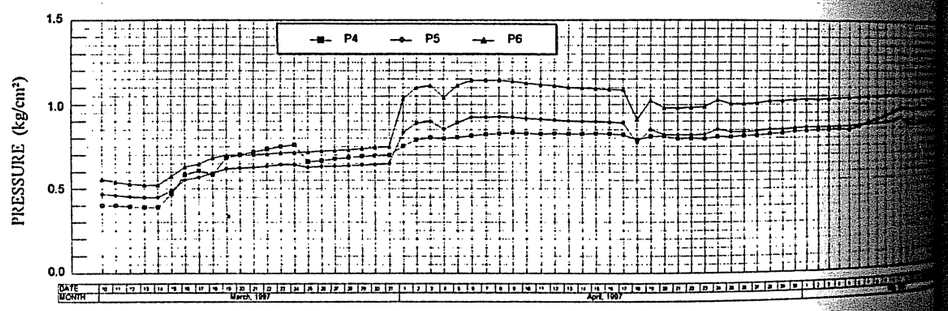
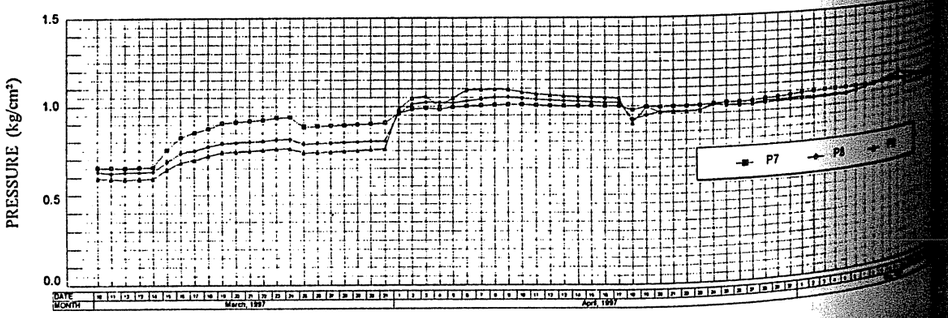


Fig. 2.9.9 (1) Measurement Records of Porepressure Meters







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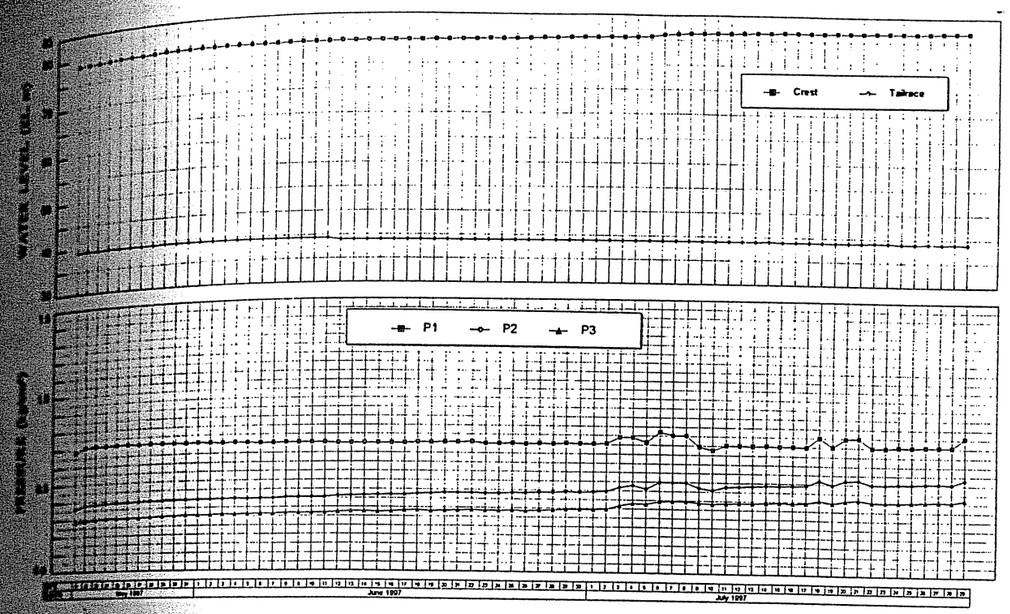

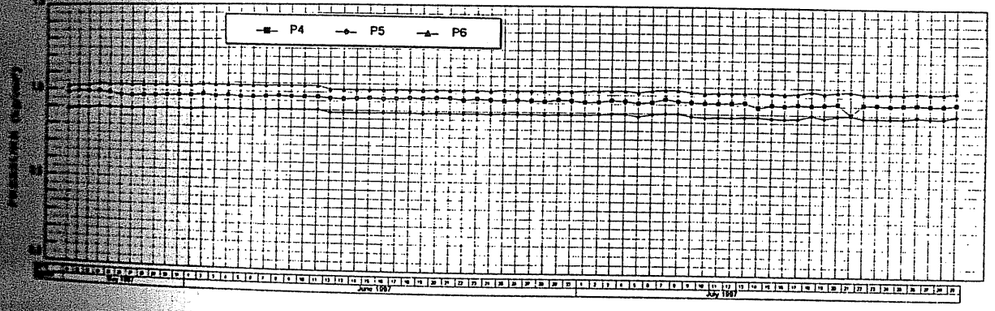

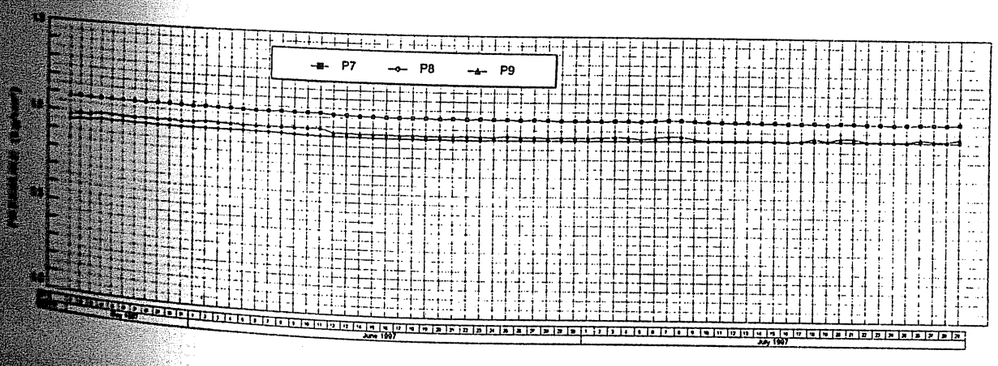

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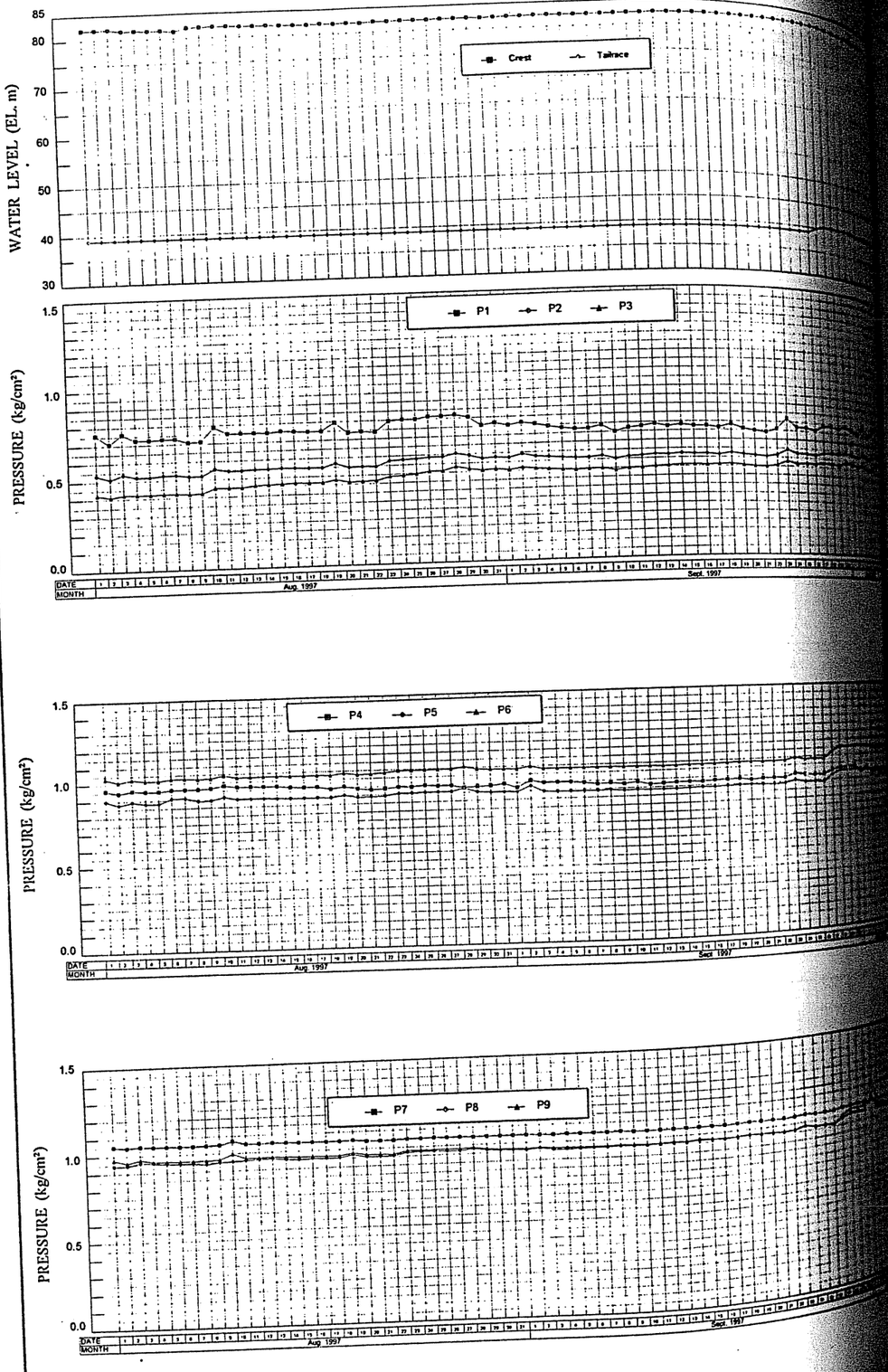

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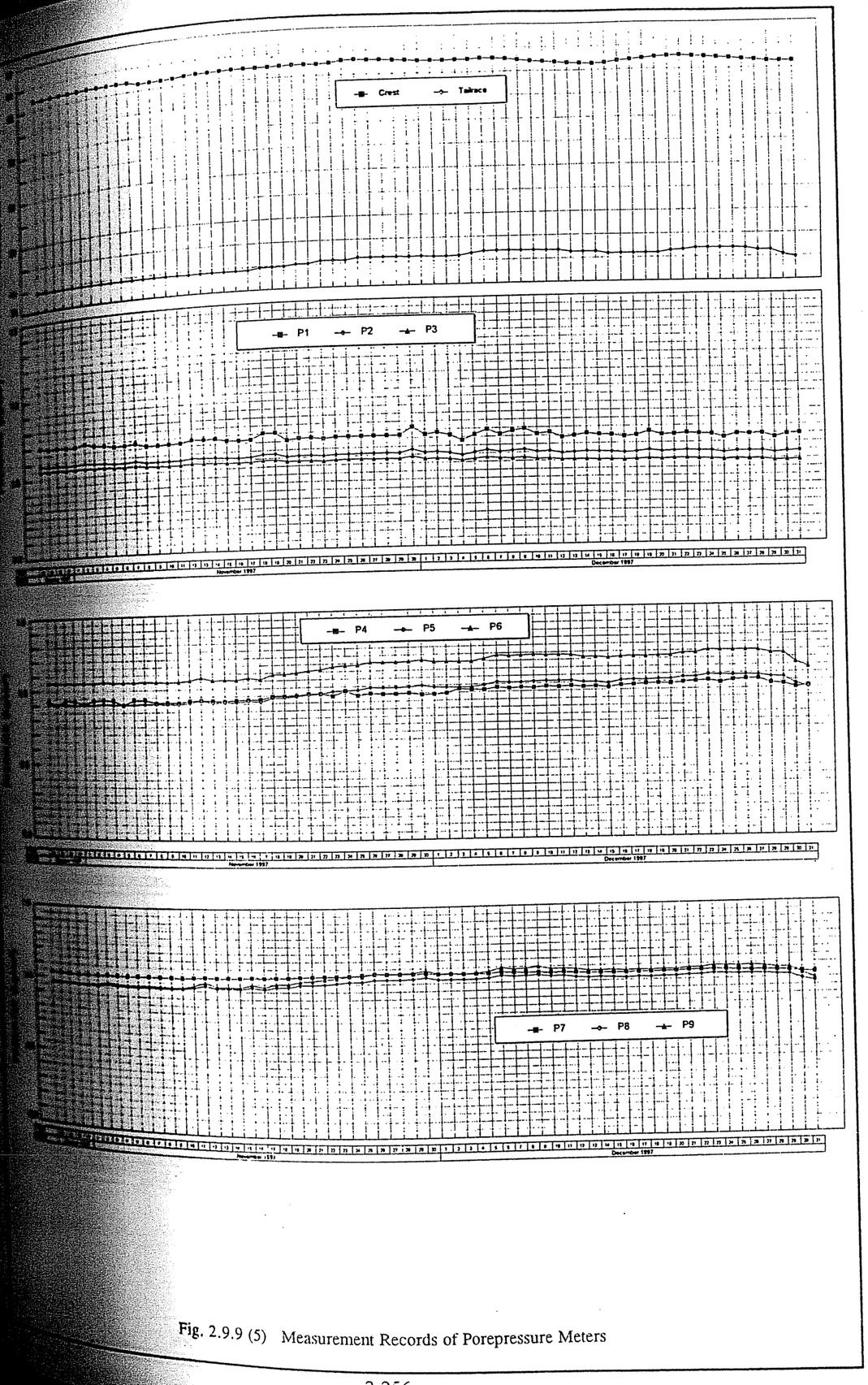

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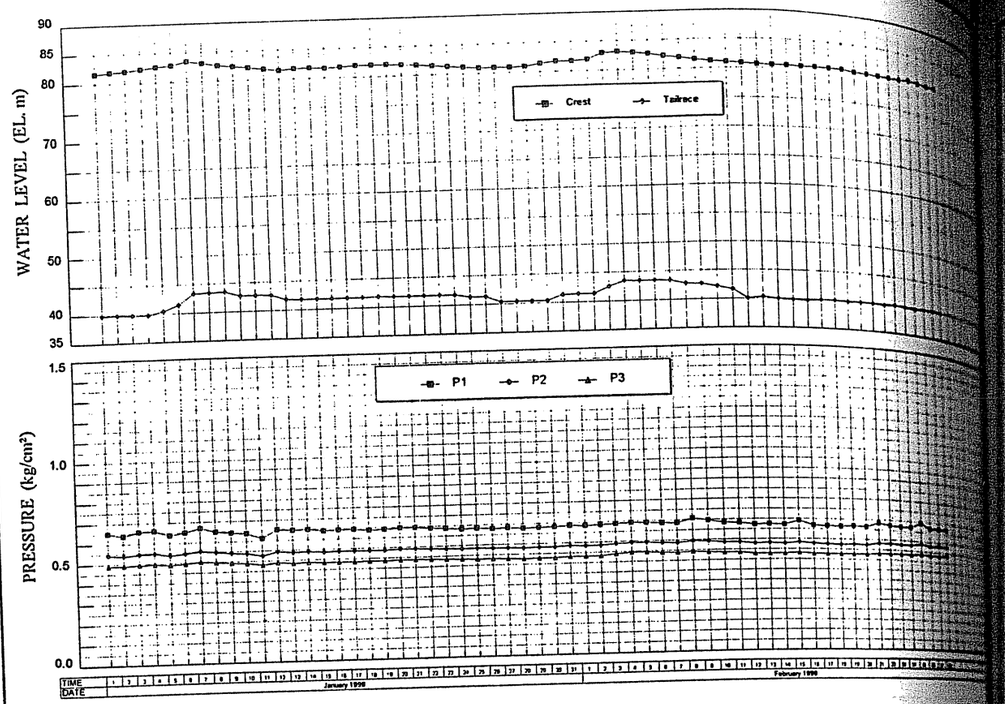
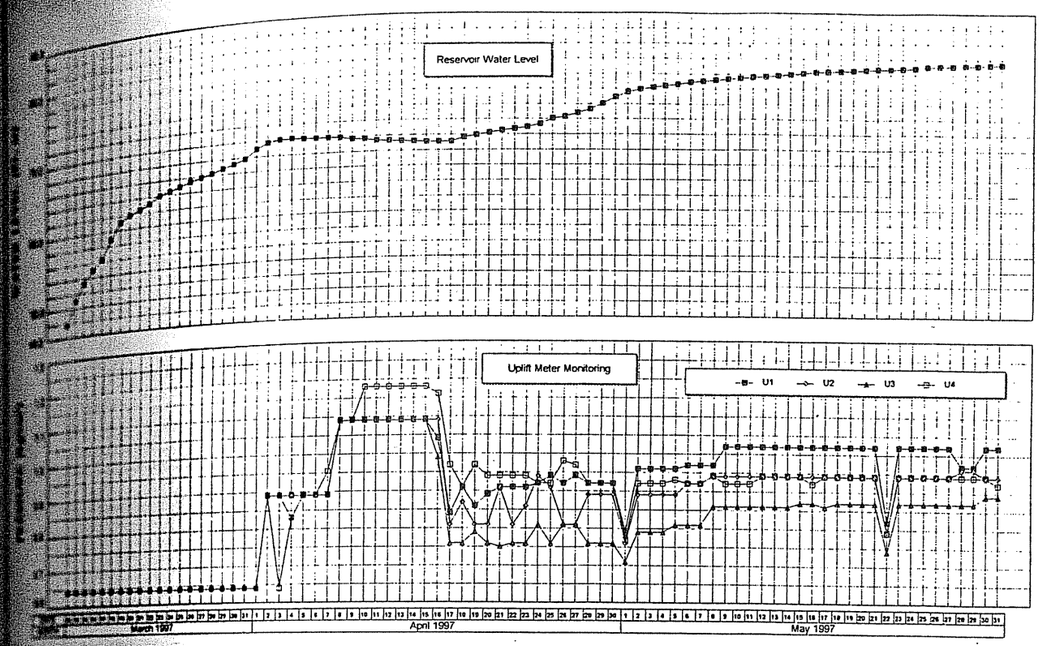


Fig. 2.9.9 (6) Measurement Records of Porepressure Meters



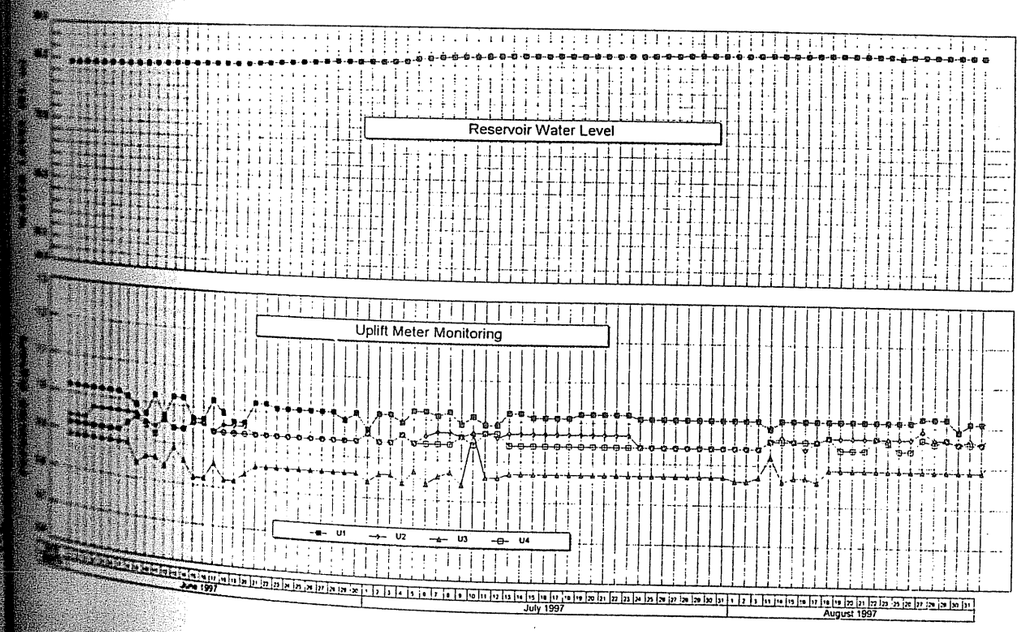
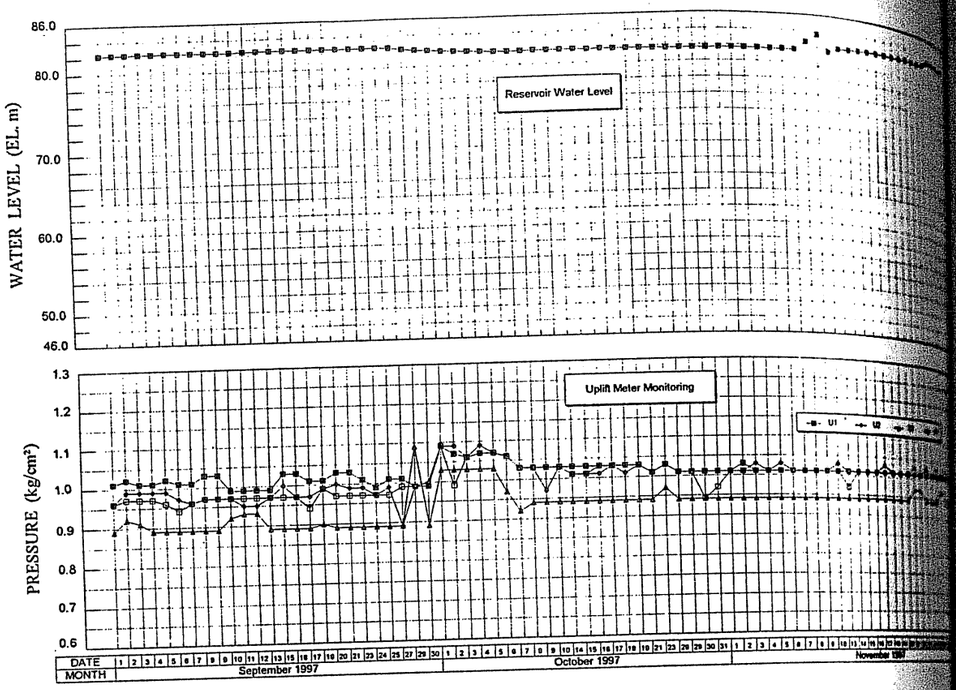


Fig. 2.9.10 (1) Measurement Records of Uplift Meters



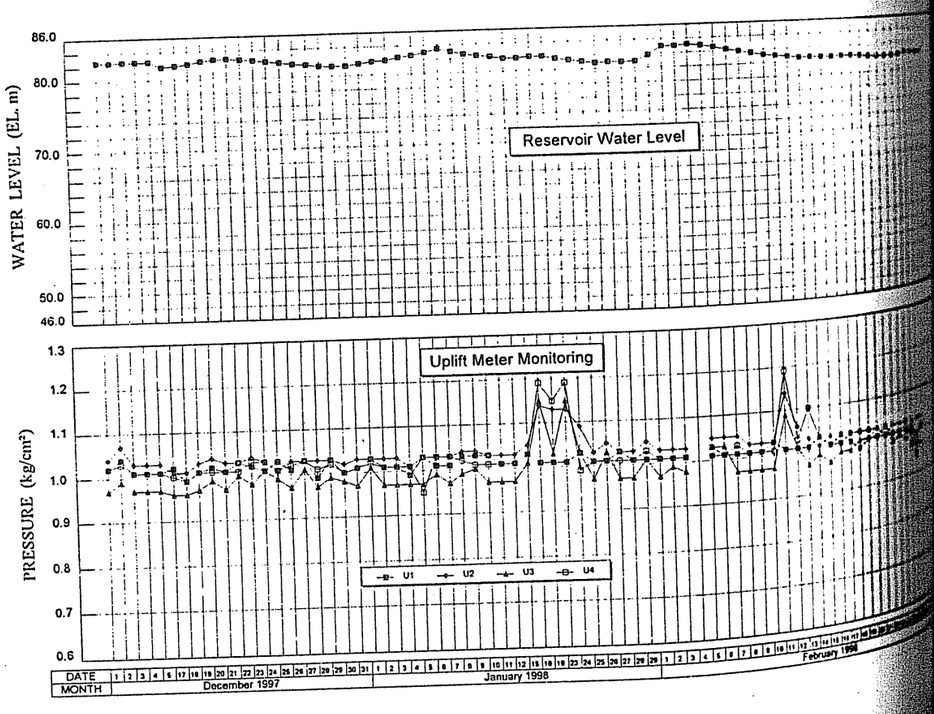
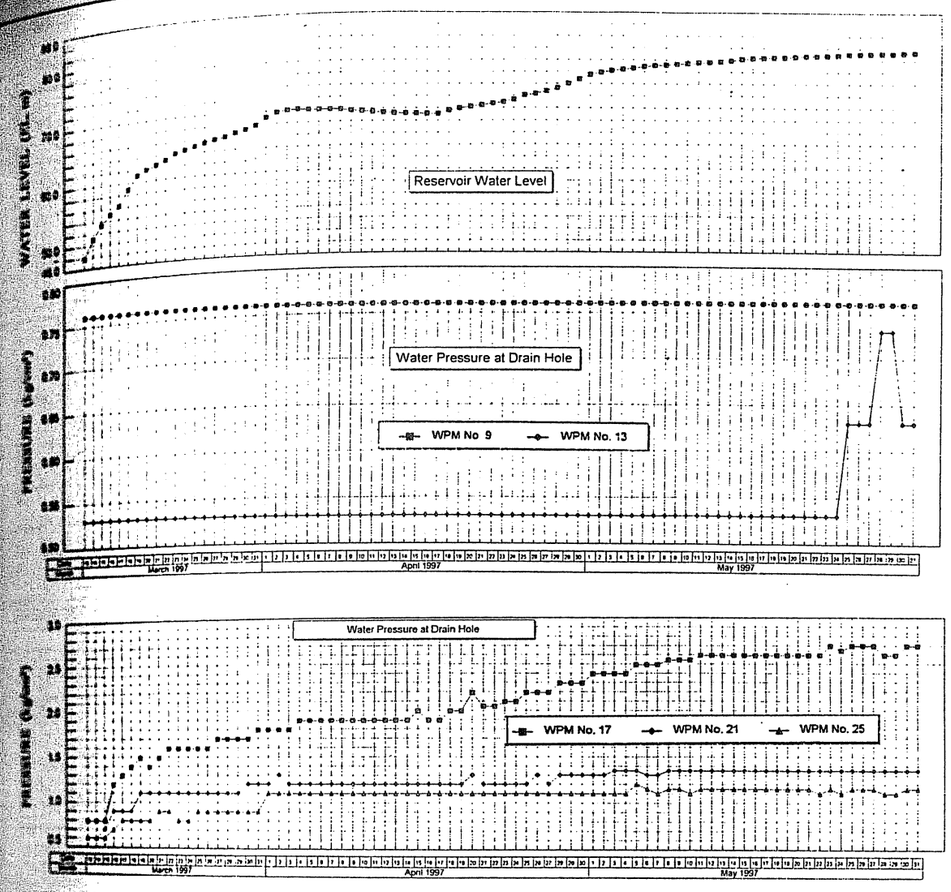


Fig. 2.9.10 (2) Measurement Records of Uplift Meters



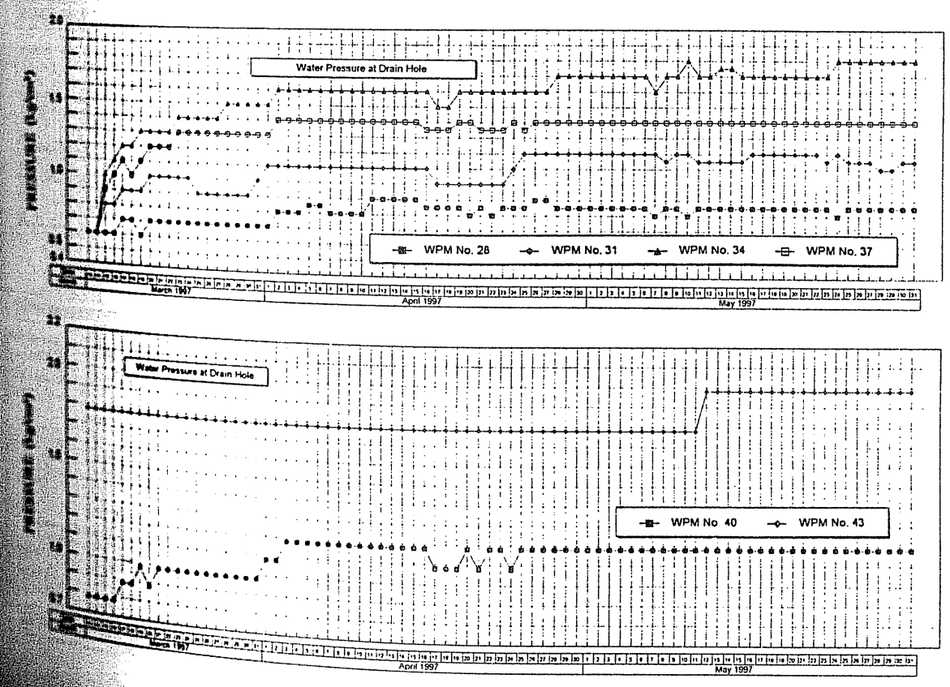
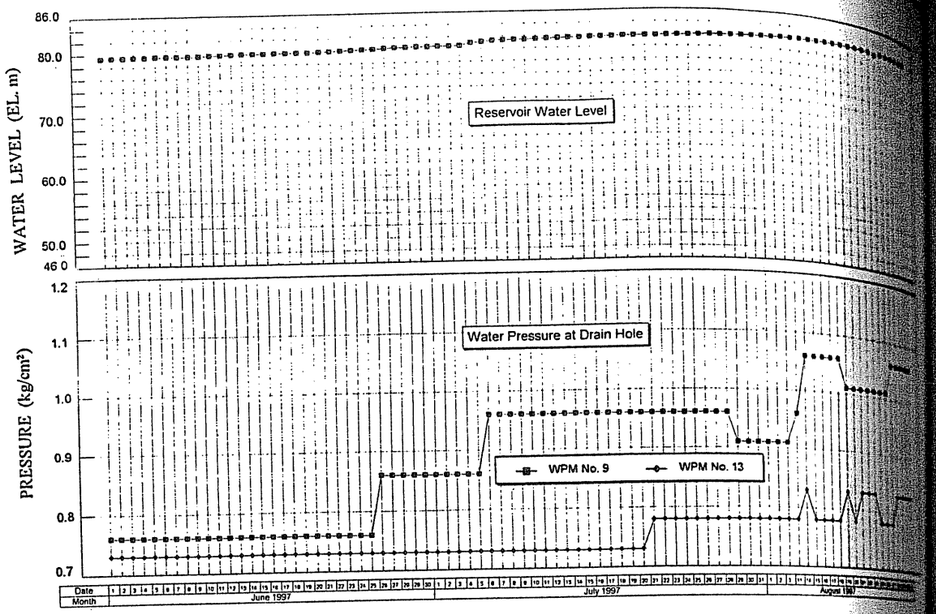


Fig. 2.9.11 (1) Measurement Records of Water Pressure Meters



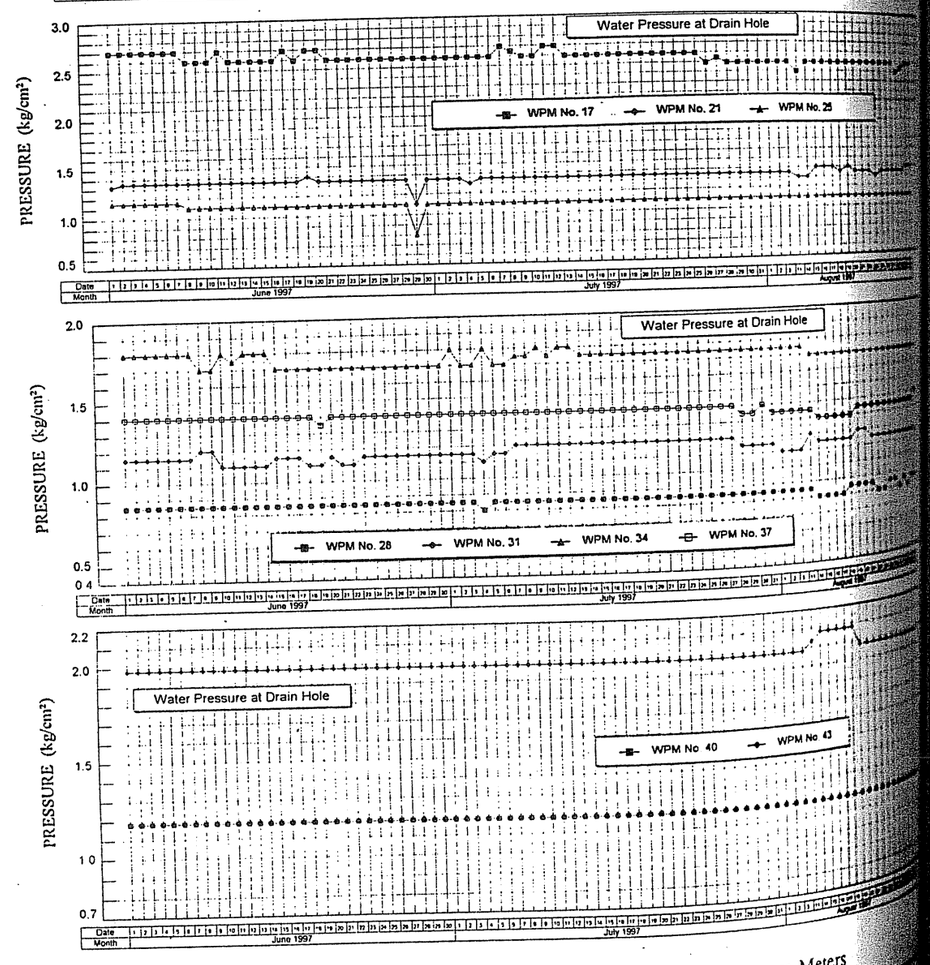


Fig. 2.9.11 (2) Measurement Records of Water Pressure Meters

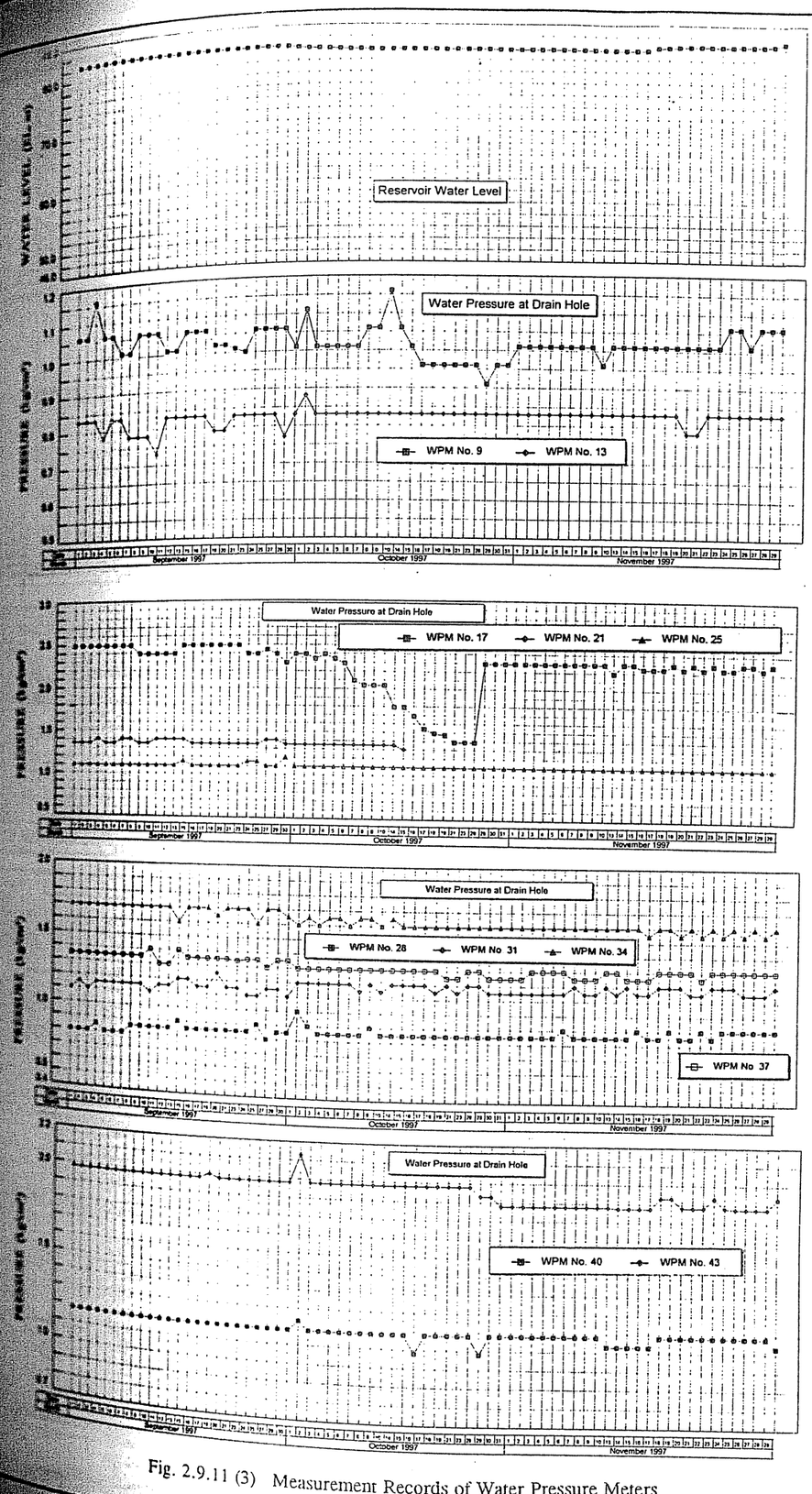


Fig. 2.9.11 (3) Measurement Records of Water Pressure Meters

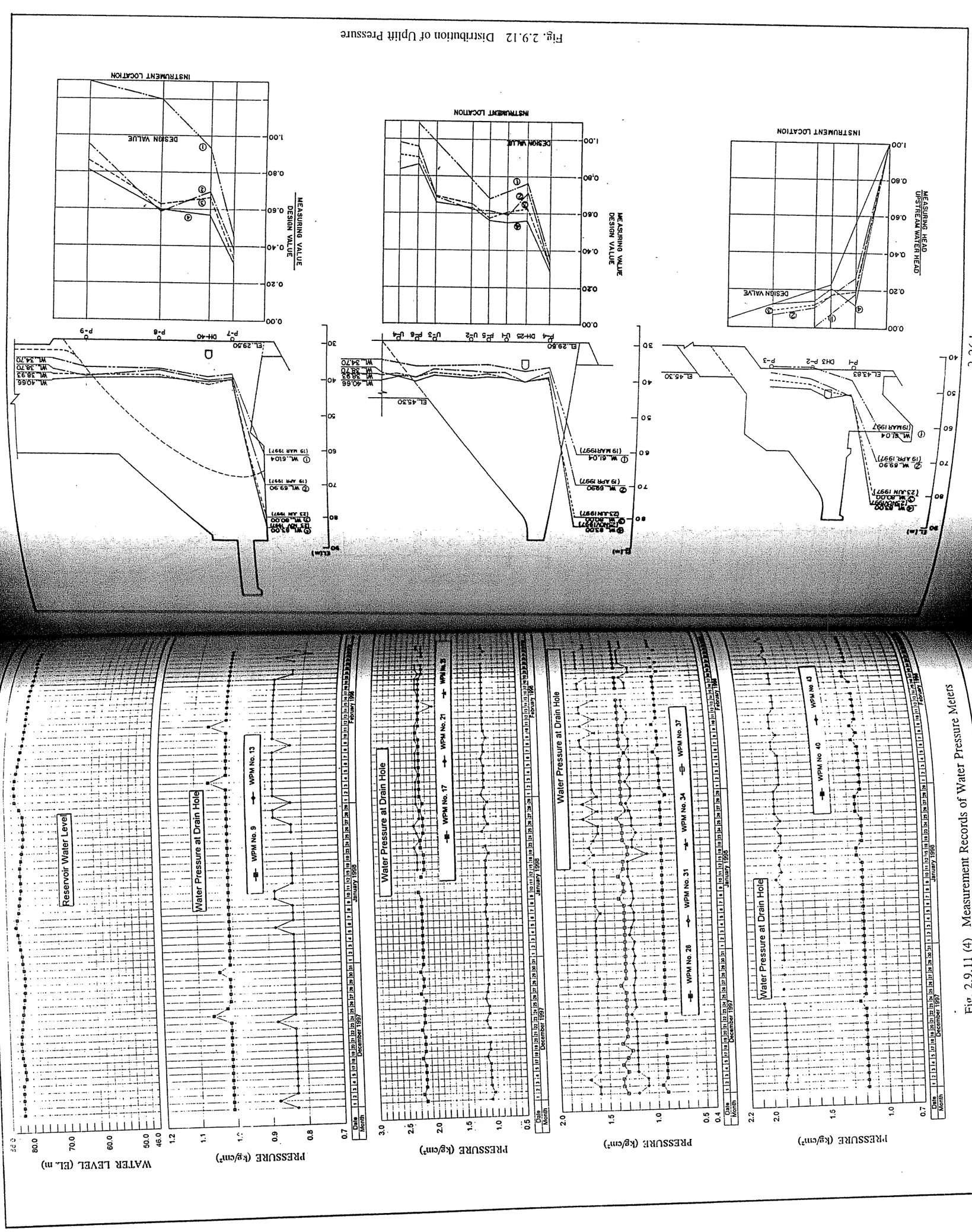


Fig. 2.9.12 Distribution of Uplift Pressure

Fig. 2.9.11 (4) Measurement Records of Water Pressure Meters

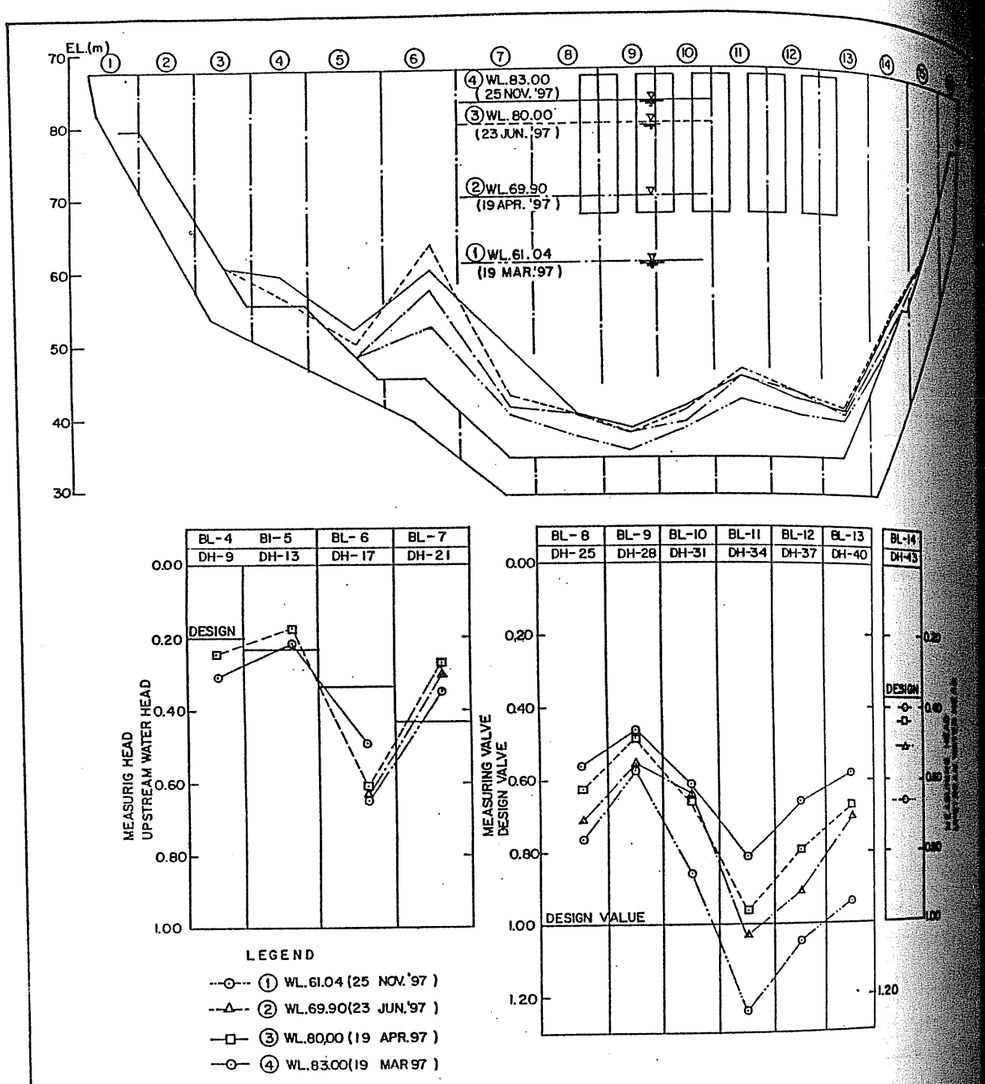


Fig. 2.9.13 Distribution of Uplift Pressure at Each Drainhole

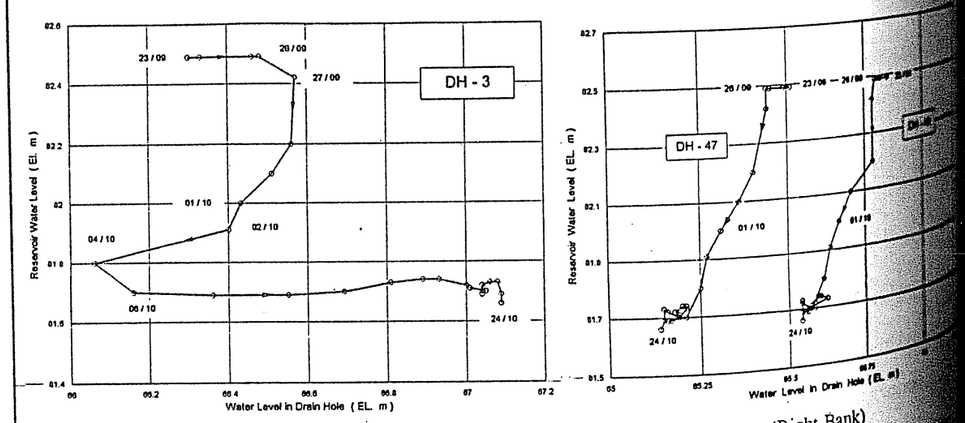


Fig. 2.9.14 Measurement Records of Water Level in Drainhole

PART 3

ARCHITECTURAL WORKS

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Chapter 1 INTRODUCTION

Architectural work in Lot I covers the building structure, finishes, mechanical and electrical works of the powerhouse including control block and relay house as well as outdoor planting works miscellaneous works around the powerhouse. Moreover, architectural work in Lot 6 covers the building structure, finishes, mechanical and electrical work of Pekanbaru substation building. The features of the architectural works are as follows:

Powerhouse

The generator room, where two 85ton overhead cranes can be operated and erection by is located beside, was arranged on the ground floor of the powerhouse. The control block is located beside the generator room in Powerhouse. The central control room and computer relay room were arranged on the second floor and the office rooms were arranged on the third floor of control block of Powerhouse. There is a elevator which has 1.5 ton capacity and serves 5 landings including basement floors beside the entrance hall of the control block in Powerhouse.

The roof ventilators were arranged on the roof of generator room of Powerhouse. The rooms such as the equipment room on basements floor are also ventilated by fans. Main rooms of the control block are air conditioned. The utility water supply for some rooms and the fire hydrant water supply for all area were arranged in Powerhouse.

The lighting fixtures were provided all rooms from basement floor to top roof ceiling including emergency lamps so that the illumination level could reached to the suitable level. The heat detector and smoke detectors were arranged for all rooms in order to protect the equipment and people from the fire.

The relay room situated beside the switchyard is located about 500m far from the Powerhouse. The relay room is one story building and houses equipment for control of switchyard. Air conditioning system was arranged for the electric panels. The fire protection system was also arranged and the signal is transferred to the central control room of Powerhouse. The lavatory was accommodated in this building.

Pekanbaru Substation Building

The substation building is located in Pekanbaru. The building is two story building and houses equipment for control of substation. Air conditioning system was arranged for the electric panels. The fire protection was also arranged. The lavatory was accommodated in this building.

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Chapter 2 DESIGN

2.1 Basic Design

2.1.1 Scope of Architectural Works

Powerhouse : (i) Superstructure (steel structure, siding and roofing and finishing) and (ii) Building Facilities (ventilation and air conditioning system, lighting system, indoor fire protection system, plumbing system , elevator).

Switchyard Relay House : (i) Substructure, (ii) Superstructure (reinforced concrete, Finishing) and (iii) Building Facilities (ventilation and air conditioning system, lighting system, indoor fire protection system, plumbing system).

Pekanbaru Substation Building : (i) Substructure, (ii) Superstructure (reinforced concrete, finishing) and (iii) Building Facilities (ventilation and air conditioning system, lighting system, indoor fire protection system, plumbing system).

The building works were assigned to the Lot I International Tender for Powerhouse & Switchyard Relayhouse and Lot-6 Local Tender for Pekanbaru Substation.

2.1.2 Summary of Design

(1) Powerhouse Superstructure

Building dimensions

• Story	3 stories	• Height of floor	
• Area		First floor	FL + 0.0 (EL + 45.500)
First floor	2,740.32 m²	Second floor	FL + 4.500
Second floor	871.20 m²	Third floor	FL + 9.000
Third floor	871.20 m²	Low roof	FL + 18.160
Penthouse	60.50 m²	High roof (G. Room)	FL + 17.700
Total floor area	4,543.22 m²	• Building volume	44,596.3 m³
		• Crane span	22.88 m

Structural planning

Main structure	:	Structural steel
Framing method	:	Vertical bracing for longitudinal direction, Rigid frame for transverse direction
Roof	:	Reinforced concrete with metal deck
Exterior wall	:	RC wainscot and precast concrete panel

Building Facilities

Ventilation and air conditioning system, Lighting system, Drinking water system, Sanitary and drainage, Indoor fire protection system, Elevator (Remarks : Lightning protection system is not provided in for the height of building less than 20 m in accordance with Japanese standards).

(2) Pekanbaru Substation Building

Building dimensions

• Story	2 stories	• Height of each floor	
• Area		First floor	FL + 0.0 GL + 0.4 m
First floor	570.2 m²	Second floor	FL + 4.7
Second floor	456.0 m²	Roof	FL + 9.1
Total floor area	1,026.0 m²	• Building volume	4,685.4 m³

Structural planning

Substructure	:	Reinforced concrete footing
Superstructure	:	Reinforced concrete
Roof and floor	:	Reinforced concrete slab
Exterior wall	:	Brick masonry

Building Facilities

Ventilation and air conditioning system, Indoor lightning system, Sanitary and drainage, Fire protection system and Drinking water supply system.

(3) Kotapanjang Switchyard Relay House

Building dimensions

• Story	1 story	• Height of each floor	
• Area		First floor	FL + 0.0 GL + 0.4 m
First floor	195.0 m²	Roof	FL + 4.2 m
Total floor area	195.0 m²	• Building volume	819.0 m³

Structural planning

Substructure	:	Reinforced concrete footing with PC panels
Superstructure	:	Reinforced concrete
Roof and floor	:	Reinforced concrete slab
Exterior wall	:	Brick masonry

Building Facilities

Ventilation and air conditioning system, Indoor lighting system, Sanitary and drainage, Fire protection system and Drinking water supply system.

Finishing schedule of the powerhouse, substation building and switchyard relayhouse is shown in the Table 3.2.1.

Table 3.2.1(1) Powerhouse Finishing Schedule

Room	Floor		Base		Wall			Ceiling
	Mat	Fin.	Mat	Fin.	Mat	Fin	Mat	
Basement Floors	Conc.	S. T. F			Conc.		Conc.	
Battery Room	Mortar S. T. F	A. P. V. T	Conc.	A. P. V. B	Do.	A. P. P	Do.	
Generator Room	Mortar	Hardener F.			Do.	Paint	Metal Deck	
Entrance Hall	Do.	T. B	Mortar	T. B	Mortar	G. W. T	P.B	
Communication Equip Room	Do.	Do.	Do.	Do.	C. P	Paint	Do.	
Cable Treatment Room	Do.	S. T. F			Brick	C. P	Metal Deck	
A/C Machine Room								
Central Control Room	F. A. F	P. C. T	A. C. B	V. B	A. C. B	V.C	P.B	
Computer / Relay Room	Do.	Do.	Do.	Do.	Do.	Do.	Do.	
Office Room	Mortar	T. B	Mortar	T. B	C. P	Paint	Do.	
Conference Room	Do.	Do.	Do.	Do.	Do.	Do.	Do.	
Superintendent Room	Mortar S. T. F	Carpet	Do.	Do.	A. C. B	V.C	Do.	
Reception Room	Do.	Do.	Do.	Do.	Do.	Do.	Do.	
Corridor	Mortar	T. B	Do.	Do.	C. P	Paint	A.C.B	
Lavatory	A. W. / L. C	C. M. T	....		Mortar	G. W. T	Do.	
Staircase	Mortar	T. B	Mortar	T. B	C. P	Paint	Do.	

Finish Schedule Abbreviations

A. B	Acoustic Board	C. H. B	Concrete Hollow Block	P. B	Plaster Board
A. C. B	Asbestos Cement Board	C. M. T	Ceramic Mosaic Tile	T. B	Terrazzo Block
A. P. P	Acidproof Paint	L. C	Lean Concrete	P. C. T	Polyvinyl Chloride Asbestos Tile
A. P. V. B	Acidproof Vinyl Base	G. W. T	Glazed Wall Tile	V. B	Vinyl Base
A. P. V. T	Acidproof Vinyl Tile	F. A. F	Free Access Floor	V. C	Vinyl Cloth
A. W	Asphalt Waterproof	S. T. F	Steel Trowel Finish	C. P	Cement Plaster

Table 3.2.1(2) Substation Building Finishing Schedule

Room	Floor		Base		Wall			Ceiling
	Mat	Fin.	Mat	Fin.	Mat	Fin	Mat	
Entrance Hall	Mortar	T. B	Mortar	T. B	Mortar	Paint	S.C	
Transformer Room	L. C / Mortar	S. T. F	Do.	Paint	Do.	Do.	Concrete	
Battery Room	Mortar S. T. F	A. P. V. T	Do.	A. P. V. B	Do.	A. P. P	Do.	
20 kV Cubicle Room	L. C / Mortar	Hardener F.	Do.	Paint	Do.	Paint	Do.	
Storage Room	Mortar	S. T. F	Do.	.....	B. M		Do.	
Control / PLC Room	L. C / Mortar	T. B	Do.	T. B	Mortar	Paint	S.C	
Meeting Room	Mortar	Do.	Do.	Do.	Do.	Do.	Do.	
Manager's Room	Mortar S. T. F	Carpet	Do.	Do.	Do.	V.C	Do.	
Praying Room	Do.	Do.	Do.	Do.	Do.	Do.	Do.	
Lavatory	L. C / Mortar	C. M. T	.....	.....	Do.	G. W. T	A.C.B	
Kitchenette	Mortar	T. B	Mortar	T. B	Do.	Paint	Do.	
Stair	Do.	Do.	Do.	Do.	Do.	Do.	Mortar	

Finish Schedule Abbreviations

A. B	Acoustic Board	A. P. V. T	Acidproof Vinyl Tile	S. T. F	Steel Trowel Finish
A. C. B	Asbestos Cement Board	B. M	Brick Masonry	S. C	Suspended Ceiling
A. P. P	Acidproof Paint	C. M. T	Ceramic Mosaic Tile	T. B	Terrazzo Block
A. P. V. B	Acidproof Vinyl Base	L. C	Lean Concrete	V. B	Vinyl Base
A. W	Asphalt Waterproof	G. W. T	Glazed Wall Tile	V. C	Vinyl Cloth

Table 3.2.1(3) Kotapanjang Switchyard Finishing Schedule

Room	Floor		Base		Wall			Ceiling
	Mat	Fin.	Mat	Fin.	Mat	Fin	Mat	
Relay Room	L. C / Mortar	Hardener F.	Mortar	Paint	Mortar	Paint	P.B	
PLC Room	Do.	Do.	Do.	Do.	Do.	Do.	Do.	
Battery and Charger Room	Mortar S. T. F	A. P. V. T	Do.	A. P. V. B	Do.	A. P. P	Concrete	
20 kV Cubicle Room	L. C / Mortar	Hardener F.	Do.	Paint	Do.	Paint	Do.	
Storage Room	Mortar	S. T. F	Brick	.....	B. M		Do.	
Lavatory	L. C / Mortar	C. M. T	.....	.....	Mortar	G. W. T	A.C.B	

Finish Schedule Abbreviations

A. B	Acoustic Board	A. P. V. T	Acidproof Vinyl Tile	S. T. F	Steel Trowel Finish
A. C. B	Asbestos Cement Board	B. M	Brick Masonry	P. B	Plaster Board
A. P. P	Acidproof Paint	C. M. T	Ceramic Mosaic Tile	T. B	Terrazzo Block
A. P. V. B	Acidproof Vinyl Base	L. C	Lean Concrete	V. B	Vinyl Base
A. W	Asphalt Waterproof	G. W. T	Glazed Wall Tile	V. C	Vinyl Cloth

2.1.3 Design Criteria, Conditions and Major Materials to be Used

(I) Design Conditions

i) Ambient temperature : Maximum 39.5^{0}C, Minimum 17.5^{0}C and Monthly Average is Maximum 27.9^{0}C, Minimum 23.9^{0}C. Humidity is Average 84%.

ii) Live Load : The live load as shown in the table 3.2.2 is applied.

iii) Dead Load : The dead load has been calculated by the following specific gravity for each material. Reinforced concrete 2.4 t/m^3, Plain concrete 2.3 t/m^3, Fill and lean concrete 1.8 t/m^3, Mortar 2.0 t/m^3, Steel 7.85 t/m^3 and Brick 0.45 t/m^2.

(iv) Wind Load : Design wind velocity 25 m/sec.

(v) Seismic Load : The design seismic load is determined by the story shear coefficient of earthquake on a portion of building at a certain height.

(vi) Crane Load : The vertical on crane runaways is not less than 110 the lateral force on crane runways is not less than 10 percent of the maximum wheel loads of the crane. The longitudinal force on the crane runway is not less than 15 percent of the braking wheel loads of the crane, and is applied at the top of rail.

(vii) Combined Design Force

Long term		D.L (Dead load) + L.L (Live load) + C.L (Crane operation load)
Short term	Case - 1	D.L (Dead load) + L.L (Live load) + C.D.L (Crane dead load) + W.L (Wind load)
	Case - 2	D.L (Dead load) + L.L (Live load) + C.D.L (Crane dead load) + S.L (Seismic load)

Table 3.2.2 Live Load

kg/m^2

Building	Room	Live Load				Seismic Load	Remarks
		Floor	Beam	Girder	Col. Found.		
Powerhouse, Substation Building and Relay House	Roof	90	80	70	70	30	
	Central Control Rm. Computer / Relay Rm. Chief Operation Rm.	400	360	300	300	200	Excluding machine loads
	Office Rm.	300	240	180	180	80	
	Lavatory	200	130	130	130	60	
	Battery Rm.	500	500	500	350	350	
	Others	200	130	130	130	60	
Gatehouse	Stair	300	240	180	180	80	
	Grating Floor	200	130	130	130	60	

(2) Allowable Unit Stress

(i) Allowable Unit Stresses of Concrete (kg/cm²)

Materials \ Stresses		Permanent Stresses					Temporary Stresses		
		Compress	Shear	Bond			Compress	Shear	Bond
				A	B	C			
Ordinary Concrete C-210	Plain bar	70	7.0	8.4	12.6	8.4	Permanent Stresses x 2.0	Permanent Stresses x 1.5	
	Deformed bar			14.0	21.0	14.0			
Ordinary Concrete C-270	Plain bar	90	9.0	9.0	13.5	9.0			
	Deformed bar			16.2	24.3	16.2			
Ordinary Concrete C-180	Plain bar	60	6.0	7.2	10.8	7.2			
	Deformed bar			12.0	18.0	12.0			

* Remarks A : Top bar of flexural members, B : Bar, except "Item A", of flexural members, C : Anchors and lap splices.

(ii) Allowable Unit Stresses of Reinforced Bars (kg/cm²)

Stresses Materials	Permanent Stresses		Temporary Stresses	
	Tension Compression	Shear Reinforcement	Tension Compression	Shear Reinforcement
SD 30	2,000	2,000	2,000	3,000
SD 35	2,200	2,000	2,000	3,000

(iii) Allowable Unit Stresses of Steel (ton /m²)

Type of Steel	For General Structures SS 41	For Welded Structures SM 41A
Thickness not more than 40 mm, t = 40 mm	2.4	2.4
Thickness more than 40 mm, t = 40 mm	2.2	2.2

(iv) Allowable Strength per High Strength Solt (Friction Type)

Materials	Bolt Nominal Dia.	Dia. of Bolt Hole (mm)	Bolt Effective Area (cm²)	Bolt Allowable Tensile Stress	Permanent Strength			Temporary Strength (t)
					Shear (t)		Tension (t)	
					Single Friction	Double Friction		
F10T	M16	17.5	1.52	10.3	3.02	6.03	6.23	Permanent Strength × 1.5
	M20	21.5	2.38	16.1	4.71	9.42	9.73	
	M22	23.5	2.95	20.0	5.70	11.4	11.8	
	M24	25.5	3.42	23.1	6.78	13.6	14.0	
F11T	M16	17.5	1.52	10.9	3.22	6.43	6.63	
	M20	21.5	2.38	17.0	5.02	10.0	10.4	
	M22	23.5	2.95	21.1	6.08	12.2	12.5	
	M24	23.5	3.42	24.4	7.23	14.5	14.9	

Note : High strength bolts shall be used for all main connections and joints.

(v) Allowable Strength per Medium Bolt

(Unfinished medium bolts – Bearing Type)

Materials	Bolt Nominal Dia.	Dis. of Bolt Hole (mm.)	Bolt Gross Area (cm²)	Permanent Strength			Temporary Strength (t)
				Shear (t)		Tension (t)	
				Single Shear	Double Shear		
SS41	M12	12.5	1.13	1.02	2.03	1.36	Permanent Strength × 1.5
	M16	16.5	2.01	1.81	3.62	2.41	
	M20	20.5	3.14	2.83	5.65	3.77	
	M22	22.5	3.80	3.42	6.84	4.56	
	M24	24.5	4.52	4.07	8.14	5.42	

(vi) Allowable Unit Stresses in Welded Joints (t / cm²)

Stresses \ Welding Positions \ Materials		Permanent Stresses				Fillet Weld	Temporary Stresses
		Groove Weld					
		Tension	Compress	Bending	Shear		
SS41	(2)	1.2	1.2	1.2	0.7	0.7	Permanent Stress × 1.5
	(1)	1.4	1.4	1.4	0.8	0.8	
SM41A	(2)	1.4	1.4	1.4	0.8	0.9	
	(1)	1.6	1.6	1.6	0.9	0.9	

(1) Flat or horizontal

(2) Overhead or vertical

(3) Major Material to be Used

(a) Structural Materials to be Used

Plain and reinforced concrete : Leveling concrete and lean concrete 120 kg/cm^2 and reinforced concrete for building sub/super structures, equipment foundation 210 kg/cm^2 for required 28 day compressive strength.

Reinforcing bars : SR 24 (round bar) 2400 kg/cm^2 and SD 30 (Deformed bar) 3,000 kg/cm^2.

Structural steel : SS 41, SM 41A for framing, crane girder, supporting steel for platform and stairs 2400 kg/cm^2 and bolts for structural steel is high strength bolt JIS B1186 F10T.

Miscellaneous steel : Same as structural steel.

Brick (Non-load bearing type) : compressive strength 100 kg/cm^2.

Metal decking sheet for floor and roof slab concrete : V-Type galvanized decking sheet, thickness 1.6 m/m and weight 17.8 kg/m^2.

(b) Architectural Material

Architectural material is based on of the following specified quality, and was adjusted and/or modified in accordance with climatic conditions at the site.

Precast concrete panel : Thickness 150 mm

Checkered plate : Thickness 6 m/m and/or 4.5 m/m and reinforcing angle L-40x40x6.

Grating : Galvanized steel bearing bars . The bearing bars are 32 mm thick and the space is be not more than 25 mm from center to center and the grating can withstand a live load of 200 kg/m^2.

Metal suspension furring : Furring channels were galvanized light gauge angle steel.

Metal flashing for pipe penetration : Steel Plate with thickness 4.5 m/m.

Hand railing : Steel pipe for hand railing 40 , Post 32 , Intermediate railing 25).

Aluminium windows and louvers : All Aluminium windows were completed with hardware, mullions, mullion covers, anchors and accessories. All metal items were anodized finish. Window and anchorage shall withstand a wind pressure of 35 kg/cm^2.

Steel door : All hollow core steel doors were completed with hardware and accessories. All metal items were galvanized steel metal. The thickness 40 m/m and all exterior doors can withstand a wind pressure of 150 kg/cm^2.

Steel roll-up door (Shutter) : Roll-up doors were of galvanized steel, motor-operated roll-up type, and the door can withstand a wind pressure of 150 kg/m^2.

Wooden door : Interior wooden doors were flush, hollow-core doors covered with water proof plywood, and was 35 mm in thickness.

Glass and glazing : For exterior windows, 6.8 mm thick, wire reinforced, polished plate, for interior windows, 6.8 mm thick, wire reinforced, polished plate, for other interior windows, 3 mm thick, polished plate.

Calking and sealing : For glazing (oil-base glazing compound), for masonry and others inside (oil calking) and for PC panel and others out (poly sulfide sealing compound side).

Tile work : Ceramic mosaic tile was 50mm x 50mm x 6mm. Vinyl asbestos tile was 300mm x 300mm x 3.2mm and the glazed wall tile was 108mm x 108mm x 8.0mm.

Asbestos cement board : Asbestos cement board was 910mm wide x 6.3mm. Moisture content was not more than 12%.

Painting : For metal (rust preventive paint and oil paint), for masonry and as-besots cement board surfaces (vinyl paint) and for wooden surface (oil Paint).

Water proofing for roof : Water proofing shall be of built-up asphalt roofing consisting of 4 layers including perforated asphalt roofing.

2.1.4 Design Concept, Arrangement and Selection for Superstructure

(1) Selection of Main Structural Materials

From the past experiences, structural steel was selected as the powerhouse superstructure based upon the following reasons : i) generally, powerhouse building has a long span, and heavy equipment (overhead traveling crane) were arranged at the high place of the building, consequently, light weight and strong framing members were required and (ii) moreover, the construction period to erect a steel frame structure was shorter than to erect a reinforced concrete one.

(2) Arrangement of the Respective Rooms

2nd Floor : The central control room has been arranged on the 2nd floor in consideration that close relation and convenient communication with the other respective floors were ensured and that the entire generator room can be looked over from this floor. Since the computer and relay rooms are closely related to the central control room in view of the nature of equipment, these rooms have also been arranged on the same floor.

3rd Floor : Generally, the space where routine work is constantly performed should be located on the above ground floor where natural lighting and ventilation is ensured. Since many workers are engaged in their jobs in an office room, therefore, the room has been arranged adjacent to utilities to ensure the convenience for the workers. The Superintendent room has been arranged behind the office room and next to the conference room in order not to permit direct access of persons from outside.

(3) Selection of Exterior Wall Materials

(a) Precasted Concrete Panel : i) Excellent in durability and weatherability, ii) This material ensures saving of manpower for the work and reduction of work period when compared with cast-in-place concrete, iii) This material is incombustible because of concrete product and iv) Excellent in thermal ans sound insulation effects, small in air conditioning loss and optimum for control building where almost all rooms are air-conditioned.

(4) Selection of Interior Materials

(a) Central Control Room, Computer Room and Relay Room

Floor (Free access floor) : This free access floor has been selected because of the convenience for cable ling (requirement from electrical equipment).

Wall (Vinyl cloth) : i) Free from flaw and contamination (fouling), ii) Excellent in durability, iii) This material is incombustible, iv) Abundant in color tones, patterns and other finishing grades and v) The atmosphere of room is softened.

Ceiling (Acoustic board) : i) Rich in appearance, ii) High in sound absorption, iii) Incombustible, iv) Low in thermal conductivity and high in thermal insulation effect and v) Excellent in workability.

(b) Office Room

Floor (Terrazzo block) : i) This material is locally procurable and local workers are familiar with this material, ii) Low in cost and iii) Less susceptible to flow and contamination.

Wall (Paint finished) : i) Low in cost, ii) Locally procurable, iii) Excellent in workability and iv) Rich in color tones.

Ceiling (Acoustic board) : Refer to Central Control Room in Clause (a).

(c) Superintendent Room

Floor (Carpeted floor) : i) Excellent in walking comfort ability, ii) Excellent in interior appearance and a relaxing atmosphere produced and iii) High in sound absorption effect.

Wall (Vinyl cloth) : Refer to Central Control Room in Clause (a)

Ceiling (Acoustic board) : Refer to Central Control Room in Clause (a).

(5) Selection of Structural Method for the Powerhouse

The three kinds of structural methods, that is, steel structure, reinforced concrete with steel truss ans steel framed reinforced concrete structure with steel truss, were proposed for study and evaluation. The evaluation of these three structure methods was made in consideration of procedure, work content, period and cost of each method. The following contains a explanation of the advantages and/or disadvantages of the three methods.

(a) Steel Structure

Construction procedures and contents of the work are very simple as the steel members are shop-fabricated. The field work comprises only erection and installation of siding and roofing, with the least amount of concrete for the superstructure. Construction term is the shortest among the tree kinds of structural methods.

Construction cost is slightly higher than those of the other two. It is not necessary to re-paint the surface of the steel structure in the Powerhouse at least for 10 years or longer because a good quality rust-prevent paint is applied two coats on it in a shop and a good quality vinyl resin paint is also applied two coats on it at the site.

(b) Reinforced Concrete Structure

When the concrete work on superstructure starts, the ground floor slab in the Generator Room will not yet be completed. Therefore, it is rather difficult to provide scaffolding, to install the forms and to take the necessary safety measures. The work is rather complex, as there are many embedded items such as anchor bolts, steel brackets, plates, etc. for connection of roof trusses an crane girders to the concrete columns or girders. It is necessary to provide a large capacity crane for erection of roof truss ; the necessary capacity is 127 tons or large, with the boom length 70m or longer. Construction term is the longest among the three kinds of structural methods : Construction cost is the lowest among the three methods.

(c) Steal Framed reinforced Concrete Structure

Construction term is almost same as the reinforced concrete structure method. (Term from the construction start of superstructure until O.H. crane operation is 13.0 months). Construction cost is lightly lower than the steel structure method.

Judging from the explanation previously given, the steel structure is the most suitable as the structural method for the Powerhouse based on the following standpoints :

i) Shortening the construction term of superstructure is the most advantageous factor for successful completion of the entire project.

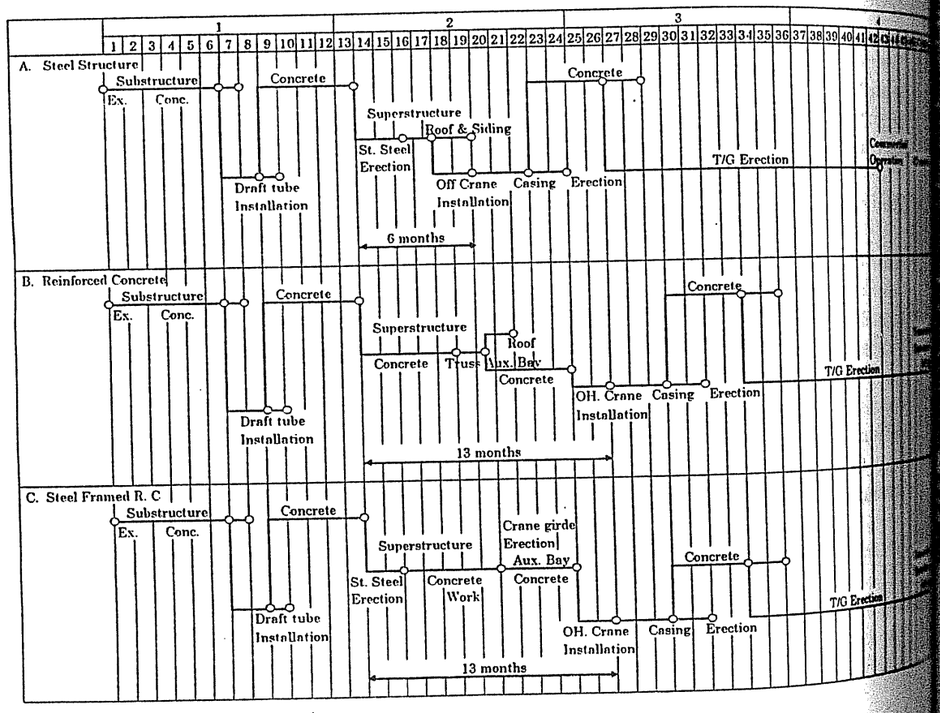
ii) Variations in the construction cost of the architectural work is not considered to be an important factor, as the ratio of construction cost of the architectural work to the entire project cost is quite low.

ii) Variations in the construction cost of the architectural work is not considered to be an important factor, as the ratio of construction cost of the architectural work to the entire project cost is quite low.

iii) Construction cost in case of steel structure is Rp. 503 x 10^6 higher than the cost in case of reinforced concrete structure. However, increase of income due to the early operation start (7 months earlier than reinforced concrete construction. Becomes Rp. 18.2 x 10^9.

The steel structure method is recommendable as the most adequate structural method for the Powerhouse as it enables time/labor saving and safety construction.

Table 3.2.3 Comparison of Overall Schedule of Powerhouse



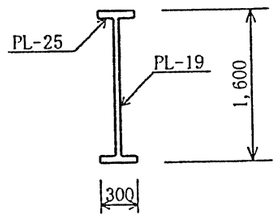
Steel Structure W/Steel Roofing & Steel Siding	Erection of Structural Steel from Gane. Room to Auxi. Bldg. Necessity of a Medium Capacity Crane for Erection.	Installation of Roofing / Siding Concreting Floor Slab & Installation of PC Panel for Wall. Necessity of Scaffolding around outside of Generator Room.	Key Date Structural Steel Erection, tightening bolts & paint Concrete Work Wall up to FL + 3.0 m in Generator Room Floor slab in Auxiliary Build Water proofing Roofing Siding PC Panel Fabrication Installation including Roof gutter 2 Months	170,000,000	175,000,000	Convert into RP. 1,920,000,000	○ ○ × ○
Reinforced Conc. Structure Roof: Steel Truss, Steel Deck & Reinforced Conc.	Concrete Work for Gane. Room (Column, Girder & Wall) Necessity of Scaffolding Around in & outside of Generator Room	Erection of Crane Girder & Roof Truss, Laying of Deck Plate & Casting of Concrete for Roof Necessity of a Large Capacity Crane for Erection	Key Date Structural Steel Erection, tightening Bolts & painting Concrete Work Col., Girder & Wall in Generator Room 2nd, 3rd floor, Roof & PH. Roof Water Proofing 15 Months	957,000,000	46,000,000	Convert into RP. 1,417,000,000	× × × ○
Steel Framed Reinforced Conc. Structure Roof: Steel Truss, Steel Deck & Reinforced Conc.	Erection of Structural Steel Necessary of a Medium Capacity Crane for Erection.	Concrete Work for Generator Room, Erection of Crane Girder, Laying of Deck Plate & Casting of Conc. for Roof Slab Necessity of Scaffolding around in-outside of Generator Room	Key Date Structural Steel Erection, tightening bolts & Paint Crane Girder Concrete Work Col., Girder, Wall & Roof Slab in Generator Room 2nd, 3rd, floor, Roof & PH. Roof Water proofing 14.5 Months	790,000,000	92,000,000	Convert into RP. 1,710,000,000	× × × ×

Notes
1. Construction Cost indicates direct cost.
2. Construction Cost excludes finishing and building services cost.

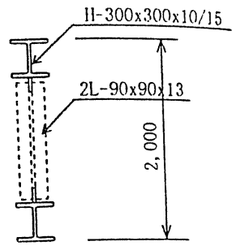
(6) Roof Beam of Powerhouse

From the comparison of single beam and truss beam, steel weight of truss beam 16.5t is less than that of single beam (8.24t). Moreover, the unit price of truss bean is nearly the same as that of built -up single beam. Then, the cost of truss beam is less than that of single beam. Therefore, the truss beam was selected for roof beam of the powerhouse.

Single beam



Truss beam



(7) Design Concept of Building Facilities

(a) Sanitary and Plumbing System

This scope covers drinking water system, waste water plumbing and battery room signataries, dose not include rainfall and osmatic water drainage system.

Water supply system : Drinking water is led from Law Water tank that is provided by Civil work, and distribute for each cock and respective valve by means of water supply pump. Moreover, strainers ans sterilizing equipment shall be provided in the drinking water line. The water supply pump unit consists of a compact type pressure tank, two pumps, two motors and control panel an designed so as to enable automatic operation either alternately or simultaneously. The sterilizing equipment is a chlorine sterilizing system designed to inject the chemical in proportion to the flowrate of drinking water.

Sanitary equipment : Water closet (western style), Water closet (Eastern style), Wash basin, Lavatory sink, Cock, Hot water heater, Water cooler, Kitchen unit, Shower set (Only Powerhouse).

Waste water system : Soil water flows by gravity and is led into a septic installed adjacent to the building. Settling water from the septic tank flows into the waste ditch together with rain water and miscellaneous waste water. The septic tank consists of a blower pump and an FRP tank. The quality of water to be discharged is B.O.D. 90 ppm or less.

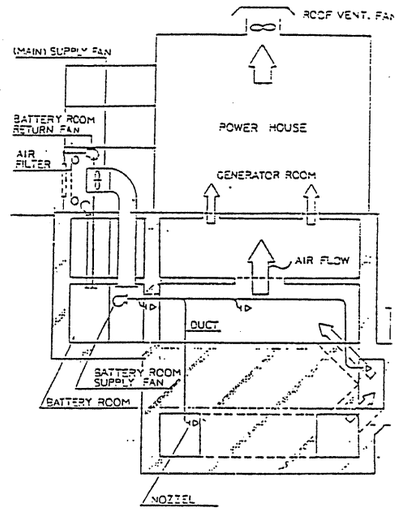
Water hydrant system (Only Powerhouse) : The scope of this section includes water hydrant system and piping, pump of drencher system expect for CO^2 hydrant system for generator. The water for hydrant system is led from draft tube by piping and supplied into the respective indoor hydrant boxes and respective value of drencher system by means of hydrant pump unit and drencher hydrant pump. The indoor hydrant box is equipped on each floor and is designed so as to include a built-in fire alarm device and have built-in automatic flow control valve, hoses, nozzle and hangers.

(b) Ventilation and Air Conditioning System

Local ventilating system of Powerhouse

Room	Methods of Ventilation	Air changes (time/hour)
Kitchen	by exhaust fan	10
Lavatory	ditto	20
Storage	ditto	5
Shower rm.	ditto	10
Battery Rm.	By supply and exhaust fan	10

Considering of the site location and the result of the site survey, the outdoor design condition is decided. The ventilating system consists of main ventilating system ad local ventilating system. Main ventilating system shall cover almost all parts of Powerhouse and perform by means of Supply/Exhaust fans and induction nozzle units.

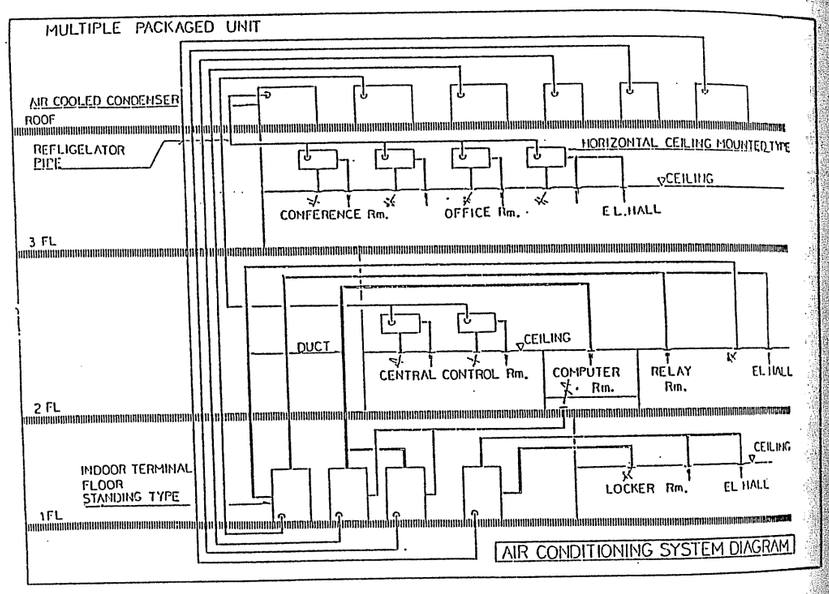


Ventilation system of Pekanbaru control building and Relay House

Room	Methods of Ventilation	Air changes (time/hour)
Cubicle Rm.	by supply and exhaust fan	*
Battery Rm.	ditto	10
Transformer Rm.	ditto	*
Toilet	by exhaust fan	20
Storage	ditto	5
Hall	ditto	5
Cable treatment Rm.	by supply and exhaust fan	more than 5

The air-conditioning system of the powerhouse is intended to perform air-conditioning is the respective rooms as shown in Table 7-2-z, and is Multiple Packaged units system. Two packaged air conditioner's (100% x 2) is provided for the air conditioning system of computer room. The packaged unit consists of cabinet compressor air cooled condenser, evaporator, air fans, air filter and control devices, and is factory-suit and factory tested. In addition, the air condition for control room is a teo-compressor type in order to serve higher free trouble.

The air-conditioning system of Pekanbaru control building and relay house is unitary packaged air conditioner system same as powerhouse. But in this case, package air-conditioner terminal is horizontal ceiling-mounted type to the utmost.



Secondary electrical wiring and automatic control system

Secondary electrical equipment is supplied to receive the main power supplied from the primary side at the distribution panels (for air conditioners, ventilators, etc.), and to distribute power to the respective facilities and equipment. The distribution panels are included in this scope of the works. In addition, it includes all wrings of control devices in order to provide automatic functioning of the equipment.

Table 3.2.5 Powerhouse Building Facilities List

Level	Room	Water Supply	Hot Water Supply	Drainage	Venti-lation	Air-condi-tioning
	Stock R.				○	
EL	Superintendent Rm.					○
54,500	Reception Rm.					○
3rd FL	Conference Rm.					○
	Office Rm.					○
	Prayer Rm.					○
	Kitchen	○	○	○	○	
	Lavatory (M)	○		○	○	
	Lavatory (W)	○		○	○	
	Storage Rm.				○	
	Elevator Hall					○
EL	Rest Rm.					○
50,000	Central Control Rm.					○
2nd FL	Computer R. & Relay Rm.					○
	Kitchen	○	○	○	○	
	Lavatory	○		○	○	
	Meeting Rm.					○
	Elevator Hall					○
EL	Cable Treatment Rm.				○	
45,500	Emergency Generator Rm.				○	
1st FL	Communication Equip. Rm.					○
	Spare Rm.					○
	Locker Rm. (M)					○
	Shower Rm. (M)	○		○	○	
	Lavatory (M)	○		○	○	
	Kitchen	○	○	○	○	
	Locker Rm. (W)					○
	Shower Rm. (W)	○		○	○	
	Lavatory (W)	○		○	○	
	Entrance Hall					○
	Generator Rm.	○		○	○	
		Water Cooler				
	Election Bay				○	
EL	Power Source Rm.				○	
40,900	Spare Rm.				○	
	Inverter				○	
	Battery Rm.				○	
	Storage Rm.				○	
EL	Storage Rm.				○	
37,600	Spare Rm.				○	
	Inverter				○	
	Battery Rm.				○	
	Sundry Rm.	○		○	○	
EL					○	
33,200	26,200				○	

Table 3.2.6 Pekanbaru Control Building Facilities List

Level	Room	Water Supply	Hot Water Supply	Drainage	Venti-lation	Air-condi-tioning
1F	Cubicle				○	
	Spare Rm.				○	
	Transformer Rm.				○	
	Toilet	○		○	○	
	Storage				○	
	Meeting Rm.					○
2F	Control R.					○
	PLC. R.					○
	Power Supply Rm.					○
	Battery & Charger Rm.				○	
	Meeting Rm.					○
	Manager Rm.					○
	Hall				○	
	Kitchen	○	○	○	○	
	Premises	○	–	○	–	–

Table 3.2.7 Kotapanjang Switchyard Relayhouse Facilities List

Level	Room	Water Supply	Hot Water Supply	Drainage	Venti-lation	Air-condi-tioning
1FL	Relay Rm.					○
	PLC Rm.					○
	Battery & Charger Rm.				○	
	20 kV Cubicle Rm.				○	
	Toilet	○		○	○	
	Storage				○	

(2) Indoor Lighting System

(a) Powerhouse

This section covers supply and installation of electrical panels and other necessary accessories for the indoor lighting system.

Lighting illumination level (Lx) of each room

Level	Room	Normal (Lx)	Emergency (Lx)
EL 54.500	Superintendent	300	50
	Conference Rm.	300	50
	Office	300	10
	Lavatory	200	10
	Elevator Hall	300	10
	Others	100	5
EL 50.000	Rest Rm.	300	10
	Central Control Rm.	500	50
	Computer & Relay Rm.	500	50
	Lavatory	200	10
	Waiting Rm.	300	50
	Elevator Hall	300	10
	Chief Operation Rm.	300	10
	Others	100	5
EL 45.500	Air Conditioning Machine Rm.	100	5
	Emergency Generator Rm.	300	10
	Communication equipment Rm.	300	10
	Locker Rm.	200	10
	Shower Rm.	200	10
	Lavatory	200	10
	Entrance Hall	300	10
	Generator Rm.	300	10
	Others	100	5
EL 40.900	Power Source Rm.	300	10
	Spare Rm.	300	10
	Battery Rm.	100	5
	Others	100	5
EL 37.600	Spare Rm.	300	10
	Battery Rm.	100	5
	Sundry Rm.	300	10
	Others	100	5

Rust Protect : The terminals and wires of the lighting panels shall be tin-toasted.

Power source : The source voltage for normal lighting, emergency lighting, convenient outlets and power outlet are : AC 1ϕ 2W 220V : For normal lighting convenient outlet and power outlet and DC 110V: For emergency lighting. AC power and DC power are supplied from 220V, 110V MCC panel and plant DC panel of primary side, respectively.

(b) Pekanbaru Substation Building and Relay House

Lighting illumination level (Lx) of each room

Room	Normal (Lx)	Emergency (Lx)
Toilet	200	10
Cubicle Rm.	200	10
Meeting Rm.	300	50
Spare Rm.	300	10
Control Rm.	500	50
Manager Rm.	300	50
PLC Rm.	300	10
Others	100	5

Power source : The source voltage for normal lighting, emergency lighting, convenient outlets and power outlet are as : AC 1ϕ 2W 220V : For normal lighting convenient outlet and power outlet and DC 110V: For emergency lighting. AC power and DC power are supplied from 220V, 110V MCC panel and plant DC panel of primary side, respectively.

(4) Indoor Fire Protection System (Powerhouse, Pekanbaru Substation Building and Relay House)

This section covers supply and installation of starters, pilot lamps, bells, electrical conduit pipes, smoke detector, heat detector and other necessary accessories for the indoor fire protection system.

Design bases : Power source voltage (Fire protection system DC 110V, Pilot lamps DC 24V, Bell DC 24V, Heat detector DC 24V Smoke detector DC 24V.

System description : The heat detector and the smoke detector mounted on the ceiling. The heat detector senses the temperature rise and the smoke detector senses the smoke density rise. By pushing the starter of the fire alarm devices mounted on the water hydrant box, or by working the heat detector, or by working the smoke detector, the bell will sound, the indication lamp on the alarm panel in the control room will light on, and automatically water hydrant pump will start. The sound level of the bell is more than 90 phones at 1.0m away from the center of devices. Electrical conduit pipes basement floor of Powerhouse shall be separated form the wall for water-proof purpose.

Elevator : Capacity 1,500 kg, Speed 45 m/min, Dimension Floor 2,200 mm x 2,400 mm, Stops 5 floors (including 2 basement fls), Motor AC 15kW 3ϕ 440V 50Hz and Floor landing fl : 2nd floor.

2.2 Detailed Design of Building Structure and Facilities

2.2.1 Powerhouse

(1) General

Powerhouse superstructure houses one big room for generator and several rooms of control blocks. There is a 80 ton overhead crane in the generator room. The steel structure is selected because of that the span of overhead crane is 23.6m between columns. The framing method of transverse direction is selected rigid frame because of the room arrangement and overhead crane. The framing method of longitudinal direction is selected vertical bracing frame because of the seismic resistance capacity.

(2) Framing Plan and Structural Elevation

Framing plans are indicated at each level from the high roof to the second floor (Refer to Fig. 3.2.1. Structural elevations are indicated at each frame from line 1 to 12 and from line A to C (Refer to Fig. 3.2.2.

(3) Design Load

Floor Load for Design : Floor load is calculated from dead load and live load. Dead load is composed of the weight of structural member and the weight of finished materials. Live load is selected from the typical load decided by the purpose of the room. Also, the values of live load are different among the structural members. Moreover, the live load at seismic time is different from the others. The value of floor load is dead load plus live load. The floor load is indicated in Table 3.2.8.

Table 3.2.8 Floor Load for Design

Room name		Slab	Beam	Girder	Column	Seismic	Note
High Roof	DL	475	495	515	515	515	RC+Asphalt Roofing
	LL	90	80	70	70	30	
	TL	565	575	585	585	545	
Low Roof	DL	500	515	540	540	540	Including ceiling
	LL	90	80	70	70	30	
	TL	590	595	610	610	570	
Elevator Machine Rm.	DL	565	580	600	600	600	Excluding Machine load
	LL	180	160	130	130	60	
	TL	745	740	730	730	660	
Office Conference Room EV. Hall, Corridor	DL	475	490	515	515	515	3rd Floor
	LL	300	240	180	180	80	
	TL	775	730	695	695	595	
Central Control Room Computer Room Chief Operation Rm.	DL	435	450	480	480	480	2nd Floor Including Free Access Floor
	LL	400	360	300	300	200	
	TL	835	810	780	780	680	
Waiting Room Meeting Room EV. Hall, Corridor	DL	475	490	520	520	520	2nd Floor
	LL	300	240	180	180	80	
	TL	775	730	700	700	600	
Steel Stair	DL	245	260	280	280	280	Including Terrazzo block for floor finish
	LL	300	240	180	180	80	
	TL	545	500	460	460	360	

Note: DL --- Dead Load, LL --- Live Load, TL --- Total Load (DL + LL)

Seismic load : Seismic load is calculated from floor load at seismic time plus other load (Wi) ans story shear coefficient of earthquake (Ci) at the ith floor. Seismic load at each floor level is indicated in Table 3.2.9.

Table 3.2.9(1) Seismic Load

Transverse Direction

Floor	Wi (t)	Σ Wi (t)	α i	Ai	Ci	Qi
High Roof	164.4	164.4	0.295	1.635	0.164	27.0
Low Roof	114.2	278.6	0.500	1.376	0.138	38.4
3rd Floor	142.6	421.2	0.757	1.161	0.116	48.9
2nd Floor	135.8	557.0	1.000	1.000	0.100	55.7

Note : This value is a horizontal force loaded to one frame.

Table 3.2.9(2) Seismic Load

Longitudinal Direction

Floor	Line	Wi (t)	Σ Wi (t)	α i	Ai	Ci	Qi
High Roof	A	-	-	-	-	-	-
	B	761.3	761.3	0.266	1.688	0.169	128.7
	C	681.0	681.0	0.524	1.352	0.135	91.9
Low Roof	A	452.0	452.0	0.272	1.676	0.168	75.9
	B	652.0	1,413.3	0.493	1.383	0.138	195.0
	C	172.1	853.1	0.657	1.237	0.124	105.8
3rd Floor	A	565.5	1,017.5	0.613	1.273	0.127	129.2
	B	713.9	2,127.2	0.742	1.172	0.117	248.9
	C	211.1	1,064.2	0.819	1.118	0.112	119.2
2nd Floor	A	642.8	1,660.3	1.000	1.000	0.100	166.0
	B	738.0	2,865.2	1.000	1.000	0.100	286.5
	C	234.5	1,298.7	1.000	1.000	0.100	129.9

Note : This value is a horizontal force loaded to each frame.

(4) Design of Overhead Crane Supporting Girder

The supporting girder is designed from the maximum vertical wheel load of overhead crane at full load. Also, the stress of girder is calculated in consideration of the position of the overhead crane in order that the stress becomes maximum one. Moreover, the horizontal wheel load is considered to design the girder. The load of overhead crane at seismic time is only the weight of crane itself without the lifting weight.

Maximum wheel load	:	30.5 ton at full load
Minimum wheel load	:	12.8 ton at full load
Number of wheel	:	4 (One side), 8 (Total)
Distance of wheel	:	1.2m + 2.9m + 1.2m
Member of girder	:	BH-1000 x 300 x 16 x 28 (Built-up H shape)

(5) Results of Stress Analysis

(a) Frame stress at each case (Refer to Fig. 3.2.3)

Case 1 : Vertical load
Case 2 : Crane vertical load at full load (Lifting weight near B line)
Case 3 : Crane vertical load at full load (Lifting weight near C line)
Case 4 : Seismic load
Case 5 : Crane seismic load
Case 6 : Wind load (Wind direction from A to C line)
Case 7 : Wind load (Wind direction from C to A line)
Case 8 : Crane vertical load without lifting weight)

(b) Combination of stress

The stress for each structural member is considered as follows. The stress at long term is combination 1 in consideration of crane full load. The stress at seismic conditions is combination 2 in consideration of crane load without lifting weight. The stress at wind conditions is combination 3 in consideration of crane load without lifting weight.

Combination 1	:	Case 1 + 2 or 3 (Long term)
Combination 2	:	Case 1 + 4 + 5 + 8 (Short term)
Combination 3	:	Case 1 + 6 or 7 + 8 (Short term)

(6) Calculation of Each Structural Member

(a) Column at B and C line

Above crane level		Below crane level
Member	BH-500 x 400 x 16 x 28	BH-1000 x 400 x 19 x 28
Stress	Ml = 109.1tm, Ql = 23.5t, Nl = 85.3t	Ml = 150.2tm, Ql = 33.3t, Nl = 219.9t
Stress ratio	0.96	0.74

Note : Subscript l = long term, s = short term

(b) Girder

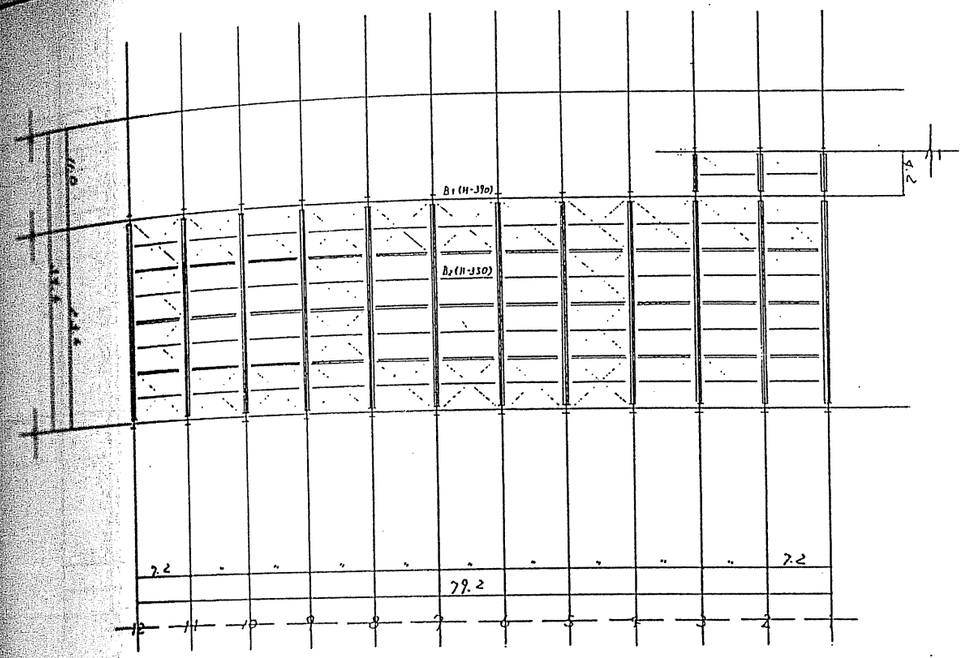
Roof truss			Low roof girder	3 F girder	2F girder
Main Member	:	H-300x300x10x15	H-700x300x13x24	H-700x300x13x24	H-700x300x13x24
Stress	:	Ml=246.6tm, Ql=43.8t	Ml=54.4tm, Ql=1.3t	Ms=124.6tm, Qs=42.7t	Ms=117.3tm, Qs=41.8
Stress ratio	:	0.82	0.69	0.90	0.85
Diagonal Member	:	2L-90 x 90 x 13			
Stress	:	Nl = 60.1t			
Stress ratio	:	0.99			
Vertical Member	:	2L-75 x 75 x 12			
Stress	:	Nl = 45.4t			
Stress ratio	:	0.94			

(c) Vertical Brace at B frame

Roof vertical brace			3F vertical brace	2F vertical brace	Ground floor vertical brace
Member	:	2L-75x75x12	2L-100x100x13	2L-130x130x12	2L-130x130x13
Stress	:	N=56.9t	N=85.8t	N=248.9t	N=286.5t
Stress ratio	:	0.64	0.84	0.95	0.87

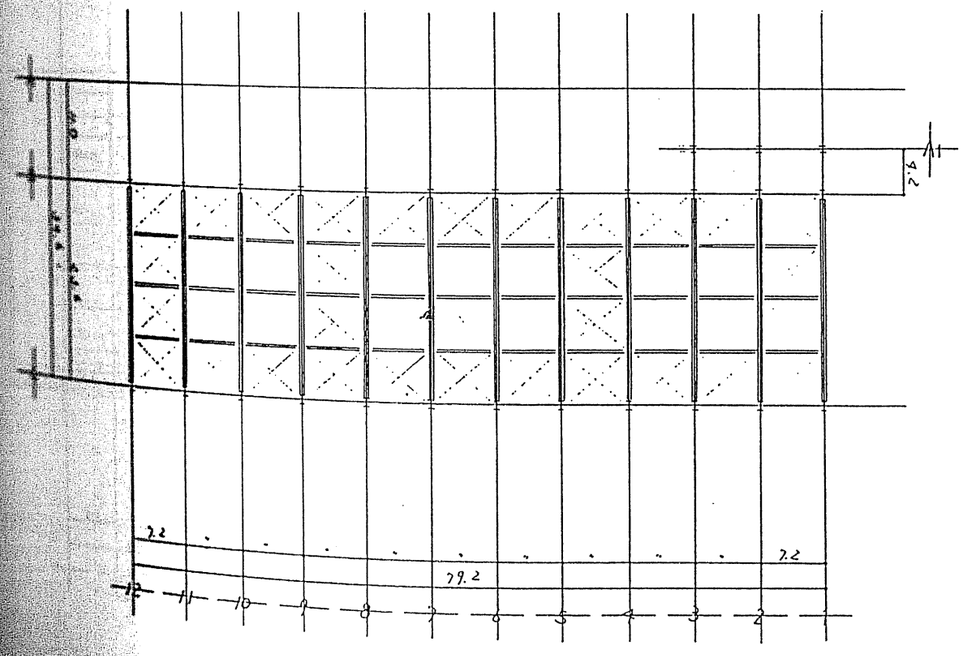
(7) Results of Deformation

The horizontal deformation is calculated at seismic time in transverse direction and calculated relative deformation at each level. The deformation/each floor height is Roof roof 1.96cm/450cm = 1/230, Low roof 1.47cm/443cm = 1/301, 3rd floor 1.58cm/45 = 1/285 and 2nd floor 0.55cm/422cm = 1/767. Therefore the relative deformation is not more than 1/200.



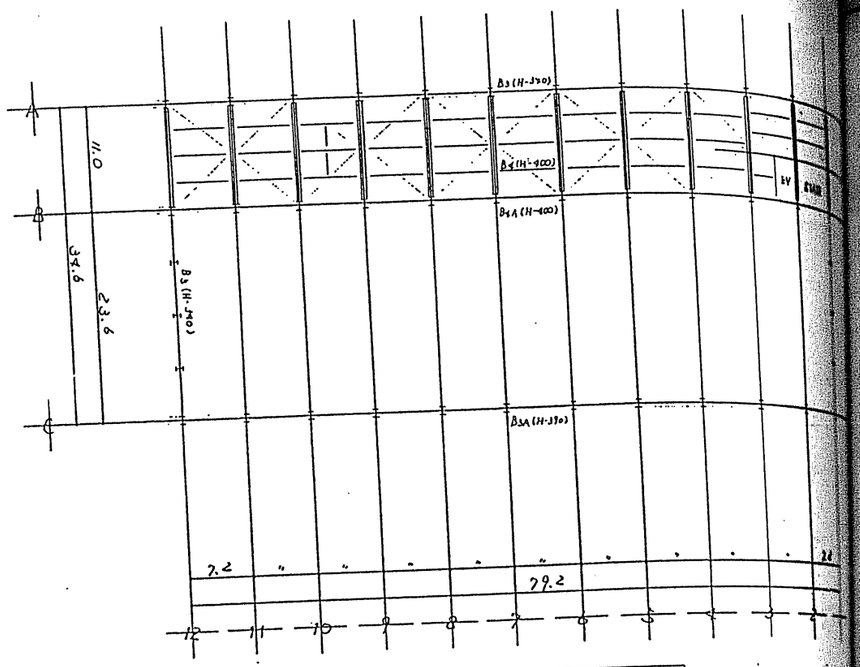
High Roof Plan T/S = EL.+63.200 (Truss : Top Chord)

Fig. 3.2.1(1) Powerhouse Framing Plan



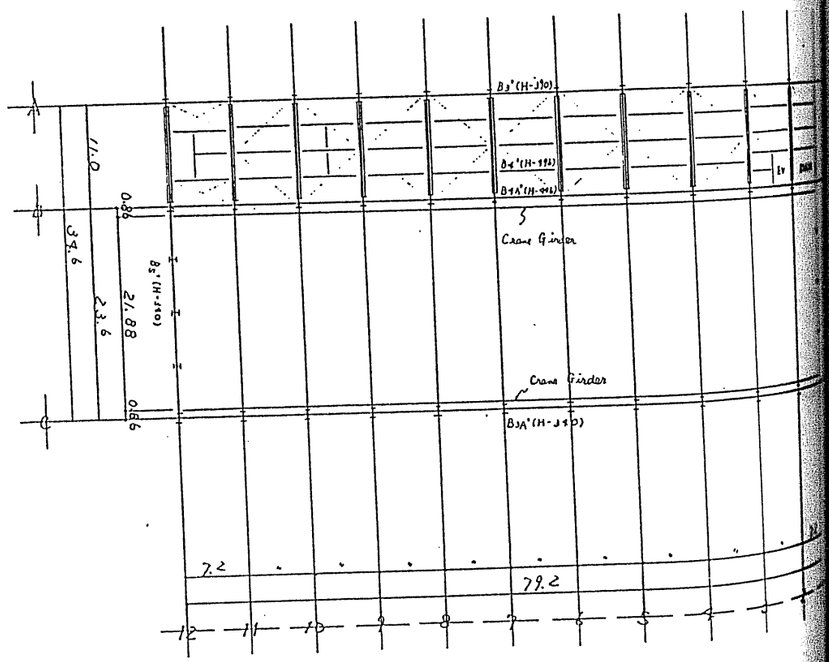
High Roof Plan (Truss : Bott. Chord)

Fig. 3.2.1(2) Powerhouse Framing Plan



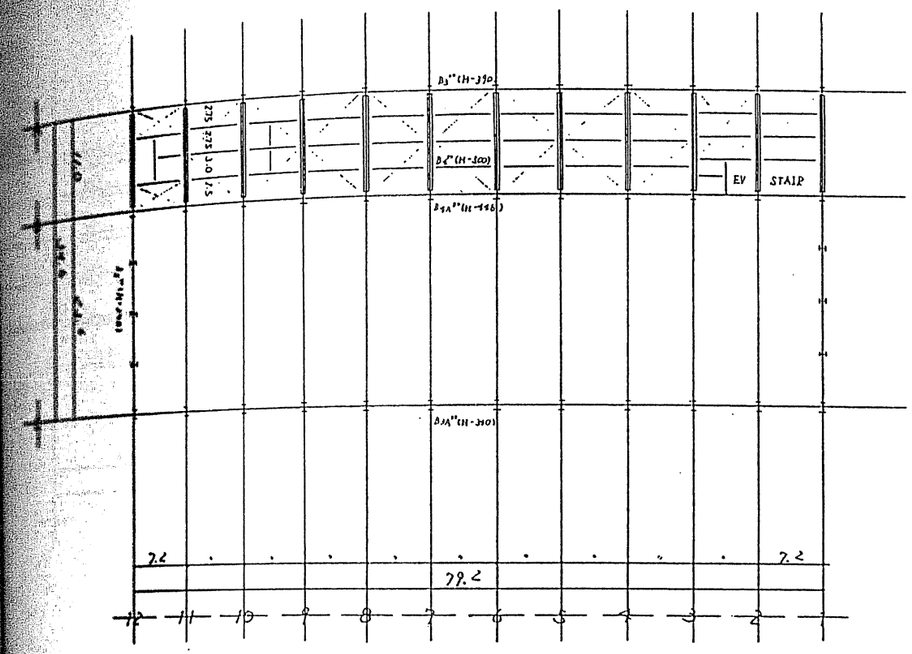
Low Roof Plan T/S = EL..+58.7

Fig. 3.2.1(3) Powerhouse Framing Plan



3rd Floor Plan T/S = EL.+54.27

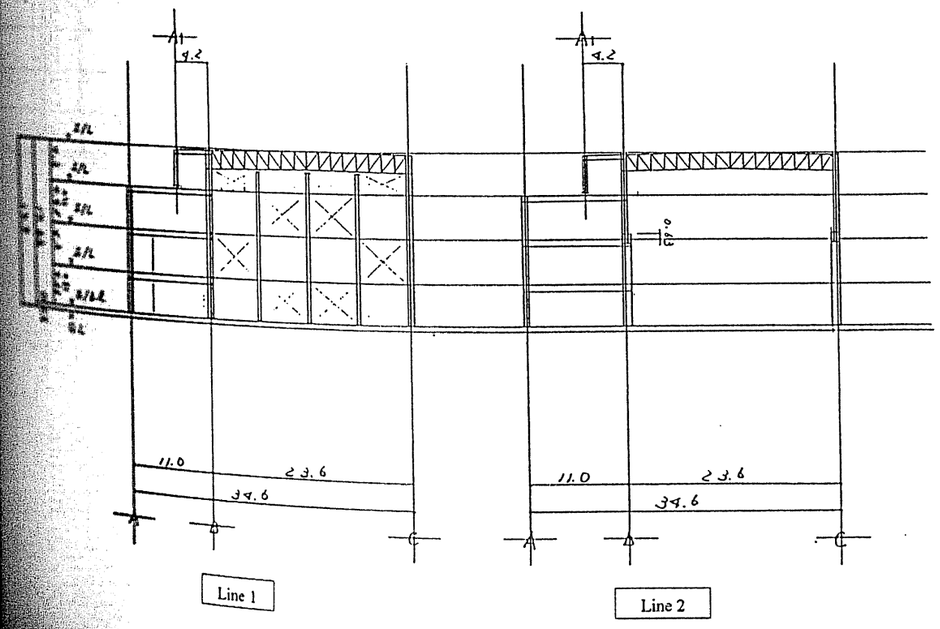
Fig. 3.2.1(4) Powerhouse Framing Plan



2nd Floor Plan T/S = EL.+49.36 , EL.+49.77

Line 1

Fig. 3.2.1(5) Powerhouse Framing Plan



Line 1

Line 2

Fig. 3.2.2(1) Structural Elevation

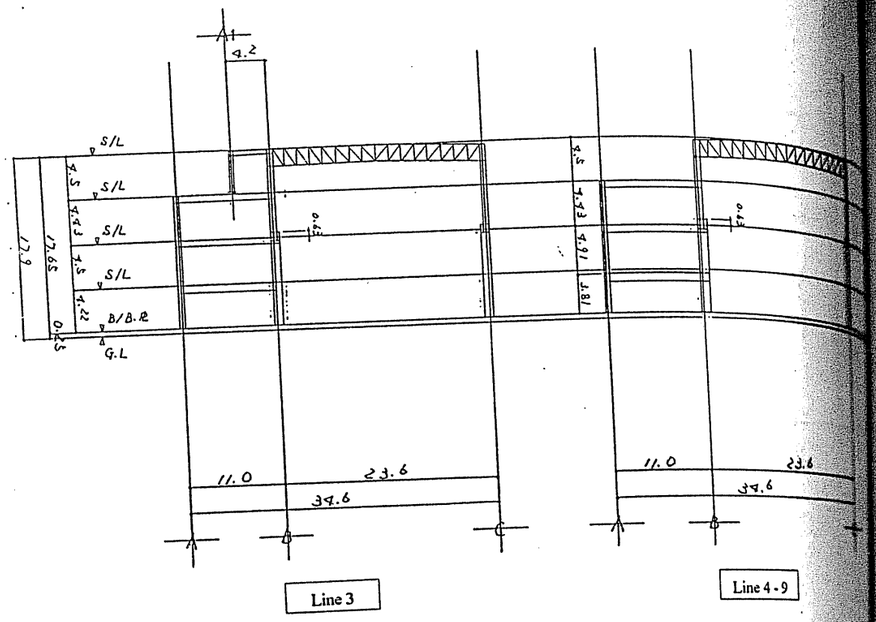


Fig. 3.2.2(2) Structural Elevation

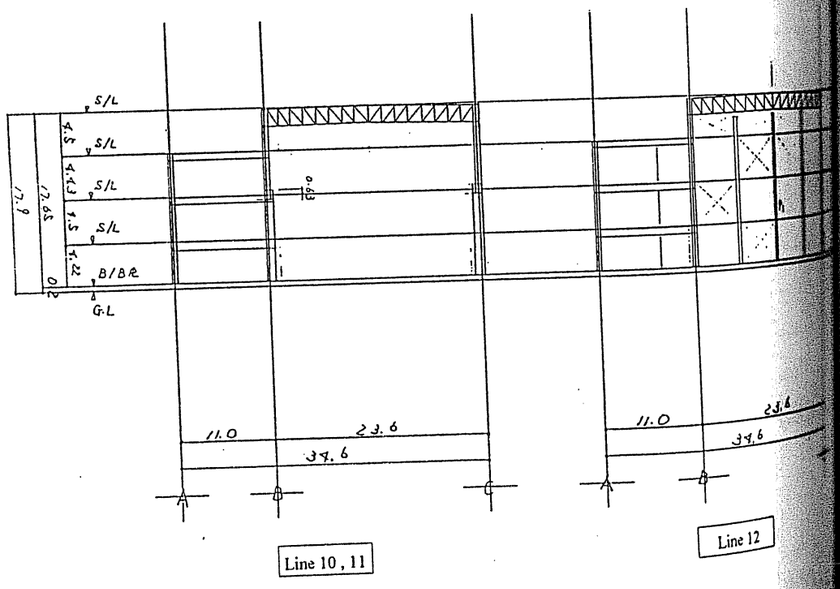


Fig. 3.2.2(3) Structural Elevation

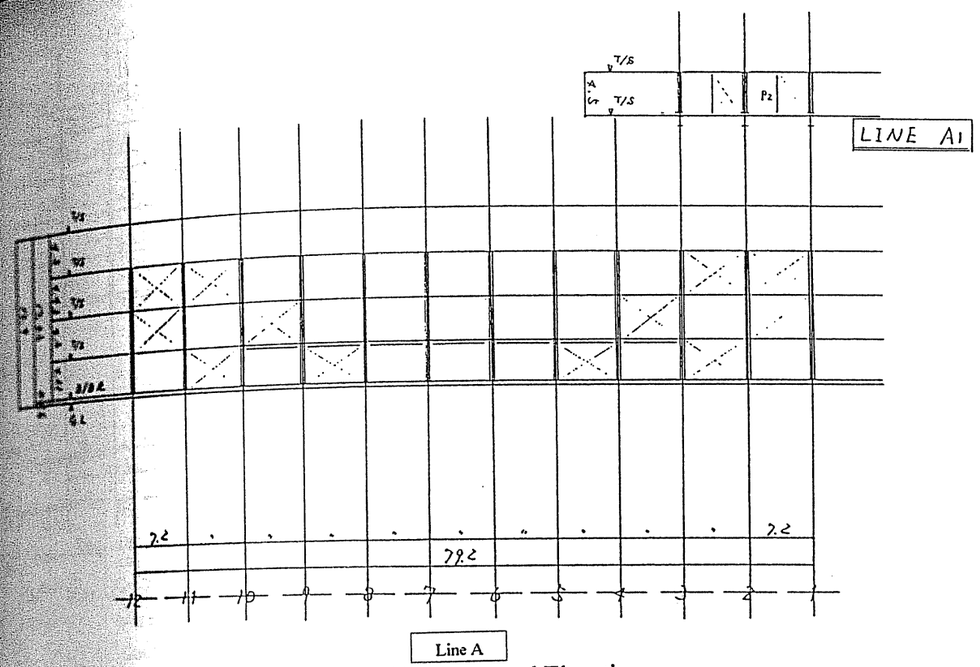


Fig. 3.2.2(4) Structural Elevation

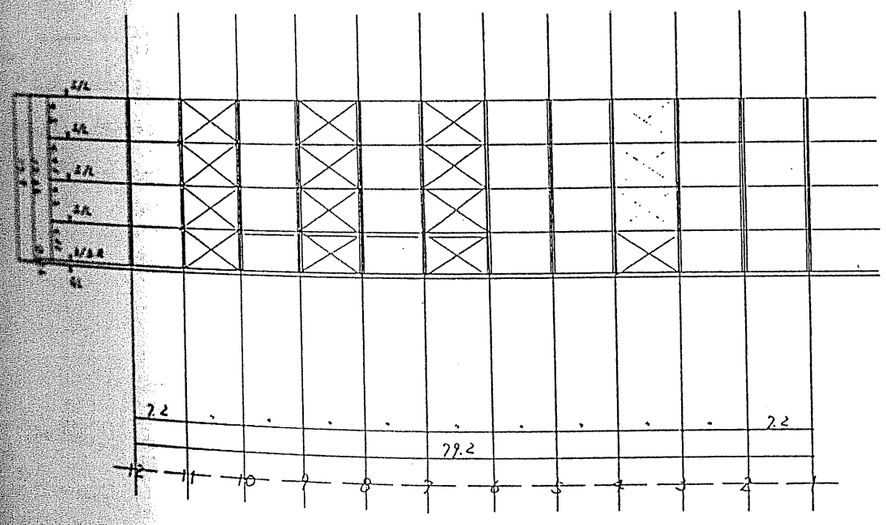


Fig. 3.2.2(5) Structural Elevation

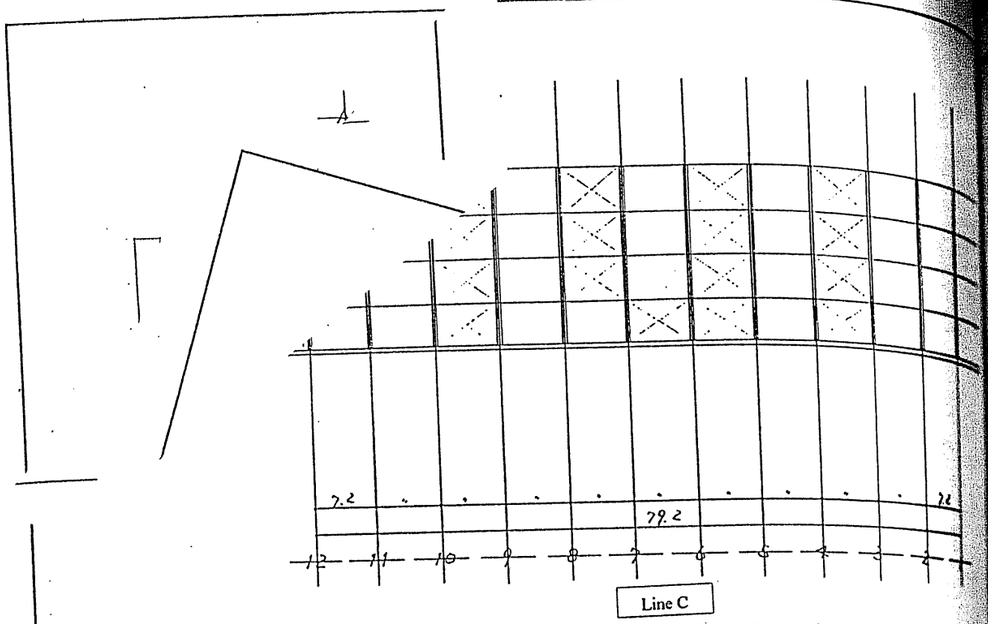
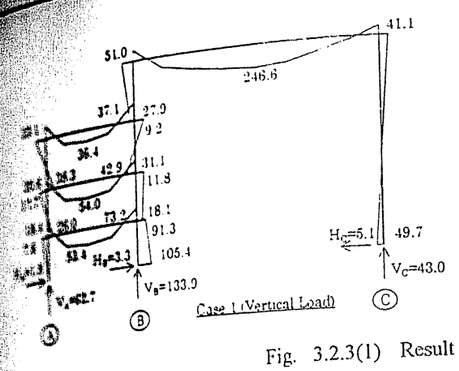


Fig. 3.2.2(6) Structural Elevation



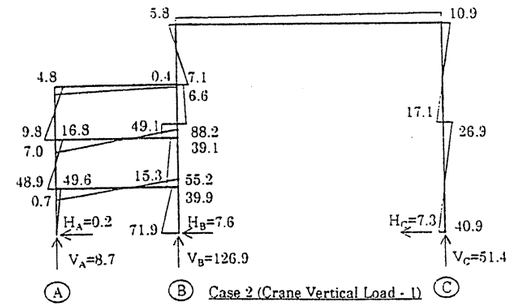
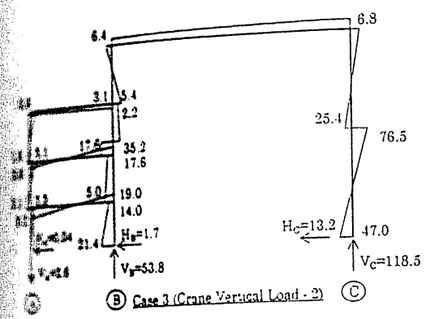


Fig. 3.2.3(1) Result of Powerhouse Stress Analysis



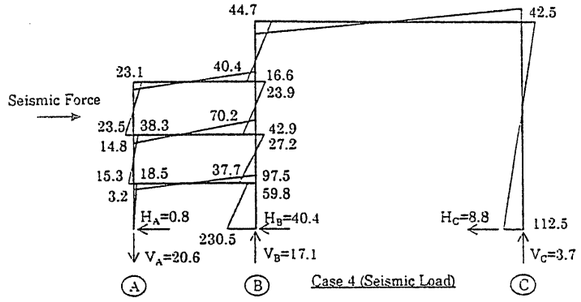
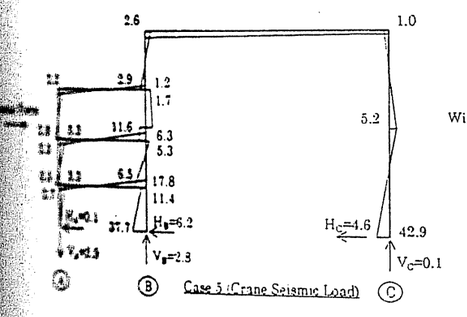


Fig. 3.2.3(2) Result of Powerhouse Stress Analysis



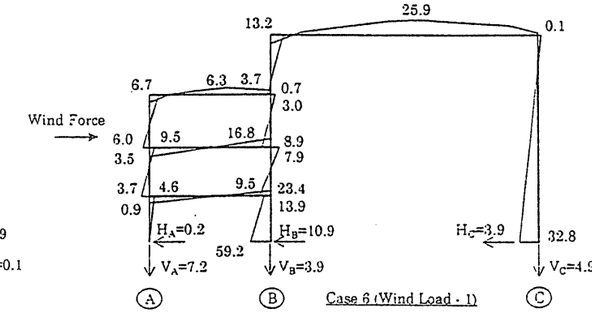
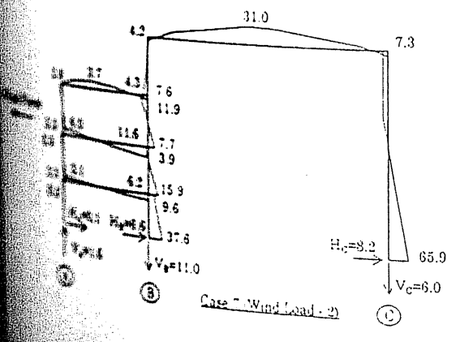


Fig. 3.2.3(3) Result of Powerhouse Stress Analysis



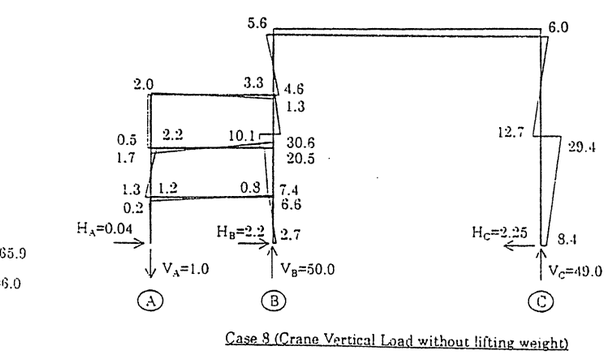


Fig. 3.2.3(4) Result of Powerhouse Stress Analysis

2.2.2 Switchyard Relay House

(1) General

Switchyard relay house houses relay panel, 20kV cubicle, PLC panel and battery. Also, lavatory is provided in this building. Maximum ceiling height is 3m. Then, the reinforced concrete structure is selected for this building. The cable duct is provided below the cubicle. The framing method of this building is rigid frame without seismic bearing wall. The bearing capacity of the soil is 10 ton per square meter and the direct foundation system is selected. The floor load for ground floor is supported by the soil directly.

(2) Framing Plan and Structural Elevation.

Framing plans are indicated at each level including foundation level (Refer to Fig. 3.2.4). Structural elevations are indicated at each frame (Refer to Fig. 3.2.5).

(3) Design Load

Floor Load for Design : Dead Load 620kg/m^2) and Live Load 60kg/m^2 for frame, 30kg/m^2 at seismic time) and Total Floor Load 680kg/m^2 for frame, 650kg/m^2 for frame).

Seismic : The calculation method is same as that of Powerhouse. The result of the horizontal seismic load is 29.58t at roof level.

(4) Results of Stress Analysis and Calculation of Each Structural Member

Results of stress analysis is shown in Fig. 3.2.6.

Column		Girder	Slab (thickness 15cm)	Beam
Size	: 45m x 45cm	35cm x 60cm (B line)		25cm x 50cm
Stress	: NI=25t, MI=8.5tm, QI=3.3t	MI=14.5tm, QI=10.5t		M=5.8tm, Q=3
Main Reinforcing bar	: 8-D22	4-D22 and 3-D22	D10 and D13@200 (Double at each direction)	4-D22 and 3-D22

Foundation	
Size	: 2.8m x 2.8m x 0.45m (Depth)
Stress	: N=55t(8t/m2)
Main Reinforcing bar	: D16 @200

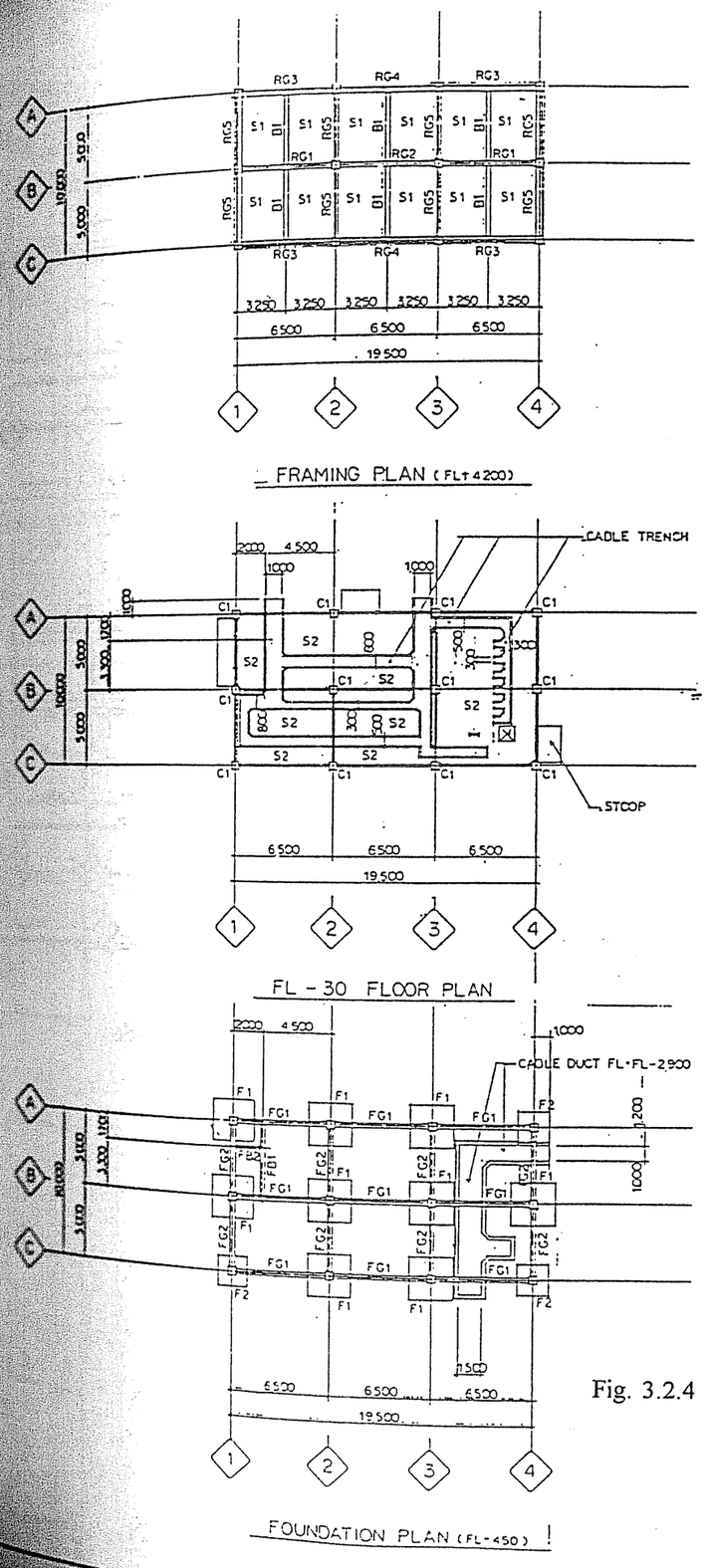


Fig. 3.2.4 Switchyard Relayhouse Framing Plan

2.2.3 Pekanbaru Substation Building

(1) General

Pekanbaru substation building houses transformer and 20kV cubicle at ground floor, its control devices, PLC panel and battery at second floor. Also, manager room, meeting room and lavatory is provided in this building. The reinforced concrete structure is selected for this building. The cable duct is provided below the cubicle. The two story is selected in consideration of the room arrangement. The framing method of this building is rigid frame without seismic bearing wall. The PC pile foundation is selected because that the bearing capacity of the soil is not enough for the weight of the building. The floor load for ground floor is supported by the piles through foundation girders.

(2) Flaming Plan and Structural Elevation

Framing plans are indicated at each level including foundation level (Refer to Fig. 3.2.7). Structural elevations are indicated at each frame (Refer to Fig. 3.2.8).

(3) Design Load

Floor Load for Design : Floor load is calculated from dead and live load. Dead load is composed of the weight of structural member and the weight of finished materials. Live load is selected from the typical load decided by the purpose of the room. Also, the value of live are different among the structural members. The value load is dead load plus live load. The floor load is indicated in Table 3.2.10.

Seismic Load : The calculation method is same as that of Powerhouse. The result of the horizontal seismic load is 76.62t at roof level and 176.33t at 2nd floor level.

(4) Results of Stress Analysis and Calculation of Each Structural Member

Results of stress analysis is shown in Fig. 3.2.9.

Column		2nd Floor	Ground Floor
Size	:	55m x 55cm	55cm x 55cm
Stress	:	Ns=28.9t, Ms=14.3tm, Qs=5.1t	Ns=79.1tm, Ms=20.3tm, Qs=4
Main Reinforcing bar	:	8-D22	8-D22

Roof		Grider	2nd Floor	Ground Floor
Size	:	35cm x 65cm (B line)	45cm x 85cm (C line)	45cm x 10
Stress	:	Ml=16.5t, Ql=12.5t	Ml=39.4tm, Ql=30.4t	Ml=56.0tm, Q
Main Reinforcing bar	:	5-D22 and 3-D22	8-D22 and 5-D22	7-D25 and 4-D

Roof (Slab)		2nd Floor Control room and Ground Floor (Slab)
Thickness	: 15cm	18cm
Main Reinforcing bar	: D10 and D13 @200	D13 @200

Roof (Beam)		2nd Floor Control room	Ground Floor (Beam)	Foundation (F-6)
Size	: 35cm x 60cm	40cm x 65cm	40cm x 85cm	3.0m x 3.0m
Stress	: Ml=11.8tm, Ql=8.4t	Ml=25.0tm, Ql=16.4t	M=43.2tm, Q=27.3t	N=243.7t
Main Reinforcing bar	: 5-D22 and 3-D22	7-D22 and 4-D22	7-D25 and 5-D25	D22 @150

Table 3.2.10 Floor Load for Design

Room name		Slab	Beam	Girder	Column	Seismic	Note
Roof	DL	620	620	620	620	620	RC+Asphalt Roofing
	LL	60	60	60	60	30	
	TL	680	680	680	680	650	
Control Room	DL	1,130	1,130	1,130	1,130	1,130	Including lean concrete
	LL	500	500	500	500	350	
	TL	1,630	1,630	1,630	1,630	1,480	
Praying Room	DL	420	420	420	420	420	
	LL	300	300	180	180	80	
	TL	720	720	600	600	500	

Note: DL --- Dead Load, LL --- Live Load, TL --- Total Load (DL + LL)

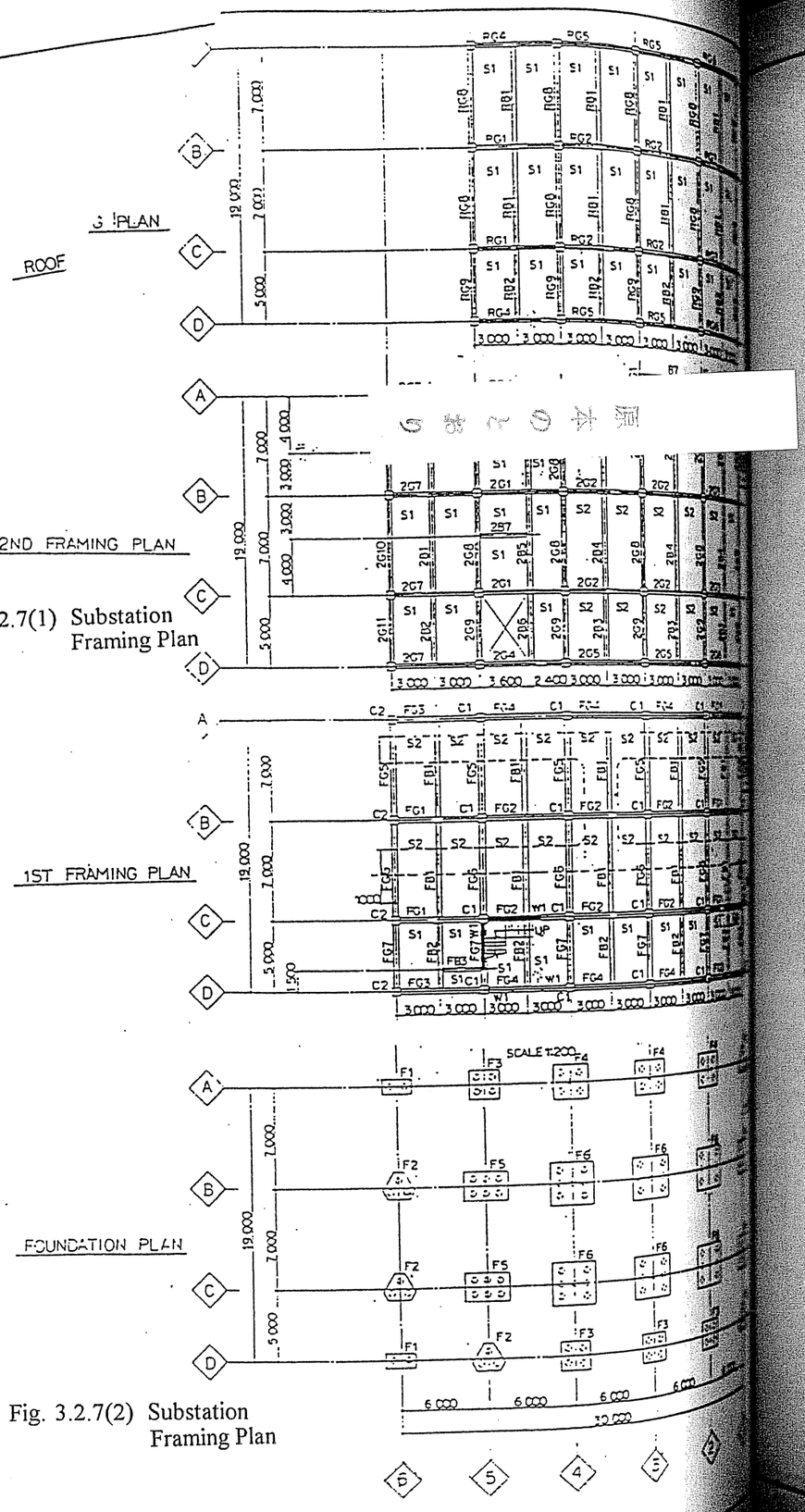


Fig. 3.2.7(1) Substation Framing Plan

Fig. 3.2.7(2) Substation Framing Plan

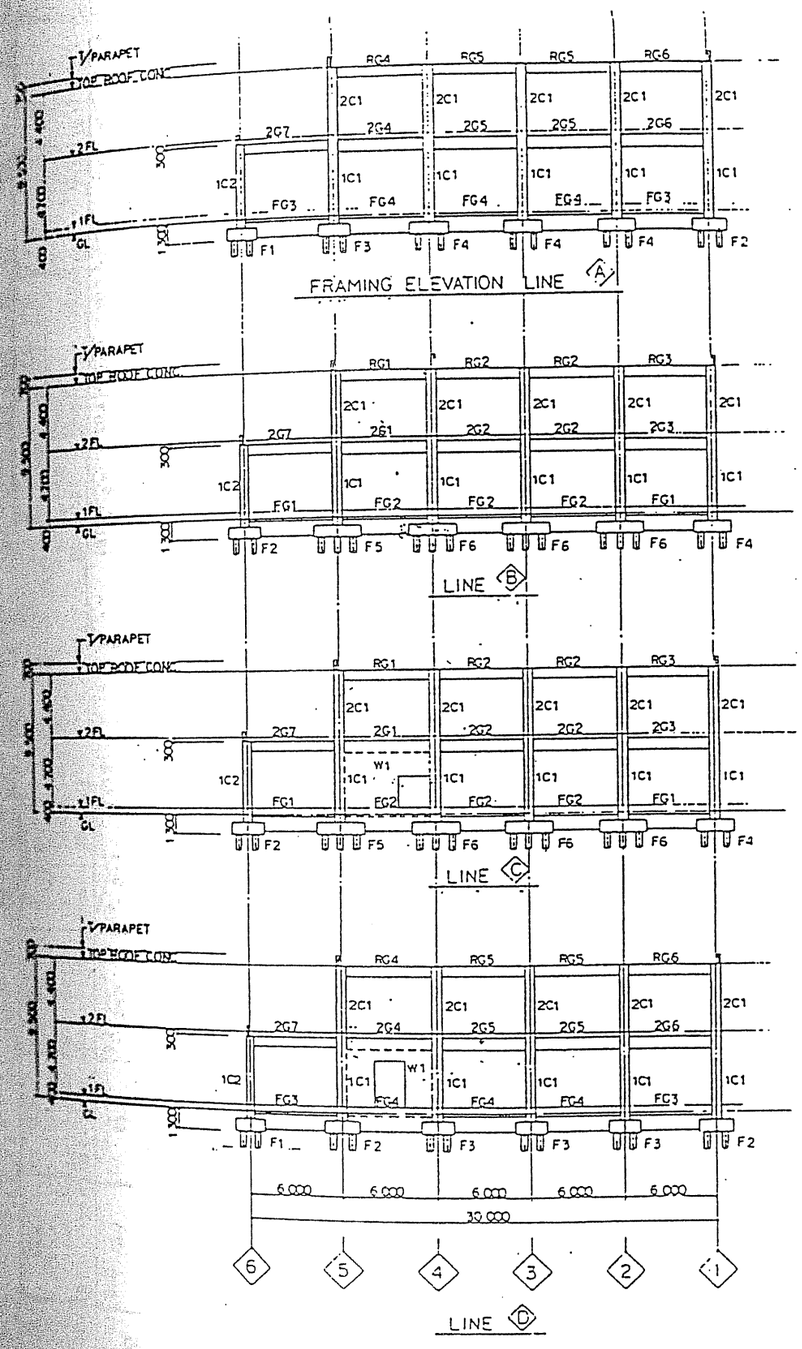


Fig. 3.2.8(1) Substation Structural Elevation

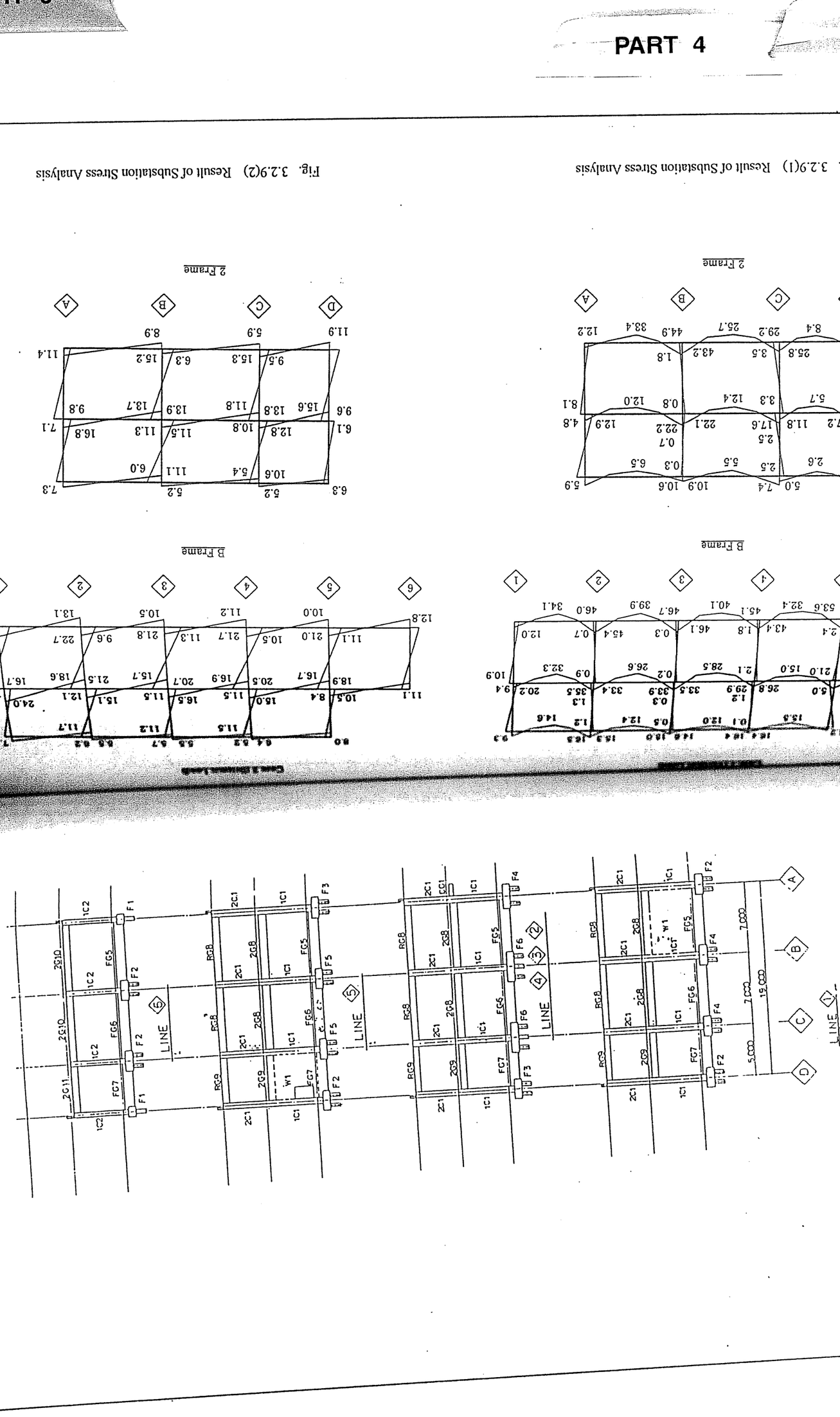


Fig. 3.2.8(2) Substation Structural Elevation

Fig. 3.2.9(1) Result of Substation Stress Analysis

Fig. 3.2.9(2) Result of Substation Stress Analysis

2.3 Detailed Design of Building Facilities

2.3.1 Powerhouse

(1) Plumbing System

The treated water and fire hydrant water are distributed from Water supply facility house on EL. 63200, to each demand point by gravity flow.

Design conditions of water supply system

Population requires water supply	:	92 numbers
Design Supply Water Quantity per day (Qd)	:	11,000 litter/day
Average per Hour (Qh)	:	1380 litter/hr
Total treated water quantity	:	69 litter/min
- Elevated water tank	:	90 litter/min
- Cooling water (Lot IIIB)	:	159 litter/min
- Sub total		
Fire hydrant system water quantity	:	1,300 litter/min
- Fire drencher	:	340 litter/min
- Fire hydrant system	:	10.0mm Aq
Water supply pressure		

Water dispenser : Water dispensers are installed in the Powerhouse.

Chemical feed pump : A chemical feed pump for discharge the sterilization chemi... located at EL. 63,200. The quantity of discharge is in proportion to water demand.

Pressure pump : A pressure pump unit is installed in order to stand by for emerge... Because after pump house, elevated water tank is located on the top of Powerhou...

Cooling water pipe : The cooling water pipe of Electrical Power plant system h... from water supply main pipe in Powerhouse after the connection with main pipe... water supply facility.

Water hydrant : Water hydrant boxes are installed in suitable place in Powerhou... piping comes from Water supply facility as exclusive piping for Hydrant system. ... cock located on roof. Water supply pressure of water hydrant system is always pro... because of gravity pressure. Thus when use water hydrant, it is simply operating ... valve in Box.

Hot water supply : Electrical hot water heater is instant type, and to supply hot w... Shower rooms. The temperature of hot water is controlled automatically by therm... installed in the machine.

Air-Conditioning System

Air conditioning system of Power house is consisted with several Air souse cooling Packaged air conditioners these are located in machine room and above ceiling. These are intended to provide favorable working conditioning for mankind and electrical machines. Design conditions and required cooling capacities are as following tables.

GROUP "A" (GROUND FLOOR FOR AC-1)

ROOM NAME	CALCULATED SENSIBLE LOAD	ESTIMATED SENSIBLE LOAD	%	ESTIMATED TOTAL LOAD	AIR QUANTITY
• Spare Room	2,309	2,540	15.3	3,730	1,100
• Communication Equipment Room	4,728	5,200	32.3	6,260	2,330
• Locker Room (M)	3,349	3,680	22.2	6,510	1,600
• Locker Room (W)	517	570	3.4	1,360	260
• Guard Room	445	470	3.0	800	250
• Entrance	3,729	4,100	28.7	9,180	1,660
TOTAL	15,007	16,580	100	27,840	7,200

SHF = 0.59

GROUP "B" 12ND FLOOR FOR AC-2)

ROOM NAME	CALCULATED SENSIBLE LOAD	ESTIMATED SENSIBLE LOAD	%	ESTIMATED TOTAL LOAD	AIR QUANTITY
• Central Control Room	20,551	22,600	87	30,700	
• Chief Operator Room	3,008	3,300	13	5,840	
TOTAL	23,559	25,900	100	36,540	

SHF = 0.708

GROUP "C" 12ND FLOOR FOR AC-3 COMPUTER & RELAY ROOM)

ROOM NAME	CALCULATED SENSIBLE LOAD	ESTIMATED SENSIBLE LOAD	%	ESTIMATED TOTAL LOAD	AIR QUANTITY
• Computer and Relay Room	23,395	25,700		32,300	12,600

GROUP "D" (FOR 2nd FLOOR FOR AC-4)

ROOM NAME	CALCULATED SENSIBLE LOAD	ESTIMATED SENSIBLE LOAD	%	ESTIMATED TOTAL LOAD	AIR QUANTITY
▸ Waiting Room	3,893	4,280	44.2	6,710	
▸ Kitchenette	207	230	2.3	230	
▸ EV. Hall	1,159	1,270	13.3	1,270	
▸ Meeting Room	3,546	3,900	40.4	9,560	
TOTAL	8,805	9,680	100	17,770	

GROUP "E" (FOR 3rd FLOOR FOR AC-5)

ROOM NAME	CALCULATED SENSIBLE LOAD	ESTIMATED SENSIBLE LOAD	%	ESTIMATED TOTAL LOAD	AIR QUANTITY
▸ Reception Room	4,052	4,460	8.2	7,310	
▸ Superintendent Room	4,714	5,190	9.6	8,500	
▸ Office Room	19,003	20,900	38.8	34,330	
▸ Conference Room	12,358	13,800	25.5	31,270	
▸ EV. Hall	3,817	4,200	7.8	4,200	
▸ Kitchenette	640	700	1.3	700	
▸ Prayer Room	4,357	4,790	8.8	9,080	
TOTAL	48,941	54,040	100	95,390	

SHF = 0.57

5) STATIC PRESSURE OF ROOM FANS

		"A"	"C"	"B" "D" AND "C"	FOR STAIR
(1)	FRICTION LOSSES STRAIGHT DUCT	600 x 0.2 = 18.0	155 x 0.1 = 15.5	12 x 0.2 = 2.4	30 x 0.2 = 6.0
(2)	ELBOW LOSSES	0.6 x 9 x 6 x 0.2 = 6.5	1.0 x 9 x 12 x 0.1 = 10.8	0.3 x 9 x 1 x 0.2 = 0.5	1 x 9 x 1 = 3.6
(3)	AREA CHANGES LOSSES	0.000166 x 7200 = 2.0	0.000166 x 12600 = 2.1	0.000166 x 4800 = 0.8	0.000166 x 19800 = 3.3
(4)	DAMPERS LOSS	2.5	3 x 2.5 = 7.5	2.5	2 x 2.5 = 5.0
(5)	OUTLET INTAKE	4.2	4.2	4.2	4.2
	TOTAL	33.2	40.1	10.4	22.1

(3) Ventilating System

Main Supply fan is installed on the roof, and sub supply fans are installed at EL 40,700 beside the wall on the side of Tailrace. Air Filters are installed in main ventilating root and sub ventilating system too. The intention of Ventilating is principally decrease inside temperature. Air quantity of ventilating fan shall be calculated as follows.

Room	Method of Ventilation	Air charges per hour
Cubicle room	By supply and exhaust fans	Air volume of ventilation is calculated by a formula
Generator room (Including Turbine room, Garry and Auxiliary equipment room)	ditto	5

In case of Powerhouse too much volume of ventilating, it is not preference mater. Because during day time, the temperature of outside air often be higher than basement level (EL 26,000 - 41,000) especially day time. This Ventilation system is intended to use the low temperature air of lower level, so it is not more presence that air volume is more bigger air than necessaries quantity because the outside air temperature is higher than inside air.

Air quantity of Main ventilating fan

SF-1 : Design heat gain is 180kW, required air quantity to Maintain inside positive pressure 10% are exceed from exhaust air quantity. 96,800 m^3/h, 35mmAq for Multi-Blade Double Suction fan.

SF-2 : Required Air Quantity total required Air quantity same as Main root 96,800 m^3/h, devid into 3 then 33,000 m^3/h, 30mmAq.

2.3.2 Relayhouse in Switchyard

(1) Plumbing Work

Water Supply System has two ways. One is elevated water tank system by gravity, the other is the pressure pump unit system in case of electrical supply failed and for Energy conservation. The normal use, the pressure of water will get enough only by gravity except for using a water cock in premise. In case of altering water supply system, valves are operated by manual.

Septic tank is installed in premise, and it is modified as no blower pump for air ration, made of F.R.P. (Fiber reinforced Plastic). The capacity is 5 persons.

(2) Air-Conditioning and Ventilating System

Air-conditioning work : Individual Air-conditioning machines are installed instead of Multiple packaged type. The Air-conditioning machine will run automatically by Weekly timer or will run by manual operation.

Ventilating system : Ventilating system of Switchyard relay house is individual ventil
system that is for adequate quantity of fresh air for maintain room temperature or cle
dirty air. All of ventilating machine will run by manual operation.

2.4 Detailed Design of Building Electrical Facilities

2.4.1 Powerhouse Building

(1) Indoor Lighting System

The Indoor Lighting System covers supply and installation of lighting fixtures, l
panels, switch, receptacles, electrical conduit pipes, wires and other necessary acces

(a) Distribution Panel Plan

Distribution Panels for the lighting system and Ventilation system is indicated eac
from EL. 31,800 to EL. 58,660 (Refer to Fig. 3.2.10). The panels were fabricated
steel plate with a thickness of at least 1.6mm for the body and 2.3mm for the pane
The rate of branch breaker is mainly used MCB 1 pole 50AF 15AT for the
fixtures and power outlets. The loads of each branch are balanced among R, S
phases.

(b) Main Line Plan

Power source : Main Line Plans are indicated at each level from the high vol
basement floor (Refer to Fig. 3.2.9). The normal circuits are supplied from
COMMON MCC No. 1 and No. 2 through the panel of PCOP and LCOP. The em
circuits are supplied from the Generator through the Panel of Lot IIIA Contai
No. 1 and CVCF through the Panel of Lot IIIB 220V SAFE AC DISTRIB
PANEL.

The Power Source Voltage : The power source voltage of the normal circui
phase 4 wires 380V/220V50Hz. The power Source Voltages of the emergency
are AC 1 phase 2 wires 220V by CVCF and 3 phase wires 380V/220V by Ge

(c) Lighting Fixture Plan

Main Illumination Levels : Control Room 500 lux, Office 300 lux, Generator
lux, Cubicle Room 200 lux and Storage Room 100 lux.

Fluorescent Lamps : Standard fluorescent lamps were provided for the control
all other rooms and designed to be manually made active by means of switc
entrance of each room.

Discharge lamps : Large wattage discharge lamps were provided in the Ge

Emergency Lighting : The emergency lighting is used incandescent lamp of 25W, 40W or 60W rating. The lamps for emergency lighting in the control room, corridor, office and all electrical rooms were automatically switched "ON" in the case of AC failure.

Exit Sign Lamps : The exit sign lights were mounted above all exits of the building.

Lighting fixtures for battery room : Acid-proof type.

(d) Equipment and Materials

Wiring materials : The wire of type-NYA is used lighting fixture, switch and receptacle circuit. It is allowable Max. Temp. Of conductor is 70 degree centigrade. The cable of type-N2XY is used for main line. It is allowable Max. Temp. of conductor is 90 degree centigrade. This specification is same as CV cable. The metal conduit and fittings are heavy gauge type.

Receptacle and Switches : For save energy, each rooms or areas are able to turn on and off its lighting fixtures by using 1-way, 3-way and 4-way switches (Refer to Fig. 3.2.20).

(2) Indoor Fire Protection System

The Fire Protection System covers the supply and installation of starters, location lamps, bells, electrical conduit pipes, smoke detectors, heat detectors and other necessary materials.

(a) Fire Alarm Control Panel

The layout, electrical characteristics, types, the number of warning lamps and the system operation of the control panel submitted to the Engineer for approval (refer to Fig. 3.2.21).

The Alarm Control Panel material : The fire alarm control panel is fabricated from steel plate with a thickness of at least 1.2mm for the body and 1.6mm for the door.

The Number of Zones and class : The number o zones is 30 and class 1.

(b) Indoor Fire Protection System Diagram

(c) Equipment and materials

Wires and Conduit : The wires are used Hp cables protected by heavy gauge conduit.

Detectors : The smoke and heat detectors are confirmed its performance by maker's test.

2.4.2 Switchyard Relay House and Pekanbaru Substation Building

(1) Indoor Lighting System

Lighting Panel Plan : (Ordinary Ac 3 phase 4 wires 380V/220V50Hz, Emergency Ac [illegible]. These are distributed from Powerhouse.

Main Line Plan : The main line for the lighting panel is distributed from Powerhouse.

Lighting Fixture Plan

i) Main Illumination Levels : Relay Room (500 lux), Cubicle room (200 lux), [illegible] Room (300 lux) and Storage room (100 lux).

ii) Fluorescent Lamps : Standard fluorescent lamps were provided for all rooms are designed to by manually made active by means of switches at the entrance of each room.

iii) Emergency Lighting : The emergency lighting is used incandescent lamp of 25W, 40W or 60W rating. The lamps for emergency lighting were automatically switches "ON" in the case of AC failure.

iv) Exit Sign Lamps : The exit sign lights were mounted above all main exits of the building.

v) Lighting fixtures for battery room : Acid-proof type.

Equipment and materials

i) Wiring Materials : The wire of type-NYA is used for lighting fixture, switch and receptacle circuit. It is allowable Max. Temp. Of conductor is 70 degree centigrade. The metal conduit and fittings are heavy gauge type.

ii) Receptacle and Switches : For save energy, each rooms or areas are able to turn on and off its lighting fixtures by using 1-way and 3-way switched.

(2) Indoor Fire Protection System

This Fire Alarm Control Panel connect and inform the condition of the building to the Fire Alarm Control Panel at Powerhouse.

i) The Fire Alarm Control Panel material : The fire alarm control panel is fabricated from steel plate with a thickness of at least 1.2mm for the body and 1.6mm for the door.

ii) The Number of Zones and class : The number of zones is 5 and class [illegible] (Switchyard Relay House)

Equipment Materials

i) Wires and Conduit : The wires are used HP cables protected by heavy gauge conduit.

ii) Detectors : The smoke and heat detectors are confirmed its performance by maker's test.

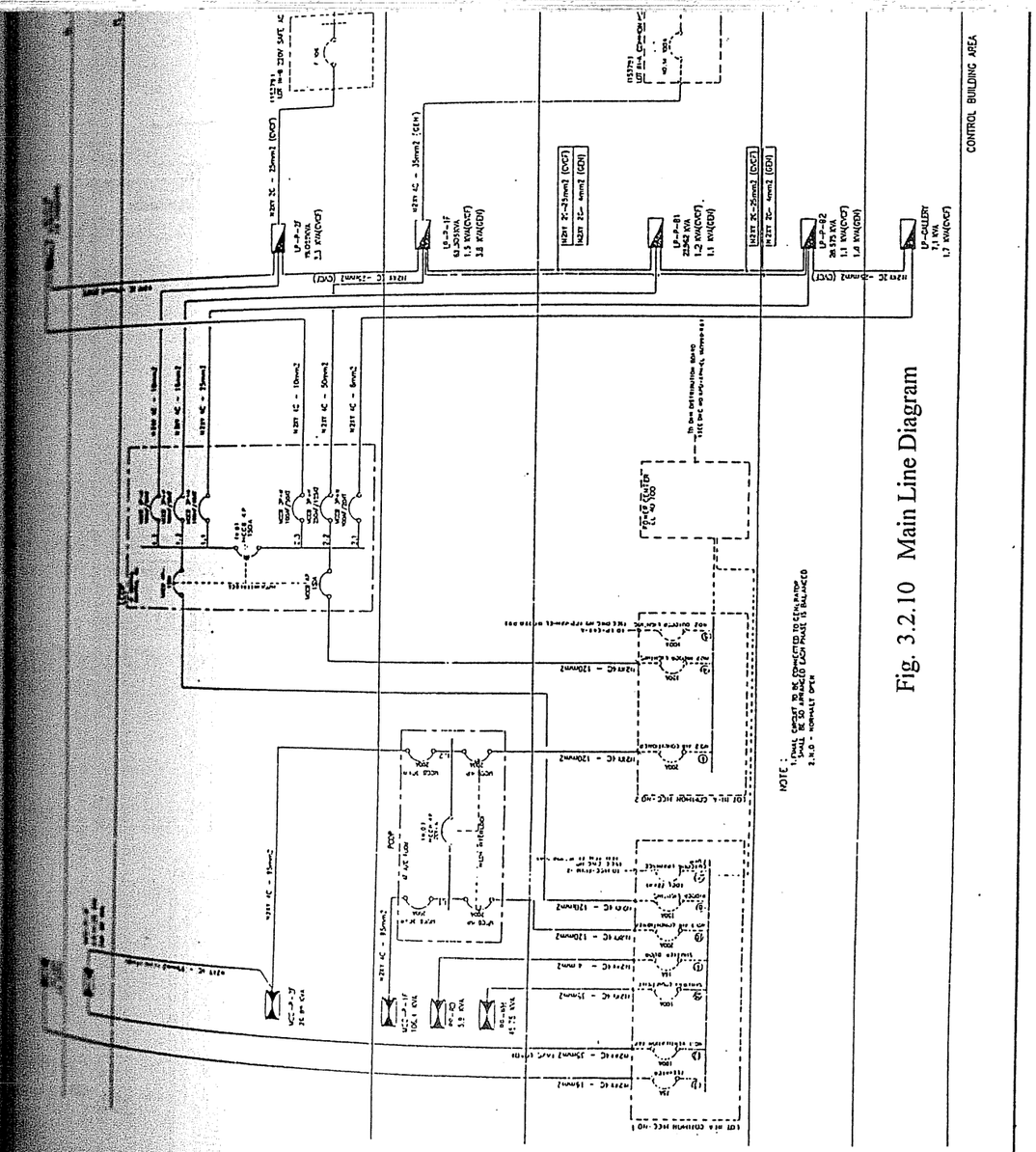


Fig. 3.2.10 Main Line Diagram

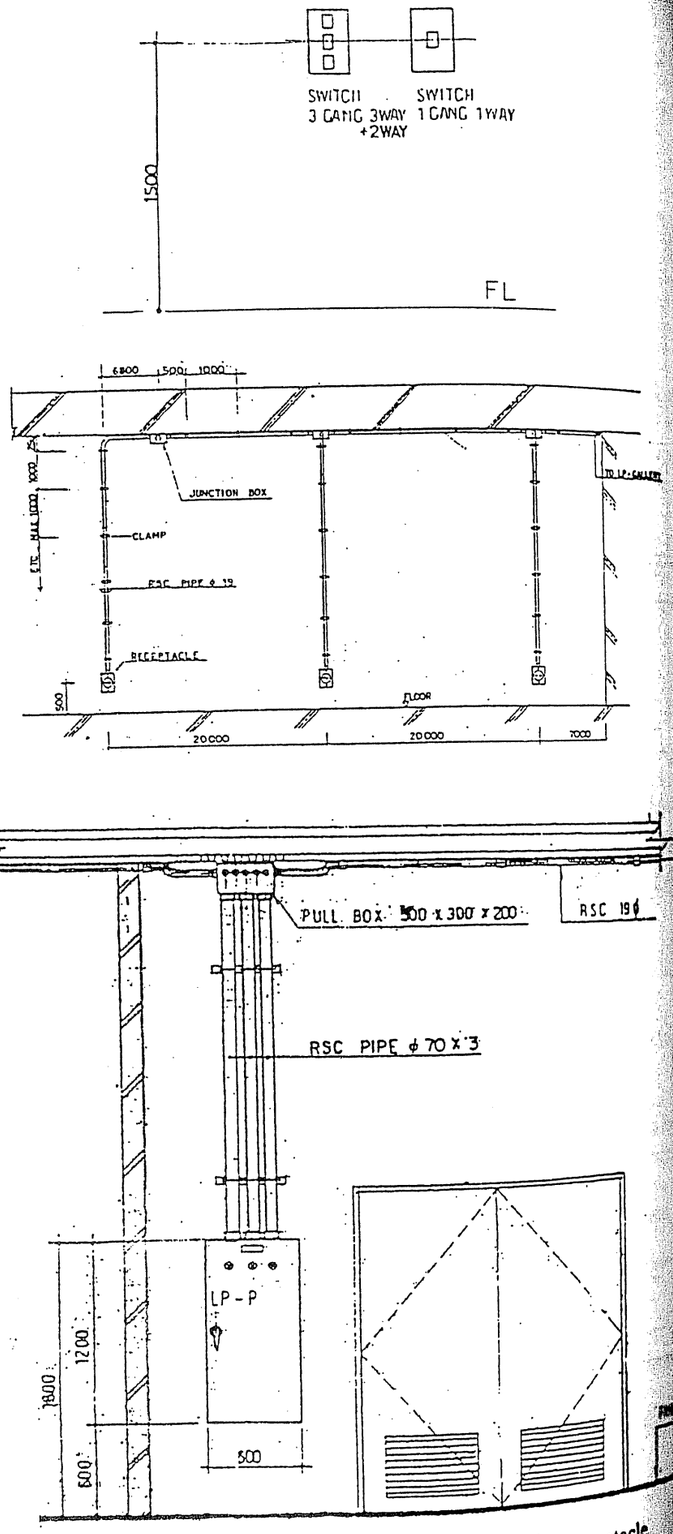


Fig. 3.2.11 Typical Installation Detail of Switch and Receptacle

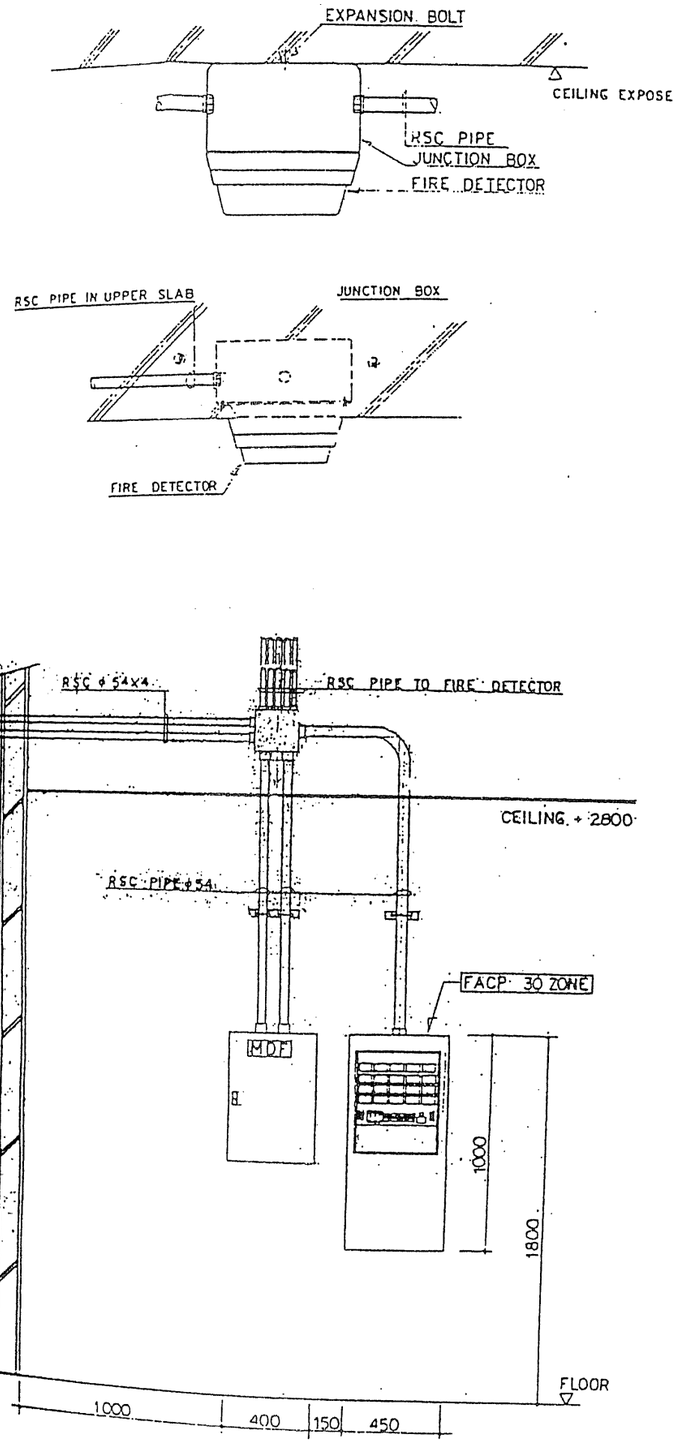


Fig. 3.2.12 Typical Detail of Control Panel and Detector

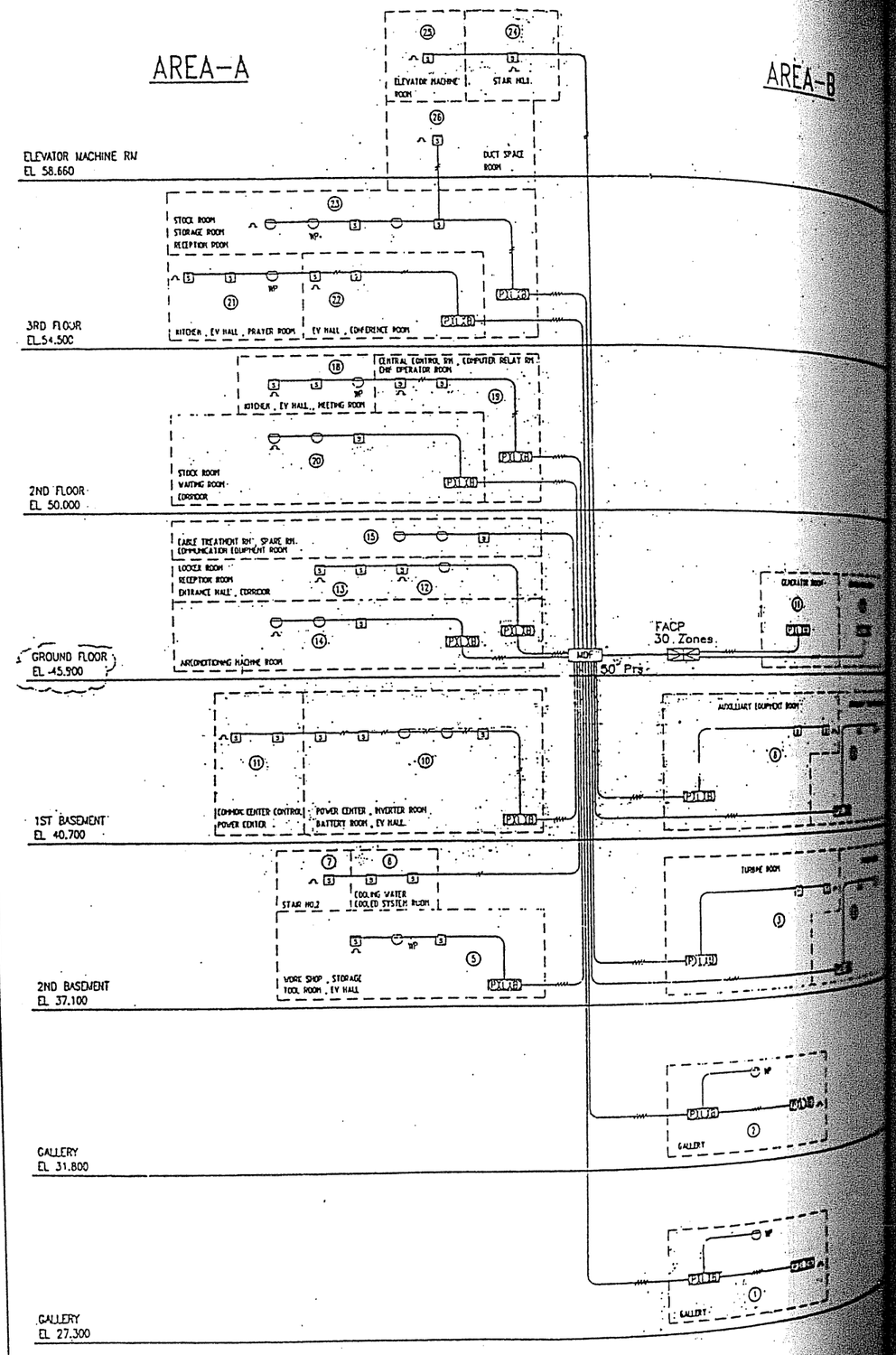


Fig. 3.2.13 Indoor Fire Protection System Diagram

Chapter 3 CONSTRUCTION WORKS

3.1 Construction Schedule

The construction schedule of the switchyard relay house is indicated below. After erection of the steel structure and pre-cast concrete wall, the finishing works of the powerhouse building were executed at the schedule as indicated in Table 3.3.2.

Fig. 3.3.1 Construction Schedule for Switchyard Relay House

Description	1995									1996					
	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6
Foundation, Excavation & concrete	■	■	■	■	■										
Steel structure erection				■	■	■									
Interior & exterior finishing					■	■	■	■	■	■	■	■	■	■	■

3.2 Construction Method of Major Works

The construction method of the following major works are indicated.

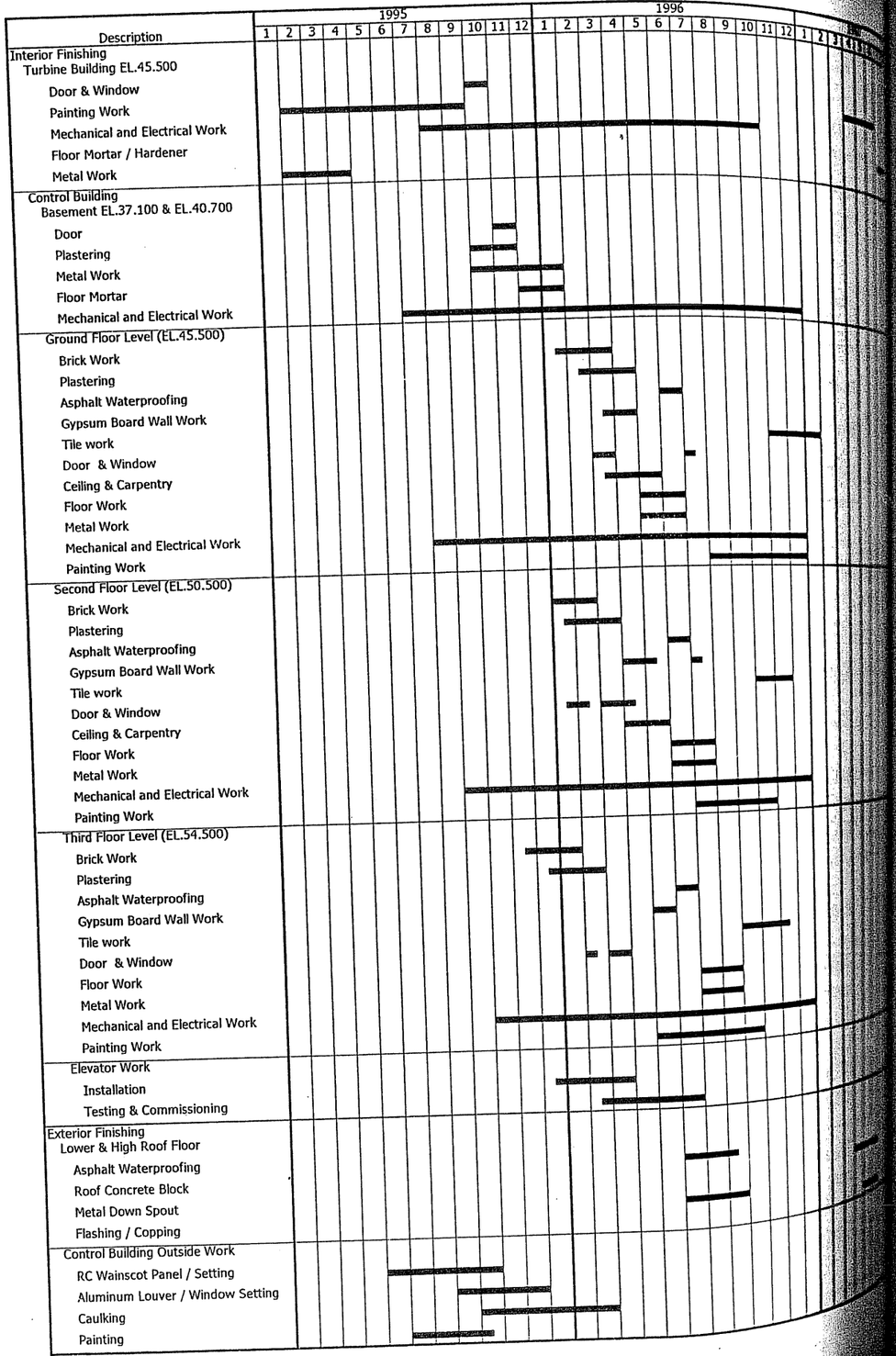
(1) Erection Method of Steel Structure for Powerhouse
(2) Fabrication and Erection Method of PC panel for Wall
(3) Work procedure of Built-up Asphalt Roofing for Powerhouse
(4) Work procedure of Free Access Floor for Control Room

3.2.1 Erection Method of Steel Structure for Powerhouse

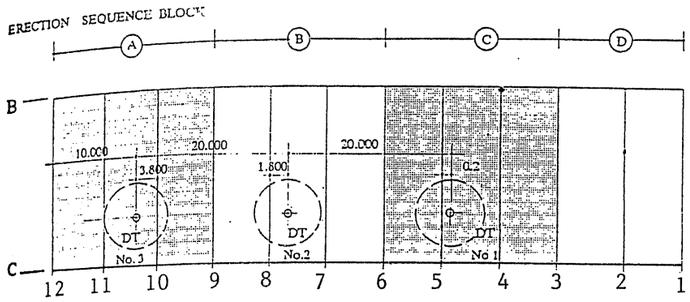
This method statement covered the erection of the steel framework of the powerhouse and control building to meet the target of quality, safety and smooth progress. The nature of this work requires extensive safety measure. This powerhouse work interfaced with other Lots. To avoid delay it was necessary that all framework plus the precast panel of grid line "C" and line 12, should be installed between mid January and end of June 1995. During this period, draft tubes were to be installed by Lot IIIA.

The steel structure was mainly erected by a 80 ton crawler crane. Due to limits of the crane capacity, the structure was divided into six erection zones, A, B, C, D, E and F, and the crane was arranged at seven locations to erect the all structure except columns at Line C between Line 4 to Line 9, which were erected by a tower crane.

Fig. 3.3.2 Construction Schedule for Powerhouse Building

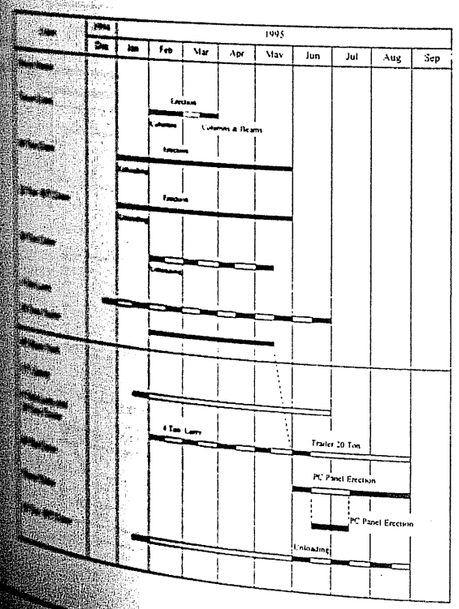


Work schedule for Steel Structure



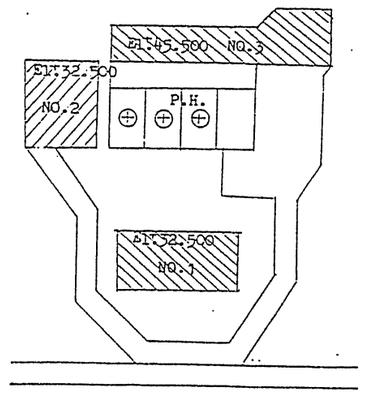
ITEM	12	1	2	3	4	5	6	7	8
Work	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug
1. DT Work	DT no.3		DT no 2						
By Others			DT no.1					Over Head Crane	
2. Painting Work			Generator Room A B C		D	Control Bldg			Crane
	Preparation						Touch Up After Erection (A,B,C,D, Control Bld)		
3. Erection Work	Erection		A	B	C D				
	Preparation & Painting					RC Wainscot RC Slab			
4.P.C.Panel work	P C Work Approval						PC Panel		
	Shop Drawing			PC Panel Production			C – Line 12 –Line	Other	
5. T/R Works								Civil	
					T/R Gate Installation by Tower Crane (By Others)			T/R Start	

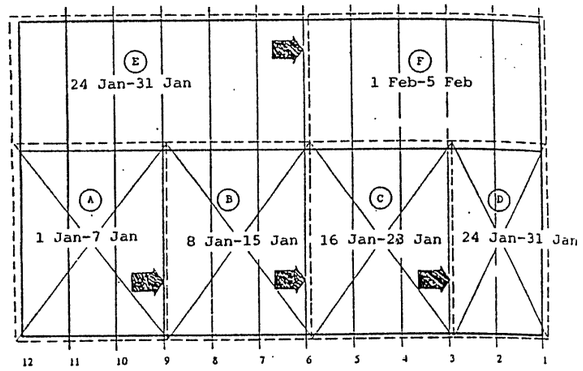
Crane Schedule



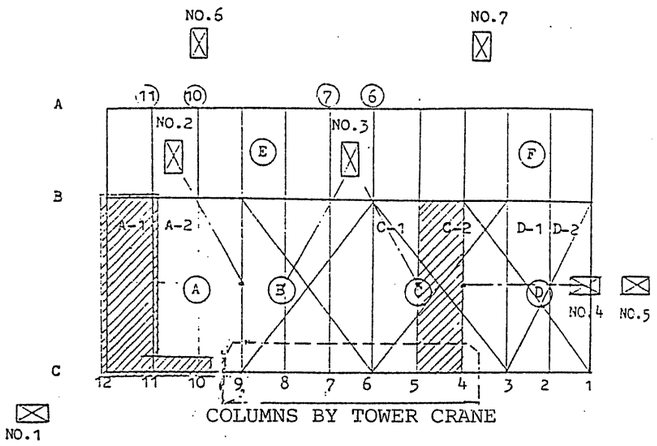
Stockyard Schedule

	For Steel Works	For PC Panel
No. 1	1st Jan - 30th Apr	
No. 2	15th Jan - 28th Feb 1st May - 30th Jun	
No. 3	1st Feb - 31st May	1st Jun - 31st Aug





Delivery Sequence



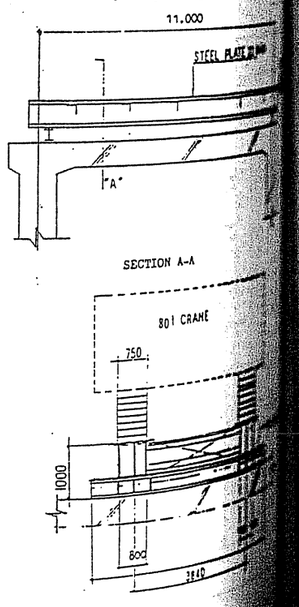
80 Ton Crawler Crane Location and Sequence

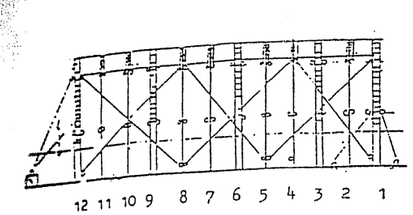
Erection Programme for 80 Ton Crane

Crane Location	Working Block	Working Area & Items
No. 1	A - 1	i) Line C ~ 12 - 11 - 10 - 9 columns ii) Line 12 Columns between Line B & C iii) Beams of Line 12 Line C - 12, 11, 10, 9
No. 2	A - 2	i) Line - B ~ 12 - 11 - 10 - 9 columns ii) Trusses & beams from Line 12 ~ Line 9
No. 3	B	i) From Line 8 to Line 6 columns truss ii) Wall & roof beams from Line 9 to Line 6
	C - 1	i) Roof beams from Line 6 to Line 5 ii) Columns of Line 5 B and C iii) Line 5 truss
No. 4	C - 2	i) Line 4, 3, 2 columns of Line B & C, Line 4 truss ii) Wall & roof beams Line 5 - 4
	D - 1	i) Line 3, 2 Trusses ii) Wall & roof beams from Line 4 to Line 2
No. 5	D - 2	i) Line 1 : B - C Columns, truss & beams ii) Beams Line 2 & 1, B - C
No. 6	E	All, excepting Line B beams and column
No. 7	F	All, excepting Line B beams and column
Columns at Line C between Line 4 to Line 9 were erected by tower cran		

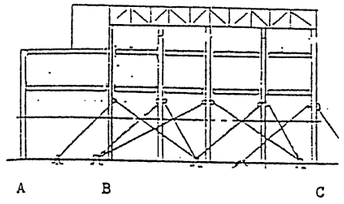
Note : Location No. 2 and 3 require bridge above the slab of the control building

Temporary Bridge for 80 Ton Crane

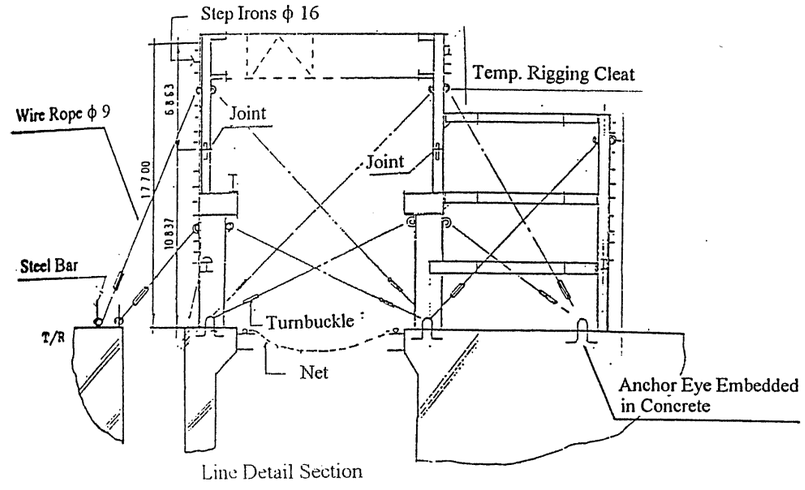




Elevation at Line C



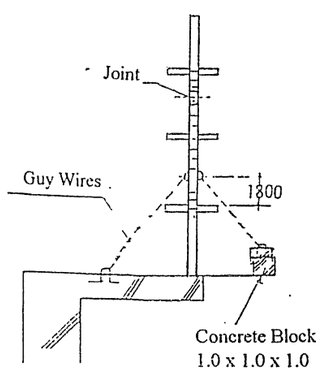
Elevation at Line 12

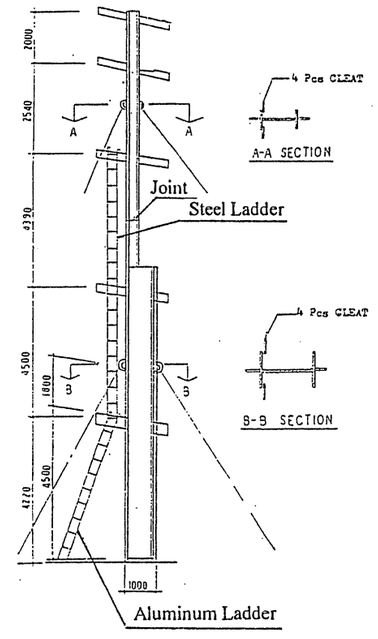


Line Detail Section

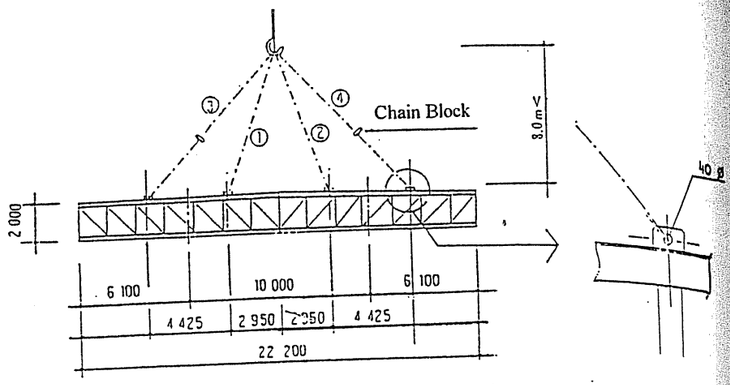
Erection and Alignment Items

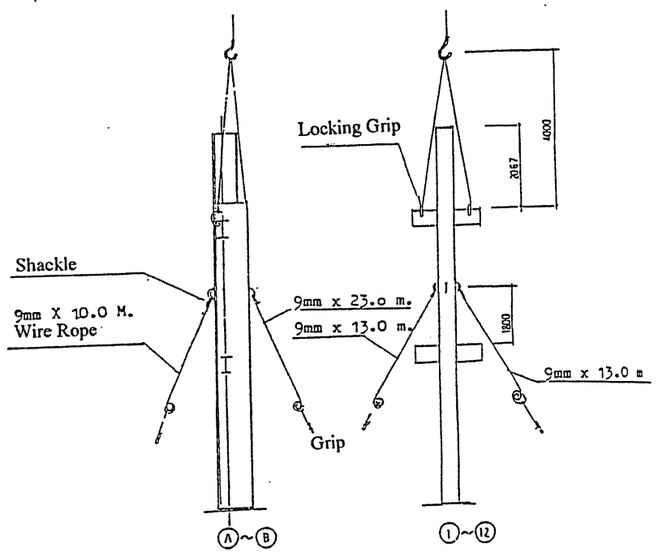




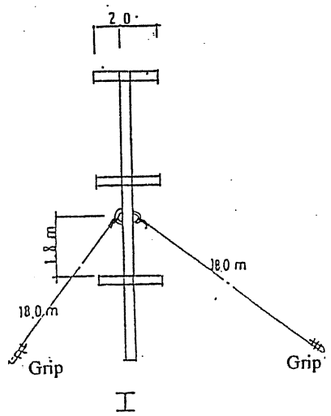


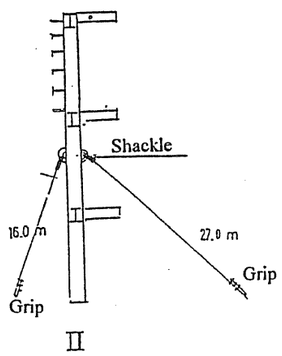
Alignment Method

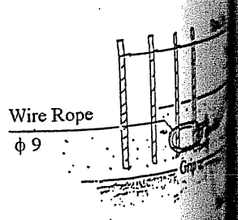




Truss and Column Slinging

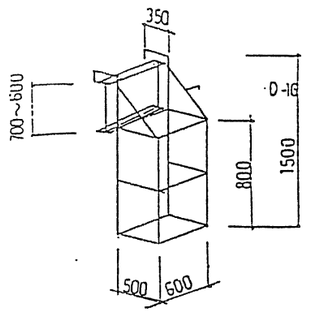




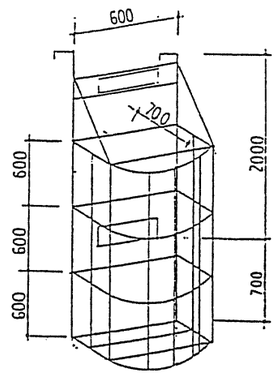


Column Guy and Slinging Wires

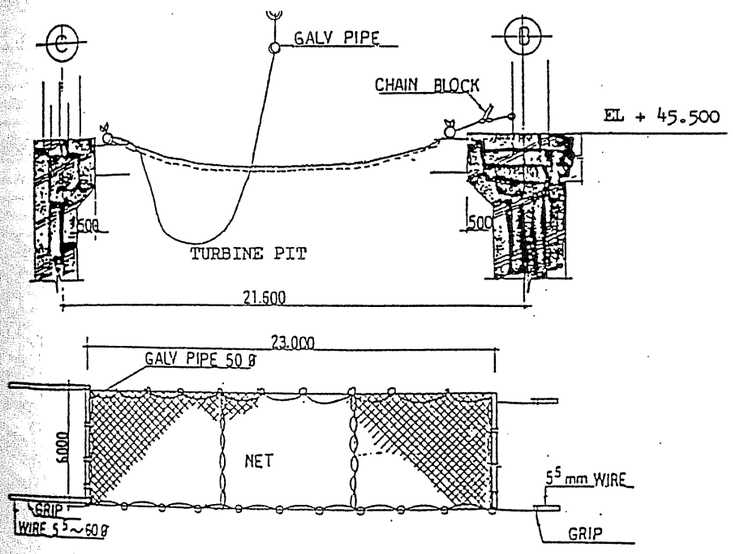




Platform for Beam Joint



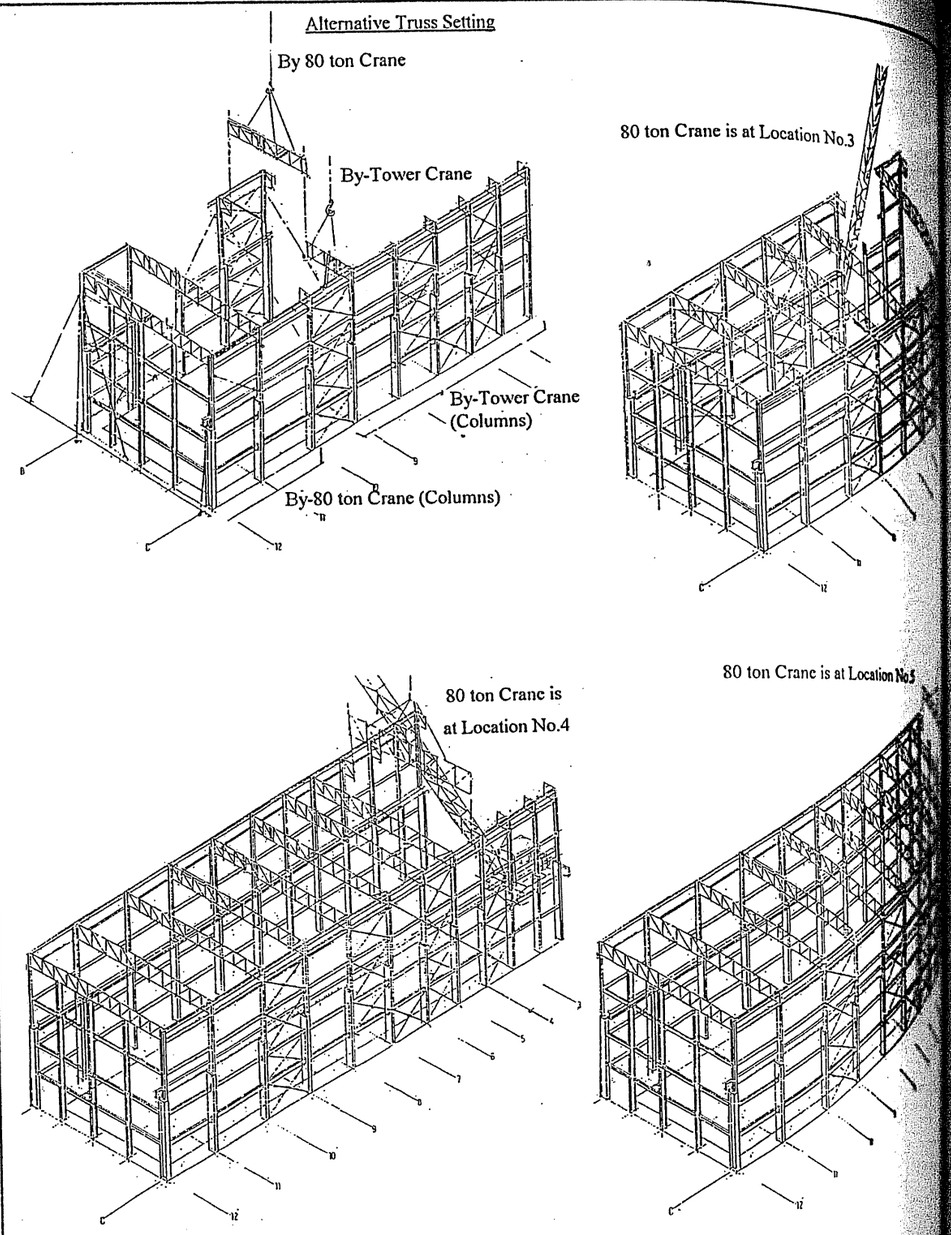
Platform for Truss



Safety Net

Duration for Erection Block A

Working Items	Erection Items				
	Columns	Beams	Trusses	Roof Beams	Working Days
Installation Pieces Nos	22	48	4	70	-
Installation Days Nos	1.0	2.5	2	3.5	9
Erect Crane Bridge					3
Relocate Crane					1
Truss Assembly					3
Columns Assembly					1.5
Rainy Season	1.5 hours/day loss				3.5
Holiday					3.0
Total (Per 3 spans, Line 9 - 12)					24 days

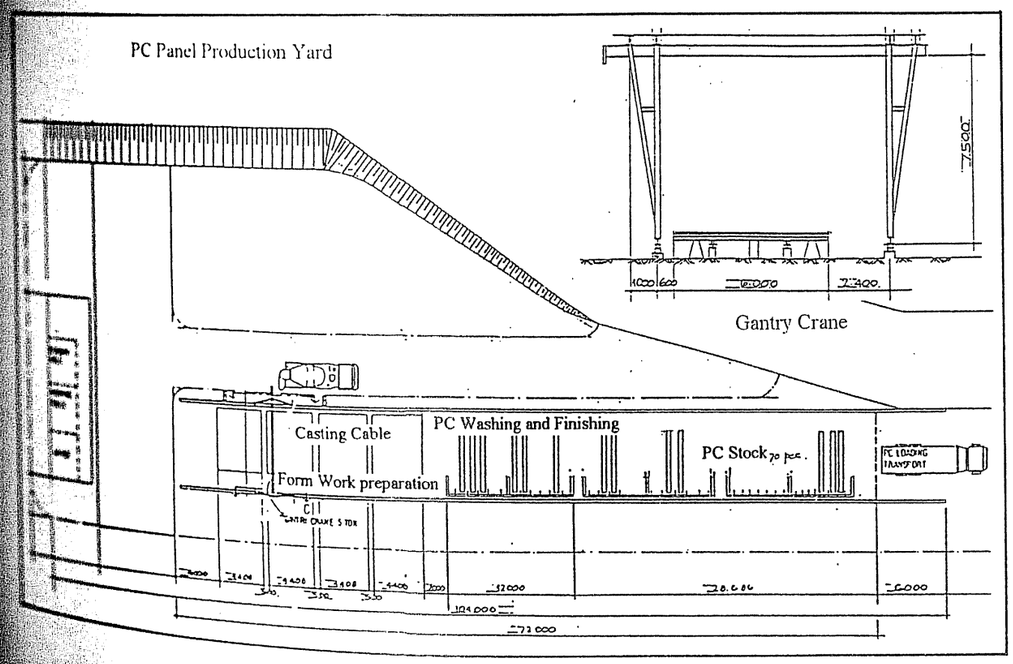


Painting Yard

12.2 Fabrication and Erection Method of PC panel for Wall

(1) Working Schedule and Production Yard

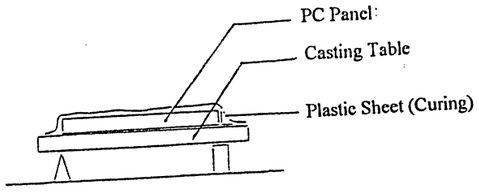
Item	1995 Jan	Feb	Mar	Apr	May	Jun	Jul
I. Preparation							
Moulding Preparation							
Fastener Fabrication							
Mobilization							
II. PC Production Yard & Gantry Crane							
Preparation							
PC Production Facilities							
III. Installation							
Moulding Setting							
Trial Production							
PC Production				(Total Panel = 400 Panel) 27 Panel / Week			
PC Erection					5 Panel / Day		
Sealant / Caulking							



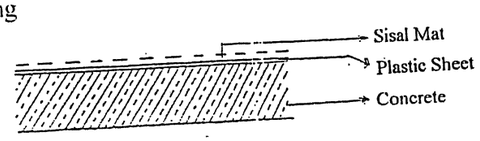
(2) Casting and Panel Transportation

(i) Concrete casting was done after 15.00 hrs. No direct sunlight and curing cover with plastic sheet. If concreting done before noon time checking of form ri steel bed temperature is required.

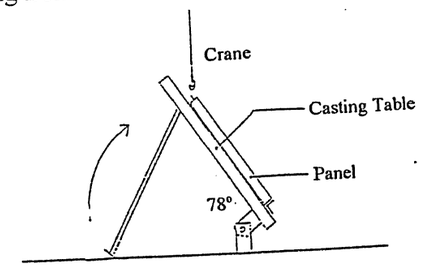
(ii) Casting table position before lifting



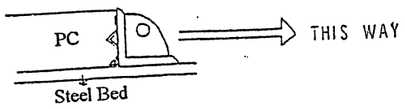
(iii) Curing



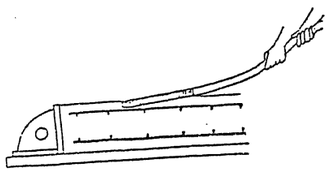
(iv) Panel lifting from table



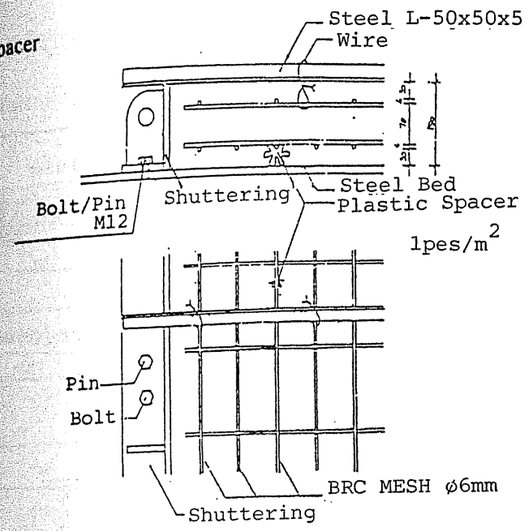
(v) Removal of shuttering



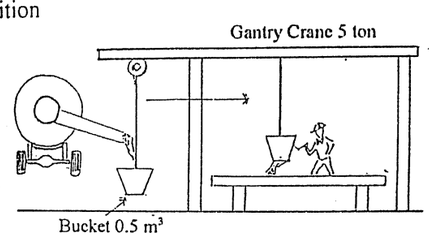
(vi) Concrete vibration

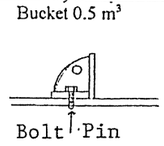


(vii) Spacer



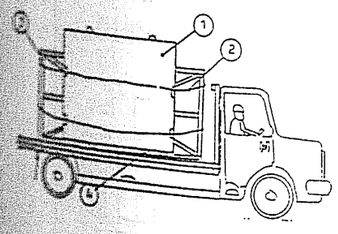
(viii) Casting position

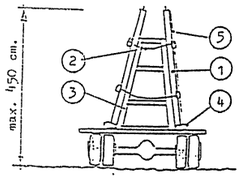




- Loosen of bolt and pin after 2 hours
- Removal of Shuttering after 6 hours
- PC panel lifting after concrete strength reach K 105 ± 12 hours

(ix) PC panel transportation

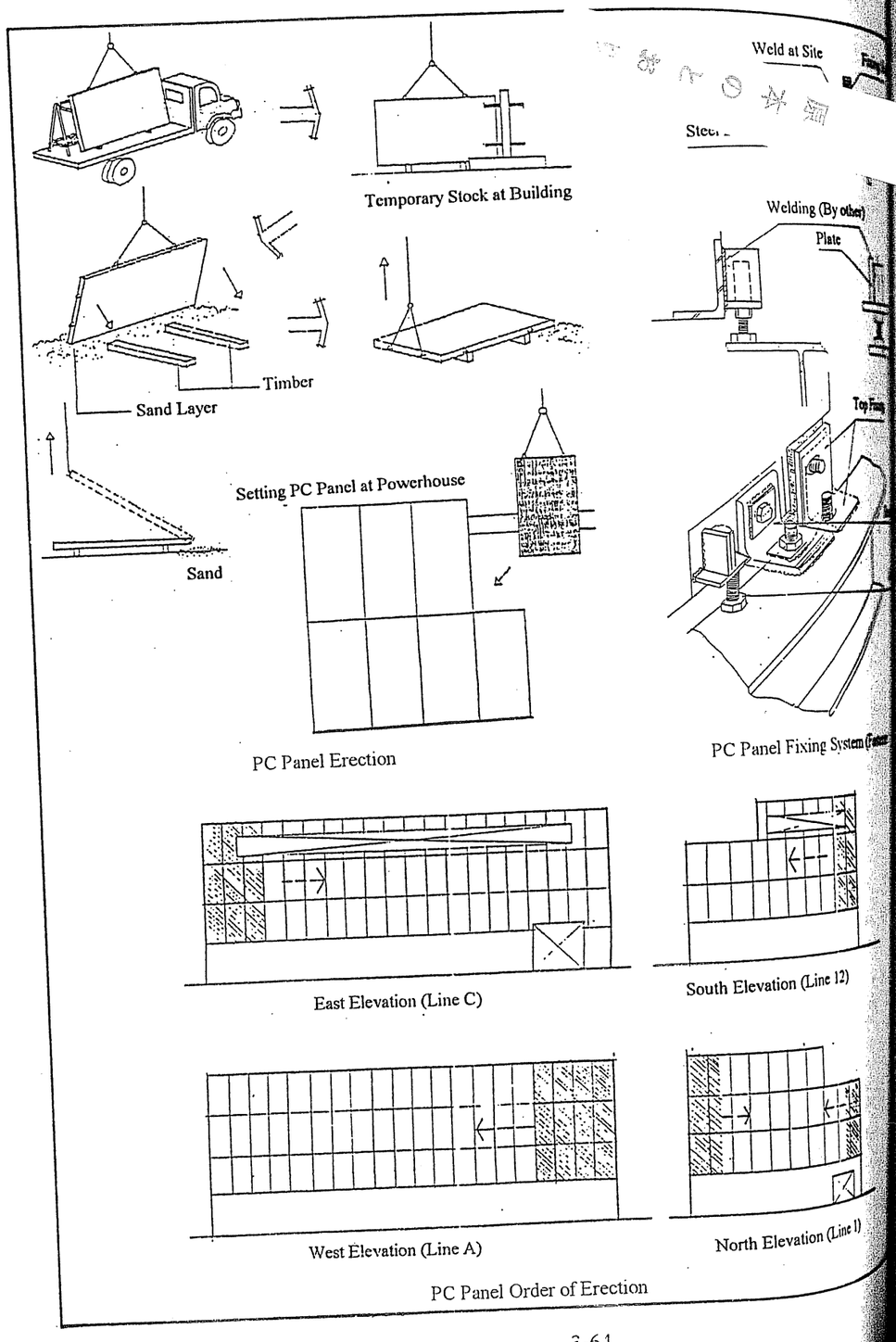




LEGEND

1. PC PANEL
2. SAFETY STRING
3. RACK
4. SAND SACK
5. GRANAITE PROTECTION, SHEET WITH WATERBASE ADHESIVE

(3) Installation Procedure



3.2.3 Work Procedure of Built-up Asphalt Roofing for Powerhouse

(1) Waterproofing Materials

Class "A"

Layer No.	Specification	Waterproofing Material
01	Asphalt primer (0.3 l/m²)	Asphalt primer (Mastiseal) (0.3 l/cm²)
02	Perforated roofing with thin layer of sand	Glass hole roofing
03	Asphalt (1.2 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)
04	Special roofing	Special roofing (Meltan 200 SP)
05	Asphalt (1.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)
06	Special roofing	Special roofing (Meltan 200 SP)
07	Asphalt (1.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)
08	Asphalt roofing	Weld roofing
09	Asphalt (2.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)

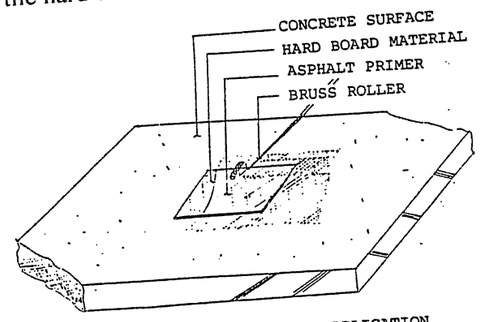
Class "B"

Layer No.	Specification	Waterproofing Material
01	Asphalt primer (0.3 l/m²)	Asphalt primer (Mastiseal) (0.3 l/cm²)
02	Asphalt primer (1.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)
03	Asphalt roofing	Weld roofing
04	Asphalt primer (1.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)
05	Special roofing	Special roofing (Meltan 200 SP)
06	Asphalt primer (1.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)
07	Asphalt roofing	Weld roofing
08	Asphalt primer (1.0 kg/m²)	Asphalt compound (Oxidized Bitumen 115/15) (1.2 kg/cm²)

(2) Asphalt Waterproofing Quantity Checking

(a) Asphalt Primer (REQ. 0.3 Lt/m^2)

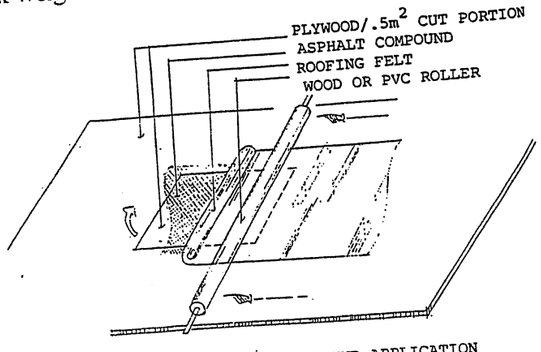
(i) Check the weigh of hardboard material (0.5 m^2).
(ii) Set the hardboard to clean surface concrete.
(iii) Apply asphalt primer on the surface of hardboard material same with concrete surface.
(iv) Cylinder glass use for checking of how much quantity consume for the required surface.
(v) Applicator will check the surface applied with primer if enough, if still not enough apply again up to reach the amount required for the surface.
(vi) Take out the hard board material & check the weigh by using weighing machine.



SAMPLE FOR ASPHALT PRIMER APPLICATION

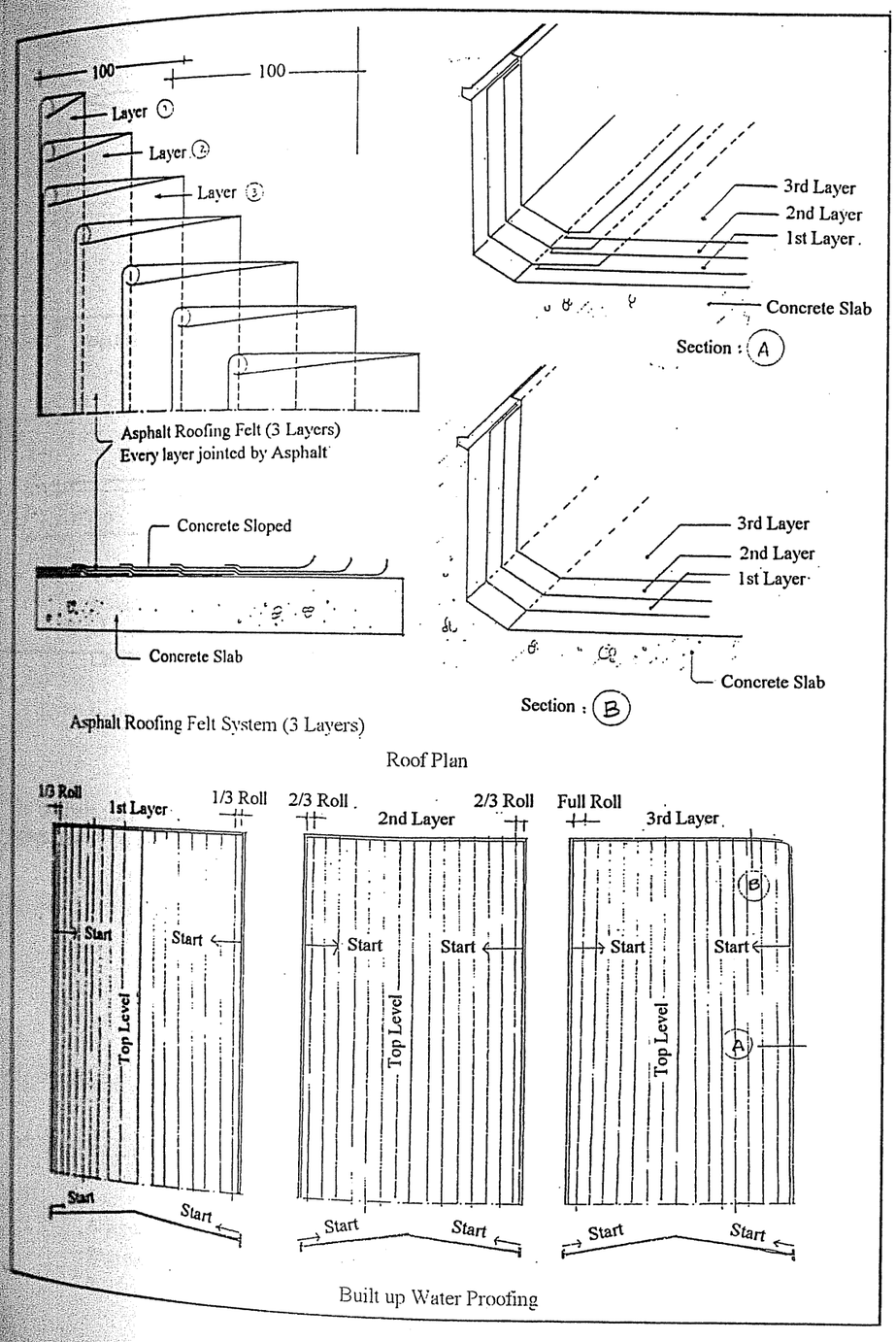
(b) Asphalt Compound

(i) Check the weigh of 5 m^2 cut portion of the plywood platform.
(ii) Setting kettle for hot asphalt compound heated by wood and shall heated with temperature loading 170 - 200 ^{0}C to melting point of asphalt.
(iii) Application of compound as shown on sketch.
(iv) After asphalt compound application take out the cut portion of plywood platform and check weight.

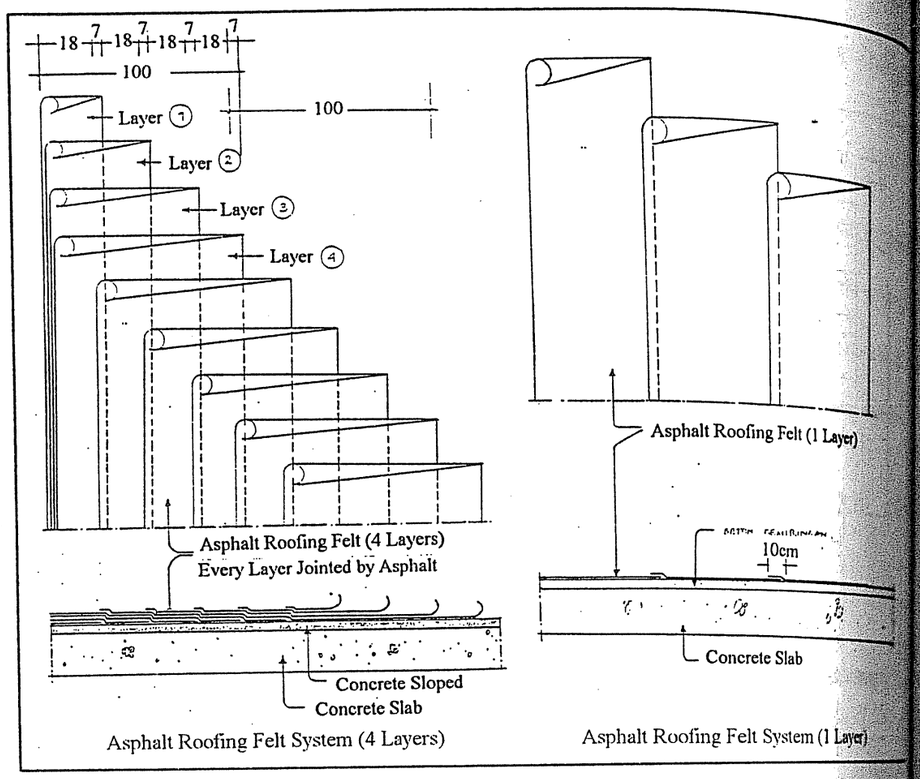


SAMPLE FOR ASPHALT COMPOUND APPLICATION

(3) Working Procedure



Built up Water Proofing



3.2.4 Work procedure of Free Access Floor for Control Room

(1) General

This method statement deals the requirements for the installation of free access floor at Cental Control Room. Chief Operator Room and Computer Relay Room of Powerhouse Control Building.

(2) Preparation and Pre-cleaning

All surface were cleaned from dust, debris etc..

(3) Installation Procedure

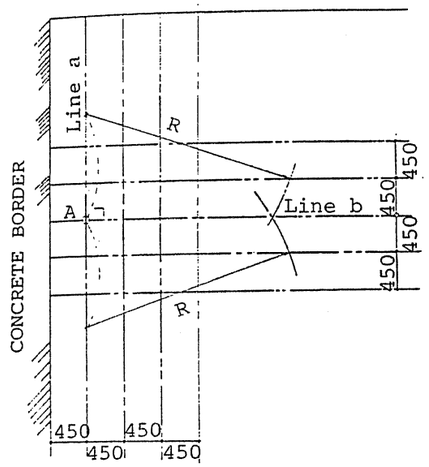
(a) Marking & Layout

(i) Draw "Line a" parallel to the concrete border. This is the standard line which all the other lines shall be based on. The width is 455 mm.

(ii) On the line a settle the base point A.

(iii) From the point "A" draw "Line b" which is perpendicular to "Line a"

(iv) Draw parallel lines with the lines a and b. The width shall be 450 mm.

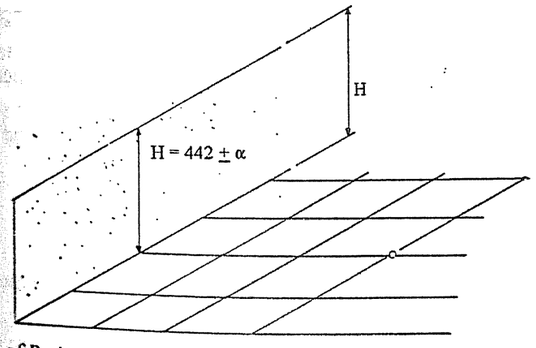


(b) Levelling

(i) Check and confirm the basic level line. Check the level line of 1m height on the column etc..

(ii) Check the height of the concrete borders at several points : H = 442 mm ± 15 mm.

(iii) In the same way check the level around the center of room.

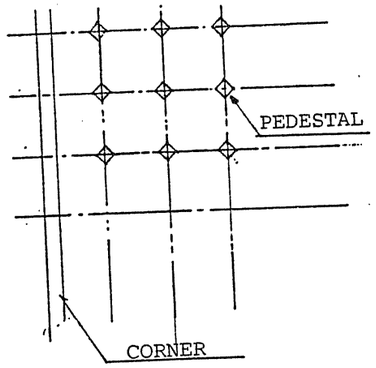


(c) Installation of Pedestals

(i) Spread adhesive (N-4568K) on the backside of each pedestal leg with a spatula. The quantity of the adhesive is approx. 60g per pc.

(ii) A pedestal leg shall be put on each cross of marked line. The angle shall be 45 degree except the pedestal for borders and corners shall be put parallel with the border.

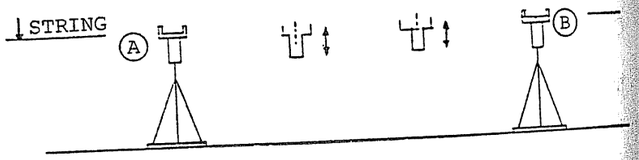
(iii) Push the pedestal leg to the base floor and adjust it to the correct position. The panels shall be installed together with the pedestals.
Do not install all the pedestals first and then panels.



(d) Installation of Panels

(i) Setting of pedestals

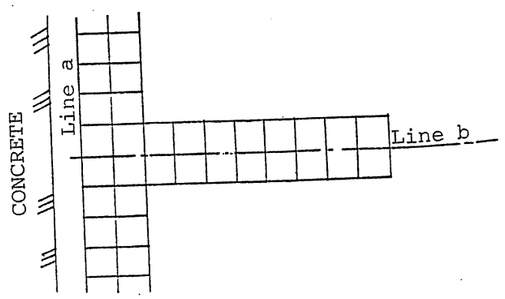
Set pedestals leg on the starting point and set the pedestals head.
Adjust the height of starting point to Finished Height 416.5 mm ..... (Pedestals A)
Adjust another pedestals over several ones in the same way as above. This shall be the 2nd starting point ..... (Pedestals B)



Stretch a black string between A and B, and adjust the height of pedestal heads to the height of string.
After the heights of pedestals are settled, fix the adjustable nut and the set-bolt of the head.

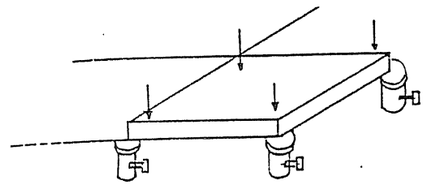
(ii) Process of installation of panels

From the starting line a - b, install panels as " ⊢ " or " + " as shown in the drawing below. Afterwards. spread from the basis panels of above.



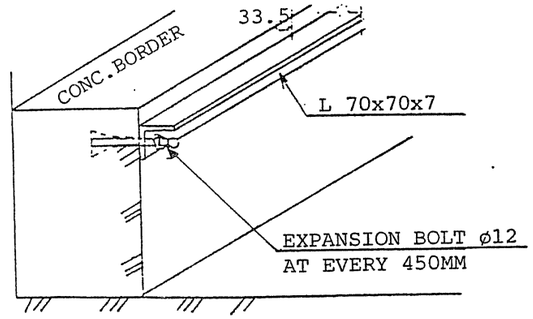
(iii) Clatter of a panel

Push the 4 corners of an installed panel to check if any clatter occurs.



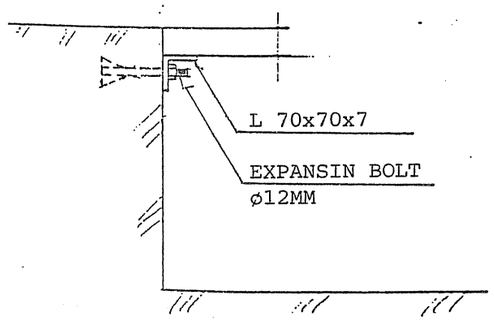
If there is any clatter adjust the height of pedestal head by loosing the adjustable nut of leg and set-bolt of head.

(iv) How to install the angles at the borders - Angle : L 70 x 70 x 7 mm

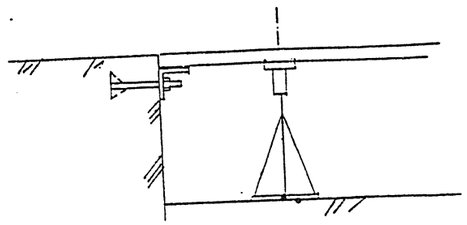


On the end of angles, put an aluminium "liner" total 4.0mm thick, adjust the height and set the panel.

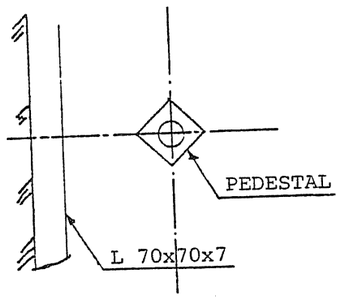
If the full sized panel (450mm x 450mm) cannot be installed, a side of the panel shall be cut out.



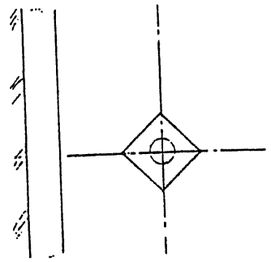
(v) How to set the pedestals near to the borders



(a) L > 270 mm
270 mm



(b) 240 mm < L >



3.3 List of Material

The list of the material for the architectural works and its fabricators are indicated in Table 3.3.1

Table 3.3.1 List of Material (1)

ITEM OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
			SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
[illegible]	• POWER HOUSE	• Aluminum Aloy 500 Kg/m2	Same as Design	HITACHI METAL TECHNO LTD	----	----	----
[illegible]	• POWER HOUSE	• Statistic Electricity Roof Tile	Same as Design	SUMBER INDAH PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	• Anvinyl Tile	Same as Design	GUBIN INTERNATIONAL PT	----	----	----
[illegible]	• POWER HOUSE	• Vinyl CloridеLeather	• Pholyvinil Choride (PVC)	HANEDA DECORATIONS	• Not Available in the Local Market & for Future Maintenance	• Contractor Lot 1	• Base on Submitted Quotation
[illegible]	• POWER HOUSE • RELAY HOUSE	• Combustible Rock wool 12 mm Thk	Same as Design	SUMBER SETIA ABADI PT	----	----	----
[illegible]	• POWER HOUSE	• Colored Aluminum Sheet	Same as Design	ARGASS MANUGGAL SEJATI PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	• Extruded Aluminum Shape	Same as Design	ARGASS MANUGGAL SEJATI PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	• Extruded Aluminum Shape JH 4100	Same as Design	ARGASS MANUGGAL SEJATI PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	• Builtup Roofing	• Builtup Roofing	RANGGALO NUSANTARA JAYA PT	----	----	----

Table 3.3.1 List of Material (2)

ITEM OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
			SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
[illegible]	• DAM	• 2 Unit 40 Hp • 2 Unit 150 Hp	Same as Design	FIBRITE FIBER GLASS PT	----	----	----
[illegible]	• DAM	• 40 Hp • 150 Hp	• 40 Hp • 150 Hp	JASA MARINE	----	----	----
[illegible]	• POWER HOUSE	• Wool Carpet	• Wool Carpet	SUMBER INDAH	----	----	----
[illegible]	• WATER SUPPLY • BOAT W. HOUSE	• Colored Metal Sheet Roofing	• Same As Design	REMASCO SEJATI UTAMA PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	• Master Key System	• Master Key System	CITRA AMANTOSA SEJATI PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	----	• Hold Open Paralel Alarm	CITRA AMANTOSA SEJATI PT	----	----	----
[illegible]	• POWER HOUSE	----	• DT 22 Floor Springs Alarm	CITRA AMANTOSA SEJATI PT	----	----	----
[illegible]	• POWER HOUSE • RELAY HOUSE	----	• S-6166-4 70 Kg	CITRA AMANTOSA SEJATI PT	----	----	----
[illegible]	• POWER HOUSE	----	• ST-5L84-1 Stainless Steel US 32D	CITRA AMANTOSA SEJATI PT	----	----	----

Table 3.3.1 List of Material (3)

WORKS : FINISHING / CIVIL

N0.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
	- DOOR STOPPER	- POWER HOUSE	----	- D-900 Wall	CITRA AMANTOSA SEJATI PT	----	----	----
	- DOOR PUSH & PULL PLATES	- POWER HOUSE	---	- ACE SUS 304 300 x 210	CITRA AMANTOSA SEJATI PT	----	----	----
	- FLUSH BOLT	- POWER HOUSE - RELAY HOUSE	---	- PR 002 RY 051 RB 002 R	CITRA AMANTOSA SEJATI PT	----	----	----
12.	ELEVATOR	- POWER HOUSE	- 1500 Kg Capacity 2 Panel Opening	Same as Design	JAYA KENCANA PT	----	----	----
13.	FLAG POLE	- POWER HOUSE - RELAY HOUSE	- Aluminum Aloy	- Galvanized Metal	PHILIPS RALIN ELETRONIC	Material Aluminum Aloy Not available in the Local Market	Let Contractor	
14.	FLOOR HARDENER	- POWER HOUSE - RELAY HOUSE	- Metallic Hardener Floor Finish	Same as Design	METROPOLE TRADING CO.	----	----	----
15.	GLASS	- POWER HOUSE - RELAY HOUSE	- Jis R 3201 - Jis R 3204 - Jis R 3205	Same as Design	ASAHI FLAT GLASS CO. LTD	----	----	----
16.	LIGHT WIEGHT CONCRETE	- POWER HOUSE	- Specific Grafity of Course Aggrelate	Same as Design	BHUMI MARDANA PT	----	----	----
17.	LOCK SETS (MASTER KEY SYSTEM)	- POWER HOUSE - RELAY HOUSE	----	- Hold Open Paralel Alarm	CITRA AMANTOSA SEJATI PT	----	----	----

Table 3.3.1 List of Material (4)

WORKS : FINISHING / CIVIL

N0.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
18.	PAINT	- POWER HOUSE - RELAY HOUSE	- Vinyl Chloride Resin Enamel	- Acrylin Resin Enamel	NIPPON PAINT INDONESIA CO. LTD	TEPSCO ENGR DECISION	ENGINEERING	
		- POWER HOUSE	- Ackyd Resin Type	Same as Design	POLAR SAKTI JAYA PT	----	----	----
19.	PLANTING	- POWER HOUSE			HAZAMA BRANTAS J.O			
20.	SIGN BOARD	- POWER HOUSE	- Cast Brounze Metal	- Cast Brounze Metal	TIRAI KENCANA	----	----	----
21.	STEEL DOORS	- POWER HOUSE - RELAY HOUSE	- Double Hallow Core	- Double Hallow Core	SAIKI SUGIYANTO SEJATI PT	----	----	----
22.	TILES	- POWER HOUSE - RELAY HOUSE	- Ceramic Tiles	- Ceramic Tiles	ASIA VICTORY INDUSTRI LTD	----	----	----
23.	WOODEN DOORS	- POWER HOUSE	- Push Type Door W/ Cornica	Same as Design	SAIKI SUGIYANTO SEJATI PT	----	----	----

Table 3.3.1 List of Material (5)

NO.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
		- POWER HOUSE - RELAY HOUSE	- Air Cooled Type	- Air Cooled Type	DAIKIN AIRCON PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	----	----	UNI MAKMUR ELEKTRIKA PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Washable Type	- Washable Type	TAMURA AIR CONDITINING PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Manual	- Manual	TAMURA AIR CONDITINING PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Round Type	- Round Type	TAMURA AIR CONDITINING PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Fixed Type	- Fixed Type	TAMURA AIR CONDITINING PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Centrifugal, Axeal Wall Mounted & Ceiling Mounted	- Centrifugal, Axeal Wall Mounted & Ceiling Mounted	NUCLEUS ENGINEERS & SUPPLIERS PT	----	----	----
		- POWER HOUSE	- Gravity System	- Gravity System	PALMAS ENTRACO PT	----	----	----
		- POWER HOUSE - DAM GALLERY - TAIL RACE - PUMP ROOM	- Submersible, Centrifugal & Vertical Type	- Submersible, Centrifugal & Vertical Type	TERALINDO LESTARY PT	----	----	----

Table 3.3.1 List of Material (6)

NO.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
		- POWER HOUSE - RELAY HOUSE	- Vitruos China	- Vitruos China	SURYA PERTIWI PT	----	----	----
		- POWER HOUSE - BUILDING	- Injection Type	- Injection Type	SOUTHERN CHEMICAL INC	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Water Cooler	- Water Dispencer	WIRA BUANA INTRENT PT	----	----	----
		- POWER HOUSE - BUILDING	- Wallhung Type	- Wallhung Type	WEST BURNE - INTERNATIONAL PT ADITYA SARANA GRAHA PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Cap 45 Persons	- Cap 45 Persons	MAIZERINDO TUNAS - PERKASA PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Cap 5 Persons	- Cap 5 Persons	MUGI MAXITHERM - INDUSTRIES PT			
		- PUMP ROOM	- Use Silica & Anthracite	- Use Silica & Anthracite	BETA PRAMESTI PT	----	----	----
		- PUMP RDOM	- Working Pressure 14 Kg / Cm2	- Working Pressure 14 Kg / Cm2	GUNA ELECTRO PT	----	----	----
		- POWER HOUSE - RELAY HOUSE	- Cap 3 M3 & Cap 1 M3	- Cap 3 M3 & Cap 1 M3	FIBRITE FIBREGLASS PT	----	----	----

Table 3.3.1 List of Material (7)

WORKS : MECHANICAL

NO.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL	
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY
15.	GALVANIZED IRON SHEET	- POWER HOUSE - RELAY HOUSE	- Various Thickness	- Various Thickness	CAHAYA BENTENG MAS PT	----	-
16.	GATE VALVE , CHECK VALVE, STAINER	- POWER HOUSE - RELAY HOUSE - PUMP ROOM	- Pressure 10 Kg/Cm2 20Kg/Cm2	- Pressure 10 Kg/Cm2 & 20Kg/Cm2	SINAR MAS SAKTI PT	----	-
17.	ELBOW , FLEXIBLE JOINT, TEE	- POWER HOUSE - RELAY HOUSE - PUMP ROOM	- Screw & Weld Type	- Screw & Weld Type	CIPTA KORELASI UTAMA PT	----	-
18.	GALVANIZED STEEL PIPE	- POWER HOUSE - RELAY HOUSE - PUMP ROOM	- Medium Class & Sch 40	- Medium Class & Sch 40	IKA RAHARJO GRAHA PT	----	-
19.	PLAIN END GALVANIZED STEEL PIPE / W PVC LINNING	- POWER HOUSE - RELAY HOUSE	- Medium Class	- Medium Class	ESLON SINGAPORE PTE LTD	----	-
20.	CAST IRON PIPE & FITTINGS	- POWER HOUSE - RELAY HOUSE	- Inside Coated Tarepoxy	- Inside Coated Tarepoxy	SAMUDRA ARTA SAKA PT	----	-
21.	DP EXTENDED SURFACE AIR FILTER	- POWER HOUSE - BUILDING	- Washable Type	- Washable Type	DUTA INSTRUMENT ALFI - SAKTI PT	----	-
22.	FLOAT SWITCH	- DAM GALLERY - TAIL RACE	- Adjustable Type	- Adjustable Type	WASKITA TEKNIK PT	----	-
23.	CAST IRON MANHOLE COVER	- POWER HOUSE - RELAY HOUSE	- Used for External Are	- Used for External Area	AMAL JAYA TOKO	----	-

Table 3.3.1 List of Material (8)

WORKS : MECHANICAL

NO.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL	
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY
24.	FLEXIBLE DUCT ALIMINIUM & CU PIPE INSULATION	- POWER HOUSE - RELAY HOUSE	- Heat Ressistant Type	- Heat Ressistant Type	SENTRAL TOKO	----	-
25.	MOTORIZED DAMPER	- POWER HOUSE - BUILDING	- Interlock Type	- Interlock Type	BERCA INDONESIA PT	----	-
26.	DUCT INSULATION	- POWER HOUSE - RELAY HOUSE - BUILDING	- Glass Wool 25 T	- Glass Wool 25 T	EKA WARNA KIMIA PT	----	-

Table 3.3.1 List of Material (9)

NO.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
	[illegible]	. POWER HOUSE . SWITCHYARD	Body : Thickness at Least 1,6 m/m Door : Thickness at Least 2,3 m/m - Jis C 8370 - Jis C 8371	Same as Original Design	UNI MAKMUR ELETRIKA PT	----	----	----
	[illegible]	. POWER HOUSE	Body : Thickness at Lesst 1,6 m/m Door : Thickness at Lesst 2,3 m/m - Jis C 8370 - Jis C 8371	Same as Original Design	UNI MAKMUR ELETRIKA PT	----	----	----
	[illegible]	. POWER HOUSE . SWITCHYARD	- Jis C 7601 - Jis C 7603 - Jis C 8106 - Jis C 8108 - Jis C 7501 - BCJ	Same as Original Design	DIAN MURNI GEMILANG PT	----	----	----
	[illegible]	. OUT DOOR	- Jis G 3101	Same as Original Design	PHILIPS RALIN ELECTRONIC PT	----	----	----
	[illegible]	. POWER HOUSE . SWITCHYARD	- Jis G 8303 - Jis G 8304 - Jis G 8305	Same as Original Design	GUNA ELECTRO PT	----	----	----

Table 3.3.1 List of Material (10)

NO.	NAME OF MATERIAL	APPLIED LOCATION	SPECIFICATION (ORIGINAL DESIGN)	ACTUALY USED		CHANGE OF MATERIAL		
				SPECIFICATION	FABRICATOR'S NAME	REASON	REQUESTED BY	HOW TO AJUST THE COST
	[illegible]	. POWER HOUSE . SWITCHYARD	- Steel and Coated With Protective Paint	Same as Original Design	SERINDO DARMA JAYA PT	----	----	----
	[illegible]	. POWER HOUSE . SWITCHYARD	- Steel and Coated With Protective Paint	Same as Original Design	SERINDO DARMA JAYA PT	----	----	----
	[illegible]	. POWER HOUSE . SWITCHYARD	Body : Thickness at Least 1,6 m/m Door : Thickness at Least 2,3 m/m - Heat Detector - Smoke Detector	Same as Original Design	JAYA TEKNIK INDONESIA PT	----	----	----
	[illegible]	. POWER HOUSE . SWITCHYARD	Body : Thickness at Lesst 1,6 m/m Door : Thickness at Lesst 2,3 m/m Type : Heat Detector Smoke Detector	Same as Original Design	JAYA TEKNIK INDONESIA PT	----	----	----
	[illegible]	. POWER HOUSE . SWITCHYARD . OUT DOOR	- Jis C 3307 - Jis C 3317 - Jis G 3605 - SPLN	Same as Original Design	IKI INDAH KABEL INDONESIA PT JAWA ELECTRIC PRIMA PT	---- ----	---- ----	---- ----

3.4 Color List of Finishing Material

The color of interior and exterior finishing is listed in Table 3.3.1 for the powerhouse and Table 3.3.5 for the switchyard relay house.

Table 3.3.2 Color List of Powerhouse (1)

WORKS : FINISHING / CIVIL

NO.	APPLIED LOCATION		NAME OF MATERIAL AND COLOUR				
	AREA	ROOM	FLOOR	BASE BOARD	WALL	CEILING	REMARKS
1	PUBLIC AREA EL.45.500	ENTRANCE	ASIA TILE 400 x 400 LAZIO GREY	ASIA TILE 400 x 100 LAZIO GREY	ASIA TILE 200 x 250 ALFA BROWN	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	ELEVATOR ENTRANCE DOOR JAMB : IVORY BS 4 - 063 DOOR : IVORY BS 4 - 063
		CORRIDOR	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT SATIN GLO ORCHID WHITE P 017	NIPPON PAINT SATIN GLO BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
2	PRIVATE AREA EL.45.500	GUARD ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT SATIN GLO ORCHID WHITE P 017	NIPPON PAINT SATIN GLO BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
3	CONTROL AREA EL.45.500	AIR CONDITIONER MACHINE ROOM	STEEL TROWEL FINISH	EXPOSED CEMENT PLASTER	EXPOSED CEMENT PLASTER	EXPOSED CEILING	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		CABLE TREATMENT ROOM	STEEL TROWEL FINISH	EXPOSED CEMENT PLASTER	EXPOSED CEMENT PLASTER	EXPOSED CEILING	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		EMERGENCY GENERATOR ROOM	STEEL TROWEL FINISH	EXPOSED CEMENT PLASTER	EXPOSED CEMENT PLASTER	EXPOSED CEILING	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		SPARE ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPONT PAINT SATIN GLO JASMINE WHITE P 009	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		COMMUNICATION EQUIP ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPONT PAINT SATIN GLO JASMINE WHITE P 009	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063

Table 3.3.2 Color List of Powerhouse (2)

WORKS : FINISHING / CIVIL

NO.	APPLIED LOCATION		NAME OF MATERIAL AND COLOUR				
	AREA	ROOM	FLOOR	BASE BOARD	WALL	CEILING	REMARKS
		GENERATOR ROOM ERECTION BAY	STEEL TROWEL FINISH WITH HARDENER CHICIBU C - 7		NIPPON PAINT LIGHT GRAYISH BLUE GREEN	METAL DECK JOTUN PAINT H - 8,5	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
4	SERVICE AREA EL.45.500	SHOWER	ASIA TILE 200 x 200 VENUS PINK		ASIA TILE 200 x 200 ANABEL PINK	NIPPONT PAINT BRILLIANT WHITE P 001	WOODEN DOOR NIPPON PAINT JAMB : MILKY GRAY 132 DOOR : IVORY (FORMICA)
		LAVATORY	ASIA TILE 200 x 200 VENUS PINK		ASIA TILE 200 x 200 ANABEL PINK	NIPPONT PAINT BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		LOCKER	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPONT PAINT BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		STORAGE	STEEL TROWEL FINISH	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHIDE WHITE P 017	NIPPONT PAINT BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
5	PUBLIC AREA EL.50.000	ELEVATOR HALL AND CORRIDOR	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHIDE WHITE P 017	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE) BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
6	SEMI PUBLIC / SEMI PRIVATE AREA EL.50.000	MEETING ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT BARLEY WHITE P 004	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063
		WAITING ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT BARLEY WHITE P 004	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 063 DOOR : IVORY BS 4 - 063

Table 3.3.2 Color List of Powerhouse (3)

NO.	APPLIED LOCATION		NAME OF MATERIAL AND COLOUR					PROCEDURE OF DECISION
	AREA	ROOM	FLOOR	BASE BOARD	WALL	CEILING	REMARKS	OF COLOUR
		STOCK ROOM	STEEL TROWEL FINISH	NA	NIPPON PAINT BARLEY WHITE P 004	EXPOSED METAL DECK	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		SUPERMETTE	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 011	NA	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		LAVATORY (MAN)	ASIA TILE 200 x 200 VENUS PINK	NA	ASIA TILE 200 x 200 ANABEL PINK	NIPPON PAINT BRILLIANT WHITE P 011	WOODEN DOOR NIPPON PAINT JAMB : MILKY GAY 132 DOOR : IVORY (FORMICA)	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		LAVATORY (WOMAN)	ASIA TILE 200 x 200 VENUS PINK	NA	ASIA TILE 200 x 200 ANABEL PINK	NIPPON PAINT BRILLIANT WHITE P 011	WOODEN DOOR NIPPON PAINT JAMB : MILKY GAY 132 DOOR : IVORY (FORMICA)	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		GENERAL CONTROL ROOM	ACCESS FLOOR + VINYL TILE TAJIMA	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT BARLEY WHITE	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		COMPUTER & RELAY ROOM	ACCESS FLOOR + VINYL TILE TAJIMA	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT BARLEY WHITE	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		CHIEF OPERATOR ROOM	ACCESS FLOOR + VINYL TILE TAJIMA	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT BARLEY WHITE	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		ELEVATOR HALL AND CORRIDOR	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ARCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 001	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO

Table 3.3.2 Color List of Powerhouse (4)

NO.	APPLIED LOCATION		NAME OF MATERIAL AND COLOUR					PROCEDURE OF DECISION
	AREA	ROOM	FLOOR	BASE BOARD	WALL	CEILING	REMARKS	OF COLOUR
		OFFICE ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT IRIS WHITE P 013	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		CONFERENCE ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT IRIS WHITE P 013	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		SUPERINTENDENT ROOM	CARPET HERINGTON RAINBOW H2	HARD WOOD H = 100 NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		RECEPTION ROOM	CARPET HERINGTON RAINBOW H2	HARD WOOD H = 100 NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		PANTRY ROOM	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	ACOUSTIC BOARD NATIONAL NEW NDF (WHITE)	STEEL DOOR JAMB : HONEY WOOD 662 DOOR : IVORY BS 4 - 052	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		SUPERMETTE	ASIA TILE 300 x 300 PICAZO GREY	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 001	NA	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		LAVATORY (MAN)	ASIA TILE 200 x 200 VENUS PINK	NA	ASIA TILE 200 x 200 ANABEL PINK	NIPPON PAINT BRILLIANT WHITE P 001	WOODEN DOOR NIPPON PAINT JAMB : MILKY GRAY 132 DOOR : IVORY (FORMICA)	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO
		LAVATORY (WOMAN)	ASIA TILE 200 x 200 VENUS PINK	NA	ASIA TILE 200 x 200 ANABEL PINK	NIPPON PAINT BRILLIANT WHITE P 001	WOODEN DOOR NIPPON PAINT JAMB : MILKY GRAY 132 DOOR : IVORY (FORMICA)	COLOR SELECTION IN BUKITTINGGI PLN PROPOSED BY LOT I CONTRACTOR / TEPSCO

Table 3.3.2 Color List of Powerhouse (5)

WORKS : FINISHING / CIVIL

NO.	APPLIED LOCATION		NAME OF MATERIAL AND COLOUR				
	AREA	ROOM	FLOOR	BASE BOARD	WALL	CEILING	REMARKS
14	SERVICE AREA EL.54.500	STOCK ROOM	STEEL TROWEL FINISH	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	GALVANIZED METAL EXPOSED	STEEL DOOR COLOR JAMB : HONEY WOOD BS DOOR : IVORY BS 4 - BS
		STORAGE ROOM	STEEL TROWEL FINISH	CEMENT PLASTER NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	GALVANIZED METAL EXPOSED	STEEL DOOR COLOR JAMB : HONEY WOOD BS DOOR : IVORY BS 4 - BS
15	CONTROL AREA EL.59.550	ELEVATOR MACHINE ROOM	STEEL TROWEL FINISH	NA	NIPPON PAINT ORCHID WHITE P 017	GALVANIZED METAL EXPOSED	STEEL DOOR COLOR JAMB : HONEY WOOD BS DOOR : IVORY BS 4 - BS
16	SERVICE AREA EL.59.550	STAIR CASE	ASIA TILE 300 x 300 PICAZO GREY WITH NON SLIP NOSING	NA	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 011	STEEL DOOR COLOR JAMB : HONEY WOOD BS DOOR : IVORY BS 4 - BS HANDRILL & STEEL PAINTED JOTUN PL01 C COLOR : GREY 7.5

Table 3.3.3 Color List of Switchyard Relay House

WORKS : FINISHING / CIVIL

NO.	APPLIED LOCATION		NAME OF MATERIAL AND COLOUR				
	AREA	ROOM	FLOOR	BASE BOARD	WALL	CEILING	REMARKS
1	EXTERIOR	-	-	-	NIPPON PAINT ENAMEL RESIN BEIGE NP3 - 035	-	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS
2	INTERIOR	RELAY ROOM	STEEL TROWEL FINISH HARDENER CHICIBU GREEN C - 7	NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 011	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS
		PCL ROOM	STEEL TROWEL FINISH HARDENER CHICIBU GREEN C - 7	NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 011	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS
		LAVATORY	ASIA TILE 200 x 200 VENUS PINK	NIPPON PAINT CONGO BROWN	ASIA TILE 200 x 200 VENUS PINK	NIPPON PAINT BRILLIANT WHITE P 011	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS
		BATTERY AND CHARGER ROOM	VINIL TILE " POLY FLOOR " WOOD SEIGE 2120	NIPPON PAINT CONGO BROWN	NIPPON PAINT OXSIDE GREEN	NIPPON PAINT BRILLIANT WHITE P 011	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS
		20 KVA CUBICLE ROOM	STEEL TROWEL FINISH HARDENER CHICIBU GREEN C - 7	NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 011	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS
		STORAGE ROOM	STEEL TROWEL FINISH HARDENER CHICIBU GREEN C - 7	NIPPON PAINT CONGO BROWN	NIPPON PAINT ORCHID WHITE P 017	NIPPON PAINT BRILLIANT WHITE P 011	DOOR : NIPPON PAINT BEE BRAND IVORY BS 4 - BS

PART 6

PART 4

FLOOD FORECASTING AND WARNING SYSTEM AND TELEMETERING SYSTEM

PART 4

PART 5

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Chapter 1 INTRODUCTION

1.1 Outline of the Works

The Flood Forecasting and Warning System and Telemetering System Works Contract Lot V was awarded on 4th June 1997 and concluded on 22nd August 1997 between PT. PLN (PERSERO) and the Consortium of PT. Gistec Prima Corporation, ELPRO Technology Pty. Ltd, and Greenspan Technology Pty. Ltd, under the Contract No. 041.PJP/922/1997/M.

The contract period was 12 months from 9th October, 1997 to 8th October, 1998 in original schedule. After the commencement of the work, the completion date of the work was revised to 10th March 1999 due to delay of Government approval for the radio frequency. The works was completed on 10th March 1999 for the original Contract. Continuously additional works mentioned later was commenced on 11th March 1999 and was to be completed by 30th September 1999. The overall schedule of the works is shown in Fig.4.1.1.

In order to carry out the construction work under the Contract, the Consortium made an association with the Hydro-Electric-Commission (HEC) of Tasmania for supplying hydrology, modeling software including water management and proving hydrological expertise for the Project.

The work was composed of the design, manufacturing, testing, finishing, painting, packing for shipping, shipping, unloading, clearing through customs, delivery to the site, building and civil works, erection and testing and commissioning of the Plant on site. The main works are divided into the following four (4) items:

(i) A flood forecasting and telemetering system for effective utilization of power and spillway gate operation.
(ii) A Warning system for ensuring the security of downstream river basin.
(iii) A dam data processing system for operation of powerhouse and effective utilization of power.
(iv) Civil construction works for the station house of telemetering gauging station, repeater station and warning station including electrical works.

1.2 Outline of System and Civil Works

(i) Flood Forecasting and Telemetering System

A telemetering system for the purpose of flood forecasting was installed to collect the necessary hydrology data, grasp the inflow volume to the reservoir and to control discharge of the dam reservoir. Rainfall data of the main geographical points in the catchment area and river water-level data are collected by radio links or wired.

A common telemetering system consists of gauging stations that observe the rainfall and water-level data and a master station that controls the gauging stations and collects the data. The data are transmitted between the stations through radio links and/or wired connection.

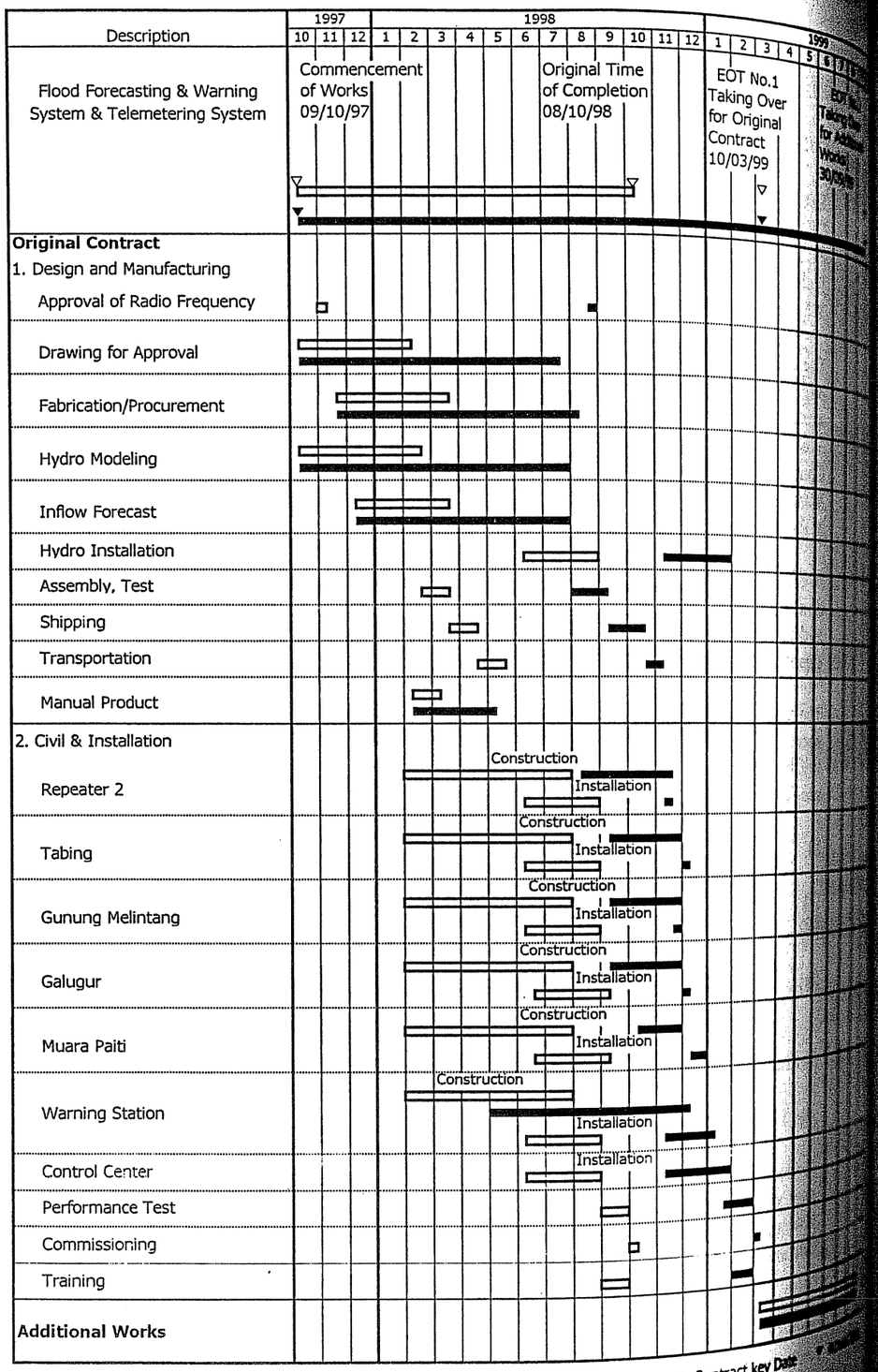


Fig. 4.1.1 Lot V Work Progress

The rainfall and water level gauging stations installed in the project catchment area can transmit data to the master station at the central room in the powerhouse via the repeater stations located Batu Putih and the dam site.

The telemetering system is a controlled observation type. The telemetering supervisory equipment in the central room can gather the accumulated rainfall and water-level data and print and display them. The gathered data output are sent to the dam data processing system. The telemetering equipment of the gauging station automatically transmit the observation data from rainfall gauges and/or water-level gauge when the calling signal is dispatched from the supervisory equipment in the central room.

(2) Warning System

In order to safely and smoothly control the dam the control equipment contains not only alarm and communication devices, but instrumental, monitoring, controlling and power devices which streamline dam control, and observation devices.

The flood warning system is in general indispensable for the control of dams and barrages. Warning issuance for spillway discharge can prevent the people from the accidents caused by a sudden rise of river. Dams with reservoir or pondage are commonly required to give effect to a warning when an excessive change of water flow occurs in the downstream of the dam.

To prevent accidents, this warning system informs the river flow condition of the residents in the downstream basin by siren and/or voice broadcast. This system is introduced and illustrated in Fig. 4.1.2.

The warning system was installed extending to a town of Bangkinang located about 20 km downstream of the dam and 50 guide board were also installed in this area. The warning station was provided with a control equipment in the central control room for controlling the warning operation.

When the gate operation is necessary due to the flood forecast by the flood forecasting and telemetering system and discharge increase, etc., an alarm will be issued using the downstream warning system 30 minutes before the start of discharge for the safety of residents living in the downstream area.

The master station controls the warning stations to perform warning operations such as siren sounding, artificial siren sounding and voice broadcasting. In addition, the master station prints out and displays the result of the operation based on the received response from the controlled warning stations.

When an order for the artificial sound broadcast is received in the warning station power is supplied to the amplifier in the station and a mixed tone of 523.5Hz and 526.5Hz is broadcast through the speaker. The pattern of the artificial sound broadcast is the same as that of the siren sounding.

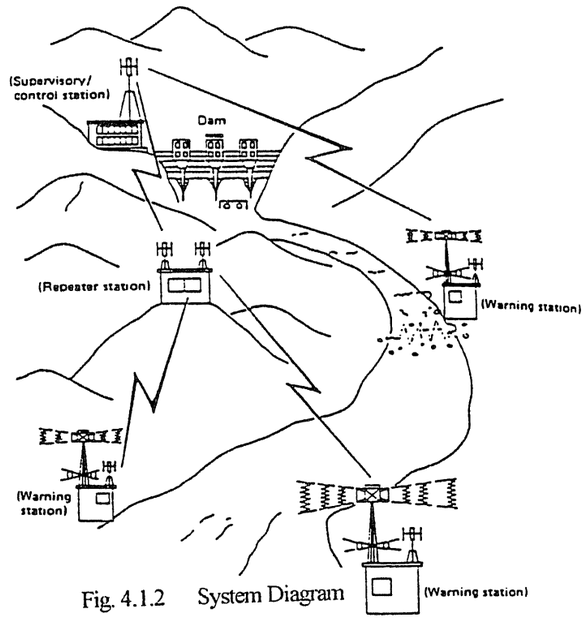


Fig. 4.1.2 System Diagram

(3) Dam Data Processing System

Dam data processing system consists of the data processing equipment, relay terminal equipment, CRT display equipment, printer, etc. It inputs the dam water level, tailrace water level spillway, gate opening (five gates), generating inflow, effective storage volume, hourly rainfall, cumulative rainfall etc. They are displayed on the CRT display, etc. and printed by printer.

In the dam site area, reservoir and tailrace water level gauge, rainfall gauge and meteorological measuring station were installed to send data to the central control room by means of wired connection. The reservoir and tailrace water level gauge were installed by the other contractor.

(a) System requirement for dam management

- The collection of hydraulic, hydrological and meteorological information.
- Data processing, flood forecasting and supervision necessary for making for flood control and water utilization.
- Making operational plans and judgment for determining time and amount of discharge.
- Advance warnings and announcements of discharge based on the operational plans.
- Observation and recording of conditions based on the operational plans.
- Data transfer of hydraulic and hydrological information to the office concerned.

(b) Relay terminal equipment

Protect the system from lightning surge by the electric isolation between control equipment and dam data processing equipment.

(c) CRT display equipment

- Display hydraulic and hydrological data and dam data with time in graphics and tables.
- Alarm indication
- Map of the river basin
- Forecast inflow and determine discharge volume
- Operating of displays, alarm indicatings and control operations for dam data processing
- File processing
- Print out processing

(d) Meteorological observation equipment

Meteorological observation equipment includes following items: i) Wind direction, ii) Wind speed, iii) Temperature, iv) Humidity, v) Atmospheric pressure, vi) Analog recording.

(4) Building and Civil Works

Building and civil works included design of the repeater stations, the gauging stations, the warning stations and warning boards, planning the equipment layout, covering the whole scope of supply and installation of the station. Details works are shown as follows:

a) Preparatory works: cleaning and site leveling.
b) Soil works: foundation, excavation, backfill, compacting and sand bedding underneath foundation and floor bedding.
c) Concrete works: concreting the building and site works complete with reinforcement steel, form work, finishing and related works.
d) Masonry works: the stone foundation, brick foundation, waterproof bricklaying, ordinary bricklaying, waterproof plastering and ordinary plastering.
e) Carpentry Works and hardwares: rough carpentry, finish carpentry, door panel, window frame and hinge, locking device foe door.
f) Roofing and ceiling works: roofing, roof ridge, gutter board and ceiling including frames.
g) Wall and floor finishing: ceramic tile, and cement mortar for wall and floor.
h) Metal work: steel structural frame, steel plate door and wire mesh, etc.
i) Glass and painting works: door and window panel varnish and glass for door window and wall paint, etc.
j) Station equipment: name plate, etc.
k) Fence works in the surrounding of the site and gate works were conducted based on the following specification.

Post	: Steel (2.00 m space)	Gate works	
Wire	: barbed wire (0.15 m space)	Gate	: Steel frame and wire net 2.00m
Height of fence	: 2.00 m	Height	: 2.00m

Chapter 2 PLANNING AND DESIGN

2.1 Original Design

The original plan of each station is shown in Table 4.2.1 and their arrangement in the river basin is shown in Fig 4.2.1.

(1) Gauging station

Two water level and rainfall gauging stations and two rainfall gauging stations were installed in the catchment area of the reservoir. These gauging stations are located at Gunung Melintang (35 km), Tanjung (35 km), Muara Paiti (37km), Galugur (61 km) respectively in a straight line via the Repeater stations from the central control room in the powerhouse.

Ten warning stations were installed along the river in the range of approximately 30 km from the dam site to Bangkinang as shown in Fig. 4.2.2. The gauging stations above-mentioned were linked with the Master station in the powerhouse by the radio link through Repeaters.

At the dam site, one rainfall station, one warning station and one meteorological measuring station were installed. These stations were linked with the Master station by wiring.

(2) Radio Link

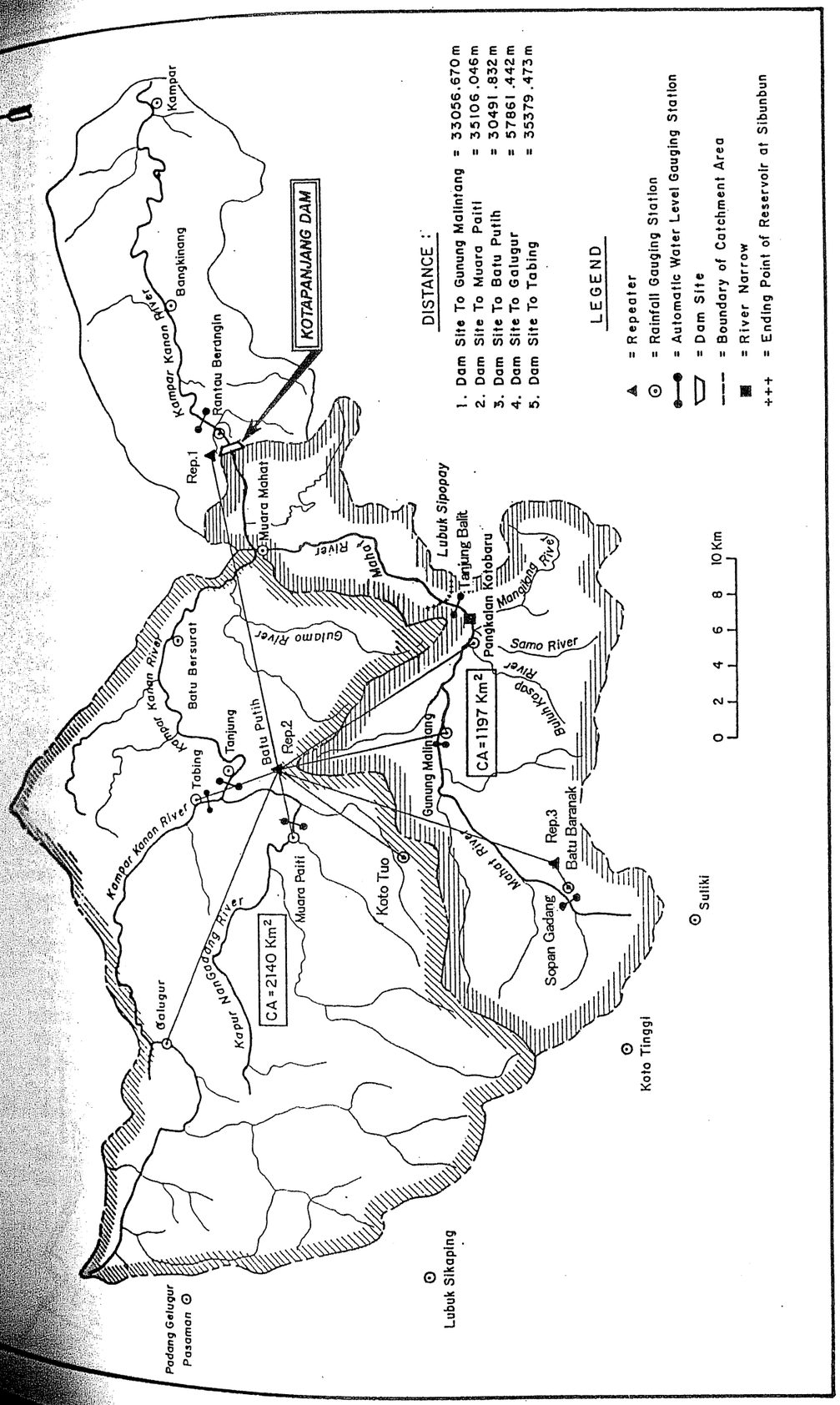
A Master station (Control center) was installed at the central control room.

One repeater station was installed near the dam site and two stations were installed in the river basin to link the Master station and gauging stations and warning stations. The location of these stations was decided based on the result of the desk survey. The desk survey was conducted using a map of 1/50,000 with contour, and total loss and signal level for each span were calculated taking account of the topographic profile. The function of the each Repeater station was described as follows.

A Repeater station was built at the upper side near the powerhouse in order to link the Master station to each telemetering gauging station and warning station through the Repeater stations for following reasons. The powerhouse is located in the valley (level of the powerhouse roof top: EL.63.2m) and surrounded by the steep mountains, consequently the Master station was not able to link them directly and/or through Repeater stations.

An already-existing telephone system repeater station house (GL.174m) with a 30m high guyed tower is located in front of the switchyard entrance and therefore the repeater station was planned to utilize for the Repeater station named Dam site repeater 1 in order to save the construction cost of new station.

Based on the desk survey result, two Repeater stations were required in the high mountain area chained to Batu Putih (EL.625m) in order to avoid obstructions and to link the Dam site repeater 1 to each telemetering gauging station. Therefore Repeater



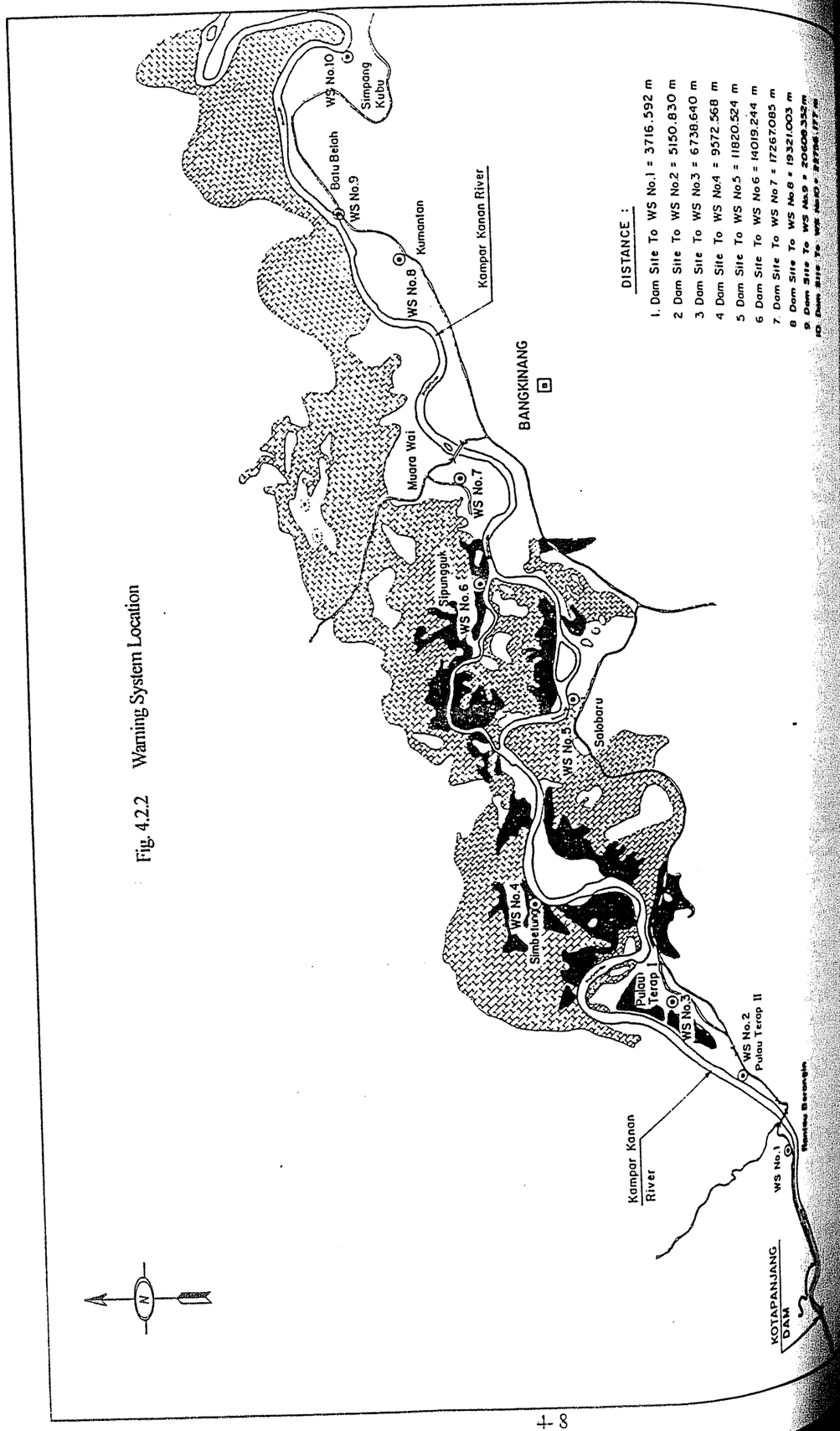


Fig. 4.2.2 Warning System Location

2 and Repeater 3 were planned to link Dam site repeater 1 to Galugur, Tanjung, Muara Paiti and Gunung Melintang gauging station respectively.

Table 4.2.1 Original and Modified Plan

		Original Plan		Modified Plan	
		Q'ty	Location	Q'ty	Location
Master Station (Control Center)		1	Central Control Room and Computer and Relay Room in the Powerhouse	1	Central Control Room and Computer and Relay Room in the Powerhouse
Gauging Station Rainfall	Radio station	4	Gunung Melintang, Tanjung, Muara Paiti, Galugus	4	Gunung Melintang, Tabing, Muara Paiti, Galugus
	Wired station	1	Dam	1	Dam
Water level	Radio station	2	Gunung Melintang, Tanjung	2	Gunung Melintang, Tabing, Muara Paiti
Warning Station	Radio station	10	Downstream	10	Downstream
	Wired station	1	Dam	1	Dam
Repeater Station		3	Repeater station (2), Dam site (1)	3	Repeater station (1), Dam site (1)
Meteorological Measuring station	Wired Station	1	Dam	1	Dam

2.2 Design Modification

After the site survey and the radio path propagation test were carried out, some design modification was made based on the site condition and the propagation test. The design modification are described in detail below and summarized in Table 4.2.1.

(1) Water Level Gauging Station

(a) Tanjung

Tanjung water level gauging station was originally planned at just downstream of the confluence of Kampar Kanan river and Kapurnan Gadang river which can gauge the total inflow to the reservoir from the both river. However the gauging station site was found to be unfit to observe the inflow accurately. Therefore the location of the water level gauging station was changed from Tanjung to Tabing which is located at approximately 7 km upstream of Tanjung site along the Kampar Kanan river.

The valley of the proposed gauging site forms very gentle slope especially on the right bank and was found to be topographically unfavourable for gauging station. Also the water level at the gauging site will be affected by the backwater caused by the reservoir.

(b) Tabing

Tabing site is suitable for the water level gauging based on the following site condition and also there is no influence by the backwater.

- Riverbed gradient is sufficient for development of accurate rating curve.
- Flood flows will be contained within the river channel.
- Down stream control (gravel bar) appears stable.
- Radio path fade margin is acceptable.
- Station house will be free from the flood flow.
- Float well will be required to be built close to left bank (main channel) and station house will be located on the top of the left bank out of the flood range.
- Main channel is close to the left bank and hence low water flow measurement in dry season will also be able to be done.
- Additional distance of station upstream from Tanjung equates to greater time for flood warning.
- Accuracy of hydraulic model will be improved as a result of station being located at Tabing.

On the other hand, the Tabing water level gauging site is not able to gauge the inflow from a large tributary Kapurnan Gadang river because the gauging site is located upstream of the confluence of the Kampar Kanan river and the Kapurnan Gadang river. An additional water level gauging site is required for measurement the inflow from the Kapurnan Gadang river.

(c) Additional water gauging station

In original planning, a rainfall gauging station was planned to install at Muara Paiti in the basin of the Kapurnan Gadang river. An already-existing water level gauging station was situated at Muara Paiti and considered to be the best site for measuring water level and rainfall along the river. Accordingly, an additional water level gauge especially for Kotapanjang project was newly installed at Muara Paiti combined with a rainfall gauge station to save the cost of the station house facilities.

- A new additional water level gauge located on the same property as the existing project station.
- A station house on the top of right bank
- Minimal interference to the rubber plantation to be achieved by confining removal of rubber tree to immediate station surrounds.

(2) Omission of Repeater 3

Based on the results of the radio path survey, it was confirmed that Gunung Malintang gauging station was able to link the Dam site repeater 1 via the Repeater 2 with sufficient signal level. Therefore the Repeater station 3 was omitted.

Modified configuration and radio link diagrams are shown in Fig.4.2.3.

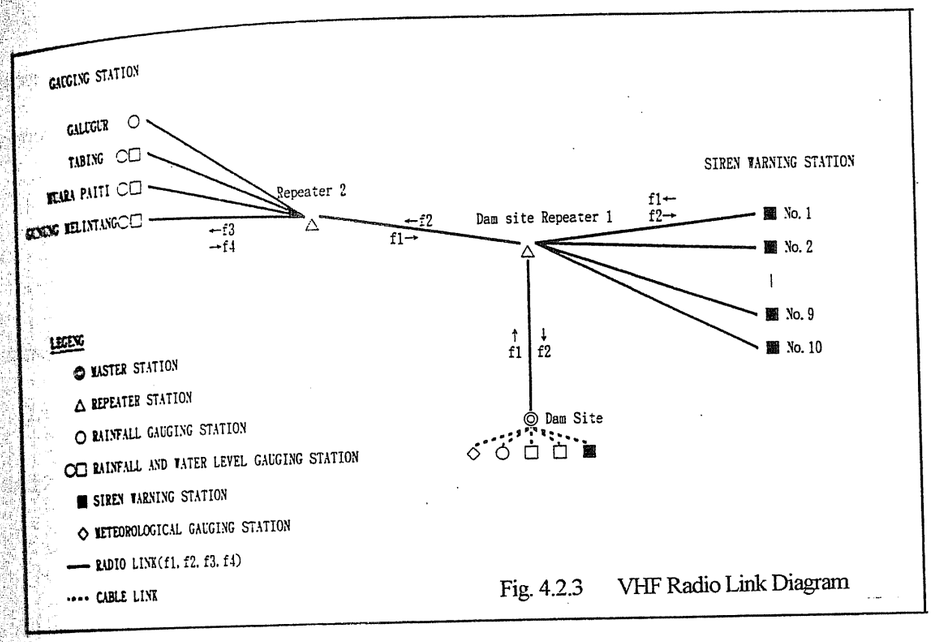


Fig. 4.2.3 VHF Radio Link Diagram

(3) Radio Frequency of this system

70 MHz band was originally selected to use for this system because 70 MHz propagation characteristics are better than other frequency bands in the mountain area for telemetering. However, based on the radio path propagation, the radio frequency for this system was changed from 70 MHz to 150 MHz by the following reasons.

- The Military Force is using 70 MHz band in this area. Therefore there are some possibilities for interference. After the installation of this system, if the interference happen, the countermeasure will be difficult and expensive.
- There is no large difference between 70 MHz to keep the stable radio link.
- High gain antenna is available for 150 MHz and can recover the signal level.

2.3 Site Selection

Site survey and radio path propagation test were carried out to check up on the site condition and the design value. The site selection was carried out based on the following procedure.

(1) Preparatory Work

Examination of 1/50,000 topographic maps was undertaken in order to: i) assess the catchment topographic features, ii) select provisional locations for warning stations, repeater stations and gauging stations, iii) conduct desktop radio path survey based on items i) and ii), adjust provisional locations based on results of iii).

(2) Site Survey

The following procedure for final site selection was employed during the site survey.

1) Site identified the preparatory work (as marked on the 1/50,000 topographic maps) was physically identified during the site survey.

2) Site was selected based on the following criteria:

Warning Stations

- Accessibility
- Social impact (for examples: consideration regarding proximity of warning station to dwelling places)
- Distance from the 20kV power lines (in the case of the warning stations)
- Ease of construction
- Concurrence of the local village head and local inhabitants regarding possession of the site
- Radio path test results confirming an acceptable fade margin.

Repeater Stations (2 and 3)

- Accessibility
- Ease of construction
- Concurrence of the local village head and local inhabitants regarding possession of the site
- Radio path test results confirming an adequate for the path Repeater 1 to Repeater 2 and Repeater 1 to Repeater 3
- Vegetation density (with regard to sunlight penetration for solar charging)

Gauging Stations

- Accessibility
- Hydrological consideration
  - Location (if any) of a suitable control for discharge rating purposes
  - Bank and stream bed stability
  - Maximum flood range
  - Availability of historical data (for correlation purposes)
  - Impact on Hydraulic modelling
- Ease of construction
- Concurrence of the local village head and local inhabitants regarding possession of the site
- Radio path test results confirming an acceptable fade margin
- Vegetation density – solar power considerations

(3) Site Locations

The longitude and latitude of each station selected are shown in the following table.

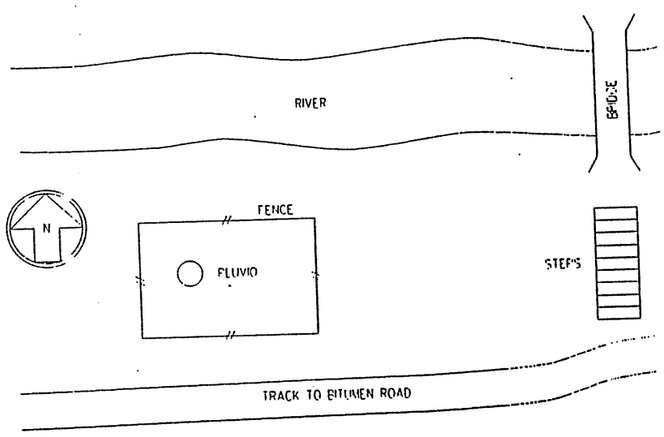
Longitude and Latitude of Each Station

Site	Latitude	Longitude
Dam Site	0°17'29"	100°52'53"
Warning Station 1	0°17'55"	100°54'31"
Warning Station 2	0°18'55"	100°54'19"
Warning Station 3	0°19'26"	100°55'47"
Warning Station 4	0°20'22"	100°56'51"
Warning Station 5	0°20'58"	100°58'28"
Warning Station 6	0°19'53"	100°58'29"
Warning Station 7	0°20'44"	100°59'42"
Warning Station 8	0°21'10"	100°01'08"
Warning Station 9	0°21'48"	100°02'54"
Warning Station 10	0°21'40"	100°04'18"
Repeater 1	0°17'36"	100°52'50"
Repeater 2	0°15'12"	100°36'34"
Repeater 3	0°08'41"	100°44'09"
Muara Paiti	0°14'31"	100°32'41"
Gunung Melintang	0°07'08"	100°38'09"
Tabing	0°19'53"	100°33'50"
Galugur	0°21'24"	100°21'53"

Note : Repeater 3 was omitted.

The typical site data in all stations mentioned above is shown in the following figure.

SITE NAME: Gunungmelintang



GRID CO-ORDINATES

NORTHING:	013133	RAYLINE:	350°T
EASTING:	682056	DISTANCE:	15km to Rptr 2
ZONE:	47	FREQUENCY:	150MHz 70MHz
MEASURED PATH LOSS:	137dB Rptr 2 120dB Rptr3	FREE SPACE LOSS:	95dB 92dB

MAP NAME/ No: Pangkalan Kotobaru 0816-11

POWER AVAILABILITY: Nil - Good Solar Clearance

ACCESSABILITY

Bitumen Road - 200m walk up steps and along level ground

FOREGROUND INSTRUCTIONS: Nil

AERIAL MOUNTING REQUIREMENTS: New Site

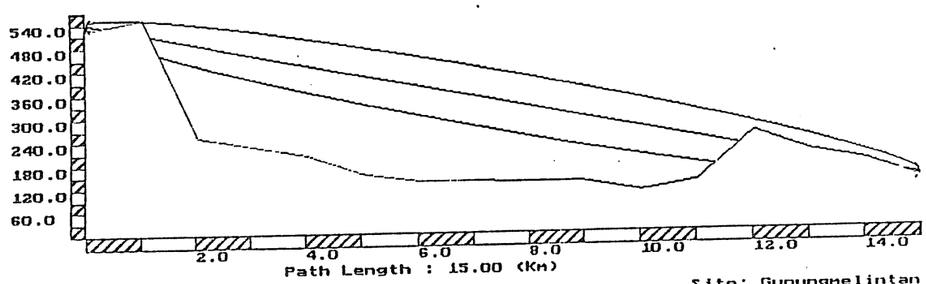
EQUIPMENT MOUNTING REQUIREMENTS: New Site

COMMENTS

NAME: DATE:

Path Profiles at 70 MHz

Fresnel Zone is 99.09m below the terrain.



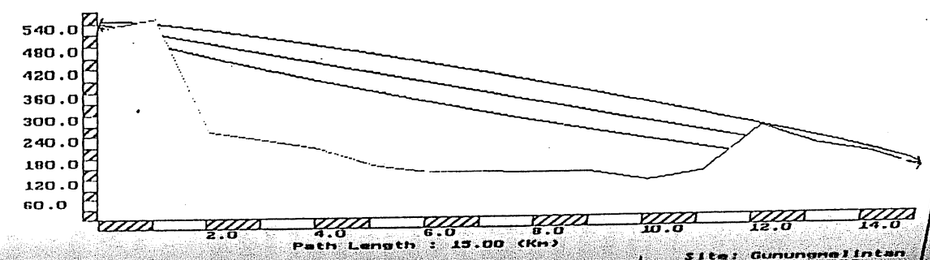
Site: Rptr 2
Lat : 000°00'00?
Long: 000°00'00?
Elev: 525.00
Hgt : 15.00

Site: Gunungmelintan
Lat : 000°00'00?
Long: 000°00'00?
Elev: 126.39
Hgt : 6.00

Project Name. Kotopanjang	Path 122dB
Link Name. Rptr 2 - Gunungmel	Date: 23-09-97

Path Profiles at 150 MHz

Fresnel Zone is 79.84m below the terrain.



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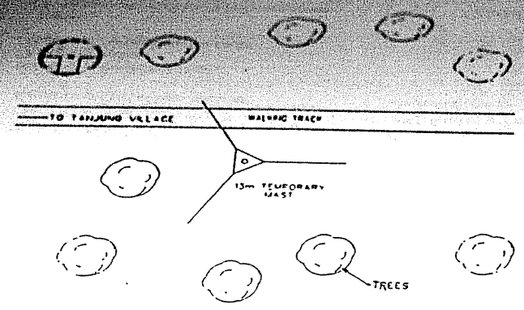


GRID CO-ORDINATES

NORTHING:	028019	RAYLINE:	N/A Omnidirectional
EASTING:	679122	DISTANCE:	29km to Rptr 1
ZONE:	47	FREQUENCY:	150MHz 70MHz
MAP NAME/ No:	Kuok 0816-14	FREE SPACE LOSS:	105dB 98dB

MEASURED PATH LOSS: 133dB to Rptr 1

POWER AVAILABILITY: Nil

ACCESSABILITY

4WD to Tanjung Village

4 hour walking track to site

FOREGROUND INSTRUCTIONS: Batuputim peak SW direction

AERIAL MOUNTING REQUIREMENTS: New Mast required.

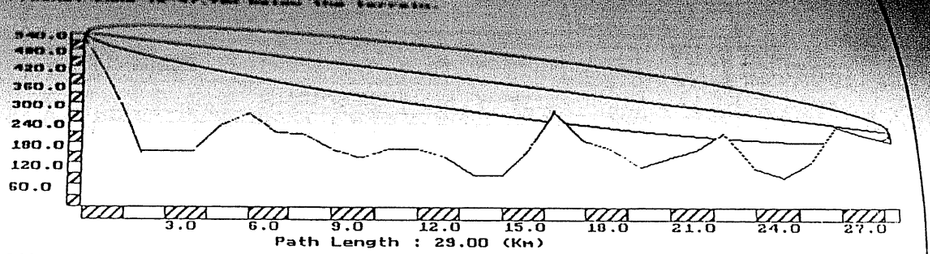
EQUIPMENT MOUNTING REQUIREMENTS: New Building required

.COMMENTS

15m trees on site to be cleared for mast and solar

Could not get to the peak because no track through the jungle

NAME: B. Anderson DATE: 29/08/97



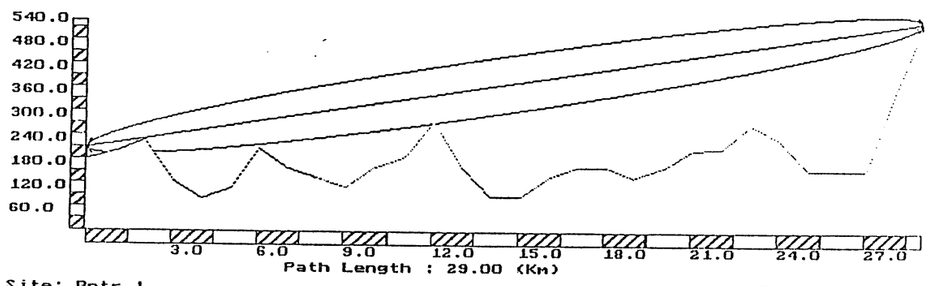
Site: Rptr 2
Lat : 000°00'00?
Long: 000°00'00?
Elev: 525.00
Hgt : 15.00

Site: Rptr 1
Lat : 000°00'00?
Long: 000°00'00?
Elev: 181.28
Hgt : 30.00

Project Name. Kotopanjang	Path 119dB
Link Name. Rptr 1 - Rptr 2	Date: 23-09-97

Path Profiles at 150 MHz

Fresnel Zone is 32.68m below the terrain.



Site: Rptr 1
Lat : 000°00'00?
Long: 000°00'00?
Elev: 180.00
Hgt : 30.00

Site: Rptr 2
Lat : 000°00'00?
Long: 000°00'00?
Elev: 514.63
Hgt : 15.00

Project Name. Kotopanjang	Path 117dB
Link Name. Rptr 1 - Rptr 2	Date: 23-09-97

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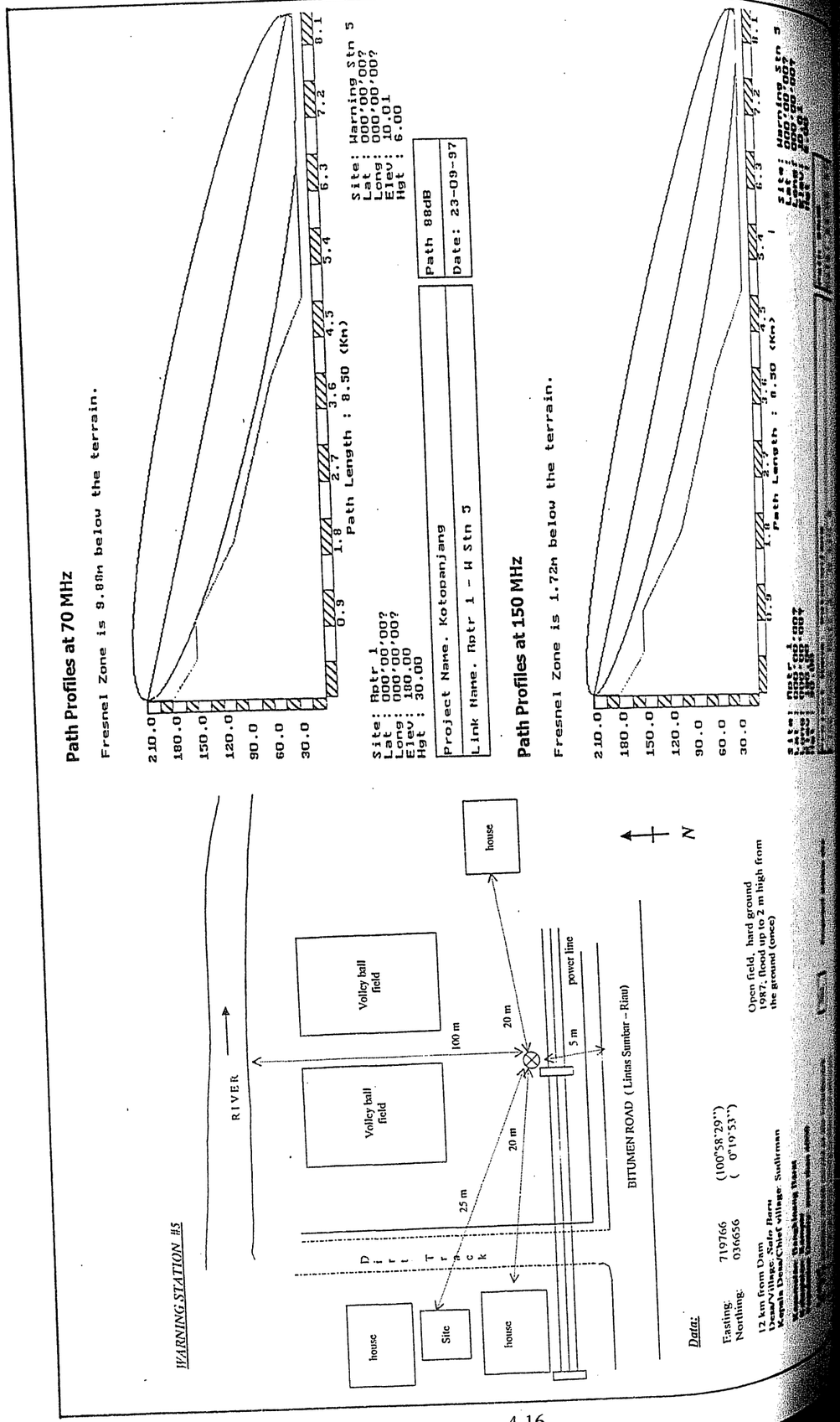
Fig. 4.2.4 (2) Site Data and Path Profile for One of Repeaters



24 Radio Path Propagation

Radio path propagation test had been carried out both 70MHz and 150MHz. Result of measured signal level of 70MHz and 150MHz are shown in Table 4.2.2.

Table 4.2.2 Comparison of 70MHz and 150MHz Measured Signal Level

No	Measured Site	Repeater Site	70MHZ Measured (dBm)	150MHZ Measured (dBm)	Difference 150MHz used (dB)	Receiving volt 150MHz (dB/μV)
1	Master Station	Repeater 1	-55	-60	-5	53
2	Warning Station 1	Repeater 1	-55	-60	-5	53
3	Warning Station 2	Repeater 1	-58	-62	-4	51
4	Warning Station 3	Repeater 1	-78	-88	-10	25
5	Warning Station 4	Repeater 1	-84	-87	-3	26
6	Warning Station 5	Repeater 1	-87	-90	-3	23
7	Warning Station 6	Repeater 1	-72	-76	-4	37
8	Warning Station 7	Repeater 1	-84	-86	-2	28
9	Warning Station 8	Repeater 1	-85	-88	-3	25
10	Warning Station 9	Repeater 1	-89	-92	-3	21
11	Warning Station 10	Repeater 1	-96	-96	0	17
12	Warning Station 11	Repeater 1	-89	-90	-1	23
13	Repeater 2	Repeater 1	-94	-92	+2	21
14	Repeater 3	Repeater 1	-104	-104	0	9
15	Tabing	Repeater 2	-79	-83	-4	34
16	Muara Paiti	Repeater 2	-56	-60	-4	53
17	Galugur	Repeater 2	-106	-105	-2	8
18	G. Melintang	Repeater 2	-93	-98	-5	15
19	G. Melintang	Repeater 3	-82	-79	+4	34

Used Equipment for survey: 10 Watt Test Transponder, 3 dB Collinear Antenna, 10 Watt interrogator, 0.5 μV Receiver sensitivity (-114 dBm) and RG-213 Coaxial cable.

2.5 Technical Particular

2.5.1 Telemetry System and Warning System

(1) Telemetry Supervisory Equipment

(a)	Name of the Manufacturer	ELPRO Technologies Pty.Ltd. Windsor, Queensland, Australia
(b)	Type	Self-standing cabinet or wall- Mounting cabinet
(c)	Capacity of station calling	250
(d)	Calling system	automatic/manual
(e)	Calling time intervals	Adjustable for each station 1 to 1440 minutes
(f)	Calling signal system	HDLC ELPRO protocol
(g)	Data code system	NRZI code
(h)	Transmission speed	300
(i)	Modulation system	Subcarrier frequency shift
(j)	Power requirement:	
	- Nominal voltage	AC220V, 50Hz or DC12V
	- Power consumption	30VA
(k)	Dimensions	Self-standing 1400 x 600 x 400 Wall mounting 1000 x 600 x 300
(l)	Weight	Self-standing 65 Wall mounting 50

(2) Warning Supervisory/Control Equipment

(a)	Name of the Manufacturer	ELPRO Technologies Pty.Ltd. Windsor, Queensland, Australia
(b)	Type	To be installed into the same cabinet as Telemetry Supervisory Equipment
(c)	Capacity of station controlling	250
(d)	Communication system	Semi-duplex communication
(e)	Control signal system	HDLC ELPRO protocol
(f)	Transmission speed	300
(g)	Modulation system	Subcarrier frequency shift
(h)	Power requirement:	
	- Nominal voltage	AC220V, 50Hz or DC12V
	- Power consumption	30VA
(i)	Dimensions	To be installed into the same cabinet as Telemetry Supervisory Equipment

(3) Operating Console for Telemetry System and Warning System

(a)	Name of the Manufacturer	ELPRO Technologies Pty.Ltd. Windsor, Queensland, Australia
(b)	Type	Self-standing console type
(c)	Capacity of station calling (For telemetry system)	} Total 250 stations
(d)	Capacity of station controlling (For warning system)	
(e)	Display for telemetry	Digital indication of measured data, receiving signals failure, status of repeater station, power supply voltage drop for gauging station, status displays for each warning station, other necessary display
(f)	Power requirement:	
	- Nominal voltage	AC220V
	- Power consumption	400VA
(g)	Dimensions	To match existing consoles
(h)	Weight	Approx 100

(4) Printer for Telemetry System

Micro line 590

(a)	Name of the Manufacturer	OKI, Japan
(b)	Printing System	Impact dot matrix
(c)	Number of printing head wire	24 lines
(d)	Diameter of printing head wire	0.2 mm
(e)	Printing speed (elite)	450 cps max. chr/s
(f)	Dimensions	430 x 378x 165 (W x D x H) mm
(g)	Weight	7.5 kg

(5) Printer for Warning System

Micro line 590 Turbo

(a)	Name of the Manufacturer	OKI, Japan
(b)	Printing System	Impact dot matrix
(c)	Number of printing head wire	24 lines
(d)	Diameter of printing head wire	0.2 mm
(e)	Printing speed (elite)	450 cps max. chr/s
(f)	Dimensions	430 x 378x 165 (W x D x H) mm
(g)	Weight	7.5 kg

(6) Master Clock

(a)	Name of the Manufacturer	Compaq Computer Corp. USA
(b)	Type	Proliant 1500
(c)	Accuracy	0.7 sec/day
(d)	Output signal	TTL Pulse
(e)	Number of Output	Server internal clock
(f)	Power source	
	- Nominal	C 5 V with battery back up v
(g)	Dimensions	TBA mm
(h)	Weight	TBA kg

(7) Repeater Equipment

(a)	Name of the Manufacturer	ELPRO Technologies Pty.Ltd. Windsor, Queensland, Australia
(b)	Type	IP66 Moisture reproof Self-standing cabinet or wall-mounting cabinet
(c)	Control Item	HDLC ELPRO protocol
(d)	Status return signal system	NRZI code
(e)	Items of status return	Repeater working, Tx No.1 working, Tx No.2 working, Tx. No.1 fail, Tx. No.2 fail, Rx. No.1 fail, Rx. No.2 fail, Mains supply fail, Battery voltage low
(f)	Power consumption	
	- Stand by	100 mA
	- Receiving	0.42 A
	- Transmitting	65.1 A
(g)	Power source	
	- Nominal	12 V
	- Ratings – guaranteed range	10.8 – 16 V
	- Operating range	10.8 – 16 V
(h)	Dimensions	Self-standing 1000 x 600 x 400 mm
(i)	Weight	Wall mounting 800 x 600 x 300 Self-standing 80 kg Wall mounting 70

(8) Telemetry Equipment

(a)	Name of the Manufacturer	ELPRO Technologies Pty.Ltd. Windsor, Queensland, Australia
(b)	Type	IP66 Moisture reproof wall-mounting cabinet
(c)	Capacity of measurement/station	32
(d)	Telemetering signal	HDLC ELPRO protocol
(e)	Transmission speed	300 bps
(f)	Power consumption	

	- Stand by - Receiving - Transmitting	450	mA
(g)	Power source - Nominal - Ratings – guaranteed range - Operating range	3.5 DC 12 10.8 – 16 11.2 – 15	A V V V
(h)	Dimensions	400 x 600 x 300	mm
(i)	Weight	20	kg
(9) Rain Gauge			
(a)	Name of the Manufacturer	Hydrological Services Australia	
(b)	Type	Tipping Bucket Rain Gauge 3	
(c)	Inlet of rainfall collector	200	mm
(d)	Measuring unit	1.0	mm
(e)	Accuracy	+/- 2% 25 - 500 mm/hr	mm/100m m/Hr
(f)	Dimensions	342 height	mm
(g)	Weight	3	kg
(10) Automatic Rainfall Recorder			
(a)	Name of the Manufacturer	Hydrological Services Australia	
(b)	Type	AUSI/R	
(c)	Operation period, maximum	90	days
(d)	Chart width	250	mm
(e)	Chart speed	6	mm/Hr
(f)	Clock: - Type - Accuracy	Battery ± 3mm	
(g)	Recording pen shift	1 pulse/mm	
(h)	Accuracy	± 3mm	
(i)	Dimensions	280(H) x 560(W) x 318 (D)	
(j)	Weight	22	kg
(11) Water Level Gauge			
(a)	Name of the Manufacturer	Hydrological Services Australia	
(b)	Type	Float	
(c)	Size of float	200	mm
(d)	Measuring range	up to 10	m
(e)	Accuracy	± 1	cm
(f)	Recording method	Pen	
(g)	Operation period, maximum	90	days
(h)	Chart width	250	mm
(i)	Chart speed	6	mm/Hr
(j)	Clock: - Type - Accuracy	Battery 5	
(k)	Dimensions	280 x 560 x 318 (H x W x D)	
(l)	Weight	22	kg
(12) Warning Equipment			
(a)	Name of the Manufacture	ELPRO Technologies Pty.Ltd. Windsor, Queensland, Australia	
(b)	Type	IP66 Moisture reproof Self-standing cabinet or wall-mounting cabinet	
(c)	Communication System	Semi-duplex communication	
(d)	Control signal System	HDLC ELPRO protocol	
(e)	Confirmation System	Serial return of audible tone and confirmation signal	
(f)	Response signal length	audible tone 5 ± 0.1 sec.	

		confirmation signal subcarrier frequency shift	
(g)	Confirmation signal system	300	bps
(h)	Transmission speed		
(i)	Power consumption: (excluding amplifier unit) - Stand by - Receiving - Transmitting	450 3.5	mA A
(j)	Power source: - Nominal - Ratings-guaranteed range - Operating range	C 12 10.8 – 16 11.5 – 15	V V V
(k)	Power requirement of amplifier unit - Input voltage - Power consumption	24 <10A	V A
(l)	Output amplifier unit	2 x 100 W	W/unit
(m)	Frequency characteristic of amplifier unit	150 Hz to 6000 Hz	
(n)	Dimensions	Self-standing 1000 x 600 x 400 Wall mounting 800 x 600 x 300	mm
(o)	Weight	Self-standing 60 Wall mounting 50	kg
(p)	Weight	22	
(13) Motor Siren			
(a)	Name of the Manufacture	GRIFFO, Australia	
(b)	Type	999	
(c)	Operating voltage	380 VAC/50Hz	V
(d)	Capacity	3.0	kW
(e)	Sounding radius	Up to 8 km	km
(f)	Dimensions	430 (H) x 580 (L) x 370 (Dia)	mm
(g)	Weight	65	kg
(14) Siren Control Board			
(a)	Name of the Manufacture	Midcoast Switchboards Australia	
(b)	Type	Wall mount enclosure	
(c)	Dimensions	1600 x 1600	mm
(d)	Weight	TBA	kg
(15) Loudspeaker			
(a)	Name of the Manufacture	TOA corp., Japan	
(b)	Type	TH-60UL/TU50M/YS151S Dual Driver Water Proof Reflex Horn	
(c)	Output power/set	100W/unit (50W x 2)	w
(d)	Nominal impedance (voice coil)	8	ohm
(e)	Nominal impedance (matching transformer)	8	ohm
(f)	Output level	110 dB W/1M	dB
(g)	Frequency range	150 to 6000	Hz
(h)	Transmission speed	300	bps
(i)	Weight	TBA	kg
(16) Speaker Junction Box			
(a)	Name of the Manufacture	Midcoast Switchboards Australia	
(b)	Type	304 SS Enclosure	
(c)	Dimensions	600 x 450 x 150	mm
(d)	Weight	12	kg

(17) Sound Collector		
(a) Name of the Manufacture	TOA corp., Japan	
(b) Type	TC151TE Water-proof Reflex horn	
(c) Nominal impedance	600	ohm
(d) Nominal sensitivity	45	dB
(e) Frequency range	450 to 3500	Hz
(f) Dimensions	TBA	Mm
(g) Weight	TBA	Kg
(18) Isolation Transformer		
(a) Name of the Manufacture	Star Delta, Australia	
(b) Type	IKO-220-220 (Enclosed)	
(c) Rating	Continuous	
(d) Phase	Single	
(e) Frequency	50	Hz
(f) Voltage	220	V
(g) Capacity	1	kVA
(h) AC withstand voltage	To AS 3108 Standard	
(i) Impulse withstand voltage	To AS 3108 Standard	
(j) Insulation resistance	300 M ohm or more with DC 1000 V	
(k) Dimensions	172 (L) x 147 (D) x 258 (H) mm	
(l) Weight	18	Kg
(19) DC Power Supply Equipment		
- Charge:		
(a) Name of the Manufacture	Antex Electrocnics, Australia	
(b) Type	ZX Charger	ISO 9002
(c) AC Input		
- Phase	Single	
- Voltage	240	V
- Frequency	50	Hz
(d) DC output		
- Voltage rating	13.4	V
- Voltage adjustment range	+/-4%	
- Current rating	18/50	A
(e) Load		
- Voltage	13.5	V
- Current rating	10/40	A
- Current variation range	18/50	A
(f) Dimensions	300 x 190 x 63	mm
(g) Weight	2	kg
- Battery		
(a) Name of the Manufacture	BP Solar, Australia	
(b) Type	Maintenance Free Lead Acid	
(c) Nominal Voltage	12	V
(d) Rated Capacity	120 for Repeater Station 240 for Warning Station	AH
(e) Dimensions	320 (L) x 174 (W) x 196 (H) mm	
(f) Weight	26.5	kg
(20) Solar Cells Supply equipment		
- Solar Cells:		
(a) Name of the Manufacture	BP Solar, Australia	
(b) Type	BP 160 – 60	
(c) Type of cells	Silicon solar cells	
(d) Output characteristic (at 1000w/m^2, 25^{0}C AM 1.5)		
Max. Output capacity	75	W
Max. Output current	4.4	A
Max. Output voltage	17.1	V

(e) Dimensions	1180 (L) x 530 (W) x 38 (D) mm	
(f) Weight	8	kg
- Power Distribution Board		
(a) Name of the Manufacture	BP Solar, Australia	
(b) Type	Moisture proof wall mounting Type FBR240	
(c) Measuring range for each measuring item		
- Output Current of solar cells	0 – 40	A
- Output Voltage of solar cells	0 – 18	V
- Load current	0 – 100	A
- Battery voltage	0 – 16	V
(d) Dimensions	155 (H) x 140 (W) x (83 (D) mm	
(e) Weight	0.6	kg
- Battery:		
(a) Name of the Manufacture	BP Solar, Australia	
(b) Type	Maintenance free lead acid	
(c) Nominal voltage	12	V
(d) Rates capacity	120	AH
(e) Dimensions	320 (L) x 174 (W) x 196 (H) mm	
(f) Weight	26.5	kg
(21) Radio Equipment		
(a) Name of the Manufacture	Tait Electronics, New Zealand	
(b) Type	Incorporated into telemetry and warning system cabinets	
(c) Transmitter and power amplifier		
- Type of emission	F2D/F3E	
- Rated output	adjustable 5 – 25	W
- Frequency band	150	HHZ band
- Antenna Impedance	50	ohm
- Modulation System	Frequency modulation	
- Frequency stability	5 x 10^{-6} (5 ppm)	
- Maximum frequency deviation	± 5	KHz
- Occupied bandwidth	15	Khz
- Dimensions	150 x 150 x 45	mm
- Weight	1.2	kg
(d) Receiver		
- Frequency band	150	MHZ band
- Antenna Impedance	50	ohm
- Receiving system	superheterodyne	
- Selectivity	<25 kHz @ -70dB	
- Sensitivity	<3dB/µV (20dB NQ) >-113Db @ 20dB SINAD	
- Dimensions - Weight	included in transmitter unit	
(22) Coaxial Arrester		
(a) Name of the Manufacture	Greenspar, UK	
(b) Type	Feeder type	
(c) Impedance	50	ohm
(d) Frequency range	DC blocked – 600	MHZ
(e) Insertion loss	0.1	dB
(f) VSWR	1.1	
(g) Dimensions	65 x 25 x 25	mm
(h) Weight	0.12	kg
(23) Antenna		
- 6 – element Yagi antenna (broad type)		
(a) Name of the Manufacture	RF Industries, Australia	
(b) Frequency band	150	MHZ band
(c) Gain	11	dB

(d) Impedance	50	ohm
(e) VSWR	< 1.5	
Polarization	Vertical	
(g) Dimensions	5000 x 2100 x 200	mm
(h) Weight	8	kg
- Sleeve antenna		
(a) Name of the Manufacture	RF Industries, Australia	
(b) Frequency band	150	MHZ
(c) Gain	2	dB
(d) Impedance	50	ohm
(e) VSWR	< 1.5	
(f) Polarization	Vertical	
(g) Dimensions	3000 x 25 x 25	mm
(h) Weight	1	kg
(24) Coaxial cable with connectors (for telemetry system)		
(a) Name of the Manufacture	Belden Wire & cable, USA	
(b) Frequency band	10 – 1000	MHZ
(c) Attenuation at 70 MHZ	40	dB/km
(d) Impedance	50	ohm
(25) Cable Protector		
- for Master Station		
(a) Name of the Manufacture	ELPRO Technologies Pty. Ltd., Australia	
(b) Frequency band	0 – 500	kHz
(c) Transmission frequency band 0-500		kHz
(d) Impulse withstand voltage	10 kV (1.2 x 50 µsec)	
(e) Equipped line capacity	1 line/ 1 line	
(f) Dimensions	75 x 40 x 50	mm
(g) Weight	0.1	kg
- for Warning Station		
(a) Name of the Manufacturer	ELPRO Technologies Pty. Ltd., Australia	
(b) Frequency band	3 stage gas discharge	kHz
(c) Transmission frequency band	0 - 500	
(d) Impulse withstand voltage	10 kV (1.2 x 50 µsec)	
(e) Equipped line capacity	1 line/ 1 line	
(f) Dimensions	75 x 40 x 50	mm
(g) Weight	0.1	kg

2.5.2 Dam Data Processing System

(1) Data Processing Equipment		
(a) Name of the Manufacturer	Compaq Computer Corp. USA	
(b) Type	Prosignia 200PII/233-1	
(c) Power requirement		
- Nominal voltage	220 – 240	V
- Power caption	325 typical	VA
(d) Functions and ratings		
- Memory	Error Correction Memory	
Element	32	Bit
Word length	64 MB (expandable to 256 MB)	Mbyte
Capacity	Intel Pentium Processor 233 MHZ	
- Microcessor unit (MPU)	488	
Number of instruction	8	
General registers	16	
Index register	64 bit Data bus	
- I/O Operation	Up to 256 RTU per system serial port.	
Number of I/O devices		

Transfer rate	Readily support 2 serial ports and optimally 4 serial ports	
	Support various industry standards: MB/s	
	Per SCSI channel – 20 MB/s	
	PCI Bus – 133 MB/s	
	EISA Bus – 33 MB/s	
- Interrupt level	16	
- Error detection	Citec Software watchdog control	
- Error handling	Up to 16 CPU server clusters are Supported by Windows NT	
- Multi – MPU configuration	Up to 16 CPU server clusters are Supported by Windows NT	
Connection	Flex SMP System Architecture	
Exclusive – processing	Standard	
- Unit mounting capacity	5 Drive Bays for hot – luggable drivers	
	3 Drive Bays for removable media drivers	
	5 EISA clots, 2 PCI slots, I EISA/PCI solar (shared)	
(c) Dimensions	655.3 x 410.2 x 569.0	mm
(2) Relay Terminal		
(a) Name of the Manufacturer	ELPRO Technologies Pty. Ltd., Windsor, Queensland, Australia	
(b) Type	PLC	
(c) Power requirement		
Nominal voltage	AC 220	V
Power consumption	100	VA
(d) Functions and ratings		
- Input data		
Generating discharge	5 BCD	digits
Generating output	5 BCD	digits
Generating unit status	6 status	points
Gate opening	8 x 2 analogue	points
Gate open/close status	8 x 3 status	points
Dam water level	4 BCD or 16 bit analogue 4 - 20	digits mA
- Output data		
Dam water level	4 BCD	digits
Effective storage volume	4 BCD	
Gate Discharge	5 x 4 BCD	digits
Inflow	4 BCD	digits
- Telemeter Interface		
Input interface	Relay coil 0.015 A 24 V	
Output interface	Relay contact 2 A 24 V	
Data format	RS232C	
Bit signal	1 (contact close)/0 (contact open)	
Terminal condition	Common voltage/ground	
- Digital interface		
Input interface	12 V to G switch input	
Bit signal	1 (contact close)/0 (contact open)	
Terminal condition	Common voltage/ground	
- Gate opening interface		
Input interface	4 – 20 mA DC	
	0 –12m/0-1m	
Characteristic curve	Linear	
- Arrester element		
Discharge starting voltage	DC 440 ± 15%	V
Nominal discharge current	6000	A
(e) Dimensions	600 x 400 x 240	mm
(f) Weight	7	kg

(3) CRT Display Equipment

(a)	Name of the Manufacturer	View Sonic Corp., USA
(b)	Frequency band	17 PS
(c)	Power requirement	
	- Nominal voltage	220
	- Power consumption	150
(d)	Functions and ratings	
-	Characteristics	
	Viewer	Controlled by Software (Microsoft Windows NT Environment)
	Display system	VGA & Super VGA up to 1,600 x 1,280 dots non-interlaced
	Number of display	No limit. Controlled by advanced Bit-mapped and font sealing Technology from Windows NT
	Kinds of display characters	Depends on available fonts installed in Windows NT/Windows 95
	Composition of characters	Bit-mapped technology allows Character and picture to be intermixed (Windows NT/Windows 95)
	Vector display	Bit-mapped non-interlaced
	Colors	Configurable within Windows NT/95 From 16 color to millions of colors
	Characters code	ASCII
	CRT editing function	Windows NT/95 (mouse & keyboard)
	No. of viewer	Multi window viewers (sessions)
-	Function	
	Superimpose function	Multi window sessions (windows NT/95)
	CRT display movement	Bit mapped, memory addressable refresh
(e)	Dimensions	410mm W x 415mm H x 438mm D
(f)	Weight	18
(g)	Submission of CRT display drawing	See Citec & Hydro Brochure

(4) Hard Copy Equipment

(a)	Name of the Manufacturer	Hewlett Packard, USA
(b)	Type	Color LaserJet 5
(c)	Copy system	Four toner dry-type laser Electrophography
(d)	Copy size	A4 (A3 for B & W only)
(e)	Copy time	3 pages/minute (Color), 10 pages/minute (B & W)
(f)	Sensitive paper	Not required
(g)	Ambient conditions	
	- Temperature	Operating 10 to 28
	- Humidity	Operating 20% to 70% Relative Huminity
(h)	Power requirement	
	- Nominal Voltage	220 – 240
	- Power Consumption	Max 1,200 W Typical 475 W (B/W) Ready 150 W Standby 45 W
(i)	Dimensions	620 x 490 x 375
(j)	Weight	46.5

(5) Printer

(a)	Name of the Manufacturer	OKI, Japan
(b)	Type	Micro line 591 Turbo
(c)	Printer system	Impact Dot matrix
(d)	Character spacing	10/12/15/17.1/20 & Proportional
(e)	Line spacing	6 & 8
(f)	Number of columns	136 columns (10 cpi) 272 columns (20 cpi)
(g)	Paper width	419
(h)	Printing speed	450cps (max.)
(i)	Dimensions	584 x 378 x 165
(j)	Weight	9

(6) Isolation Transformer (for Master Station)

(a)	Name of the Manufacturer	Star Delta, Australia AS 3108
(b)	Type	IKO-220-220 (Enclosed
(c)	Rating	Continuous
(d)	Phase	Single
(e)	Frequency	50 Hz
(f)	Voltage	220 V
(g)	Capacity	10 kVA
(h)	AC withstand voltage	To AS 3108 Standard
(i)	Impulse withstand	To AS 3108 Standard
(j)	Insulation resistance	300 M ohm or more with DC 1000 V
(k)	Dimensions	172 (L) x 147 (D) x 158 (H) mm
(l)	Weight	18 kg

(7) LBP (Lase Beam Printer)

(a)	Name of the Manufacturer	Hewlett Packard, USA
(b)	Type	LasetJet 4V
(c)	Printing system	Dry-type laser Electro photography
(d)	Character spacing	Variable Chr/inch
(e)	Line spacing	600 dots per inch (DPI) Line/inch
(f)	Number of columns	Variable /line
(g)	Paper width	A3, A4, B3, B4 mm
(h)	Printing speed	Up to 16 pages per Character/ minute
(i)	Dimensions	459 x 522 x 317 mm
(j)	Weight	23.2 kg

(8) Meteorological Observation Equipment

(a)	Combined anemometer and wind vane	
	- Name of manufacturer	Viasala, Finland
	- Wind direction Transmitting section	WAD15A/WAV15A
	- Measuring range	
	wind direction	0 to 360 degrees
	wind speed	0.4 to 75 m/s
	- Accuracy	
	wind direction	better than +/- 3 degrees degrees
	wind speed	+/- 0.17 m/s
	- Dimensions	WAA15A : 240 (H) x 86 mm Dia. Sweep radius : 91 WAA15A : 300 (H) x 90 Dia. Sweep radius : 172
	- Weight	WAA15A : 0.56 kg WAA15A : 0.56
(b)	Thermometer	Viasala, Finland
	- Name of manufacturer	Platinum temperature Measuring resistor
	- Sensitive section	
	- Measuring range	0 to 100 degree C
	- Accuracy	± 0.2% degree C
	- Dimensions	312 x 100 x 100 mm

	- Weight	0.1 kg
(c)	Barometer	
	- Name of manufacturer	Vizsla, Finland
	- Measuring range	800 – 1060 hPa
	- Accuracy	± 0.3 %
	- Dimensions	60 x 88 x 21 mm
	- Weight	0.1 kg
(d)	Barometer	
	- Name of manufacturer	Vizsla, Finland
	- Measuring range	0 – 100 %
	- Range of temperature used	-40 to +80 degree C
	- Accuracy	+ 2 %
	- Dimensions	312 x 100 x 100 mm
	- Weight	0.1 kg
(e)	Recorder	
	- Name of manufacturer	Yokogawa Electric Corp., Japan
	- Model	µR 1800
	- Auto balancing recorder	
	Recording period	> 30 days
	Number of recording pen	4 pen
	Recording paper width	180 mm
	- Dimensions	279 (L) x 288 (W) x 220 (H) mm
	- Weight	TBA kg

Chapter 3 MANUFACTURING, CONSTRUCTION AND INSTALLATION

3.1 Manufacturing and Delivery

The plant was supplied and tested by three Australian companies. The Radio and Telemetering system with Citect Software and Warning System were supplied by ELPRO Technologies in Brisbane, Queensland. Flood Forecasting system with HYDROL Hydraulic Modeling software was supplied by HECEC (Hydro-Electric Corporation) in Hobart, Tasmania. After each section was tested, the entire system was assembled, adjusted and tested by Greenspan Technology in Coffs Harbour, New South Wales.

(1) Factory Test

Witness Factory Test for the following equipment was carried out from 24th August to 28th August 1998 in accordance with the approved Factory Test Procedure at the factory of Greenspan Technology in Coffs Harbour, New South Wales Australia, in the presence of the Engineer.

(a) Telemetry Master Station Equipment
(b) Gauging Station Equipment for Muara Paiti
(c) Gauging Station Equipment for Tabing
(d) Gauging Station Equipment for Gunung Malintang
(e) Gauging Station Equipment for Galugur
(f) Meteorological Observation Equipment
(g) Dam Data Processing System (Server and Workstations)
(h) Warning System Master Station Equipment
(i) Warning Station Equipment (10 units)
(j) Repeater Station Equipment (2 units)
(k) Antenna Masts
(l) Sirens

The above equipments were assembled and tested without difficulty. Signal from other lot contractor were emulated using current injectors. For Dam data processing system, dummy data were used to construct a flood forecasting mode.

All plant was checked for conformance with the Contract Specifications.
Test result was satisfactory except some minor correction which was instructed to complete before shipment and Inspection/Test Certificate issuance.

Engineer requested the following corrections:

(a) A desk-stand microphone should be provided at the console of Control Center for warning station announcement without additional charge.

(b) On Warning Station cabinet door, the lamp indicators for 3-phase power (R-S-T) should be replaced with the ones of higher intensity as the blue indicator was not bright enough and loaded like there was no power in that phase/line without additional charge.

(2) Delivery

(a) Shipment of Goods

After the Factory Test, the anchor bolt and template for antenna mast foundation were shipped from Australia on September 4th 1998 and arrived at KHPP site office in Bangkinang Riau on October 5th, 1998.

Other all goods were packaged following agreed-upon material and all crates were packed into 2 (two) shipping containers, 1 of 20 foot 6,200 kg and 1 of 40 foot 11,700 kg. Both shipping containers were loaded onto the Ship MSC China (Mediterranean Shipping Company Geneva) on September 28th with Bill of Lading issued (B/L No. MSCU-B140869-5) dated September 29th 1998. Certificate of Australian Origin document was issued by the Australian Chamber of Commerce and Industry. The Ship MSC China arrived at Tanjung Priok Jakarta on October 21th, 1996.

After custom clearance, shipping containers departed for site on October 31st by trailer and arrived at site, no lacking item and no damage were reported.

(b) Site Delivery

One container was stored in the PLTA Kotapanjang premises near the reservoir and the others were stored in the yard of Contractor's site office in Bangkinang and delivered to each site by the truck.

13 Construction and Installation

(1) Building and Civil Works

The building and civil works for the following stations were commenced at the end of May 1998 and completed by the end of February 1999.

(a) Gauging Station

The gauging stations were constructed in the upstream project catchment area of the Kampar Kanan river, Kapurnan Gadang river and Mahat river at Galugur (Rainfall Gauge), Tabing (R/G and Water level Gauge), Muara Paiti (R/G & W/L) and Gunung Melintang (R/G & W/L). The water level sensors at Muara Paiti and Gunung Melintang gauging stations were changed from the float well type to the gas purge pressure type due to the collapse of bank and the existing bridge renovation respectively.

(b) Warning Station

Ten (10) radio control warning stations were constructed in the downstream area of the dam along the Kampar Kanan river and one (1) wired warning station was constructed at the powerhouse.

(c) Repeater station

The existing telephone system repeater station is located in front of the switchyard supplied by PLN and this station is used for the Dam site repeater 1. The repeater 2 was constructed newly at the top of the mountain Batu Putih (EL.640m) located at the near the center of the project catchment area.

(2) Installation Works

In accordance with the approved drawings and the installation procedure, the equipment installed for each station is shown in Table 4.3.1. Equipment installation work was commenced on November 6th,1998 and completed by the end of February 1999. The general arrangement of the typical site in all stations is shown in Fig. 4.3.1.

(3) Contractor's Organization and Manpower Deployment

Organization for the execution of this works is shown in Fig.4.3.2. Manpower deployment for the works is shown in Table 4.3.2.

(4) Accidents

During construction works, no accident occurred.

Table 4.3.1 Main Equipment Installed for Each Station

Master station			Repeater station		Warning station	Gauging station		
Central Control Room	Computer & Relay Room	Roof of Powerhouse	No.1	No.2	No1-10	Galung	Tabing	Muara Paiti & Gunung Melintang
-Telemetering Desk -Data Processing Desk -Annuncial Console -CRT Display Equipment X2 -Printer for Telemetering -Printer for Data Processing -Printer for Warning -Microphone	-Warning Master Station -Relay Terminal and Telemetering Master Station -Computer Server Cabinet -Dam Data Processing Equipment -Isolation Transformer (10 kVA) -UPS for computer -Dam Warning Station -Meteorological Cabinet -Recorder Cabinet	-Siren with Pedestal -Antenna Mast with Lightning Rod -Yagi Antenna -Loud Speaker X4 -Speaker Junction Box -Sound Collector -Combined Anemometer & Wind vane -Barometer -Thermometer -Hygrometer -Steel Post for Meteorological Cabinet -Flooring Basket	-Repeater Equipment -Isolation Transformer -DC Power Supply Equipment with Battery -Omni-Antenna -Coaxial Cable with Connector -Coaxial Arrester -Antenna Mast -Grounding system	-Repeater Equipment -Solar Panel & Battery -Power Distriubution Board -Omni-Antenna -Coaxial Cable with Connector -Coaxial Arrester -Guyed tower with Lightning Rod -Grounding system	-Warning Equipment -Siren with Pedestal -Loud Speaker -Speaker Junction Box -Sound Collector -Switch Board -DC Power Supply Equipment with Battery -Yagi Antenna -Coaxial Arrester -Antenna Mast with Lightning Rod -Grounding system	-Telemetering Equipment -Rain Gauge -Rainfall Recorder -Solar Panel & Battery -Power Distriubution Board -Yagi Antenna -Antenna Mast with Lightning Rod -Coaxial Cable with Connector -Coaxial Arrester -Grounding system	-Telemetering Equipment -Rain Gauge -Rainfall Recorder -Float well Water Level Gauge -Solar Panel & Battery -Power Distriubution Board -Yagi Antenna -Antenna Mast with Lightning Rod -Coaxial Cable with Connector -Coaxial Arrester -Grounding system	-Telemetering Equipment -Rain Gauge -Rainfall Recorder -Gas Purge Water Level Gauge -Solar Panel & Battery -Power Distriubution Board -Yagi Antenna -Antenna Mast with Lightning Rod -Coaxial Cable with Connector -Coaxial Arrester -Grounding system

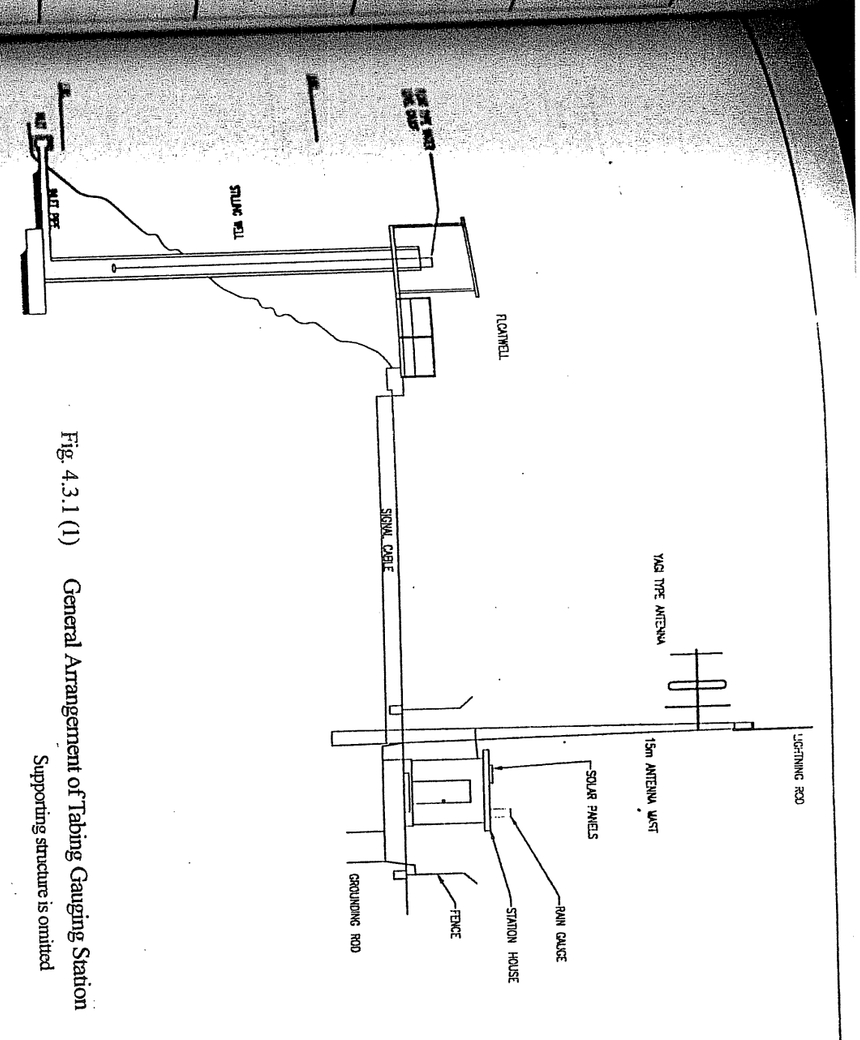


Fig. 4.3.1 (1) General Arrangement of Tabing Gauging Station
Supporting structure is omitted

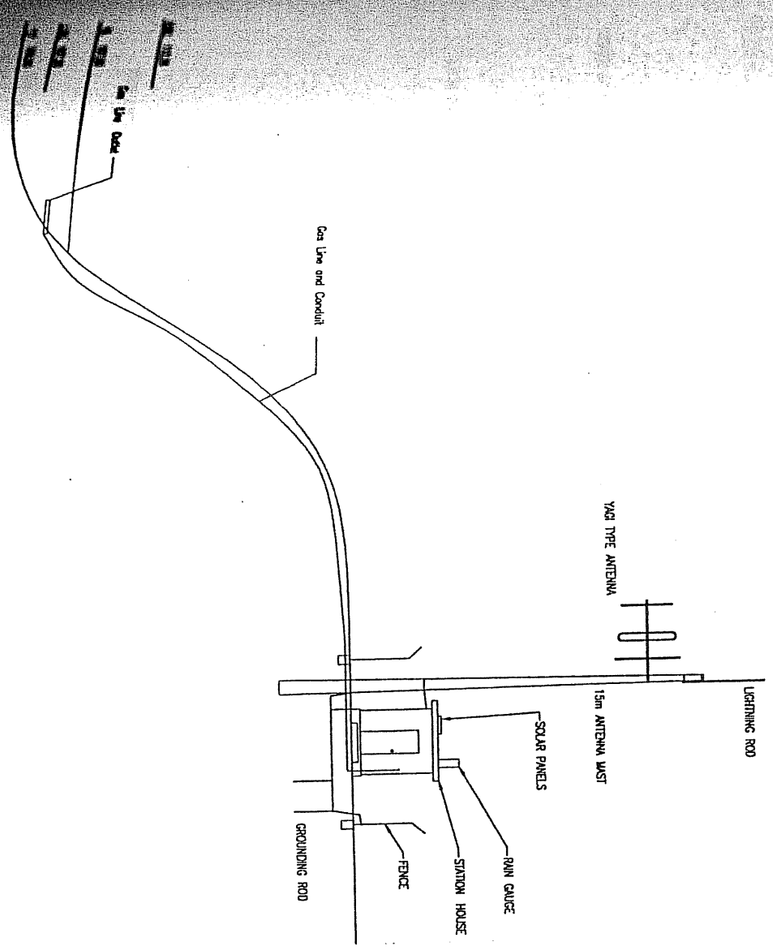


Fig. 4.3.1 (2) General Arrangement of Gunung Melintang Gauging Station

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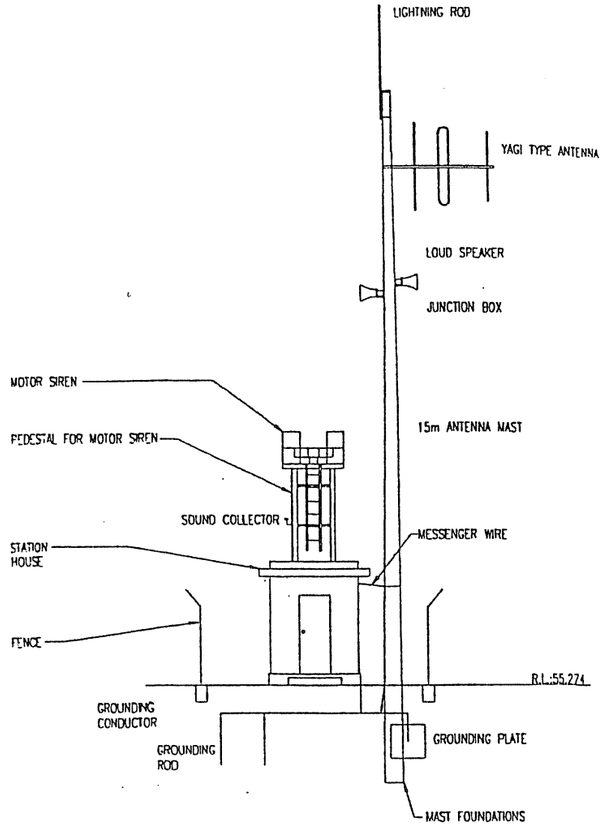


Fig. 4.3.1 (3) General Arrangement of Repeater Station 2

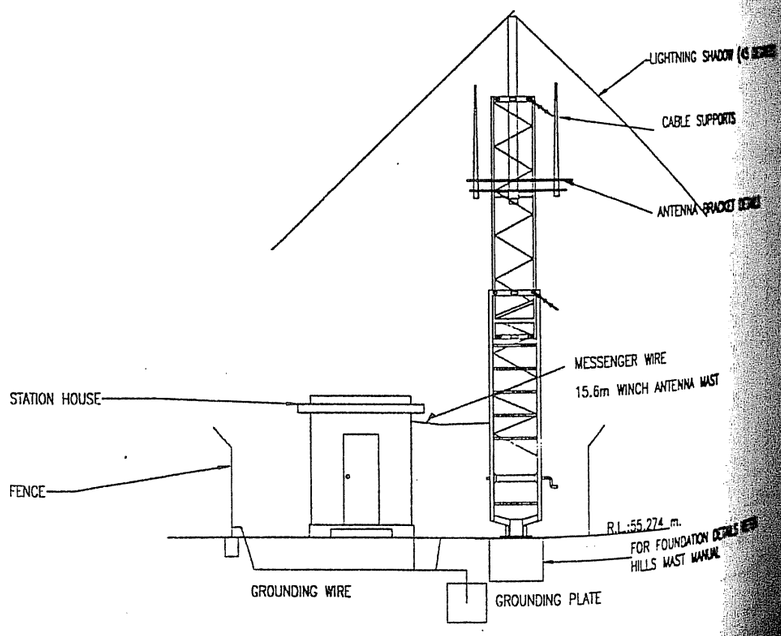
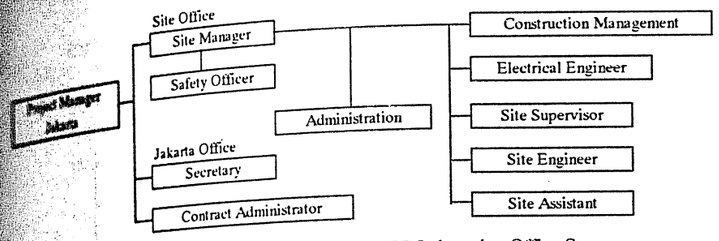
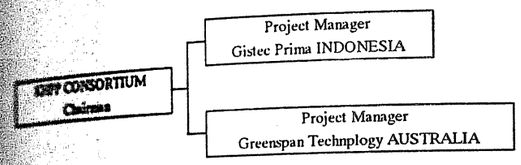


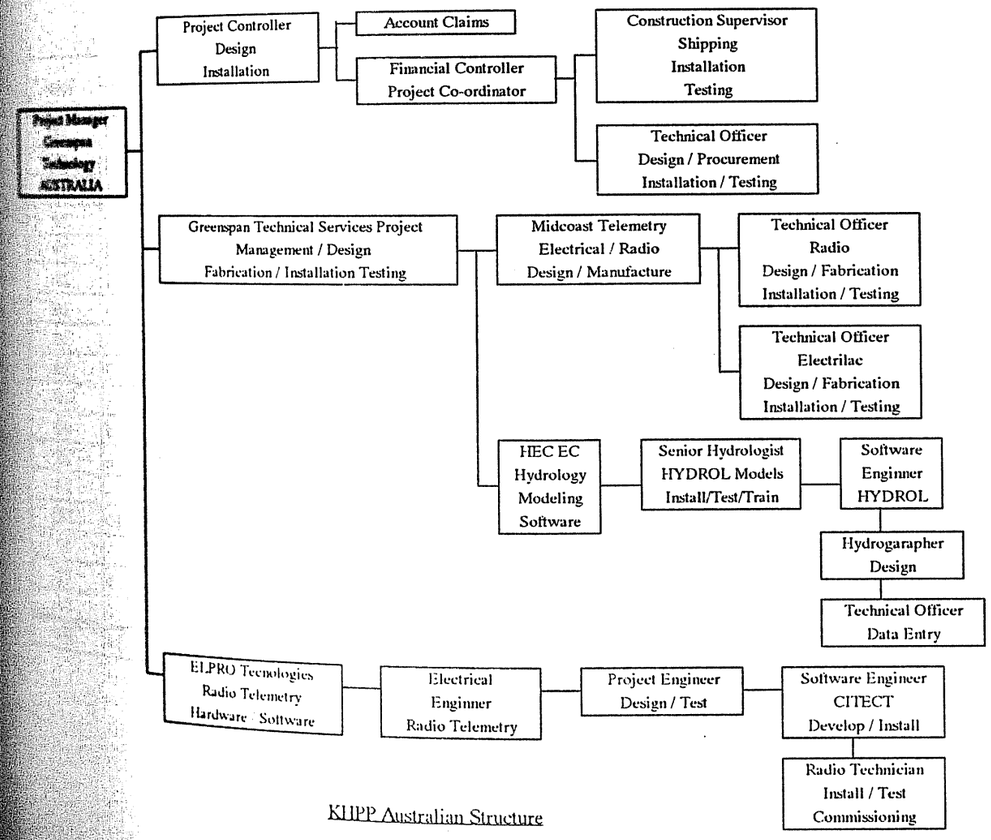
Fig. 4.3.1 (4) General Arrangement of Radio Warning Station No.1



Site Organization KHPP Indonesian Office Structure



KHPP Consortium Structure



KHPP Australian Structure

Fig. 4.3.2 Organization for Works

Table 4.3.2 Manpower Development

Expert Manpower

PROFESSION	1998			1999		Total
	Oct	Nov	Dec	Jan	Feb	
Project Manager	1	1	1	1	1	5
Electric Engineer	1	1	1	1	1	5
Radio Engineer	1	1	1	1	2	6
Computer Software Engineer	1	1	1	1	2	6
Construction Manager	-	-	-	-	2	2
Telemetering Engineer	-	-	-	-	1	1
Hydrology Engineer	2	2	2	2	1	9
Amount	6	6	6	6	10	34

Local Manpower

PROFESSION	1998												1999		Total
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	
Project Manager	1	1	1	1	1	1	1	1	1	1	1	1	1	1	14
Site Manager	-	-	-	-	1	1	1	1	1	1	1	1	1	1	10
Site Assistant	1	1	1	1	1	1	1	1	1	1	1	1	1	1	14
Construction Manager	-	-	-	-	1	1	1	1	1	1	1	1	1	1	10
Administration	-	-	-	-	1	1	1	1	1	1	1	1	1	1	10
Electrical Engineer	-	-	-	-	1	1	1	1	1	1	1	1	1	1	10
Technician	-	-	-	-	2	2	3	3	3	3	-	-	-	-	16
Civil Engineer	1	1	1	1	1	1	1	1	1	1	1	1	1	1	14
Supervisor	-	-	-	-	1	2	2	2	2	2	2	2	2	2	19
Draftsman	-	-	-	-	1	1	1	1	1	1	1	1	1	1	10
Foreman	-	-	-	1	2	5	5	5	5	5	5	5	5	5	61
Skill	-	-	-	4	6	7	10	10	10	8	6	6	6	6	79
Welder	-	-	-	-	-	-	2	2	2	2	2	2	2	-	14
Helper	2	2	2	2	20	20	38	42	49	37	24	17	15	10	362
Office boy	-	-	-	-	1	1	1	1	1	1	1	1	1	1	11
Driver	-	-	-	1	1	1	1	1	1	1	1	1	1	1	14
Cleaning Service	1	1	1	1	1	1	1	1	1	1	1	1	1	1	14
Amount	6	6	6	12	42	47	71	75	82	68	50	43	41	34	583

Chapter 4 FIELD TEST AND TRAINING

4.1 Field Test and Taking Over

After the installation and adjustment of all equipment, the field test was conducted following the schedule in accordance with the Specification and the approved test procedures in the presence of PLN Project and Sector personnel.

The result of the field test was satisfactory. Therefore, the taking over certificate was issued on March 10th, 1999 for the original Contract. For the additional work mentioned in Chapter 8, the TOC is to be issued separately later on.

(1) Electrical and Mechanical Individual Test

Date		Description
January 26, 1999	:	Powerhouse Master station, Repeater No.1 Warning Station No.1, No.2, No.3
January 27, 1999	:	Warning Station No.4, No.5, No.6, No.7, No.10 No. 9, No.8

(2) Radio, Solar and Gauging Individual Test

January 30, 1999	:	Galugur Rainfall Gauging Station
January 31, 1999	:	Tabing Rainfall and Water Level Gauging Station

(3) Total System Test and Individual Test

February 9, 1999	:	Mater Station, Repeater No.1
February 10, 1999	:	Repeater No.2 Batu Putih
February 11, 1999	:	Tabing Rainfall and Water Level Gauging Station Warning Station
February 12, 1999	:	Gunung Malintang and Muara Paiti Rainfall and Water Level Gauging Station
February 13, 1999	:	Warning Station No.4, No.6, No.7, No.8, No.9
February 14, 1999	:	Warning Station No.10, No.2, No.3, No.5, No.
February 15, 1999	:	Warning Station Dam Site
February 16, 1999	:	Master Station Polling and Warning Broadcast Sequence
February 17, 1999	:	Galugur Rainfall Gauging Station and Tabing Rainfall and Water Level Gauging Station
February 18, 1999	:	Total System Operation, Alarms annunciator, Data And Report Printing Out
March 9, 1999	:	Checking of Spare Parts, Spare Equipment, Measuring Instrument and Tools

4.2 Training and Operation/Maintenance Guidance

For the purpose of keeping the safe and efficient operation and maintenance of the system, the training for the PLN Project and Sector personnel was conducted by the foreign expert and Indonesian system engineer using a training manual. The training was implemented in the form of seminar in the powerhouse, then conducted at the Repeater station, Warning station and Gauging station as on-the-job training.

(1) Trainee

The trainee consisted of PLN Sector Operation 6 persons, PLN Sector Maintenance persons and PLN Project Kotapanjang 6 persons, total 15 persons and some observers.

(2) Training Program

The training was conducted simultaneously with the field testing period in accordance with the Specification. The training was started on February 2nd, 1999 and finished on February 19th, 1999. The training program is shown in detail in the following table.

Stage 1 Gauging and Warning station

Date	Place	Description
2 Feb. 1999	Powerhouse	System Overview, Introduction to Gauging station Instrumentation including principles of operation
3 Feb. 1999	Powerhouse & Warning Stn. No.3	Meteorological station equipment Warning station equipment
4 Feb. 1999	G.Malintang & Tabing Gauging Stn.	Solar system
5 Feb. 1999	Powerhouse	Discussion, question and answer

Stage 2 Telemetering and Warning system CPU Hardware & Software Dam processing Hydraulic Modeling

Date	Place	Description
16 Feb. 1999	Powerhouse & Warning Stn.	Master station polling & Warning sequence group & single Voice message broadcasting, Alarm screen on Citect
17 Feb. 1999	Powerhouse	Introduction of System design and operation Introduction of Citect/Radio system Description of Gauging, Warning Repeater and its Operation System Modeling Introduction of Relay terminal equipment Block system operation of Warning
18 Feb. 1999	Powerhouse & Warning Stn.	Description of main screen on Citect Description of Warning station data Interface of data from Citect-Hydrol Operation and function of Radio network Description / Fault finding of alarm Maintenance of Citect / Back up / Restore file and data Operation of Sirens at each Warning station Exercise for Operators Visit to Warning station No.1 Description of RTU Operation and maintenance of TLC equipment Fault finding – TLC menu operation and diagnostics Generating of report

ADDITIONAL WORKS

(1) Background

The flood occurred on January 6th,1998 and on February 2nd ,1998 and submerged a part of Pangkalan Kotabaru area and the traffic link from Bukittinggi to Pekanbaru was suspended. The flooding area of Pangkalan Kotabaru was situated about 7 km upstream of the reservoir boundary.

Referring to the original plan, one unit of the water level and rainfall gauging station was installed in the Mahat river basin at Gunung Melintang, about 20 km upstream of Pangkalan Kotabaru. However this gauging station can not cover extensively. Also the distance is very close to Pangkalan Kotabaru and edge of impounding area for earlier observation. Therefore additional observation stations and related works were planned to mitigate yearly flood condition effect on the residents and public infrastructures at Pangkalan Kotabaru.

(2) Outline of Additional Works

The contract period of the additional works was 204 days from 11th March 1999 to 30th September 1999 and all of the Engineering works for the additional works were conducted by PLN after 1st April 1999. The outline and reasons for the additional works are as follows.

(a) One water level + rainfall gauging station at Sopan Gadang village in the upper Mahat, junction of Mahat river and Batang Penawan and Air Dingin.
(b) One repeater station at Bukit Baranak close to the Sopan Gadang station (if necessary) depending on the result of radio propagation test.
(c) One water level gauging station at Tanjung Balit.
(d) Theses three stations were planned to provide early data on river discharge and rain fall in the upper catchment area necessary to provide advance warning and determining time and amount of the discharge.

(e) One rainfall gauging station at Koto Tuo village, at the boundary of Mahat river basin and Kampar Kanan river basin, for additional comparison of rainfall characteristic between the two river basins.

(f) One water level gauging station at Rantau Berangin, for provision for determining the status of the actual river conditions at the downstream of the dam prior to changing the flow regime, such as adjusting spillway gates, power station discharge, and for additional check on the theoretical discharge from the spillway and/or power station allowing adjustment of the rating curve and hence more accurate and safety operating control.

(g) Modification on the computer program for data analysis related to the above
(h) Site survey and radio propagation test
(i) Temporary works
(j) Spare equipment and consumables for additional works
(k) Additional station batteries and solar panels for the Repeater 2 (Batu Putih)

PART 6

PART 5

METAL WORKS (HYDRO MECHANICAL WORKS)

PART 5

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Chapter 1 INTRODUCTION

1.1 Outline of The Metal Works

The Metal Works for the gates, stoplogs and penstocks for the Kotapanjang Hydroelectric Power Project were designed, fabricated and erected by Sumitomo Corporation as main Contractor. Sumitomo Corporation called Ishikawajima-Harima Heavy Industries Co, Ltd. (IHI) to perform all the necessary detail design for the gates, stoplogs, and penstocks. To carry out the construction work under this contract, Sumitomo Corporation made an association with a well experienced international manufacturer and its local sub-contractor. The associated manufacturer was the Ishikawajima Harima Heavy Industries Co., Ltd (IHI), and the employed local sub-contractor was PT.Barata Indonesia for shop fabrication and site installation.

The Metal Works Contract, Lot II, was awarded on 25th February 1993, was concluded on 22nd April 1993 between PT.PLN (PERSERO) and Sumitomo Corporation under the Contract No.039/PJP/922/1993/M, and was completed on 15th November 1996. The metal works contract had two key completion dates. These two key date for completion of the work on the erection of the metal works were October 30, 1995, for tailrace gates and stoplogs, including guide frames and accessories up to EL.45.50, and November 15, 1996 for guide frames and accessories of tailrace gates from EL.45.50 up to EL.60.50, tailrace gantry crane, penstocks, intake gates and stoplogs, intake trashrack, intake trashrake, and spillway gates and stoplogs. See Table 5.1.1 Contract Key Dates.

Table 5.1.1 Contract Key Date

Key Date of Completion		Contract Schedule Date	Total Delay	Actual Completion Date
Sec.1	Tailrace gates, stoplogs up to EL. 45.50	Oct. 30, 1996	None	Aug. 24, 1995
Sec.2	a. Guide frames of Tailrace from EL.45.5	Nov.15, 1996	None	Unchanged
	b. Tailrace gantry crane	Nov.15, 1996	None	Unchanged
Sec.3	Penstock	Nov.15, 1996	None	Unchanged
Sec.4	Intake gates and stoplogs	Nov.15, 1996	None	Unchanged
Sec.5	a. Intake trashrack	Nov.15, 1996	None	Unchanged
	b. Intake trashrake	Nov.15, 1996	None	Unchanged
Sec.6	Spillway gates and stoplogs	Nov.15, 1996	None	Unchanged

The Contract consisted of the designing, manufacturing, supplying, testing, finishing, primer coat, painting, packing for export, insuring, shipping, delivering to the port of Dumai or nearest port, landing or transhipment, customs clearing, transporting inland or marine from the port of Dumai to the site, erection/supervision of erection, finishing, painting, testing on completion and commissioning and remedying any defects of the following :

a. Spillway Gate	5 sets of fixed wheel type gates, with 11.0 m width and 18.0 m height
b. Spillway Stoplog	12 sets of stoplogs with dimensions of 11.0 m x 1.5 m
c. Intake Gate	3 sets of fixed wheel type gates, with 6.0 m width and 6.0 m heigh
d. Intake Stoplog	20 sets of stoplogs with dimension of 6.0 m x 1.2 m
e. Intake Trashrack	3 sets of trashracks with 10.0 m width and 23.5 m, vertical heigh
f. Intake Trashrake	1 set mechanical rake, with conveyor
g. Tailrace Gate	2 sets of sluice type gates with 6.0 m width and 4.5 m height
h. Tailrace Stoplog	20 sets of stoplogs with dimension of 6.0 m x 0.92 m
I. Tailrace Gantry Crane	1 set of electrically operated gantry crane
j. Penstock	3 lines of penstock with 5 m inside diameter and 86.905 m length

The spillway gates and stoplogs, intake gates and stoplogs, intake trashrack, intake trash rake are all located around the dam site area. The tailrace gates and stoplogs, and tailrace gantry crane are located in the outlet structures of the tailrace tunnels. The penstock consists of an upper horizontal, an inclined, and a lower horizontal section. The penstocks are 86.905 meters long each and their upstream end is positioned immediately upstream of the dam. The penstocks terminate in the powerhouse at the connection with the turbine spiral casing.

The more complicated and important parts of the works, such as control panels, hydraulic brake of intake gate hoist equipment, cyclo reducer and electro hydraulic lifter brake for the gate hoist equipment were fabricated in Japan. The intake trash rake and conveyor was fabricated at Kyowa Seisakusyo Co.Ltd, in Japan. The tailrace gantry crane was fabricated at MHE-Demag(s) Pte.,Ltd, in Singapore.

The spillway gates and stoplogs, intake gates and stoplogs, intake trashrack, tailrace gate and stoplogs were fabricated at the PT. Barata Indonesia factory in Surabaya. Steel materials used included 50 kgf/mm^2 classes. Steel materials were purchased in Japan, shipped to Surabaya, and fabricated into manufacturing goods, there after transported as assemblies to site for erection.

The penstocks were fabricated at site shop by PT. BARATA Indonesia. Steel materials used also included 40 kgf/mm^2 classes. Steel materials for the penstocks were purchased in Japan, shipped to the site, and rolled into cylindrical sections. Installation unit pipes, six meters long each, were sub assembled and transported to the erection site where they were welded into final position. No post weld heat treatment was required for the penstock steel liners as the thickness of the plates was within the allowable limits for this purpose. The local fabrication was carried out under full supervision by the specialist staff of IHI. The erection works of all equipments were carried out by PT. BARATA Indonesia under the supervision of the specialist staff of IHI and PT. Barata Indonesia. See Table 5.1.2. Metal Work Fabrication and Installation.

Table 5.1.2 Metal Work Fabrication and Installation

Major Equipment	Individual Equipment	Main Contractor & Its Associated Manufacturer	Original Supplier	Local Sub-Contractor for Fabrication and Site Installation
Spillway equipment	Spillway Roller Gate Spillway Stoplog	Sumitomo Co, IHI "	IHI "	PT. Barata, Surabaya "
Intake equipment	Intake Gate Intake Stoplog Intake Trashrack Intake Trashrake Conveyors	Sumitomo Co, IHI " " " "	IHI " " Kyowa SeisakushoCo..Ltd , Saga Japan "	PT. Barata, Surabaya " " - -
Steel penstock	-	Sumitomo Co, IHI	IHI	PT. Barata, Surabaya (Kotapanjang Site)
Tailrace equipment	Tailrace Gate Tailrace Stoplog Storage Facility Gantry Crane	Sumitomo Co, IHI " " "	IHI " " MHE-Demag Pte,Ltd Singapore	PT. Barata, Surabaya " " -
Control panel	-	Sumitomo Co, IHI	EIM System Kitakyushu, Japan	-

The pre-tender of the Contract was made by PLN by his pre-tender conference dated July 23, 1992 to 22 manufacturers prequalified from the viewpoint of the required experience of manufacturing and installation. Following the tender, the Employer and the Engineer commenced respective tender evaluation at Jakarta and Tokyo separately. After successful negotiations between PLN and Sumitomo Corporation, the letter of intent for Lot II-Metal Works was issued by PLN on 25 February 1993 and the Contract agreement was concluded on 22 April 1993. The contract price was Yen 936,155,000 (foreign component) and Rp. 6,142,400,000 (local component).

1.2 Organization and Technology of Installation Work

1.2.1 General

The entire installation of large hydromechanical equipment as like Lot II-Metal Works are divided into three main stages the following (1), (2) and (3) according to the time in which the installation work is carried out :

(1) Preparatory work was carried out before installation of the equipment began. Preparatory work represents organizational and technical readying of the equipment for installation. The Lot II - Metal Works could be divided into the following two parts (i) and (ii) in according to preparatory work :

i) Gates and Stoplogs

The gates and stoplogs were fabricated at Surabaya shop and arrived at site in the form of pre-assemblied product.

ii) Penstock

Materials for penstock arrived at site in the form of raw materials (plates). The materials was fabricated first at the site fabrication shop into semi-fabricated product namely the unit pipe. This unit pipe was be installed at site to form the penstock. For preparatory work, the Contractor established the site fabrication shop beside the site for storage and preassembly of the equipment, complete with: office, living quarters, electricity, water etc. (see Fig.5-23). Activities at site were started by fabrication and installation of penstocks, continued by installation of gates and stoplogs.

(2) Installation of the built in parts of the hydromechanical equipment.

i) The installation of the built in penstock parts in the case of simultaneous construction of powerhouse and dam body, and installation work of penstock began immediately after the concrete structure of the powerhouse had reached the required elevation at EL.29.800. There after penstock installation was continued progressively with the construction of powerhouse and dam body.

ii) The installation of the built in gates and stoplogs parts were only concerning the installation of embedded anchor bars at the block out for installing the guide frames of gates and stoplogs, and the installation of anchor pads at the hoist decks for installing the hoists.

(3) Installation of the operating mechanisms of the hydromechanical equipment.

This installations, especially concerning with gates and stoplogs, began after the structure of the tailrace, intake and spillway had been finished in the rough and mobile crane (or cable crane, or tower crane) could reach the axis of the equipment to be installed.

1.2.2 Planning of Installation Work

Installation work in Lot II hydromechanical equipment project was planned in two stage, (1) and (2) as follows :

(1) The first planning stage involved establishing organizational procedure for preparing installation work. This plan was "Work Master Schedule (Overall Programme)" and contained the following information concerning installation of the principal equipment, (i) the amount of installation work by years, (ii) sequence, speed, and method of installation work and (iii) requirement as regards workers, engineers, and technicians, as well as of basic material and technical supplied.

(2) The second planning stage consisted in establishing special plan for installing the principal equipment, based on the overall construction organization plan, drawings of the equipment, instruction of the makers, and relevant directives. This plans were "Shop Test Schedule and Procedure", "Method Statement", monthly schedule, weekly schedules and daily schedules , and "Operation and Maintenance Instruction".

1.2.3 Technical Accounting

Brief technical characteristic of the equipment with the name of the supplier can be seen in the "Operation and Maintenance Instruction" submitted during construction. This completion report is the technical account of installation of the principal equipment of the hydromechanical equipment that indicate the conditions and process of installation. Some importance records that can be used for making evaluation for another kind of project are :

i) Work Progress (Ex-Factory), see Fig 5.15.
ii) Work Progress (Erection), see Fig.5.16.
iii) Detail Erection Works Schedule, see Fig.5.28.
iv) Detail Bi-Monthly Progress Payment, see Fig.5.29.
v) Planning and Actual Utilization of Direct and Indirect Labour, see Fig.5.30.
vi) Planning and Realization of Overtime for Indirect Labour, see Fig.5.31.
vii) Planning and Realization of Overtime for Direct Labour, see Fig.5.32.

By combining the record of Fig.5.15. Work Progress (Ex-Factory), Fig.5.16 Work Progress (Erection) and Breakdown of the Contract Price, it can be evaluated the answers of the questions of (i) fabrication product quantity (ton) that can be completed by the sub-contractor per day, (ii) erection works amount (ton) that can be completely installed at site per day. (iii) local ex-factory price per ton, (iv) erection price per ton, (v) total price (ex-factory, transportation, insurance, erection etc.) per ton. See Table 5.1.3 Item of Metal Works and Evaluation.

Table 5.1.3 Items of Metal Works and Evaluation

Item	Description	Dimension W (m) x H (m)	Quantity	Weight (t) Unit	Weight (t) Total	Fabrication Time Day	Fabrication Rate Ton/Day	Erection Time Day	Erection Rate Ton/Day
1.	Spillway Gate	11.0 x 18.0	5	76.7	383.4	365	1.05	240	1.6
	Guide Frame		5	12.1	60.4	150	0.4	150	0.402
	Hoist		5	15.1	75.7	300	0.252	90	0.84
2.	Spillway Stoplog	11.0 x 1.5	12	7.4	88.5	210	0.421	30	2.95
	Guide Frame		5	3.4	16.9	150	0.113	120	0.141
3.	Intake Gate	6.0 x 6.0	3	25.9	77.6	255	0.304	90	0.862
	Guide Frame		3	9.4	28.1	150	0.187	90	0.312
	Hoist		3	13.3	39.9	240	0.166	90	0.443
4.	Intake Stoplog	6.0 x 1.2	20	3.1	62.1	150	0.414	30	2.07
	Guide Frame		3	5.4	16.3	150	0.109	90	0.181
5.	Intake Trashrack	10.0 x 23.5	3	44.7	134.1	180	0.745	120	1.118
6.	Intake Trashrake	- W 3.0	1		20.5	150	0.137	90	0.228
7.	Tailrace Gate	6.0 x 4.5	2	11.7	23.4	180	0.13	30	0.78
	Guide Frame		6	3.6	21.3	120	0.178	120	0.178
8.	Tailrace Stoplog	6.0 x 0.92	20	2.4	48.5	180	0.269	60	0.808
	Storage Facility		6	2.5	14.7	180	0.082	60	0.245
9.	Tailrace Gantry Crane		1		18.8	90	0.21	60	0.313
				Sub Total	1130.7	3200	0.353	1560	0.725
10	Penstock	Ø 5.0 x L86.905	3	186.0	558.1	-	-	510	1.094
	Total			Total	1688.8	-	-	2070	0.816

Item	Description	Local Ex-Factory Price Rp.1000,-	Local Ex-Factory Price per Ton Rp.1000,-/Ton	Erection Price Rp.1000,-	Erection Price per Ton Rp.1000,-/Ton	Total Price Rp.1000,-	Total Price per Ton Rp.1000,-/Ton	Manufacturer
1.	Spillway Gate	3,978,500	10,376.89	1,177,800	3,071.99	5,483,600	14,302.58	Barata Indonesia
	Guide Frame	768,200	12,718.54	227,600	3,768.21	1,058,400	17,523.18	"
	Hoist	784,400	10,361.96	231,200	3,054.16	1,079,200	14,256.27	"
2.	Spillway Stoplog	946,700	10,697.18	280,400	3,168.361	1,305,000	14,745.76	"
	Guide Frame	195,100	11,544.38	58,900	3,485.21	270,500	16,005.92	"
3.	Intake Gate	806,800	10,396.91	240,200	3,095.36	1,112,500	14,336.34	"
	Guide Frame	238,900	8,501.78	72,100	2,565.84	330,700	11,768.68	"
	Hoist	419,800	10,521.30	125,000	3,132.83	579,800	14,531.33	"
4.	Intake Stoplog	647,300	10,423.51	191,300	3,080.51	892,300	14,368.76	"
	Guide Frame	170,100	10,435.58	49,800	3,055.21	234,400	14,380.37	"
5.	Intake Trashrack	1,322,300	9,860.55	392,100	2,923.93	1,832,700	13,666.67	"
6.	Intake Trashrake	-	-	62,900	3,068.29	1,269,100	61,907.32	Japan
7.	Tailrace Gate	241,400	10,316.24	72,200	3,085.47	333,300	14,243.59	Barata Indonesia
	Guide Frame	359,300	16,858.54	105,800	4,967.14	495,600	23,267.60	"
8.	Tailrace Stoplog	488,100	10,063.92	144,800	2,985.57	672,800	13,872.16	"
	Storage Facility	10,800	734.69	3,200	217.69	14,700	1,000.00	"
9.	Tailrace Gantry Crane	-	-	59,700	3,175.53	715,900	38,079.79	Singapore
		-	-	3,495,000	3,091.13	17,680,500	15,636.77	"
10	Penstock	-	-	3,957,000	7,090.13	6,310,900	11,307.83	Barata Indonesia
	Total	-	-	7,452,000	4,412.60	23,991,400	14,206.18	"

Chapter 2 GATES AND STOPLOGS - DESIGN AND CONSTRUCTION

2.1 Design

2.1.1 General

Kotapanjang Hydroelectric Power dam is a concrete gravity structure with a crest length of 257.5m with the crest level of EL.87.500m and a height of 58.0m. The dam body overflow portion comprises five weir sections equipped with five fixed wheel gates with single leaf type 11m wide by 18m high named of spillway gates.

The intake turbine portion at the dam comprises three fixed-wheel intake gates 6m wide by 6m height to close the penstock. Tailrace gates were installed at the outlets of the draft tubes of the turbines at the powerhouse.

To summarize the Kotapanjang Hydroelectric Power dam is equipped with : (i) five spillway single leaf fixed wheel gates 11m x 18m high each, (ii) 1 set spillway slide stoplog comprises of identical twelve stoplogs 11m x 1.5m high each, (iii) three fixed wheel intake gates 6m x 6m high each, (iv) 1 set intake slide stoplog comprises of identical twenty stoplogs 6m x 1.2m high each, (v) 3 sets trash racks fixed type 10m x 23.5m vertical height each, (vi) 1 set intake trash rake and 1 set of belt conveyor, (vii) 2 set of tailrace slide gates 6m x 4.5 height, (viii) 4 sets of tailrace slide stoplog comprises of five stoplog blocks 6m x 0.92m height each, (ix) 1 set gantry crane of 13 ton working load. Table 5.2.1, 5.2.2, 5.2.3, 5.2.4, and 5.2.5, present a summary of design features and constructions performance of the hydromechanical facilities.

2.1.2 Selection of Gate Type

(1) Spillway Gate

The spillway gate was installed at the crest of the spillway at EL.67.500m to control discharge safely from the reservoir during flood. The maximum discharge of the spillway was determined to be at 8000m^3/s at the reservoir water level at EL. 85.000m.

Taking into account of the design discharge and the topographic configuration at the dam site, the total width of the spillway including gate pier is adopted at 71m under the design head of 17.5m to release the design discharge at HWL. EL.85.000. The total width comprises five (5) sets of gates with width of 11m each and four (4) sets of gate piers with width of 4m each.

The gate height was determined at 18m from the crest at EL.67.500m to EL. 85.500m. Design hydrostatic head load under normal condition was HWL 85.000 plus 1.370m wave height by wind. Design hydrostatic head load under seismic condition was HWL 85.000 plus 0.371m wave height due to earthquake. On account of 50% increase of allowable stress of the seismic condition, stress calculation was made under the normal condition. Hoisting and lowering of the gate is controlled by electrically driven hoist installed at the top of spillway structure, i.e at EL.108.000m.

After a comparative study for selection of a suitable gate type, a roller gate with single-leaf type was adopted for the spillway gate among other alternatives, such as a roller gate with double leaf and radial gate, taking into account of construction cost, structural characteristic, and hydraulic performance.

(2) Spillway Stoplog

One set of slide stoplog completed with their guide frame and one set of lifting beam are installed to enable maintenance of the spillway gate after impounding of reservoir. Required dimension of the stoplog is 11m in width and 18m in height, comprises of identical twelve stoplog 11m wide x 1.5 m high each. Installation and removal of the stoplog from the spillway is carried out by using a mobile crane.

(3) Intake Gate

The fixed-wheel intake gate was installed on the sill of the intake at the inlet of the penstock to close the penstock for maintenance and other requirement of the power station. The clear span and clear height of the gate is 6m each. The maximum design water level was at EL.85.000m under normal condition, at EL.85.371m due to seismic wave and at EL.86.370 due to wind wave. Hoisting and lowering of the gate is controlled by electrically driven hoist installed at the top of intake structure, i.e at EL.96.000m.

(4) Intake Stoplog

One set of stoplog, three (3) sets of guide frame and one set of lifting beam was installed and prepared to enable maintenance of the intake gates. The clear span of stoplog is 6m and the total height of the stoplog is 24m consisting 20 pieces of a stoplog with 1.2m height each. The maximum design water level was at HWL 85.000m, height of wave due to wind 1.370m, and height of wave due to earthquake 0.371m. Hoisting and lowering is executed by a mobile crane.

(5) Intake Trash Rack

Three (3) sets of fixed trash rack were installed, clear width 10.0m, vertical height 23.5m (EL.87.5m - EL.64.0m) each which are stacked obliquely along the front face of the intake. The trash bars consist of thickness 12mm, pitch 100mm (center to center), the bars connected to the tie rods and distance pieces. The trash rack was designed for the water head difference 2.0 m acting on the edge of bar elements. The supporting beams were designed for the reaction load due to the water load on the bar element and the load due to its own weight.

(6) Intake Trash Rake, and Conveyor

An intake trash rake of travelling gantry type was installed at the dam crest level of EL.87.5m for removal of debris from the front face of the intake fixed trash rack. The trash rake 3.0m width for rake operation on clear width of 10m for the trash rack.

Rake capacity was designed for lifting load of more than 250 kgf and 0.5m^3 per meter rake width. Debris from the trash rake will be dumped to horizontal belt conveyor of 600mm belt width which will carry it to slopped belt conveyor of 1:0.3 slope.

(7) Tailrace Gate

Tailrace gates were installed at the outlets of the draft tubes for maintenance and inspection of hydraulic turbines. In order to close all draft tube outlets two (2) sets of a slide gate and four (4) sets of stoplog were adopted from economical and construction point of view. The gates will be installed and removed by a gantry crane mounted on top of the tailrace structure.

(8) Tailrace Stoplog

Four (4) sets of stoplog were prepared for maintenance of the tailrace gate and closure of the draft tube. The stoplog consists of five blocks. The guide frames for the tailrace stoplogs will be commonly used with those for the tailrace gate.

(9) Gantry Crane

The gantry crane of 13 ton working load was installed at the top of the tailrace gate structure in order to lower, remove and carry the tailrace gates and the stoplogs.

Table 5.2.1 Design Data of Gate

Description	Spillway Gate	Intake Gate
1. Type of Gate	Fixed wheel gate	Fixed wheel gate
2. Quantity	5 sets	3 sets
3. Design Water Level	EL.85.000+1.370m	EL.85.000+1.370m
4. Sill Level	EL.67.500m	EL.61.000m
5. Clear Span	11.000m	6.000m
6. Height of Gate/Clear Height	18.000m	6.000m
7. Type of Hoist	Electrically driven 1-motor 2-drums wire rope hoist	Electrically driven 1-motor 2-drums wire rope hoist
8. Hoisting height	20.5m	26.8m
9. Raising Operation	-	Balance by-pass valve
10. Operating Speed	0.3m/min ±10%	0.3m/min ±10%
Normal rais/lower		4.0m/min ±10% (EL.87.50-EL.62.00)
Emergency lower		0.3m/min ±10% (EL.62.00-EL.61.00)
11. Operation	Local and remote	Local and remote

Table 5.2.2 Design Data of Stoplog

Description	Spillway Stoplog	Intake Stoplog	Tailrace Gate	Tailrace Stoplog
1.Type of stoplog/gate	Slide stoplog	Slide stoplog	Slide gate	Slide stoplog
2.Quantity	Identical 12 stoplog	Identical 20 stoplog		Identical 5 stoplog x4
Gate leaf	-	-	2 sets	-
Guide frames	5 sets	3 sets	6 sets	Same with Tailrace gate
Lifting beam	1 set	1 set	1 set	- ditto -
3.Design water level	EL.85.000 +1.370m	EL.85.000 +1.370m	FWL 54.820m	FWL 54.820m
4.Sill level	EL.67.273m	EL.61.000m	EL.26.000m	EL.26.000m
5.Clear span	11.000m	6.000m	6.000m	6.000m
6.Height of stoplog/ gate set	18.000m	24.000m	4.550m	4.600m
7.Type of hoist	Truck/crawler crane	Truck/crawler crane	Gantry crane	Same with Tailrace gate

Table 5.2.3 Design Data of Intake Trashrack

1. Type	Fixed type
2. Quantity	3 sets
3. Clear width	10.000 m
4. Vertical height	23.500 m
5. Inclination	1:0.3
6. Bar element	
Bar pitch	100 mm (center to center)
Bar thickness	12 mm2 mm
Corrosional allowance	rectangle
Shape of section	
7. Allowable deflection	1/600

Table 5.2.4 Design Data of Intake Trashrake and Conveyor

Rake	
1. Rake capacity	250 kg/m (debris load) x 3 m = 75 kg 0.5 m^3/m (rake volume) x 3 = 1.5 m^3
2. Rake width	3.0 m
3. Operating speed (all local operation) (Automatic and manual)	Hoisting : 10 m/min Lowering : 20 m/min Open/clause: 5 m/min Travelling : 10 m/min
4. Rail gauge	2.375 m
5. Concrete deck level	EL. 87.500 m
6. High water level	EL. 85.000 m
7. Low water level	EL. 73.500 m
8. Sill level	EL. 64.000 m
9. Total width of trashrack	30.0 m (10.0 m x 3 span)
10. Bar pitch	100 mm
11. Thickness of bar	12 mm
12. Inclination	1:0.3
Belt conveyor	
1. Belt width	0.60 m
2. Length	Approx. 53+ 9 m (Horizontal + Vertical/Inclined or Slope)
3. Operating speed (All local operation)	Approx. 10 m/min
4. Carrying capacity	Approx. 30 m^3/H

Table 5.2.5 Design Data of Tailrace Gantry Crane

1. Capacity	13 ton
2. Hoist type	2xDH1040 H40kV 1-2x2/1 FS
3. Hoisting speed	1.0 m/min ± 100% 2x1.55 kW 100%ED
4. Lowering speed	5 m/min ± 10% (EL. 61.00-EL.27.00)2x7kW 100% ED 1 m/min ± 10% (27.00 - EL.26.00)2x1.55kW 100% ED
5. Long travel speed	10 m/min ± 10% 2x1.55 kW 100% ED
6. Current supply	380 V 3 Phase 50 Hz
7. Control voltage	220 V 1 Phase 50 Hz
8. Crane class	Group Hz B3 DIN 15018
9. Crane control	Via push button control from desk type panel in the operating cabin
10. Operation	Local
11. Dead wt. of crane	10 tons
12. Material	Structure-ASTM A36 or Equiv., Wheel-DIN GG 65, Rail-JIS E 1101

2.1.3 Design and Fabrication Conditions

(1) General

All materials were new, the best of their respective kinds, and of such as are usual and suitable for work of like character. All materials complied with the latest Japanese Industrial Standards or equivalent ones.

The workmanship was of the highest class to ensure smooth and vibration free operation under all possible operating conditions, and the design, dimensions and materials of all parts were such that the stresses to which they may be subjected were not rendered them liable to distortion, undue wear, or damage under the most severe conditions encountered in service.

All parts conformed to the dimensions shown on the specification drawings and were built in accordance to the approved shop drawings. All joints, datum surfaces, and matching components were machined and all castings were spot faced for nuts. All screws, bolts, studs and nuts and threads for pipes conformed to the latest standards of the International Organization for Standardization (ISO) covering these components and conformed to the standards for metric sizes.

(2) Standard Specification and Approved Procedures

Standard Specification

Japanese Standards issued by the Japanese Standards Association were used for the design specifications. Design and Fabrication of structural parts in principle conformed to the applicable provisions of the "Technical Standards for Gates and Penstock" issued by the Water Gate and Penstock Association of Japan. All electrical equipment, materials and details of installation complied with the requirements and latest revisions of the following codes and standards where applicable :

a. Japanese Industrial Standards (JIS)
b. Technical Standards for Gate and Penstock, Japan
c. Standard of the Japanese Electrotechnical Committee (JEC)
d. Japanese Technical Standard for Electrical Facilities
e. Japanese Engineering Standards (JES)
f. The standard of the Japan electrical Manufacturer's Association Standards (JEM)
g. Japanese Cable Markers Association Standards (JCS)
h. Japan Gear Manufacturer's Association Standards (JGMA)
I. International Organization for Standardization (ISO)
j. American Society of Testing Materials (ASTM)
k. American Institute of Steel Construction (AISC)
l. Indonesian Industrial Standard (SII)
m. PLN Standard (SPLN)

Approved Procedures

The main items of the approved procedures were as follows :

- Work Master Schedule (Overall Procedure)
- Shop Test Schedule and Procedure
- Method Statements
- Operation and Maintenance Instruction

(3) Design Stresses

Design stress for gate, stoplog, trashrack and other steel structures are as follows :

(a) Steel Structural Members

1) Allowable Stress of Material

i) The allowable stresses under normal loading condition of structural steel members less than 40mm in thickness is shown in the following Table 5.2.6.

Table 5.2.6 Allowable Stress of Materials

Unit : kgf/cm^2

Material		SS400, SM400 (thickness < 40 mm)	SM490 (thickness < 40 mm)
1.	Axial tensile stress (per net section area)	1,200	1,600
2.	Axial compressive stress (per gross section area)	$0 < l/r \leq 110$ $1,100 - 0.048\ (l/r)^2$	$0 < l/r \leq 90$ $1,500 - 0.09\ (l/r)^2$
a.	Compressive members l=buckling length of member r=radius of gyration of area of gross section area of members	$l/r > 110$ $6,350,000/(l/r)^2$	$l/r > 90$ $6,350,000/(l/r)^2$
b.	Splice plates	1,100	1,500
3.	Bending stress	1,200	1,600
a.	Bending tensile stress (per net section area)		
b.	Bending compressive stress (per gross section area) l=flange supporting length b=flange width	$1,100\text{-}0,5(l/b)^2$ however $l/b \leq 30$	$1,500\text{-}0.9(l/b)^2$ however $l/b \leq 30$
c.	Compressive flanges directly welded or rivetted, as in the case of skin plates	1,100	1,500
4.	Shearing stress (per gross section area)	700	900

Allowable stresses under normal loading condition of the structural steel members not less than 40mm in thickness shall be within 0.92 times of the allowable stress in Table 5.2.6 in case of the steel material SS400 and SM400 and 0.94 times in case of the steel material of SM490. The allowable stresses if other materials are shown in the following Tables 5.2.7 and 5.2.8.

Table 5.2.7 Allowable Stress of Joint Materials

Unit : kgf/cm²

Material	SS 400, SM 400 SV 330	SM 490 SV 400
1. Shearing stress		
Reamed bolts	750	1,000
Anchor bolts	500	650
Rivets (shop)	950	1,200
Rivets (field)	850	1,050
2. Bearing stress		
Rivets (shop)	1,950	2,550
Rivets (fields)		
Reamed bolts	1,700	2,300

Table 5.2.8 Allowable Stress of Steel Castings

Unit : kgf/cm²

Material	SC 450
Axial tensile stress	1,200
Axial compressive stress	1,200
Bending stress	1,200
Shearing stress	700
Bearing stress	1,700

ii) The allowable combined stresses resulting from combination of biaxial or triaxial principal stresses may be 50 percent higher than those under normal loading condition. In no case, however, shall any stresses exceed 90 percent of the yield strength or elastic limit of the steel material. The combined stress was calculated by the following formula:

$$\sigma g = \sqrt{\sigma x^2 + \sigma y^2 - \sigma x . \sigma y + 3\tau^2}$$

where, σg : Combined stress (kgf/cm²)
σx : Stress on x - axis (kgf/cm²)
σy : Stress on y - axis (kgf/cm²)
τ : Shearing stress (kgf/cm²)

2) Allowable Stress of Trashrack Bar

The stresses in the bar element of trash rack were not exceed the following allowable stress.

$$fcr = 0.6fy\ (1.23 - 0.0153\ L/t)$$

where, fcr : Allowable stress (kgf/cm²)
fy : Yield strength of the material (kgf/cm²)
L : Tie bolt distance (cm)
t : Thickness of the bar element (cm)

3) Allowable Stress of Other Material

The allowable tensile stresses of other materials than those mentioned in the Table of point a.1) under normal loading condition was not exceed 50% of the yield strength of the material. The allowable stresses was computed in proportion to the allowable stresses given in the Table of point a.1) based on the yield and/or ultimate strengths of the material.

4) Increase of Allowable Stress

Allowable stresses under seismic conditions may be increased by 50% of those above mentioned. The allowable stresses for stoplog and tailrace gate may be increased 15% of those above mentioned.

(b) Mechanical Parts

All mechanical parts of the equipment subjected to normal or rated capacity loading condition were designed with the following factors of safety (FS) against the ultimate strength of the steel material.

Table 5.2.9 Factors of Safety for Mechanical Parts

Material	FS for Tensile Stress	FS for Compressive Stress	FS for Searing Stress
Rolled steel for general or welded structure	5	5	8.7
Carbon steel forgings	5	5	8.7
Carbon steel for machine structural use	5	5	8.7
Corrosion-resisting steel	5	5	8.7
Carbon steel castings	5	5	8.7
Gray iron castings	10	3.5	10
Bronze castings	8	8	10

The Unit stress was not exceed ninety (90) percent of the yield strength of the material under the loading resulting from the breakdown or locked-rotor torque of the hoist motor, whichever is greater.

(c) Mechanical Efficiency

For the design of operating machineries for the gates, mechanical efficiencies stated hereunder was applied to the individual machinery component :

Machinery	Efficiency
Spur gear and, helical gear (oil bath)	0.97
Spur gear (open gear)	0.95
Bevel gear (oil bath)	0.95
Bevel gear (open gear)	0.90
Drum	0.95
Sheave	0.95
Pulley	0.95
Worm gear	with evidence by calculation

(d) Concrete Stress

The concrete bearing and shearing stresses were not exceed 50 and 7 kgf/cm² respectively.

(e) Materials

Materials were selected of the most suitable materials for the functions of the hydromechanical mechanism/structure as shown in table below :

Code	Class	Description	Applied
JIS G 3106	SM 490 YA SM 400 B	Rolled steels for welded structure	Gate leaf, Guide Frame Trashrack Penstock Ladder, Roller, Cover
JIS G 3101	SS400	Rolled steel for general structure	
JIS G 3452	SGP	Carbon steel pipes for ordinary piping	Air pipe
JIS G 4304	SUS 304 SUS 304 N2	Hot rolled stainless steel plates, sheets and strip	Roller to plate Seal plate
JIS G 4303	SUS 304 SUS 403 SUS 402 J2	Stainless steel bars	Shaft, Bolt & Nut
JIS E 5402	SSW-Q1	Solid rolled carbon steel Wheels for railway rolling stock	Main roller
JIS G 5501	FC 250	Gray iron castings	Sheave

2.1.4 Design Details

(1) 11m W by 18mH Spillway Fixed Wheel Gates and Hoists

(a) Design Requirement

Five sets of vertical lift fixed wheel gate were installed in the spillway. They serve for emergency spillway regulation and will be normally closed except for flood periods when the reservoir water level exceeds or is likely to exceed HWL 85.000m.

The basic requirements for design of the spillway gates and hoist are as follows :

Type of Gate	:	Steel-construction fixed wheel gate
Clear span	:	11.000m
Height	:	18.000m
Water tightness	:	Three-side rubber seal
Design head	:	18.870m
Elevation of sill	:	EL.67.500
Corrosion allowance	:	1mm for skinplate
Deflection of girders	:	Less then 1/800 of the span
Type of hoist	:	Electrically driven 1-motor 2-drum wire rope hoist
Hoisting height	:	20.5m
Hoisting load	:	109.626 tf
Hoisting speed	:	0.3 m/min ±10%
Electric source	:	3 phase, 4 wire, AC 380/220V, 50Hz
Control system	:	Local and remote control
Quantity	:	5 sets

(b) Gate Design

The general arrangement and the assembly of gate leaf are shown in Figs. 5.1 and 5.2 respectively. The design head was 18.870m at HWL 85.500+1.370m wave due to wind. The maximum gate opening is 20.00m. The gate contacts the sillplate on the crest at EL.67.500.

Each gate consists of four separated leaves in order to securely transmit the water loads to the embedded steel tracks without warping by undue deflection and completed with fixed wheels. It was fabricated and shipped in twelve sections to facilitate handling, shipment, and field assembling. A skinplate was welded to the upstream side of 11 vertical ribs.

The framework of each leaf consists of two horizontal web girders, bracings into vertical girder beams at each end of the gate. Eight wheels (two wheels for each leaf) were fitted on to each side vertical beams and serve to transfer the hydraulic load from the gate to the corrosion resisting steel tracks.

The tracks were welded to steel base girders and embedded in the concrete at the downstream face of gate slots. Lateral movement of the in loaded gate is controlled by guide rollers on each end of the gate, which contact metal guides embedded in the concrete at each end of the gate slot.

Solid rubber, side type seals were provided in both sides of the upstream face of the skinplate on the gate. These seals contact stainless-steel seal seat plates embedded in the concrete at the upstream face of the slots. Solid rubber, flat seal was fastened to the bottom of the skinplate. When the gate is on the sill, the bottom seal is compressed against the stainless-steel sillplate, thereby preventing leakage along the bottom edge of the skinplate.

The gate was designed for a head of 18.87m, assumed to uniformly distribute over the entire gate leaf. The hoist connections on the gate were designed for the pull-out force of the hoist. By sectionalization to four gate blocks, the gate blocks were designed structurally independent each other to sustain the hydrostatic pressure. Two main girders and four wheels are distributed for one gate block in such a manner that they sustain equalized hydrostatic load.

The horizontal beams were assumed as simple beams supported by the vertical girder beams at each of the gate block leaf, and the vertical girder beams, as simple beams supported by main fixed wheels. The skin plate was assumed to be uniformly loaded rectangular plate, and was also assumed to act as a continuous beam with haunch (beam flanges) supported on the webs of the vertical beams. The maximum combined stress in the skinplate due to beam bending and skin plate bending was limited to 150% of the normal allowable stress of 1850 kgf/cm^2. Each wheel has self aligning roller bearings completed with grease lubricating nipple.

The guide frame consists of the side and sill sealing frames, wheel tracks and side roller paths. The sealplate and track plate of stainless steel plate is attached to sealing frames and track frames.

(c) Hoist Design

The hoist assembly was designed as a cable lift type suitably mounted on the hoist deck for raising or lowering the gate. The hoist consist of a center drive unit and two drum units, one located just above each gate wire rope sheave. The center drive unit consists of a 11kW electric motor with magnetic brake, hydraulic type brake and cycle reduction unit and 2nd reduction gearing. The output shaft was coupled through single spurgear reduction train to the drums.

The drums units consist of grease lubricated spur reduction gearing and a drum with right or left hand helical grooving. The wire ropes suitable for the hoist, ϕ37.5 mm 6x37 Class B galvanized extend from each drum to gate sheaves and other end of the wire ropes connected to wire rope adjuster located on the hoist base frames. The wire ropes are wound on the drums which were grooved to a depth of about 30% of each diameter of the wire ropes.

Anti friction bearings were used in fabrication of the hoist. The hoist was designed with a safety factor of 5 based on the ultimate strength of the materials. Also each part was designed so that the maximum working stress might be 90% or less of the yield stress of the material based on the maximum motor torque 250% of rated motor torque. The hoist wire ropes were designed with a safety factor of 8 based on the breaking strength of the wire ropes.

The hoist was provided with manual operating device which can lift-up the full design load. The manual force of handling device was less than 10kg. While the device is being operated, the electric motor will be disconnected from the system. Each hoist is operated through either local control cabinet, or a remote control cabinet located in the control room in the powerhouse.

Each local control cabinet is located near the hoist and is weather-proof outdoor construction. A remote control cabinet is adequately sized to contain the remote controls for five hoists. A mechanical gate position indicator was provide on each hoist, and electrical position indicators were mounted on the remote control cabinet.

(2) Spillway Stoplog

(a) Design Requirements

One set of stoplog comprising twelve logs, 11m wide by 18m high, vertical lift slide type, are provided in front of the spillway roller gate for the purpose of inspection and maintenance of the spillway gates, seals or embedded seating surfaces. A lifting beam for raising, lowering and moving the stoplog is provided on the dam. A mobile crane for hoisting the stoplog shall be hired from others.

The basic design requirements of the stoplog are as follows :

Type of stoplog	:	Steel-construction slide stoplog
Quantity	:	One set (12-logs)
Clear span	:	11m
Height	:	18m
Design head	:	19.097m for each log
Deflection of beams	:	Max. 1/600 of clear span
Hoisting height	:	Max.20m
Hoisting load	:	9.0 tf (under balance water pressure)

(b) Stoplog Design

The general assembly of the stoplog are shown in Fig.5.1 and 5.3 respectively. The 11m wide by 1.5m high stoplog is of welded steel construction and was fabricated of structural steel beams and a steel plate. The water load on the upstream side of the stoplog is transmitted to the concrete through the skinplate, the wide-flange-beams, the vertical beams of stoplog and embedded guide frames.

The sealing is obtained by music-note type of molded rubber seals attached on the skinplate. The own weight of stoplog compresses the bottom seal and/or seal seat on the top of each stoplog. The stoplog was designed for head of 19.097m assumed to uniformly distribute over the entire leaf and for raising and lowering under conditions of no flow in the spillway and balanced water pressure on either side of the stoplog.

The stoplog guide frame consists of sealing frames, sill beam with seal seats, track frames, side guide rails. The stainless steel seal plate attached to all sealing frames. The stoplogs are equipped with taper wedges which by contacting the guides provide initial seating before the water pressure is introduced; this initial seating prevents "bearing" from suddenly applied pressure.

(3) Intake Gate and Hoist

(a) Design Requirement

A 6m by 6m fixed wheel gate was installed at the penstock inlet. The gate is required to operate for emergency closure of the penstock in case of damage to the turbine generating set. The intake gate is also closed when dewatering the penstock for inspection and maintenance purposes. The general arrangement of the intake gate are shown in Fig.5.4 and 5.5 respectively.

The design requirement of the intake gate and hoist are as follows :

Type of gate	:	Steel construction, fixed wheel gate
Clear span	:	6m
Height	:	6m
Water tightness	:	Four-side rubber seal
Elevation of sill	:	EL.61.00m
Design head	:	25.370m
Corrosion allowance	:	1mm for skinplate
Deflection of girder	:	Max. 1/800 of clear span
Type of hoist	:	Electrically driven 1-motor 2-drum wire rope hoist
Hoisting height	:	26.8m
Hoisting load	:	30tf
Hoisting speed		
Normal raising and lowering	:	0.3m/min ±10%
Emergency lowering	:	4.0m/min ±10% (EL.87.50-EL.62.00) 0.3m/min ±10% (EL.62.00-EL.61.00)
Electric source	:	3φ, 4W, 380/220 V, 50 Hz
Control method	:	On local and remote cabinets
Raising operation	:	Balance by-pass valve
Quantity	:	3 sets

(b) Gate Design

The gate operates on vertical tracks extended from the sill at EL.61.000 to the top of the hoist deck placed at EL.96.000. The gate is normally held in the open position immediately below the hoist deck using wire rope hanging if manual emergency lowering is required. A dogging device was equipped for holding the gate, when repaired. The detailed assembly of the gate leaf is shown in Fig.5.5. and Fig.5.6.

The gate is a flat structural steel member of welded construction. It was fabricated and shipped in two separate sections to facilitate handling, shipment, and field assembling. A skinplate was welded to the downstream side of the main gate framework. The framework consists of three horizontal beams, framing into vertical girder beams at each end of the gate.

Four wheels are attached onto vertical beams and serve to transfer the hydraulic load from the gate to the stainless steel tracks. The tracks were welded to steel base girders and embedded in the concrete at the downstream face of gate slots. Lateral movement of the in loaded gate is controlled by guide shoes, on each end of the gate tracks. Solid rubber seals were provided on the top, both sides and bottom of the downstream face of the skinplate on the gate. These seals contact stainless steel seal seat plates welded to the guide frame around the bell mouth of the inlet.

The gate was designed for a head of 25.370m, assumed to uniformly distribute over the entire gate leaf. The hoist connections on the gate were designed for the pull-out force of the hoist. By sectionalization to two gate blocks, the gate blocks were designed structurally independent each other to sustain the hydrostatic pressure. Three main girders and four wheels are distributed for one gate block in such a manner that they sustain equalized hydrostatic pressure load.

The horizontal beams were assumed as simple beams supported by the vertical girder beams at each end of the gate leaf, and the vertical girder beams were assumed as continuous beams supported by main fixed wheels. The skinplate was assumed to be uniformly loaded rectangular plate, and was also assumed to act as a continuous beam with haunch (beam flanges) supported on the webs of the horizontal beams.

The maximum combined stress of the skinplate due to beam bending and skinplate bending was limited to 150% of the normal allowable stress of 1810 kgf/cm^2 (JIS SM490 YA). Each wheel has self-aligning roller bearings completed with grease lubricating nipple.

The guide frame consists of the side and lintel sealing frames, sill beam, wheel tracks with side shoe paths. The sealplate of stainless steel plate is attached to sealing frames and track frames.

(c) Hoist Design

The hoist assembly was designed as a cable lift type suitably mounted on the hoist deck for raising or lowering the gate. The hoist consists of a center drive unit located at the centerline of the gate and two drum units, one located just above each gate wire rope sheave.

The center drive unit consists of a 3.7kW electric motor with magnetic brake, hydraulic type brake, differential gear device. The output shaft was coupled through single spur gear reduction train to the drums. The drum units consists of grease-lubricated spur reduction gearing and a drum with right or left hand helical grooving.

The wire ropes suitable for the hoist, ϕ33.5 (6x37), Class A galvanized extend from each drum to gate sheaves and other end of the wire ropes connected to wire rope adjuster located on the hoist base frames. The wire rope are wound on the drums which were grooved to a depth of about 25% each diameter of the wire rope.

Antifriction bearings were used in fabrication of the hoist. The hoist was designed with a safety factor of 5 based on the ultimate strength of the materials. Also each part was designed so that the maximum working stress might be 90% or less of the yield stress of the materials based on the maximum motor torque 250% of rated motor torque.

The hoist wire ropes were designed with a safety factor of 8 based on the breaking strength of the wire ropes. The hoist is provided with a manual operating device which can lift up the gate leaf under balanced water pressure. The manual operating force of the handle was less than 10kg. The hoist is operated through a local control cabinet and remote control cabinet located in the control room. For the emergency lowering, the hoist equipped with emergency lowering device (hydraulic brake) completed with differential gear device and electro hydraulic lifter brake with speed regulator.

(4) Intake Stoplog

(a) Design Requirement

One set of stoplog comprising 20 logs, 6m wide by 24m high, vertical lift slide type are provided in front of the intake roller gate for the purpose of inspection and maintenance of the spillway gates, seals or embedded seating surface. A mobile crane for hoisting the stoplog shall be hired from others.

The basic design requirements of the stoplog are as follows :

Type of stoplog	:	Steel-construction slide stoplog
Quantity	:	One set (20-logs)
Clear span	:	6m
Height	:	24m (20 x 1.2m)
Design head	:	25.370m for each log
Deflection of beams	:	Max. 1/600 of clear span
Hoisting height	:	Max. 40m
Hoisting load	:	6.275 tf

(b) Stoplog Design

The general of the stoplog is shown in Fig.5.4 and 5.6. The 6m wide by 1.2m high stoplog is of welded steel construction and was fabricated of structural steel beams and a steel face plate. The water load on the upstream side of the stoplog is transmitted to the concrete through the skinplate, the wide-flange beams, the vertical beams of stoplog and embedded guide frames. The sealing is obtained by music-note type of molded rubber seals attached on the skinplate. The own weight of stoplog compresses the bottom seal seat and/or seal seat on the top of each stoplog.

The stoplog was designed for head of 25.370m assumed to uniformly distribute over the entire leaf and for raising and lowering under conditions of no flow in the intake.

The stoplog guide frame consists of sealing frames, sill beams with seal seats, track frame, side guide rails. The stainless steel seal plate attached to all sealing frames and truck frames. The stoplogs are equipped with taper wedges which by contacting the guides provide initial seating before the water pressure is introduced : this initial seating prevents "bearing" from suddenly applied pressure.

(5) Intake Trash Rack

The fixed trash racks installed in the intake are composed of 25 rack panels, 2m wide x 5.477m high each which are stacked obliquely along the intake front face (see Fig.5.7). The trash rack section consists of 100mmx12mm trash bars connected to the tied rods and distance pieces. The rack panels are supported by H-section steel horizontal beams, which in turn transmit the load to the side concrete wall structure.

The trash racks were designed for the water load of 2tf/m^2 acting on the edge of bar elements. The supporting beams were designed for the reaction load due to the water load on the bar element and the load due to its own weight.

The allowable stress of the bar element was calculated by the formula, critical stress:

$$\sigma c = 0.6 \times \sigma y \, (1.23 - 0.0153L/t)$$

where, σc= allowable stress
σy= yield point stress
L = laterally supported length of bar element in centimeters, subject to $L \leq 70t$
t = thickness of bar net of corrosion allowance of 2mm

The maximum allowable deflection of the supporting beam was taken at 1/600 of the clear span at the center under the full load.

(6) Intake Trash Rake and Conveyor

(a) Design Requirement

A travelling type raking equipment is equipped on the intake top floor at EL.87.500m for removal of debris from the front face of the intake fixed trash rack. The design specifications of the raking equipment are as follows :

Type	:	Travelling type		
Operating speed	:	Hoisting	:	10 m/min
(All local operation)	:	Lowering	:	20 m/min
(Automatic & Manual)	:	Open/Close	:	5 m/min
Travelling	:	10 m/min		
Lifting length	:	28.2m		
Raking capacity	:	250 kg/m (Debris load) x 3m = 750kg		
		0.5 m^3/m (Rake volume) x 3m = 1.5m^3		
Scraping length	:	Max. 25m		
Electric source	:	AC 380/220V, 50 Hz, 3 phase, 4 wire		
Control system	:	Local cabinet		
Quantity	:	1 set		

The design specifications of the conveyor are as follows :

Belt width	:	0.6m
Length	:	Approx 53+9m (Horizontal&Vertical/Inclined&Sloped)
Operating speed	:	Approx. 10 m/min
(All local operation)		
Carrying capacity	:	Approx. 30 m^3/H
Quantity	:	1 set

(b) Design Details

The raking equipment consists of rake, lifting and dumping mechanisms, travelling unit and belt conveyor. The general assembly of the raking equipment is shown in Fig.5.8.

The 3m wide (24.61m long travel) rake consists of bar elements, supporting framework, two 300mm dia. flanged roller, two 100mm dia. flanged guide rollers and a dumping arm. All the parts were designed for a lifting load of 1.810tf including self weight. The bar elements are of rectangular section with 22mm thickness. The bars were spaced at 100mm centers. All the bar elements were welded to the framework.

The hoisting equipment consists of a cycle reducer (1/59) with a 7.5 kW electric motor, roller chain coupling (17/35), driving shaft and two 500mm dia. wire drums. The dumping equipment is of motor cylinder which operated by 0.75 kW electric motor. The travelling equipment consist of a cyclo reducer with a 1.5kW electric motor and electro-magnetic brake, sprocket, driving shaft, roller chain coupling, and four wheels.

The rubber horizontal belt conveyor is placed under raking equipment. The conveyor is operated by cyclo reducer with a 1.5kW electric motor through the chain sprocket wheels and conveyor chains. The framework and rails were designed for the maximum acting load of the equipment. The control cabinet is located near the equipment.

The rubber sloped belt conveyor is placed at the end of the travelling routed of rake equipment. The sloped conveyor is also operated by cyclo reducer with a 1.5kW electric motor through the chain sprocket wheels and conveyor chains. The framework and rails were also designed for the maximum acting load of the equipment. The control cabinet is located near the equipment.

(7) Tailrace Gate

(a) Design Requirement

Two sets of tailrace gate comprising two blocks each, 6.5m wide by 4.6m high, vertical lift slide type, were installed at the tailrace. The gates are used to dewater the draft tube for inspection and maintenance purposes. The general arrangement and the structural assembly of the tailrace gate are shown in Fig.5.9 and 5.10 respectively. A gantry crane with a lifting beam for raising, lowering and moving the gate, is provided on the tailrace.

The design specification of the tailrace gates are as follows :

Type of gate	:	Slide gate (made of steel)
Clear span	:	6m
Clear height	:	4.55m
Water tightness	:	Four-side rubber seal
Design head	:	28.82m (lower gate block)
	:	26.62m (upper gate block)
Elevation of sill	:	EL.26.000
Hoisting height	:	35.0m
Hoisting load	:	12.4ton
Type of hoist	:	Gantry crane

(b) Gate Design

The gates were designed for raising and lowering only under a condition of no-flow in the draft and balanced pressure on either side of gate leaf. The gates are of welded construction, and were fabricated of structural steel beams and steel plate.

The water load is transferred to the concrete structure through the gate side bearing plates against the embedded guide frames. The sealing is obtained by a continuous music-note type of molded rubber seal attached around the gates. Water pressure compresses the side and top seals against the stainless seats, and the weight of the gate compresses the bottom seal against the embedded sill beam.

The tailrace gates are normally stored in the upper portion of the gate slots just below the grating where they are supported on both sides by dogging devices so located that the lifting lug on the gate is just below the grating and therefore easily accessible from the deck.

In order to equalize the pressure on each side of the gate, a by-pass valve is equipped in the side wall. The by-pass valve for the tailrace gate is manually operated. After allowing sufficient time to elapse to filling the draft tube and tailrace tunnel, the respective gate may be raised and stored in position. Each guide frame consists of the side frames, sill and lintel frames. The seal plate of stainless steel plate is attached to sealing frames and truck frames.

(8) Tailrace Stoplog

(a) Design Requirement

Four sets of stoplog comprising five logs each, 6.5m wide by 4.6m high, vertical lift slide type, are provided at the tailrace. The stoplogs are used to dewater the waterway for inspection and maintenance purposes. A gantry crane with a lifting beam for raising, lowering and moving the stoplog, are provided on the tailrace (the same equipment as used for the tailrace gate). The general arrangement and the structural assembly of the tailrace stoplog are shown in Fig.5.9 and 5.11.

The design specification of the tailrace stoplog are as follows :

Type of stoplog	:	Steel-construction slide stoplog
Quantity	:	Four sets (4 x 5 logs)
Clear span	:	6m
Height	:	4.6m (5 x 0.92m)
Design head	:	28.82m
Deflection of beams	:	1/600
Hoisting height	:	35.0m
Hoisting load	:	3.3 tf
Type of hoist	:	Gantry crane

(b) Stoplog Design

The general assembly of the stoplog and gantry crane are shown in Fig.5.9 and 5.11. The 6.5m wide by 0.92m high stoplog is of welded steel construction and was fabricated of structural steel beams and a steel face plate. The water load on the downstream side of the stoplog is transmitted to the concrete through the skinplate, the wide-flange-beams, the vertical beams of stoplog and embedded guide frames. The sealing is obtained by music-note type of molded rubber seals attached on the skinplate. The own weight of stoplog compresses the bottom seal seat and/or seal seat on the top of each stoplog.

The stoplog was designed for head of 28.82m assumed to uniformly distribute over the entire leaf and for raising and lowering under conditions of no flow in the tailrace and balance water pressure on either side of the stoplog. The stoplog guide frame are commonly used with those for the tailrace gate.

(9) Gantry Crane

The gantry crane, outdoor gantry crane type electric hoist is provided for the tailrace gate and stoplog operation. The equipment consists of the machine frame, electric hoist, travel motor, completed with machine house, closed cabin, control panel, control station, chair, cable reeling drum, floodlight, rails. The electric hoist consists of the reducing gear unit, electric motor with magneting brake, wire drum, wire rope and sheave. The hoist is operated by means of push-button switched at the control panel. The rails cover the full travel length of the six tailrace openings and is layed down on the top of the tailrace gate structure at EL. 60.500.

The design capacity of the hoist is 13.390tf. The hoisting and travelling speed were set at 1m/min and 10m/min respectively. Vertical lift of 35.1m.

2.2 Construction (Shop Fabrication and Site Installation)

2.2.1 Outline

The site installation work of the gates for Kotapanjang spillway, intake and tailrace structure started on April 01, 1995 by Sumitomo Corporation and its associated manufacturer, Ishikawajima-Harima Heavy Industries Co.Ltd (IHI) with its local sub Contractor PT.Barata Indonesia (Persero).

2.2.2 Construction Progress and Resource Use Record

(1) Shop Fabrication

(a) Typical Sequence of Fabrication was i) Procurement of materials, ii) Marking of steel plates, iii) Cutting of steel plates, iv) Assembling of parts, v) Welding of joints and vi) Painting of steel surface.

The most important requirements for proper operation of the gates are water tightness during closure and smooth water flow when the gate is in open position. The above can be achieved only through careful design and construction of the gates and their various components, such as the embedded parts, guide frames, hoisting equipment, and civil works water passages. In addition, proper fitting in the concrete of the first and second stage embedded parts play in important role in satisfying the above requirements. The correct fitting of the gate leaf to the guide frame was obtained by occur fabrication. All gates were of welded steel construction. Implementation of the procurement of materials can be seen in the following of Shipping Report.

5.2.10 SHIPPING REPORT

No.	Items	Vessel Name	From	ATD (ETD)	To	ATA (ETA)	Condition
1-1	Section Steel for Spillway Gate Guide Frame Spillway Stoplog (S.S) Tailrace Gate and Stoplog (T.G)	Tanjung Emas	Kobe	Feb.25,94	Surabaya	Mar.16,94	arrived at shop May 17,1994
1-2	Plate Material for Spillway Gate Guide Frame S.S and T.G	Cosmic Leader	Yokohama	Mar.10,94	Surabaya	Mar.31,94	arrived at shop Jun.02,94
2-1	All Materials and Parts for Lower Portion of Penstock	Baab Ullah	Yokohama	May 26,94	Belawan	Jun.08,94	arrived at Site June.23,94
2-2	Section Steel and Parts for Penstock (Intake Section)	Kiat Baru	Kobe	Jun.11,94	Surabaya	Jul.05,94	arrived at Shop Jul.22,94
2-3	All Materials and Parts for Spillway Gate, Gate Leaf Parts for S.G.Guide Frame, S.S and T.G	Kiat Baru	Kobe	Jun.11,94	Surabaya	Jul.05,94	arrived at Shop Oct.25,94
2-4	Plate Materials for Penstock (Intake Section)	Cosmic Master	Yokohama	Jun.29,94	Surabaya	Jul.22,94	arrived at Shop Aug.11,94 Sep.01,94
2-5	Plate Materials for Spillway Gate Leaf	Cosmic Master	Yokohama	Jun.29,94	Surabaya	Jul.22,94	arrived at shop Nov.29,94
3-1	All Materials and Parts for Intake Gate, Stoplog, Trashrack Spillway Gate Hoist	Cosmic Pioner	Yokohama Kobe	Oct.17,94	Surabaya	Oct.31,94	arrived at shop Apr.10,95
3-2	Section Steel and Parts for Upper portion of Penstock	Champion Trader	Kobe	Oct.14,94	Belawan	Nov.16,94	arrived at Site Dec.02,94
3-3	Plate Material for Upper portion of Penstock	AKEBIA	Yokohama	Oct.12,94	Belawan	Nov.16,94	arrived at Site Dec.05,94
4	Gantry Crane	Bina Indah	Singapore	Jan.15,96	Dumai	Jan.17,96	arrived at Site May.22,96
5	Trashrake, Control Panel Spare parts	Sun Pride	Kobe	Dec.20,95	Belawan	Jan.08,96	arrived at Site May.20,96

(b) Flow Chart of Fabrication and Installation

The flow charts depicting the steps employed in manufacturing of the guide frames, gate leaves and hoists, are shown in Figures 5.12, 5.13 and 5.14. They cover all the activities from the arrival of the materials at the factory or the site plant to the completion, installation and final testing and operation.

(c) Fabrication Progress

The actual fabrication work progress is shown in Fig.5.15. The shop fabrication work started at June 06, 1994 with the tailrace gate guide frames and was completed at April 04, 1996 with the intake gate. The actual starting and finishing date of fabrication activities of gates is shown in Table 5.2.11.

Table 5.2.11 Starting and Finishing Date of Gates Fabrication at Surabaya - Shop

No.	Key Description	Actual Start Date	Actual Finishing Date
1.1	Spillway gates, gate leaf	Jan,01,1995	Feb.13,1996
1.2	Spillway gates, guide frame	Jun.06,1996	Dec.15,1994
1.3	Spillway gate, hoist	Apr.20,1995	Feb.13,1996
2.1	Spillway stoplog, stoplog	Aug.08,1994	Jul.28,1995
2.2	Spillway stoplog, guide frame	Jun.20,1994	Nov.10,1994
3.1	Intake gates, gate leaf	Aug.21,1996	Apr.04,1996
3.2	Intake gates, guide frame	Apr.20,1995	Sep.29,1995
3.3	Intake gates, hoist	Jul.10,1995	Apr.04,1996
4.1	Intake stoplog, stoplogs	Sept.13, 1995	Nov.23,1995
4.2	Intake stoplog, guide frame	Sep.13,1995	Nov.23,1995
5	Trashrack	Jun.20,1995	Nov.23,1995
6	Trashrake	Jun.01,1995	Oct.28,1995
7.1	Tailrace gate, gate leaf	Aug.08,1994	Feb.09,1995
7.2	Tailrace gate, guide frame	Jun.06,1994	Oct.07,1994
8.1	Tailrace stoplog, stoplog	Jun.20,1994	Feb.10,1995
8.2	Tailrace stoplog, storage facility	Nov.01,1994	May 30,1995
9	Tailrace gantry crane	Sep.05,1995	Dec.08,1995

No.
1-1

...tion for the gates was ; (i) installation of embedded parts by the Civil Contractor under the supervision of TEPSCO, ... stage concrete by the Civil contractor, (iii) installation of gate guide frames second, lintel third, and side rail last, (iv) placement of concrete by the Civil Contractor, (v) installation of gate leaves, (vi) of hoists, (vii) installation of electric (control) equipment and (viii)

(2) Site

(a)

...ection of valves and trashrack was somewhat different but it followed similar procedure as applicable.

(b) Inspection During Installation

Typical inspection items during installation were as follows :

1) Delivery

A visual inspection was performed of the materials and finished products on arrival at the erection site for possible damage due to transportation and/or mishandling.

2) Dimensions and Workmanship

This consisted of checking the dimension during and at the end of the installation by tape and level and as required to assure that the position and the dimensions were within the tolerances required by the specifications. Visual checking of the workmanship was performed for compliance with the provisions of the specifications.

3) Welding

Quality of butt welding was checked by radiographic examination applying RT procedure.

4) Performance During Operation

Performance of the equipment during operation was examined by visual inspection and use of testing equipment. But before the equipment was put into operation, its operational performance was checked by testing. For the gate and the valve, two types of testing were conducted. One was operational test under dry conditions (dry test) and the other, under wet conditions (wet test which is remained at impounding and carried out by Employer).

The operational test under dry conditions was carried out in order to determine whether the gates could be operated properly. The main purpose of the test under wet conditions was to determine the watertightness of the gate, and in particular, proper sealing of the gate contact surfaces with the embedded steel frame of the gate. During these tests, the amount of leakage through the contact surfaces was always smaller than that was allowed by the specifications.

(c) Installation Progress

The actual site installation work progress is shown in Fig.5.15. The site erection work started at April 01 1995 with the tailrace gate guide frames and was completed at the end of October 1996.

Table 5.2.12 shows the actual starting and finishing date of installation works.

Table 5.2.12 Starting and Finishing Date of Installation Works of Gates at Site

No.	Key Description	Actual Starting Date	Actual Finishing Date	Remarks (specification Starting Date)
1.1	Spillway gates, gate leaf	Jan.04,1996	Nov.15,1996	-
1.2	Spillway gates, guide frame	Aug.15,1995	Dec.15,1996	Jan.01,1996
1.3	Spillway gate, hoist	Aug.1,1996	Nov.15,1996	-
2.1	Spillway stoplog, stoplog	Jan.11,1996	Nov.15,1996	-
2.2	Spillway stoplog, guide frame	Aug.15,1995	Dec.02,1995	Jan.01,1996
3.1	Intake gates, gate leaf	Aug.13,1996	Nov.15,1996	-
3.2	Intake gates, guide frame	Apr.18,1996	Apr.30,1996	Mar.01,199
3.3	Intake gates, hoist	Jul.30,1996	Nov.15,1996	-
4.1	Intake stoplog, stoplog		Nov.15,1996	-
4.2	Intake stoplog, guide frame	Jan.18,1996	Apr.30,1996	Mar.01,1996
5	Trashrack	Feb.01,1996	Nov.15,1996	Mar.01,1996
6	Trashrake	Jun.10,1996	Nov.15,1996	Mar.01,1996
7.1	Tailrace gate, gate leaf	Jul.10.1995	Jul.21.1995	-
7.2	Tailrace gate, guide frame up to EL.45.5	Apr.01.1995	Jun.16,1995	May 01,1995
7.3	Tailrace gate, guide frame from EL.45.5	Nov.27.1995	Jan.18.1996	Jan.01.1996
8.1	Tailrace stoplog, stoplog	Jul.04,1995	Jul.25,1995	-

8.2	Tailrace stoplog, storage facility	Jan.20,1996	May 02,1996	
9	Tailrace gantry crane	Jul.20,1996	Aug.19,1996	Jan.01,1996
10.1	Penstock Fabrication	Jul.04,1994	May 25,1995	
10.2	Penstock Installation	Aug.06,1994	Jun.20,1996	Oct.01,1994

(d) Manpower

The monthly use record of the workers is shown in Table 5.2.13. In total 53 man-months of Japanese workers and 378 man-months of Indonesian workers were used for the installation work.

Table 5.2.13 Labor Needs for Site Installation Work of Gates

Month		Japanese (M/M)	Indonesian (M/M)
1995	March	1	10
	April	1	10
	May	1	10
	June	1	10
	July	2	38
	August	3	37
	September	2	38
	October	2	52
	November	2	60
	December	2	62
1996	January	2	61
	February	2	61
	March	2	53
	April	2	56
	May	2	61
	June	2	63
	July	3	63
	August	5	69
	September	5	70
	October	4	56
	November	4	26
	December	3	12
Total		53	378

(e) Equipment Used

The type of major equipment used are shown in Table 5.2.14.

Table 5.2.14 Equipment List

No.	Description	Capacity	Quantity	Remarks
1.	Generator Set	250 KVA	1 unit	
2.	Generator Set	24 KVA	1 unit	
3.	Travo Welding Machine	400 A	9 unit	
4.	Travo Welding Machine	250 A	1 unit	
5.	Auto.Welding Machine	400 A	1 unit	
6.	Gas Cutting Machine	-	1 unit	
7.	Gantry Rail	6000 L	-	
8.	Truck Crane	30 Ton	1 unit	
9.	Truck	10 Ton	1 unit	Rental
10.	Trailler	25 Ton	1 unit	Rental
11.	Electric Winch	25 Ton	3 units	
12.	Hand Winch	5 Ton	2 unit	
13.	Air Compressor	8 BAR	1 unit	
14.	Blower	-	2 units	
15.	Container Box	20 Feet	2 units	
16.	Kijang St.Mobile	-	3 units	

(f) The Generating Electric Power Requirement

The electric power required for site installation work was supplied by the Contractor's own generating set until the end of work. The monthly electricity consumption record is shown in Table 5.2.15.

Table 5.2.15 Electricity Used for Gate Installation Work

Year/Month		KWH
1995	July	38,400
	August	25,600
	September	38,400
	October	49,000
	November	53,200
	December	42,600
1996	January	42,600
	February	57,000
	March	38,400
	April	55,400
	May	53,200
	June	38,400
	July	57,600
	August	57,600
	September	49,000
	October	42,600
	November	38,400
	December	-
Total		777,400

2.2.3 Welding Operation

(1) General

Welding was done manually by the shielded metallic arc method or automatically by the shielded arc or submerged arc method. Radiographic inspection was carried out by the Contractor when required by the specifications. All important welds which were considered to subject to the full stress induced in the adjacent or which did not appear to conform to the welding standards, were radiographed as directed by the Engineer. Suitable matters were provided to show and control the welding current and the arc voltage at all times during the welding operations.

Plates to be joined by welding were accurately cut to size, and rolled by pressure to the proper curvature. The dimensions and shape of the edges to be joined were set up to allow thorough fusion and complete penetration.

The edges of plates were properly formed to accommodate various welding conditions. The surfaces of the plates for a distance of 25mm from the edge to be welded were thoroughly cleaned of all rust, grease and scale, to bright metal.

The technique of welding employed, the appearance and quality of the welds and the methods used in correcting defective works ere performed in accordance to Japanese Industrial Standards for Gates and Penstocks, Japan. All welders and welding operators assigned to this work passed a qualification test in accordance to ASME Sec.IX.

(2) Welding Method

Welding was done manually by shielded metal arc welding (SMAW) and submerged arc welding (SAW) automatic. Each welding method was applied as shown in Table 5.2.16.

Table 5.2.16 Application of Welding Methods

Classification	Place Method Joints	Factory		Field Shop		Dam Site
		SAW	SMAW	SAW	SMAW	SMAW
Guide frame	Corner weld		o			
	Flat joint		o			o
	Stiffeners		o			
Gate leaf	Flat joint		o			o
	Stiffeners		o			
	Corner weld		o			
	Field joint skinplate					o
Girder web	Flat		o			
Girder flange	Flat		o			
	Stiffeners		o			
	Corner weld	o				

(3) Welding Electrodes and Drying Condition

The welding electrodes conformed to JIS Z3211 D4301, JIS Z3212 D5016, JIS Z3221 D308-16, JIS Z3221 D309-16. Welding electrodes were selected of the most suitable materials for the base metals as shown in Table 5.2.17. The electrodes selected for each welding method were shown in Table 5.2.18.

Table 5.2.17 Electrodes for Arc Welding

Base Metal (JIS)	Welding Position	JIS Classification	Brand Name	Manufacturer
SS400	Flat	Z 3211 D 4301 Z 3212 D 5016	B-17 LB-52	Kobe Steel
	Horizontal	Z 3221 D309-16	NC-39	
SM 490 YA	Flat	Z 3221 D 5016	LB-52	Kobe Steel
	Horizontal	Z 3183 D 5501-H	MF-44 x US-36	
SUS 304	Horizontal	Z 3221 D 308-16 Z 3221 D 309-16	NC-38 NC-39	Kobe Steel

Table 5.2.18 Welding Materials and Welding Method

Welding Method	JIS Class	Brand Name	Manufacturer
SMAW	JIS Z 3211 D 4301	B-17	Kobe Steel
	Z 3212 D 5016	LB-52	"
	Z 3221 D 308-16	NC-38	"
	Z 3221 D 309-16	NC-39	"
SAW	JIS Z 3183 D 5502-H	MF-38 x US-36	Kobe Steel
	Z 3183 D 5501-H	MF-44 x US-36	"

(4) Welders and Welding Operators

All welders and welding operator assigned to the work passed a qualification test in accordance to ASME Boiler and pressure Vessel Code Section IX.

(5) Radiographic Examination

(a) Length Examination

The welded seams were radiographed in the 20% of butt welded length at the field shop and erection site.

(b) Result of X-ray Test

The results of the radiographic tests were evaluated in accordance to JIS Z3104 "Method of Radiographic Test and Classification of Radiographic for Steel Work". There were also conducted ultrasonic test examination for fillet weld.

Objectionable defective portions were chipped off to receive re welding. Upon completion of the rewelding, such non destructive test as radiographic test was made.

Required radiographic/ultrasonic examinations were performed, and all the films photographic were inspected by the Engineer. The results of this radiographic/ultrasonic examination are given in Table 5.2.19.

Table 5.2.19 Results of Radiographic Examination

Radiographic Examination of Spillway Gate Leaf

No.	Description	Shoot	Re-shoot	Total
1.	Spillway Gate Leaf No.1	36	3	39
2.	Spillway Gate Leaf No.2	36	1	37
3.	Spillway Gate Leaf No.3	36	2	38
4.	Spillway Gate Leaf No.4	36	9	45
5.	Spillway Gate Leaf No.5	36	2	38
Total Spillway Gate Leaf		180	17	197

Ultrasonic Examination of Spillway Gate Leaf

No.	Description	Shoot	Re-shoot	Total
1.	Spillway Hoist No.1	12	-	12
2.	Spillway Hoist No.2	12	-	12
3.	Spillway Hoist No.3	12	6	18
4.	Spillway Hoist No.4	12	-	12
5.	Spillway Hoist No.5	12	4	16
Total Spillway Hoist		60	10	70

Ultrasonic Examination of Intake Gate

No.	Description	Shoot	Re-shoot	Total
1.	Hook and Pulley Bracket of Intake Gate No.1	20	-	20
2.	Hook and Pulley Bracket of Intake Gate No.2	20	-	20
3.	Hook and Pulley Bracket of Intake Gate No.3	20	3	23
Total		60	3	63

(6) Quality Control

(a) Materials

The steel materials used for fabrication of gates and stoplogs were selected after having passed the mill tests of in accordance to applicable JIS codes.

i) Mechanical property test (tension test) and ii) chemical analysis test. Certified copies of the mill test reports were submitted to the Engineer for his approval prior to shipping to the site.

(b) Welding Procedure Specification

The shape and dimension of the welding to be adopted in the welding work are shown in Appendix I.

(c) Welding Conditions

1) Drying Condition of Electrodes

All electrodes and fluxes were kept sufficient dry prior to their use. The drying temperature and time were shown in Table 5.2.20.

Table 5.2.20 Drying the Welding Material

Type	Brand Name	Drying Temperature (°C)	Drying Time (Min)
Electrode	B-17 LB-52 NC-38/NC-39	70 - 100 300 - 350 150 - 200	30 - 60 30 - 60 30 - 60
Flux	MF-38/MF-44	150 - 350	30 - 60

2) Heating Condition

The pre-heating was done according to the methods and temperatures indicated in Table 5.2.21. The others conditions are i) preheat area (preheat area is more than four times the plate thickness from the welding line). ii) check to preheat temperature (reheat temperature was checked by tempi stick or contact thermometer) and iii) interpas temperature (maximum interpas temperature less than 200°c).

Table 5.2.21 Pre-Heat and Interpas Temperature

Base Metal (JIS)	Thickness (mm)	Welding Process	
		SMAW	SAW
SS 400 SM 400 B	T < 25 25 ≤ T ≤ 38 38 ≤ T	- 50 50	- - 50
SM 490 YA	T < 25 25 ≤ T ≤ 38 38 ≤ T	- 50 50	- - 50
SUS 304 SUS 304 N	T < 25 25 ≤ T ≤ 38 38 ≤ T		

(d) Welding Control

All the welding work was controlled under the conditions as shown in Table 5.2.21.

(e) Welding Work Instruction

i) Checking of the Parts to be Welded

Prior to starting the welding work, the parts to be welded were sufficiently checked for the state of cleanliness (scale, rust, oil, moisture), the prepared edge root opening, the mounting of the tab and the existence of crack in the tack welding before starting the welding work.

ii) Multi Layer Welding

When there are 2 or more layers, care is to be taken not to concentrate the change of welding rod at the same place. More over it is also to be refrained from leaving craters and creating overlap. In particular, care utmost be taken not for leave any crater at end of bead.

In addition, it is also to be cleaned slug and sputter of each layer, and moreover to be inspected each layer for defect and made the required repair.

iii) Quotation of Welding

The welding work was carried out by taking special care to prevent occurrence, and if any to take correction practice, of defect like cracks, incomplete penetration, slag inclusion, blow hole and pinhole, pits, undercuts, overlaps, arc strike, shortage or excess of length of leg, craters etc. in the welded portion.

iv) Weld Beads

The beads were made so as to have external appearance as smooth as possible with a regular wave form, and the welding size and length conformed to the design dimension.

v) Reinforcement of Weld

All the welding joints were dressed smooth, and maximum reinforcement was limited to 10% to 15% of the plate thickness. (Care is to be taken not to grind out more than 0.5mm). When the finish is not specified, 20% of the bead width (however not exceeding 3mm).

vi) Weather Conditions

In case of rainy weather, execution of welding was in principle avoided at places subject to noxious influences of the rain. If it was confirmed, however, that there were no noxious influences, the welding was executed taking special care of the humidity by dehumidifying heating as occasion demanded.

2.2.4 Safety Precaution

Prior to any of the work being energized, the Contractor was responsible for supplying and fixing in prominent positions near to each item of the work, large multilingual temporary sign giving clear warning of danger in areas which might previously have been regarded as not safe.

During erection and commissioning, all temporary scaffolding, ladders, platforms with toe boards and handrails essential were provided for safe and convenient access of workmen, inspectors and other authorized persons. All dangerous openings or holes in floors were provided with handrails or covers; preventative measure were taken to protect workmen from falling objects.

The maximum possible safety was afforded to personnel directly engaged or those who might frequent the working area or those who in the normal course of their occupation found it necessary to utilize temporary works erected by the Contractor.

2.2.5 Protection, Cleaning and Painting

All parts of equipment to be embedded in concrete were cleaned and protected by a wash primer coating method. Before being installed they were thoroughly descaled and cleaned of all rust and adherent matters. All machined parts or bearing surface were cleaned and protected from corrosion by the application of rust-preventative grease.

The surface preparation was performed immediately prior to applying paint. The primer and finish coats of paint were applied using the approved methods and equipment. The internal surface of all pipe lines was cleaned out by approved methods

before or after installation and again prior to commissioning, to ensure free from dirt, rust, scale, welding slag, etc.

The painting of hydromechanical work included preparation of the metal surfaces, paint application, protection and drying of the paint coatings as well as supplying of all tools, labours and materials necessary for the entire painting work. Sufficient paint was provided for field painting and touch up on shop painting by the Contractor. Paint was the product of reputable manufacturer and its selection was approved by the Engineer.

2.2.6 Inspection and Test

(1) General

The inspection and test procedure to be applied to the shop fabrication and site installation was approved by the Engineer. During the fabrication and after the installation of each item of equipment, necessary inspections and tests were performed according to the procedures to establish accuracy of the assembly and/or installation and to prove adequacy of the materials and workmanship.
The following inspections and tests were made for this work.

At the shop : material and parts inspection, molding inspection, welding qualification, welding inspection, alignment and machining inspection, shop trial assembly inspection, painting inspection and check of shipping condition.

At the site : welding inspection, dimensional inspection, operation test, painting inspection and leakage inspection for the gates (remains when impounding).

Details of inspection and test procedure are given in the inspection procedure on shop trial assembly and on site erection for gates, stoplog, trash racks and steel liner prepared by the Contractor and approved by the Engineer.

(2) Details of Major Inspections and Tests

(a) Material and Parts Inspection

Material and parts inspection were performed to check type, dimension, quantities, compliance with the specifications, and other pertinent data of the materials. All principal materials (rolled steel, forgings and castings, etc) were tested at respective mills of foundries in accordance to JIS specifications and the approved procedures. Mechanical tests such as tension tests, bending test and impact test for main parts accordance to the requirements of applicable standards.

Purchased parts such as valves, flanges, pipes, indicators and electrical parts were tested and inspected at respective manufacturer's shop in accordance to approved drawings, specification and manufacturer's specification.

(b) Shop Trial Assembling Inspection and Test:

Each complete gate including seals, guides, and wheels assemblies were temporarily assembled in the shop in the approximate position. While assembled, each equipment was checked for dimensions, tolerances, and accuracy of alignment in accordance to the approved inspection procedures on shop fabrication.

Any errors and misalignment discovered during the shop assembling were promptly corrected before shipment. Parts were clearly match-marked, for installation on the job site, before disassembling for shipment.

Each hoist was completely shop-assembled and tested for smooth and proper performance. All units were tested at normal operating speeds and at the rated load and closely checked to ensure that all necessary clearances and tolerances were provided and that no binding occurred in any moving parts. All lubrication grease and oil required for performance of the tests were furnished. An operation test between the remote and local controls was made by electrically connecting both control cabines to the hoist at rated load conditions to prove specified functions in accordance with the approved inspection manuals.

(c) Painting Inspection

After finishing the painting work, the following inspection was performed. See Appendix II. Dry film thickness was measure at a rate of two portions (four points per 1 portion) at least for every ten (10)m² of coated area by an electro-magnetic film thickness gauge.

Allowable minimum film thickness was 0.42mm for upstream side of gate leaves and guide frame, 0.16mm for downstream side of gate leaves and stoplogs, 0.135mm for hoist, trash rake & gantry crane, 0.105mm for steel surface which permanently contact with oil, 0.110mm for internal oil-tanks.

(d) Packing

Checking of packing was performed visually.

(3) Witness Inspection at the Fabrication Shop

Witness inspection (attended by the Engineer and the Employer) with regard to the manufacturer of the materials to be used for construction of the equipment of spillway roller gate, spillway stoplog, intake gate, intake stoplog, intake trashrack, tailrace gate, tailrace stoplog, and storage facility were conducted at the PT.Barata Indonesia at Surabaya and Tegal, in Indonesia. Witness inspection of the manufacturer of intake trashrake, and conveyor were conducted at Kyowa Seisakusho Co.,Ltd, Saga, Japan.

Witness inspection of the manufacturer of gantry crane was conducted at MHE-Demag Pte.Ltd., Singapore. Witness inspection of the manufacture of control panels were conducted at EIM System, Kitakyushu, Japan. Witness inspection with regard to the installation and completion of the equipment under this Contract were carried out at the installation site and at the site shop of Kotapanjang. The particulars of these inspections were as follows (see Table 5.2.22) :

Table 5.2.22 Factory Inspection Record

No.	Description	Inspection Date	Place
1	Gate Materials	Sep. 1 ~ 2 '94	PT. Barata, Surabaya
2	Guide Frame for Tailrace	Oct. 6 ~ 7 '94	"
3	Guide Frame for Spillway	Nov. 10 ~ 11 '94	"
4	Guide Frame for Spillway	Dec. 15 ~ 16 '94	"
5	Tailrace Gate & Stoplog	Feb. 9 ~ 10 '95	"
6	Spillway Stoplog & Tailrace Storage Facility	May 29 ~ 30 '95	"
7	Spillway Stoplog, Lifting Beam & Intake Steel Beam	July 27 ~ 28 '95	"
	Spillway Gate, Intake Guide Frame		"
8	Trashrack Beam	Sep. 28 ~ 29 '95	"
9	Intake Trashrake	Oct. 27 ~ 28 '95	Kyowa Seisakusho, Japan
10	Control Panel	Oct. 30 ~ 31 '95	EIM System, Japan
11	Spillway Gate Trashrack Panel	Nov. 21 ~ 24 '95	PT. Barata, Surabaya
12	Intake Stoplog Trashrack Panel	Nov. 21 ~ 24 '95	PT. Barata, Tegal
13	Gantry Crane	Dec. 7 ~ 8 '95	MHE-Demag, Singapore
14	Spillway Gate, Spillway Hoist Intake Gate	Feb. 13 ~ 14 '96	PT. Barata, Surabaya
15	Intake Gate, Intake Hoist	Apr. 4 '96	"

(a) PT. Barata Indonesia Works in Indonesia

The witness inspection conducted at the PT. Barata at Surabaya Factory in Indonesia were in connection with the gates and stoplogs, and the gate hoists manufactured in Indonesia. Total of 12 trips were undertaken by the Engineer and the Employer for the above purpose. The first trip was made on September 01, 1994, and the last trip was made on April 04, 1996.

The witnessing consisted of materials inspection, full size drawing inspection, operation tests of hoists, and shop assembly. The results produced by the inspections and witnessing were quite valuable and satisfactory.

(b) Kyowa Seisakusyo Co., Ltd. in Japan

The witness inspection and functional operation tests were conducted at Kyowa Seisakusyo Co.,Ltd. at Saga in Japan, in connection with the trashrake and conveyor manufactured in Japan. The inspection and test was made on October 27 & 28, 1995 undertaken by the Engineer and the Employer.

The witnessing inspection and test :

i) Materials and articles : Mill certificates and manufactures were checked and accepted.

ii) Appearance inspection : Visual inspections were made and accepted.

iii) Measurement inspection : Randome checks were made following to the previously prepared records, and results were accepted.

iv) Limit switch operation : All of the limit switches were arranged to confirm practical operations, and results were acceptable.

v) Functional tests for the raking equipment and conveyors were made applying the control panels and the result were acceptable.

(c) EIM Control Systems Co., Ltd., Kitakyushu, Japan

The witness inspection and simulation operation test were conducted at EIM Control Systems Co., Ltd. at Kitakyushu in Japan, in connection with control panels for the and spillway gates manufactured in Japan. The inspection and test was made October 30 & 31, 1995, undertaken by the Engineer and the Employer.

The witnessing inspection and tests

i) Appearance inspection : Visual inspections were made and accepted.

ii) Measurement inspection : Randome checks were made following to the previously prepared records, and results were accepted.

iii) Simulation test : All of the switches and lamps were arranged to confirm practical operations, and results were acceptable. Local and remote control panels were carried out and the results were acceptable.

(3) MHE - DEMAG (S) Pte., Ltd., 10, Neythal Road, Singapore 2262

The witness inspection conducted at the MHE-Demag(s) Pte., Ltd., Neythal Road, Singapore 2262 were in connection with the tailrace gantry crane manufactured in Singapore. The inspection was made on December 07 & 08, 1995, undertaken by the Engineer. Results of the inspection and tests were acceptable.

(4) Witness Inspection at the Site Shop and Equipment Locations

i) The site arrival witness inspections of the materials and manufactured parts of the equipment under this contract were conducted at the site shop of Kotapanjang immediately after the arrival of the equipment. These inspections were conducted for all the equipment and materials arriving either from Japan, Singapore, or Surabaya, Indonesia.

ii) Site Assembly and Installation Inspection

The site assembly inspection were conducted after assembly installation and/or erection of the parts of the work or whole structures were completed to verify their conformance with the provisions of the specification. The site assembly inspections generally consisted of the followings :

- Assembly inspection of guide frames including top and bottom seal seat bases prior to placement of the second stage concrete.
- Visual inspection following placement of the second stage concrete.
- Assembly inspection of gate leaves and their accessories.
- Assembly inspection of hoist equipment.
- Inspection of electric wiring work.

(5) Completion Work

(a) Operation Tests Under Dry Condition (Dry Tests)

Operation tests under dry conditions were carried out after all installation work was completed in order to determine whether all gates could be operated properly.

The following items of operation and testing were checked or carried out :

- Raising and lowering of gate leaves.
- Vibration and noise of all revolving parts.
- Bearing temperature and the lubrication.
- Measurement of cable resistance.
- Manual operating device.
- Gear teeth contact.
- Limit switch operation
- Operation of interlocking devices.
- Mechanical gate position indicator.
- Local and remote control panel operation.
- Balanced condition of gate leaves.

(b) Operation Tests Under Wet Condition. (remains for Employer impounding test).

Operation tests under wet conditions were carried out for all gates and valve after the water level in the reservoir reached EL.85.00m. The intake stoplogs and the spillway stoplogs tests were carried out for the reservoir water level at EL.85.00m. The Operation test items were the same as those used for the dry conditions except that a leakage test was also added.

The maximum permissible amount of leakage with reservoir water level at EL.85.00m was as follows :

1.5 l/s for each spillway gate
1.0 l/s for each intake gate and tailrace gate
2.0 l/s for each block of stoplog

The amount of leakage observed during the tests, for each particular gate, was always smaller than the above limits.

2.2.7 Installation of Dam Spillway Fixed Wheel Gates and Hoists

(1) General

Five 11m wide by 18m high fixed wheel gates constructed of welded steel, with a vertical lift of 20.0m, are located on the ogee spillway crest.

The gate consists of four separated leaves and each leaf consists of three essential parts of 1) the skinplate on which the seal was attached, 2) the framework including two main horizontal beams and 3) the fixed-wheel assemblies. The total weight of the gate leaf was 76.5 tf. The installation began August 15, 1995 and was completed on November, 1996.

The gate leaf was fabricated in four skinplate panels gate leaf blocks with vertical beams and two frame works. The 11mm (the top gate leaf block) and the 12mm (the rest three lower gate leaf blocks) thick skinplate was supported by 9 vertical 180mm x 400mm x 12mm, wide flange girders. They were supported by 2 horizontal 320mm x 1700mm x 9mm wide flanged girders. The horizontal girders were braced by 3 vertical bracing made by L 100 x 100 x 10 and CT 125 x 125 x 6/9.

The main horizontal girders were connected to the side vertical beams by welding. Two sets of fixed-wheel assemblies and two side guide rollers were provided on each side of the four gate leaves. Each wheel and roller was provided by grease nipple lubricating bearing. The seal assembly was composed of 16mm thick L-shaped rubber seals which faced "J" - type seals. These seals seal against the stainless seal seats. The bottom seal assembly was composed of 20mm thick by 100mm high solid rubber bottom seals. The fabricated gate and guides were shop-assembled, and the dimensional inspection followed.

The hoist were furnished in the Contractor's workshop in Surabaya. The hoist capacity was 109.626tf with a 11 kW output. The total lift is 20.00m and the lifting speed is 0.3 m/min. The hoist connects to the leaf by two 37.5mm dia. JIS 6x37 class B galvanized wire rope, 81200 kgf breaking strength. Each rope clamps to the centerline of the drum at the center of the rope, loops around the drums, and connects to the gate leaf hoist attachment. The centerline of the hoist drum shafts is located at EL.109.170.

The hoist units were held in place by 20mm dia. anchor bolts. Before shipping the equipment, the painting work was performed in accordance to the approved painting specifications and procedures. The Spillway Gate Installation Procedure can be seen in Fig.5.16.

(2) Guide Frame

Prior to installation of the guide frames, the Contractor pre-assembled them at the Surabaya shop at December 15, 1994.

After the dam concrete was lifted up to the dam crest EL.67.500, the Contractor began installing sill beams and site guide frames with seal seats on August 15, 1995. The sill beams with expansion joints were aligned to a true position within an allowable flatness of ±1.5mm per 1.0m using a level scope and fixed with anchor bars embedded in the sill block out.

After the concrete was poured in the sill blockouts, the side guide frames were aligned in position and checked for vertical alignment and flatness using a transit and a plumb line suspended from the top of guide frames. The allowable flatness of tracks and seal seats was ±0.5mm per 1.0m. The installation of 5 guide frames was completed on, December 15, 1995.

(3) Gate Leaves

Assembling and erection of the gate leaves started on January 04, 1996 and was completed on October 1996 except minor adjustment and trial operation test. The final operation test was performed on November 15, 1996.

The Contractor moved the leaves for gate No.1 from the stockyard on January 04, 1996 after curing the second stage concrete around the guide frames. The Contractor began assembling and erecting the gate leaf on the temporary supports place on the spillway crest at EL.67.500.

First, the lowest gate-leaf block were placed on the supports. There after the second gate-leaf block (separated in two sections) were placed above the lowest gate leaf.

The weight of the lowest gate leaf block (Block No.1) was 13.665 tons and the weight of the second gate leaf block (Block No.2, consisting of two sections) were 11.242 ton and 3.923 ton. So for placing them would need 13.5 ton cable crane. Caused by limiting time in using 13.5t cable crane till February 1996, removal time of that crane from site, the arrival of gate leaf Blocks No.1 and No.2 for all of the five spillways should be made earlier than the rest of Blocks No.3 and No.4. So the fabrication of Blocks No.1 and No.2 also should be made to lead the fabrication of Blocks No.3 and No.4.

After adjusting setting positions, and after bolting connected the two sections, the lowest gate leaf was bolt connected to the second gate leaf. Then, the third gate leaf block (Block No.3, separated in four sections) were placed above the second gate leaf block. Moreover, followed by adjustment of setting positions, and bolting connection. The last stage was placing the top gate leaf (separated in five sections) above the third gate leaf block, followed by adjustment of setting positions, and bolting connection. Then the skinplate seams of each gate leaf were weld connected.

Prior to bolt tightening and welding of the skinplate seam, all the dimension and alignment were checked. Finally, the rubber seals were set.

(4) Hoists

The installation of the hoists started on August 1, 1996 after completing the hoist deck concrete. The hoists and controls were installed in two phases :

1) Cleaning and assembling the hoist and positioning the hoist on the hoist deck and
2) Installing the control cabinets and electric wiring and testing.

(5) Operation Tests

After the gates were test operated under dry condition, they were found to be thoroughly fit enough. The water leakage test was performed on August 1997 after impounding the dam.

2.2.8 Installation of Spillway Stoplog

(1) General

One set of the stoplog comprising twelve logs, 11 m wide by 1.5 m high each, constructed of welded steel was located in front of the spillway gate. The total weight of the stop log was 7.4 tf x 12 = 88.8 tf.

The stoplog was fabricated of structural steel beams and steel plates, the 9 mm thick skinplate was supported by two horizontal 150 mm x 1800 mm x 9 mm T - steel beams. The horizontal beams were braced vertically by 9 mm thick plate diaphragms. The log has two guide rollers on each side. The stoplog travels from the storage yards to site location by using trailer and installed at place by using 80 ton mobile crane. The log slides away down against a series of side guides.

(2) Guide Frames

Installation of the guide frames for the stoplog started on November 10, 1994 with pre-assembling work in Surabaya shop. The actual setting at site of the guide frames started on August 15, 1995.

The seal seats were erected in sequence, bolted together and aligned to finish up the stoplog frame.The sill seats were installed and fitted in the concrete block out within an allowable deviation of ±1.5mm per 1.0m flatness and accurately set with a machinist's level. After the concrete was placed in the sill block out, the side frame assemblies were set in blockouts and were bolt connected to the sill seats.

The bases of the stainless steel seats were fitted snugly on the sill to insure tight fit. The side frames were aligned by means of plumb lines. All the work was completed on December 15, 1995.

(3) Stoplog Assembly

The stoplogs arrived at the site in complete assembly except for attaching the rubber seals. The seals were clamped in position on the stoplogs using templates, stainless steel bolts and nuts in the field shop. The assembling work including repair painting was finished on November 15, 1996 except for some minor adjustment.

(4) Operation Test

An operational test of the stoplog was made by raising, lowering and transporting t storage-yard in November 1996 except water leakage test. The water leakage test w performed on August, 1997 after impounding the dam.

2.2.9 Installation of Intake Gate

(1) General

Three 6m wide by 6m high fixed-wheel gate of welded construction was installed in th intake structure. The gate consists of two separated leaves with wheel assembled, by pa valved and lifting beam, guide frame assemblies with wheel tracks, side roller paths, w seats, and an electric hoist with controls.

The gate leaf was fabricated in two blocks. The 19mm thick skinplate was supported three horizontal 800mm by 250mm by 12mm T-steel plate girders which were brac vertically with 12mm thick plate diaphragms.

The main horizontal girders were weld connected to the side vertical beams. The leaf w provided with four fixed-wheel assemblies (two for each block), and two side pa rollers, on each side of the gate. The wheels were provided with self aligning rad bearings and grease lubricating nipple respectively. The rollers were provided with ba bearings with grease lubricating nipple respectively.

The seal assembly is composed of 15mm thick music note shaped molded rubber seal a 100mm by 20mm solid rubber bottom seal. The gate was equipped with two 250mm d by pass valves operated through a lifting beam to fill the waterway with water an equalize the water pressures on both up and downstream sides of the gate before it opened. The Intake Section Installation Procedure can be seen in Fig. 5.17.

(2) Guide Frame

The Contractor began installation of the sill beam on January 18, 1996 by moving parts to the intake. Prior to the installation, such preparation works as survey, marking positions of guide frames, installation of erecting jigs and the hoist, and temporary elec power for the works were performed.

The steel bottom seat was installed, fitted to the anchor bars embedded in the conc block out within the allowable surface deviation of ±1.5mm per 1.0m from the design and accurately set with a level scope. Then the secondary concrete was placed for bottom block out. Thereafter, the lower guide frames consisting of tracks, side guide seats were hung on anchor bars on each side of the gate opening, and were bolt connec to the sill. After adjusting and fixing the lower guides, the lintel seal seat was connec set in and adjusted using control plumb wires.

The specification called for deviation of the track from a true place within ±0.5mm per 1.0m without offsets or gaps at the joints. The inspection of erected guide frame was performed on April 30, 1996, and then the secondary concrete in the block out was placed.

(3) Gate Leaf

Installation of the gate leaf started August 13, 1996 after concreting the guide frames was finished up. The intake gate arrived at the site in two separated assembled blocks except for attaching the rubber seals. The lower half leaf (lower block) and the upper half leaf (upper block) was fitted and connected on a dolly that placed on temporarily erection rails directly over the gate opening on EL.87.500.

After assembly of the gate leaf completely, measurement check was made to be within the allowable tolerance. Then field painting of joints and damaged portions was performed. The seals were placed in position on the gate leaf and fastened by seal clamps, bolts and nuts. Then the gate leaf was lowered by hoist after the wiring.

The assembling of the gate leaf was completed on October 1996 except the final adjustment and trial operation test. The final operation test was performed on November 15, 1996, and water leakage check performed on August 1997.

(4) Hoist

The hoist was set in accurate position using a mobile crane and fixed to the anchor bolts after adjusting the level using a level scope. Prior to the operation test of the gate and hoist, lubrication of bearings and gear reducers was carried out. The local control cabinet was erected on the hoist deck and the electric wiring was finished.

The hoist installation was started on July 30, 1996. Final operation test and adjustment of the seal of the gate leaf and the emergency lowering devices was completed on November 1996. The gate operated freely without any troubles.

2.2.10 Installation of Intake Stoplog

(1) General

One set of the stoplog comprising twenty logs, 6m wide by 1.2m high each, constructed of welded steel was located in front of the intake gate. The total weight of the stoplog was 3.1tf x 20 = 62tf.

The stoplog was fabricated of structural steel beams and steel plates. The 12mm thick skinplate was supported by two horizontal 900m x 9mm T-steel beams. The horizontal beams were braced vertically by 9mm thick plate diaphragms. The log has two guide rollers on each side. The stoplog travels on a system of tracks and against a series of side guides.

(2) Guide Frames

Installation of the guide frames for the stoplog started on September 29, 1995 with pre-assembling work in Surabaya. The actual setting of the guide frames started on January 18, 1996.

The seal seats were erected in sequence, bolted together and aligned to finish up the stoplog frame. The sill seats were installed and fitted in the concrete block out within an allowable deviation of ± 1.5 mm per m and accurately set with a machinist's level. After the concrete was placed in the sill block out, the side frame assemblies were set in side blockouts and were bolt-connected to the sill seats. The bases of the stainless steel seats were fitted snugly on the sill to insure tight fit. The side frames were aligned by means of plumb lines. All the work was completed on April 30, 1996

(3) Stoplog Assembly

The stoplogs arrived at the site in complete assembly expect for attaching the rubber seals. The seals were clamped in position on the stoplogs using templates, stainless steel bolts and nuts in the filed shop. The assembling work including repair painting was finished on November 15, 1996 expect for some minor adjustment.

(4) Operation Test

An operational test of the stoplog was made by raising, lowering and travelling each leg several times along its frames in November, 1996 expect water leakage test. The water leakage test was performed on August, 1997 after impounding the dam.

2.2.11 Installation of Intake Trash Rack

The 10m wide by 24.612m high obliquely trashracks were installed in the intake structure. There are 25 rack sections, each 2m wide and 5.477m high, installed in the structure on a 1 : 0.3 slope.

Eight horizontal supporting girders were fabricated with wide flange H 808 x 400 x 1630 girders. The rack sections were fabricated from 100mm by 12mm trashbars connected to the rods and distance pieces at 100mm centers. The installation work began on February 01, 1996 with installing the horizontal supports and trash racks, and was completed with repair painting on November 15, 1996.

2.2.12 Installation of Intake Trash Rake and Conveyor

A raking equipment, travelling type, with a 3.0m wide for rake operation on clear width of 10m for the trashrack, and 52m travelling length was fabricated and tested in the Contractor's shop in Japan. On January 10, 1996 the Contractor began the work with preparation of rail laying on the concrete structure. The rail surface was completely aligned and adjusted.

Installation of the raking equipment began on August 1, 1996. The work completed on October 1996. The operation test was performed on November 1996.

2.2.13 Installation of Tailrace Gate

(1) General

Two 6.5 wide by 4.6m high slide gate of welded construction with a vertical lift of 35m, and six sets of guide frames were provided at the tailrace outlet.

The gates were completely fabricated in the Contractor's work shop at Barata Surabaya's shop, and shipped to the site. The 16mm thick skinplate was supported by four horizontal 920mm x 320mm x 10mm T-steel plate girders. They were braced vertically with 10mm thick plate diaphragms. Each leaf was provided with two side guide rollers on each side of the gate.

The molded rubber, music note type, solid rubber seals were fitted on the skinplate, and flat rubber seals, fitted at the bottom of the skinplate using stainless steel bolts and nuts. The Tailrace Gate Installation Procedure can be seen in Fig.5-19, and the Gantry Crane Installation Procedure in Fig.5-19A.

(2) Guide Frame

Installation of the guide frames began on April 01, 1995 when the lower tailrace concrete structure was completed until EL.45.500. First, 6 sil beams were fixed with anchor bars, followed by erecting continuously lintel and side guide frames in the concrete block out. After embedding the concrete block out completely, the tailrace gates and the tailrace stoplog were installed at their position, functioned as cofferdam. Afterwards completion of the upper tailrace concrete structure was continued until EL.60.500, and followed by completion of erecting side guide frames in the concrete block out until EL.60.000. The erection of the guide frames was completed on January 18, 1996.

(3) Gate Leaf

The gate leaves arrived at the site in complete assembly except attaching the rubber seals. The seals were clamped in position on the gates using templates, stainless steel bolts and nuts in the field shop. The trial operation test of the gate leaves was completed on July 21, 1995, with satisfactory results.

2.2.14 Installation of Tailrace Stoplog

(1) General

Four sets of the stoplog comprising five logs each, 6.5m wide by 0.92m high each, constructed of welded steel were provided at the tailrace outlet.

The stoplog was fabricated of structural steel beams and steel plates. The 9mm thick skinplate was supported by two horizontal 800mm x 150mm x 9mm T-steel beams with both sides provided with 714mm x 150mm x 14mm T-steel beams.

The horizontal beams were braced vertically by 9mm thick plate diaphragms. The leg has two guide rollers on each side. The stoplog travels on a system of tracks and against a series of side guide.

(2) Stoplog Assembly

The stoplogs arrived at the site in complete assembly except for attaching the rubber seals. The seals were clamped in position on the stoplogs using template, stainless steel bolts and nuts in the field shop. The assembling work including repair painting was finished on July 25, 1995.

2.2.15 Installation of Gantry Crane

A gantry crane, travelling type, with a capacity of 13tons, 3.2m rail gauge, 57.7m long travel, and vertical lift of 35.5m was fabricated and tested in the Contractor's shop at Demag Singapore's shop. On January 20, 1996 the Contractor began the work with preparation of installation of sole plate for rail laying on the tailrace concrete structure. After placing the embedment concrete for sole plate, continued by laying of the steel rails for the gantry crane over the sole plates at EL.60.500. The rail surface was completely aligned and adjusted.

Installation of the gantry crane equipment began July 11, 1996. The work completed on August 19, 1996. The operation test was performed on September, 1996.

Chapter 3 PENSTOCK DESIGN AND CONSTRUCTION

3.1 Design

3.1.1 General

Three lines of steel penstock of 5.0m diameter approximate 86.905m long (curve length) were designed and installed for conveying water 348m^3/s maximum discharge, from the intake to the water turbines housed in the half-underground powerhouse. The penstock line starts at an intake portion of the upstream of dam surface, and terminates at the entrance of the turbine.

Each penstock line system consists of an upper horizontal pipe with a manhole and an air pipe with starting point in the intake, and an upper bend pipe with ending point at the downstream dam surface, both of them will be embedded in the dam concrete. Between downstream dam surface and bending point the penstock is inclined and exposed with an expansion joint completed with rocker shoe to allow movement due to temperature change, and between bending joint and entrance of the turbine the penstock is horizontal and embedded in the powerhouse concrete. The penstock shall be designed to have sufficient structural strength against load applied. The general arrangement of the penstocks is shown in Fig.5.20 and 5.21.

The intake gate serves as an emergency shut off gate or as a service gate to permit dewatering of the waterway system for inspection and maintenance of the pressure tunnel. Flow in each unit penstock line will be controlled by a turbine vane. Access to the lower part of unit penstock line can be made through the manhole provided in the turbine spiral case.

The diameter of 5.0m was determined from the following design consideration.

1) The minimum diameter of the inlet of the turbine casing is 5.0m
2) Head loss due to friction is not significant, since the length of the penstock is only 86.905m

To summarize, the penstock pipe is of rolled steel for welded structural consisting of:

1) 5.0m dia x 32.991m long x 3 lines encased in dam concrete
2) 5.0m dia x 13.439m long x 3 lines exposed in the air
3) 5.0m dia x 40.479m long x 3 lines encased in powerhouse concrete

3.1.2 Hydraulic Design Data

Fig.5.22 Shows the penstock design head diagram. The maximum flow rate per line is 348m^3/sec at the poundage HWL 85.000m. The penstocks were designed for the maximum design water level EL.86.370m (HWL + wave height), HWL : 85.000m, wave height:1.370m, and the spiral casing centre line of water turbine : EL.33.300m.

The pressure rise due to water hammer is 19.0m at the center of water turbine and [illegible] (0) meter at the beginning point of penstock and this rise is assumed to change gradually along to the penstock centerline.

3.1.3 Design Details of Penstock Pipes

The design of the penstocks and appurtenant parts conformed to the provisions of the Technical Standards for Gates and Penstocks issued by the Water Gate and Penstock Association of Japan. Table 5.3.1 presents a summary of penstock design features. Standards Specification and Approved Procedures mentioned at Section 2.1.3 (2) were also used for the design specifications.

Table 5.3.1 (1) Summary of Steel Penstock Design Features

Division	Description	Penstock No.1 (2, or 3)
	Overall length (m) (Curve length)	86.905
CP 1	Pipe length (m) Inside dimension (m) Material used Shell thickness (mm)	2.350 5.000 x 5.000 JIS.SM400B, S25C, SS400, 14.5
CP 2	Pipe length (m) Inside dimension (m) Material used Shell thickness (mm)	2.000 5.000 X 5.000 JIS.SM400B, S25C, SS400, SM490 YA 14.5
CP 3	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	4.650 5.000 JIS.SM400B 14.5
CP 4	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	6.000 5.000 JIS.SM400B 14.5
CP 5 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	3.8512 3.0064 5.000 JIS.SM400B 14.5
CP 6 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	5.041 3.9149 5.000 JIS.SM400B 14.5

Table 5.3.1 (2) Summary of Steel Penstock Design Features

Division	Description	Penstock No.1 (2, or 3)
CP 7 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	5.041 3.9149 5.000 JIS.SM400B 14.5
CP 8 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	5.041 3.9149 5.000 JIS.SM400B 14.5
CP 9	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	4.2312 3.9493 5.000 JIS.SM400B 14.5
CP 10 (expans ion joint)	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	1.720 5.000 JIS.SM400B,SUS CLAD STEEL : SUS304 & SM400B 15
CP 11	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	6.000 5.000 JIS.SM400B 14.5
CP 12	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	6.000 5.000 JIS.SM400B 14.5
CP 13	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	3.7023 3.9842 5.000 JIS.SM400B 14.5
CP 14 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	5.041 3.9149 5.000 JIS.SM400B 14.5

Table 5.3.1 (3) Summary of Steel Penstock Design Features

Division	Description	Penstock No. 1 (2, or 3)
CP 15 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	 5.041 3.9149 5.000 JIS.SM400B 14.5
CP 16 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	 5.041 3.9149 5.000 JIS.SM400B 14.5
CP 17 (angled)	Pipe length (m) - Top - Bottom Inside diameter (m) Material used Shell thickness (mm)	 4.5945 5.4393 5.000 JIS.SM400B 15.5
CP 18	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	6.000 5.000 JIS.SM400B 15.5
CP 19	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	5.850 5.000 JIS.SM400B 15.5
CP 20	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	1.430 5.000 JIS.SM400B 19
CP 21	Pipe length (m) Inside diameter (m) Material used Shell thickness (mm)	1.430 5.000 JIS.SM400B 19

The details of design requirements and assumptions are enumerated as follows :

1) Type of Penstock : Welded steel pipes with stiffener rings, part of them are encased in concrete and part of them are exposed in the air.

2) Diameter : 5.0m inside diameter

3) Length : L = 86.905m (curve length)

4) Appurtenance : One set of the following item ;

a) Stiffener rings and others closely related to the pressure lining part, Manhole (circular form of 500mm diameter, flanged type with a rubber checking, bolts nuts and other necessary components, and handling tools), Expansion joint (sleeve type with the inner sleeve is clad with corrosion-resisting steel and seals are greased hemp square packing and "0" type of rubber packings), Ring girder support (rocker type), Grouting hole where applicable, Thrust collar, Seepage collar, Air pipe, Access ladders and walkway and Anchor band and anchor bar.

5) Quantity : 3-lines

6) Design Condition :

a) Internal pressure (see Fig.5.22), External pressure were assumed to (i) external pressure for the exposed portion of the penstock is capable of resisting the pressure difference of 2.0m in water head during drainage (0.2kgf/cm^2) and ii) Grouting pressure 4kgf/cm^2, Axial forces, consisting of (i) bending stress regarding the pipe as a cantilever beam, (ii) local bending stress due to the restraint of the pipe shell bulging by means of a ring girder, (iii) stress due to pipe gradient, (iv) stress due to longitudinal component of the internal pressure acting on an expansion joint, (v) stress due to friction between pipe and support, and stress due to friction of expansion joint, (vi) stress due to temperature change of the pipe and (vii) stress due to Poisson's affect (embedded pipe), Connecting angle. (The maximum connecting angle between segments of a bending pipe is not exceed 7 degrees) and Seismic load.(The penstock is designed to be safe against the earthquake which intensity of 0.1g in any horizontal direction).

7) Materials

a) SM400B for pipe shells, stiffener rings, thrust collars and seepage collars, SS400 for grouting holes reinforcement and anchor of inlet portion, S25C for grout plug, SM490YA for stiffener rings of inlet portion and SUS CLAD STEEL (SUS304 3mm, SM400B 12mm) for inner sleeve of expansion joint.

8) Allowable Stresses

The allowable stress of steel material under normal loading condition is as shown in the following Table 5.3.2.

Table 5.3.2 Allowable Stress

Materials		Tensile Stress (Kg/cm^2)	Compressive Stress (Kg/cm^2)	Shearing Stress (Kg/cm^2)	Bearing Stress (Kg/cm^2)
Classifications	Kinds				
Rolled steel for general structure	SS400 Thickness≤40mm Thickness >40mm	1300	1300		
Rolled steel for welded structure and	SM400 Thickness SM400 Thickness >40mm	1200	1200	700	
	SM490 Thickness ≤40mm Thickness >40mm	1750 1650	1750 1650	1000 950	
Hot-rolled atmosphere corrosion resisting steel for welded structure	SM490Y Thickness ≤25mm	1950	1950	1100	
	SMA490 Thickness ≤40mm SM520 Thickness >40m	1950 1850	1950 1850	1100 1050	
	SM570 Thickness ≤40mm SMA570 Thickness >40mm	2400 2300	2400 2300	1400 1300	
Rolled steel for boiler	SB410 SB450 SB480	1250 1350 1500	1250 1350 1500	700 750 850	
Rolled steel for rivet	SV330 SV400	- -	- -	1050 1300	-
Carbon steel casting	SC410 SC450 SC480	700 750 800	700 750 800	400 400 400	
Carbon steel forging	SF390 A SF440 A SF490 A SF540 A SF590 A	1100 1250 1400 1550 1700	1100 1250 1400 1550 1700	600 700 800 900 1000	
Grey iron casting	FC200 FC250 FC300	200 250 300	200 250 300	100 150 200	

Allowable stress under seismic and wind loading conditions is increased 50% of the above table. The allowable bearing and shearing stresses of concrete are 50kgf/cm² and 7kgf/cm² respectively.

9) Loading Assumption

The steel penstocks were designed to resist the following loads.

a) Internal loads

The internal load was taken at the sum of the hydrostatic pressure and the pressure rise due to water hammer (see Fig.5.22). (i) The hydrostatic head is the difference between the centerline elevation of the penstock and the maximum design water level DWL.86.370 (HWL.85.00+wave height 1.370m). As the spiral case centre is at EL.33.300, the maximum static head is 53.070m. (ii) The pressure rise due to water hammer are 19.0m, at the centre of water turbine, and assuming linearly change along the penstock centerline.

b) External loads

The penstocks are encased in concrete and were designed to resist the following external loads when they are empty. The safety factor against buckling was taken at 1.5. (i) Embedded portion (The external pressure at the embedded portion was assumed as shown in Fig.5.22. E.Amstuz's formula gave critical buckling pressure for embedded pipe shell). (ii) Exposed portion (The differential pressure between the inside and outside of the penstock shell was assumed at $0.2kfg/cm^2$), (iii) Grout pressure (The pressure of contact grouting between the penstock shell and the surrounding concrete was taken at $4.0kgf/cm^2$).

c) Axial forces :

The penstocks are subject to the following axial forces. (i) Stress due to longitudinal component of the internal pressure acting on an expansion joint, (ii) Stress due to temperature change of the pipe, (iii) Stress due to friction between pipe and support, (iv) Stress due to friction of expansion joint, (v) Local bending stress due to the restraint of the pipe shell bulging by means of a ring girder or stiffener rings, (vi) Stress due to pipe gradient and (vii)Bending stress regarding the pipe as a cantilever beam.

The circumferential stress, longitudinal stress and the shearing stress (perpendicular stress) against the axis of the pipe, and the combined stress under normal condition were held below the allowable stresses mentioned above. For the loading case in which the bending stress caused by stiffener rings was added, the allowable stress was increased by 35% over the above value.

The combined stress was calculated by the following formula :

$$\sigma g = \sigma 1^2 + \sigma 2^2 - \sigma 1 \times \sigma 2 + 3\tau^2$$

Where, σg = Combined stress (kgf/cm^2)
σ1 = Circumferential stress (kgf/cm^2)
σ2 = Longitudinal stress (kgf/cm^2)
τ = Shearing stress (kgf/cm^2)

10) Corrosion Allowance

A thickness of 1.0mm was added to the calculated value as a corrosion allowance.

3.1.4 Design Details of Appurtenance

1) Stiffener Rings

The stiffener rings are attached to the penstock perpendicular to the direction of the penstock axis and to the penstock shell by continuous welding.

The structure of rings attached to the embedded portion are such that secondary concreting around the penstock can be securely placed.

2) Expansion Joint

The expansion joints are located as shown on Fig.5.21. The expansion joints are of the sleeve type and the outside surface of the inner sleeve is clad with corrosion-resisting steel. The corrosion-resisting steel clad is a minimum of 3.0mm in thickness and is considered as surplus thickness. The inner sleeve and the outer sleeve are smooth and uniform where they contact with the seals. The seals are greased hemp square packing and "0" type of rubber packings. The longitudinal movement is more than value calculation from the temperature variation of 80°C with an allowance of 5.0cm.

3) Ring Girder Support

The ring girder supports are located as shown on Fig.5.21. The ring girder supports are of rocker type.

The girder is attached to the penstock perpendicular to the direction of the penstock axis. The ring girders have ribs and bearing flange for the rockers and are attached to penstock shell by continuous welding. The rockers and bases are of cast steel and hinge pins are forged steel with a bushing. The friction coefficient between the penstock and rocker is 0.15.

4) Grouting Hole (where applicable)

Grouting holes are provided in the penstock shell during fabrication. These holes are closed and welded with steel plugs after grouting is completed.

5) Thrust Collar

Thrust collars are welded to penstock shell and have the strength sufficient for transferring axial thrust to surrounding concrete.

6) Seepage Collar

The penstock is provided with three parallel seepage collars at the beginning and other necessary portions of penstock.

7) Manhole

The manholes are provided as shown on Fig.5.20. The manholes are of circular form of 500mm diameter, flanged type with a rubber packing, bolts nuts and other necessary components. The suitable handling tools are provided for handling the manhole covers.

8) Air Pipe

Air pipe is provided as shown on Fig.5.20.

9) Access Ladders and Walkway

Access ladders and walkways are provided for the expansion joints.

10) Anchor Band and Anchor Bar

Anchor bands and anchor bars are provided against the external force to securely anchor the penstock to the base concrete of rock. Anchor bars are embedded in the concrete or rock.

3.2 Mobilization

(1) Preparation of Land Area

Sumitomo Corporation started mobilizing in May 1994. His site facilities were completed in July 1994.

The land area which he used for construction of the site facilities was 75 by 170 meters large. The total area was 12,750 square meters. The land was improved by providing a surface gravel, laying out of drainage trenches and pipes, and constructing all the necessary features required for manufacturing of the penstocks and their accessories.

(2) Site Facilities

The Contractor built in the above area an office building, a warehouse, a work shop office, a canteen and mushola. The above buildings were wooden frame structures and it took three months to build. The fabrication plant as described below was developed within this area with the arrangement of the manufacturing equipment as shown in Figure 23.

3.3 Construction (Shop Fabrication and Site Installation)

3.3.1 Outline

The site fabrication and installation works of the penstock for Kotapanjang HEP started on August 1994 by Sumitomo Corporation and its associated manufacturer Ishikawajima-Harima Heavy Industries Co.,Ltd (IHI) with its local Sub-Contractor PT.Barata Indonesia.

3.3.2 Construction Progress and Resource Use Record

(1) Construction Progress

The actual site installation work progress is shown in Fig.16. The pipe sections were firstly fabricated in a fabrication-shop site on a space near dam-site location (1km from dam-site), where material plates were cut, bent, fitted, and welded into circular sections 3-6m long each. The pipe sections were transported to dam site by trailer and were lowered into position by a 13.5 ton cable crane installed on the top of left and right bank of dam site location, and then were aligned and manually welded to the previously installed section. All built welds were X-ray inspected.

After lower portion consisting of 10 sections were welded together, then concrete was backfilled in the 40.479m length. Also after upper portion consisting of 8 sections were welded together, then concrete was backfilled in the 32.991m length. The remaining 3 sections was exposed in inclined position. The penstocks were constructed during August 1994 - June 1995.

The following is a listing of execution periods of major work items included in the Contract.

a.	Designing	: Aug. 1993	-	Apr. 1994
b.	Material delivery from Japan to site	: Feb. 1994	-	Sep. 1995
c.	Site preparation	: May 1994	-	Aug. 1994
d.	Site fabrication	: Aug. 1994	-	May 1995
e.	Actual erection	: Aug. 1994	-	Jun. 1995
f.	Finishing	: Apr. 1996	-	Jun. 1996

(2) Labour Needs

The monthly use record of the workers is shown in Table 5.3.3. In total 24 man-months of Japanese workers and 1309 man months of Indonesian workers were used for this construction work.

Table 5.3.3 Labor Needs for Site Fabrication and Installation Work of Penstock

Month		Japanese (M/M)	Indonesian (M/M)
1994	May	2	25
	June	2	52
	July	2	109
	August	2	135
	September	2	139
	October	2	97
	November	2	100
	December	2	119
1995	January	2	123
	February	2	121
	March	1	102
	April	1	82
	May	1	60
	June	1	45
Total		24	1309

(3) Equipment Used

As originally conceived, the field fabrication plant consisted of one gantry crane, one rolling machine, compressor equipment, generating set equipment, and flat stand for cutting. It was also provided with the necessary welding equipment. The type of major equipment used are presented in Table 5.3.4.

Table 5.3.4 Equipment List for Fabrication

No.	Description	Capacity	Quantity	Remarks
1.	Roll Machine	-	1 unit	
2.	Gantry Crane	10 Ton	1 unit	
3.	Generator Set	250 KVA	2 unit	
4.	Travo Welding Machine	400 A	17 unit	
5.	Auto.Welding Machine	400 A	2 unit	
6.	Gas Cutting Machine	-	3 unit	
7.	Turning Roller	10 Ton	10 Unit	
8.	Gantry Rail	6000 L	18 PCs	
9.	Truck Crane	30 Ton	1 unit	
10.	Truck	10 Ton	1 unit	
11.	Trailer	25 Ton	2 units	Rental
12.	Electric Winch	25 Ton	1 unit	Rental
13.	Hand Winch	5 Ton	1 unit	
14.	Air Compressor	8 BAR	2 units	
15.	Container Box	20 Feet	2 units	1 Unit Rental
16.	Kijang St.Mobile	-	3 units	
17.	Kijang Pick Up	-	1 unit	

(4) Electric Power Consumption

The electric power required for the site installation work was supplied by Die
Electric Generating set owned by the Contractor. The monthly power consump
record shown in Table 5.3.5.

Table 5.3.5 Electric Power Used for Site Work of Penstock Installation

Year/Month		Energy Consumed(kWh)
1994	June	25,200
	July	48,000
	August	102,600
	September	113,400
	October	106,600
	November	76,800
	December	87,400
1995	January	102,400
	February	59,600
	March	89,600
	April	85,200
	May	55,400
	June	42,200
Total		994,200

(5) Weight of Steel Penstocks

The tonnage of the penstock steel pipes by major items is shown in Table 5.3.6.

Table 5.3.6 Weight of Steel Penstocks

(Unit = Ton)

Description	Penstock No.1	Penstock No.2	Penstock No.3	Total
1. Penstock shell				
CP 1	7.891	7.891	7.891	23.673
CP 2	5.777	5.777	5.777	17.331
CP 3	8.647	8.647	8.647	25.941
	11.280	11.280	11.280	33.840
CP 4	6.738	6.738	6.738	20.214
CP 5	9.476	9.476	9.476	28.428
CP 6	9.476	9.476		28.428
CP 7	9.101	9.101	9.476	27.303
CP 8	7.437	7.437	9.101	22.311
CP 9	5.946	5.946	7.437	17.838
CP 10	14.554	14.554	5.946	43.662
CP 11	10.964	10.964	14.554	32.892
CP 12	7.199	7.199	10.964	21.597
CP 13	8.350	8.350	7.199	25.050
CP 14	8.350	8.350	8.350	25.050
CP 15	8.350	8.350	8.350	25.050
CP 16	9.940	9.940	8.350	29.820
CP 17	11.913	11.913	9.940	35.739
CP 18	11.557	11.557	11.913	34.671
CP 19	3.630	3.630	11.557	10.890
CP 20	4.403	4.403	3.630	13.209
CP 21	1.870	1.870	4.403	5.610
2. Manholes	1.596	1.596	1.870	4.788
3. Accessories	184.445	184.445	1.596	553.335
TOTAL			184.445	

3.3.3 Working Procedure

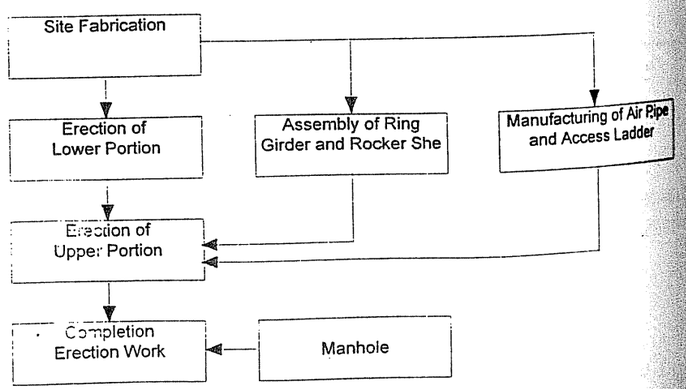
The Construction work consists of fabrication at the field work-shop, transportation and site installation. See Fig.5.24 Flow chart of penstock activities. The pipe sections were fabricated at the field shop because of inland transportation limit. On this shop were conducted, cutting of materials, edge preparation, plate rolling/bending, stiffener ring fabrication, fitting, and welding. The circular pipe sections which were produced in this shop, were delivered to and erected at the site. The general working process is shown in Table 5.3.7.

Table 5.3.7 General Working Process

Mill Maker		Factory	Field Shop	Erection Site
Penstock	Inspection of Material	Plate processing	Manufacturing	Penstock
Ring Girder and Rocker shoe		Plate processing Manufacturing Shop assembling	Field assembling	Ring Girder and Rocker Shoe
Manhole		Plate processing Manufacturing Shop assembling	-	Manhole
Air pipe and Access Ladder		-	Manufacturing	Air pipe and Access Ladder

The field installation work was carried out in the process as shown below. The erection and installation work consists of transportation, fitting, welding, inspection and painting.

Flow Chart of field Installation



3.3.4 Temporary Facilities

(1) Field Work Shop

A field work shop of about 12750m^2 (170m x 75m in area) was provided on an area of about 1km far from dam site location. This workshop comprised an office, a warehouse, assembling yards, inspection yard, stock area, a mess, a canteen, a rest room, and a chapel. Fig.5.23 shows a general layout of the work shop. The equipment used for fabrication is listed in Table 5.3.4.

(2) 13.5ton Cable Crane

For actual erection work a 13.5ton cable crane and related facilities such as rail for radial movement, travelling carriage with winching equipment, and diesel electric generating set for supplying electrical energy, were installed over the dam site location across the left and right bank of Kampar river.

In fact the main purpose of the cable crane is for placing materials for dam body, yet it can also be used for erecting penstock section pipes to its position in dam site (see Fig.5.27).

As shown in Fig.5.27, a cable crane was used for lowering and positioning the pipe sections in the penstock position. By using chain blocks and wedges, workers aligned and fitted to the previously installed section.

3.3.5 Fabrication

(1) General

The fabrication work consists of cutting, bending, fitting, and welding. The work began on July 1994 and was completed in May 1995. The main Contractor (Sumitomo Corporation) employed PT.Barata (PERSERO) as a local sub-Contractor for this work. See Fig.5.25 Fabrication Method of Penstock.

(2) Fabrication at Site Shop

Fabrication in the site shop consists of cutting the shell plates, preparing edges for welding, pre-bending of plate edges for rolling, plate rolling, fitting, welding, and examination of dimensions (including radiographic inspections). This work started in July 1994, and was completed in May 1995. Primered raw steel plated were delivered into the site for steel penstock fabrication. Delivered plates were cut and bent to semi-circular shape, and edge preparation was made for welding. Two pieces of bent semi-circular shape of plate were fitted and welded, to form a normally 3m length of unit circular section.

Two circular unit sections were jointed and welded into one "installation unit pipe" of which length is normally 6m and weight is 10.5 tons. Internal supports were provided to keep proper circular form of installation unit pipe. Installation unit pipe was transported by trailer-truck to the installation place.

(3) Fabrication at PT.Barata Shop

Some part of the penstocks were fabricated at PT.Barata Shop, Surabaya : ring girder, rocker-shoes, and CP1 and CP 2 in sections.

3.3.6 Installation

(1) General

The installation work consists of transportation, unloading, fitting, welding, inspection and painting. The work began on August 1994 and was completed at June 1996. The main Contractor (Sumitomo Corporation) employed PT.Barata (PERSERO) as a local sub-Contractor for this work.

(2) Transportation and Unloading

All unit pipes and accessories for the main penstocks were transported by trailer truck from the unit pipe storage yard at fabrication site shop to the left bank cable crane pit at dam site. They were unloaded and lowered to the erection spot using the 13.5ton cable crane. See Fig.5.26 Transportation of Penstock Unit-Pipe and Fig.5.27 Installation Method of Penstock.

(3) Site Installation

Dam slab concrete was concreted in the stair case form as shown in Fig.5.28, 5.29 and 5.30. Anchors and bars were placed at respective level of slab concrete measuring to locations by the other Contractor. Applying the cable crane, supporting stages for penstock emplacement were lowered down and were placed respectively at proper positions from the downstream end. Embedded anchors and stagings were be rotated and welded.

Each installation unit pipe (6m long each) was lowered down onto the installation place. After the fitting and adjustment, each installation unit was welded and braced to the anchor bars and supporting stages. See Fig.5.27.

Two sections at the downstream ends of steel penstock (CP 20 and CP 21) were lowered and placed by overhead crane in the powerhouse, after the completion of powerhouse. After the completion of installation of lower portion of steel penstock, expansion joint was installed.

Check of locational measurements was made for the installed expansion joint sections and at this time, tightening of bolts for stuffing of packing material was not made. Tightening of bolts was made after the completion of entire penstock line.

Respective installation unit pipe of upper portion of steel penstock was installed after the finish of each level of slab concreting and emplacement of anchor bars. Emplacement of anchor bars were made by the other Contractor. Lowering and Emplacement of supporting stages and installation unit pipes were made directly by cable crane, and penstocks were embedded by the other Contractor.

All of the internal supports were removed and painting was made for the internal and external surfaces. After the installation of spiral casing by the other Contractor, remained two sections at the end each penstock lines (CP 20 and CP 21) were fitted and welded.

(4) Painting

a) Paint Materials

Table 5.3.8 Paint Materials

Location	Type of Paint	Brand Name	Maker
(1)	(2)	(3)	(4)
Interior surface & outside surface of exposed in the gallery	Primer - Mill or shop	- Nippe Zinky 8000 (organic Zinc Rich Primer/Epoxy Polyamide Resin Zinc Rich Primer)	Nippon Paint
	1st-Shop or field shop	- Epotar HB (High Build Type Tar Epoxy-Polyamide Resin)	Nippon Paint
	Finishing - Shop or field shop	- Same as above	
Exposed parts outside of dam	Primer - Mill or shop	- Nippe Zinky 8000 (Organic Zinc Rich Primer)	Nippon Paint
	1st - shop	- Chlorinated Rubber Resin Red Lead and Red Oxide Primer	Nippon Paint
	2nd - shop	- Same as above	
	3rd - shop or after installation	- Chlorinated Rubber Resin, Intermediate coat	Nippon Paint
	Finishing - Same as above	- Same as above	
Embedded exterior surface area	Primer - Mill or shop	Nippe Zinky 8000 (Organic Zinc Rich Primer)	Nippon Paint
Access ladder	Primer - Mill or shop	- Nippe Zinky 8000 (Organic Zinc Rich Primer)	Nippon Paint
	1st - Shop	- Chlorinated Rubber Resin Red Lead and Red Oxide Primer	Nippon Paint
	2nd - Shop	- Same as above	
	3rd - Shop or after installation	- Chlorinated Rubber Resin, Intermediate Coat	Nippon Paint
	Finishing - Same as above	- Same as above	

b) Material Control

Table 5.3.9 Paint Materials Control for Epotar HB

Control Item	Specified	Control Method
- Mixing paint	Over 2 min	Control by specialist (local)
- Pot life	5 hours at 30°C after mixing	Checking paint value to be used
- Mixing ratio	Base : Hardener = 9:1 by volume	Measuring value by gauge

c) Painting Control

Table 5.3.10 Painting Operation Control Methods for Epotar HB

Control Item	Specified	Control Method
Film thickness	200μ/coat, airless spray	Measure by thickness gauge
Interval	Min. 24 hours, Max. 7 days	Controlled by specialist (Local)
Humidity	Less than 85%	Working day decided by specialist (Local)
Plate temperature	Less than 50°C	Temperature checked by gauge
Machine	Air pressure	Air pressure checked prior to work

d) Film Thickness of Paint (Especially for the interior surface of penstock)

The paint film thickness was measure at more than 4 points per 5m² and electromagnetic film thickness gauges or electrometers. The film thickness measure 420-800μ at 462 points per penstock (1386 points for total 3 penstocks for the specification requirement of 420μ or more.

3.3.7 Welding Operation

(1) Welding Method

Welding was done manually by shielded metal arc welding (SMAW) and submerged arc welding (SAW) automatic. Each welding method was applied as shown in Table 5.3.11.

Table 5.3.11 Application of Welding Methods

Classification	Place Method Joints	Factory		Field Shop		Dam Site
		SAW	SMAW	SAW	SMAW	SMAW
Pipe shell	Longitudinal joint of straight pipe				o	
	Circumferential joint of straight pipe				o	o
	Longitudinal joint of bend pipe				o	
	Circumferential joint of bend pipe				o	o
Penstock box	Flat joint	o				
	Corner weld		o		o	o
Fittings	Stiffeners, thrust collars, seepage rings				o	
	Internal support				o	
	Jig for erection				o	o
Ring girder	Shaping joint		o			
	Circumferential joint to pipe shell				o	
Rocker shoe	Shaping joint		o		o	
Ladder and walkway	Shaping joint				o	
	Installation joint					o

(2) Welding Electrodes and Drying Condition

The welding electrodes conformed to JIS Z3211 D4301, JIS Z3212 D5016, JIS Z3221 D308-16, JIS Z3221 D309-16.

Welding electrodes were selected of the most suitable materials for the base metals as shown in Table 5.3.12. The electrodes selected were kept under sufficient dry conditions as shown in Table 5.3.13 before used.

Table 5.3.12 Electrodes for Arc Welding

Base Metal (JIS)	Welding Position	JIS Classification	Brand Name	Manufacturer
SS 400	All position	Z3211 D4301 Z3212 D5016	B-17 LB-52	Kobe Steel
	Flat	Z3221 D309-16	NC-39	
SM 400B	All position	Z3211 D4301 Z3212 D5016 Z3221 D309-16	B-17 LB-52 NC-39	Kobe Steel
	Flat	Z3183 S502-H	MF-38 X US-36	
SM 490 YA	All position	Z3211 D4301 Z3212 D5016	B-17 LB-52	Kobe Steel
	Horizontal	Z3183 S501-H	MF-44 X US-36	
SUS 304	Horizontal	Z3221 D308-16 Z3221 D309-16	NC-38 NC-39	Kobe Steel
SUS 304 N2	Horizontal	Z3221 D308-16	NC-38	Kobe Steel
SUS CLAD STEEL (SUS 304 + SM 400 B)	All position	Z3212 D5016 Z3221 D309-16	LB-52 NC-39	Kobe Steel

Table 5.3.13 Drying the Welding Material

Type	Brand Name	Drying Temperature (°C)	Drying Time (Min)
Electrode	B-17 LB-52 NC-38/NC-39	70-100 300-350 150-200	30-60 30-60 30-60
Flux	MF-38/MF-44	150-350	30-60

(3) Welders and Welding Operators :

All welders and welding operator assigned to the work passed a qualification test in accordance with ASME Boiler and Pressure Vessel Code Section IX.

(4) Radiographic Examination

a) Length Examined

The welded seam of the pipe shells were radiographed in the 20% of butt welded length at the field shop and erection site.

b) Result of X-ray Test

The results of the radiographic tests were evaluated in accordance to JIS Z3140 "Method of Radiographic Test and Classification of Radiographic for Steel Work". Objectionable defective portions were chipped off to receive re welding. Upon completion of the rewelding, such non destructive test as radiographic test was made.

The required radiographic examinations were performed, and all the films photographed were inspected by the Engineer. The results of this X-ray examination are given in Table 5.3.14.

The required ultrasonic tests were also performed for fillet weld of the wrapped plate connection between penstock and spiral casing, and fillet weld of manholes. The results are also given in Table 5.3.15.

Table 5.3.14 Radiographic Film Quantity on Penstock Welding Seams

Penstock	Site Shop			Site			Surabaya Shop			Total
	Shoot	Re-Shoot	Total	Shoot	Re-Shoot	Total	Shoot	Re-Shoot	Total	
No.1	292	27	319	202	16	218	28	14	42	579
No.2	292	10	302	202	12	214	28	7	35	551
No.3	292	14	306	202	6	208	28	2	30	544
Total	876	51	927	606	34	640	684	23	107	1674

Table 5.3.15 Ultrasonic Test Report

Penstock	19mm Plate	15.4mm Plate
No.1	18	18
No.2	18	18
No.3	16	18

(5) Quality Control

(a) Materials

The steel materials used for fabrication of penstocks and appurtenances were selected after having passed the mill tests of i) mechanical property test (tension test) and ii) chemical analysis test in accordance with applicable JIS codes.

Certified copies of the mill test reports were submitted to the Engineer for his approval prior to shipping to the site.

(b) Welding Procedure Specification

The shape and dimension of the welding to be adopted in the welding work are shown in Appendix III.

(c) Welding Conditions

1) Drying Condition of Electrodes

All electrodes and fluxes were kept sufficient dry prior to their use. the drying temperature and time were shown in Table 5.3.13.

2) Heating Condition

The pre-heating was done according to the temperatures indicated in Table 5.3.16. The order conditions are i) preheat area (preheat area is more than two times the plate thickness from the welding line) ii) check to preheat temperature (preheat temperature was checked by tempi stick or contact thermometer) and iii) interpas temperature (maximum interpas temperature is less than 200°C).

Table 5.3.16 Pre-Heat and Interpas Temperature

Base Metal (JIS)	Thickness (mm)	Welding Process	
		SMAW	SAW
SS 400 SM 400 B	T < 25 25 ≤ T < 38 38 ≤ T	- 50 50	- - 50
SM 490 YA	T < 25 25 ≤ T < 38 38 ≤ T	- 50 50	- - 50
SUS 304 SUS 304 N	T < 25 25 ≤ T < 38 38 ≤ T		

(d) Welding Control

All the welding work was controlled under the conditions required.

(e) Welding Work Instruction

i) Checking of the Parts to be Welded

Prior to starting the elding work, the parts to be welded were sufficiently checked for the state of cleanliness (scale, rust, oil, moisture), the prepared edge root opening, the mounting of the tab and the existence of crack in the tack welding before starting the welding work.

ii) Multi Layer Welding

When there are 2 or more layers, care is to be taken not to concentrate the change of welding rod at the same place. More over it is also to be refrained from leaving craters and creating overlap. In particular, care utmost be taken not for leave any crater at end of bead. In addition, it is also to be cleaned slug and sputter of each layer, and moreover to be inspected each layer for defect and mad the required repair.

iii) Quotation of Welding

The welding work was carried out by taking special care to prevent occurance, and if any to take correction practice, of defect like cracks, in completed penetration, slag inclusion, blow hole and pinhole, pits, undercuts, overlaps, arc strike, shortage or excess of length of leg, craters etc. in the welded portion.

iv) Weld Beads

The beads were made so as to have external appearance as smooth as possible with a regular wave form, and the welding size and length conformed to the design dimensions.

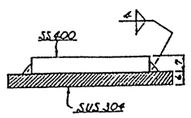
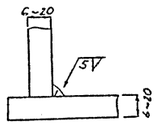
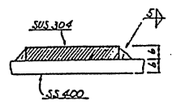
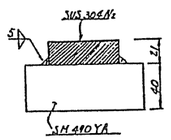
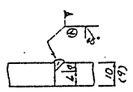
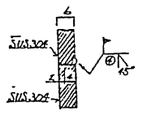
v) Reinforcement of Weld

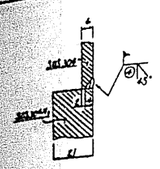
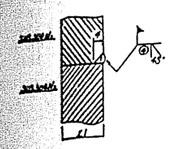
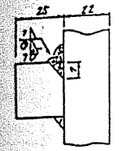
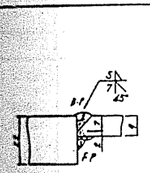
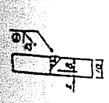
All the welding joints were dressed smooth, and maximum reinforcement was limited to 10% to 15% of the plate thickness. (Care is to be taken not to grind out more than 0.5mm). When the finish is not specified, 20% of the bead wide (however not exceeding 3mm).

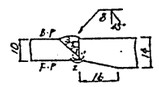
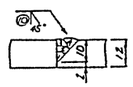
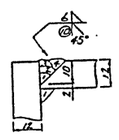
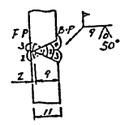
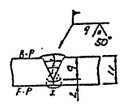
vi) Weather Conditions

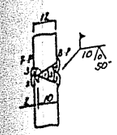
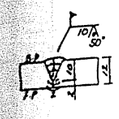
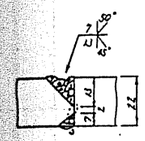
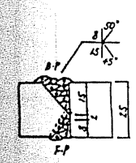
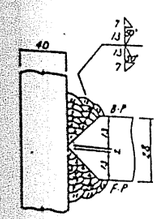
In case of rainy weather, execution of welding was in principle avoided at places subject to noxious influences of the rain. If it was confirmed, however, that there were no noxious influences, the welding was executed taking special care of the humidity by dehumidifying heating as occasion demanded.

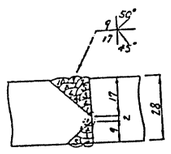
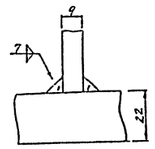
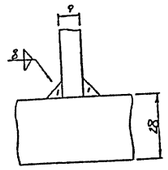
Appendix I Welding Procedure

Application	Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
					No.	Size (mm)	Brand	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	
Guide frame of the spillway gate		JIS SS400 JIS SUS304	SMAW	Horizontal	1	5.0	NC-39	
Welds which are not noted in DWG		JIS SS400 JIS SM400B JIS sM490 YA	SMAW	All	(Flat position) 1 (Vertical position) 1 (Horizontal Position) 1	5.0 4.0 4.0	LB-52 B-17 (for mild steel)	
Guide frame of the spillway gate		JIS SS400 JIS SUS304	SMAW	Horizontal	1	5.0	NC-39	
Ditto		JIS SM490 YA JIS SUS304 N2	SMAW	Horizontal	1	5.0	NC-39	
Gate leaf of spillway gate etc.		JIS SM490 YA JIS SM490 YA	SMAW	Flat	1	4.0	LB-52	
Guide frame of the spillway gate		JIS SUS 304 JIS SUS 304	SMAW	Horizontal	1	3.2	NC-38	

Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
	JIS SUS304 JIS SUS304 N2	SMAW	Horizontal	1	3.2	NC-38	90
	JIS SUS304 N2 JIS SUS304 N2	SMAW	Horizontal	1	3.2	NC-38	90
	JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4 5 1	4.0 4.0 5.0 5.0 5.0 6.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 230 300
	JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 1 2 3	4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52	180 180 180 180 180
	JIS SS400 JIS SS400	SMAW	Flat	1 2	4.0 4.0	LB-52 LB-52	180 180

Application	Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
Girder web of tailrace gate etc.		JIS SM490 YA JIS SM490 YA	SMAW	Fla	1 2 3 4	4.0 4.0 5.0 5.0	LB-52 LB-52 LB-52 LB-52	
Gate leaf of spillway stoplogs etc.		JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4	4.0 4.0 5.0 5.0	LB-52 LB-52 LB-52 LB-52	
Gate leaf of spillway gate etc.		JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4 1	4.0 4.0 5.0 5.0 6.0	LB-52 LB-52 LB-52 LB-52 LB-52	
Field joint of skin plate etc.		JIS SM490 YA JIS SM490 YA	SMAW	Horizontal	1 2 3 4 1 2 3	4.0 4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	
Ditto		JIS SM490 YA JIS SM490 YA	SMAW	Vertical	1 2 3 1 2	4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52	

Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
	JIS SM490 YA JIS SM490 YA	SMAW	Horizontal	1 2 3 4 1 2 3	4.0 4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	170 170 170 170 170 170 170
	JIS SM490 YA JIS SM490 YA	SMAW	Vertical	1 2 3 1 2	4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52	140 140 140 140 140
	JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4 5 6 7 1 2 3	4.0 4.0 5.0 5.0 5.0 5.0 5.0 4.0 4.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 230 230 230 180 180 230
	JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4 5 6 7 8 9 1 2 3 4 5 6	4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 4.0 4.0 4.0 4.0 5.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 180 180 230 230 230 230 230 180 180 180 180 230 230
	JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4 5 6 7 8 9 10 11 12 13 1 2 3 4 5 6 7 8 9 10 11 12 13	4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0 4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0	LB-52 LB-52	180 180 180 180 230 230 230 230 230 230 230 230 230 180 180 180 180 230 230 230 230 230 230 230 230

Application	Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current (A)
Girder flange of spillway gate	 This joint shall be taken the qualification tests.	JIS SM490 YA JIS SM490 YA	SMAW	Flat	1 2 3 4 5 6 7 8 9 10 1 2 3 4 5	4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 5.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	
Girder flange to web of tailrace, spillway		JIS SM490 YA JIS SM490 YA	SAW	Horizontal	1	4.8	US-36	
Ditto		JIS SM490 YA JIS SM490 YA	SAW	Horizontal	1	4.8	US-36	

A. Painting Schedule

Item		Classification Number	Paint	Color or Munsell Color	Coating Area (m^2)
Spillway Gate	Upstream side of gate leaves	1	Tar-epoxy resin paint	Black	200
	Downstream side of gate leaves	2	Chlorinated rubber paint	5B6/8	1350
	Guide Frame	1	Tar-epoxy resin paint	Black	70
	Hoists	3	Alkyd resin paint	2.5Y8/16	150
Spillway stoplog	Stoplogs & Lifting beam	2	Chlorinated rubber paint	5B6/8	2150
	Guide frames	1	Tar-epoxy resin paint	Black	15
Intake gate	Gate leaves	2	Chlorinated rubber paint	5B6/8	310
	Guide frames & Steel beams	1	Tar-epoxy resin paint	Black	60
	Hoist	3	Alkyd resin paint	2.5Y8/16	80
	Opening cover	2	Chlorinated rubber paint	5B6/8	70
Intake stoplog	Stoplog & Lifting beam	2	Chlorinated rubber paint	5B6/8	1250
	Guide frames	1	Tar-epoxy resin paint	Black	20
Trashrack	Trashracks	1	Tar-epoxy resin paint	Black	890
Trashrake	Rake of Trashrake	1	Tar-epoxy resin paint	Black	
	Trashrake	3	Alkyd resin paint	2.5YR6/13	
	Rails & fisher plates	-	No painting	-	-
Tailrace gate	Gate leaves	2	Chlorinated rubber paint	5B6/8	180
	Guide frames & Dogging devices	1	Tar-epoxy resin paint	Black	20
	By-pass valves including water filling pipes & Air pipes	1	Tar-epoxy resin paint	Black	10
	Embedded metal for opening cover	2	Chlorinated rubber paint	5B6/8	10
	Opening cover	-	Zinc plating	-	-
Tailrace stoplog	Stoplog	2	Chlorinated rubber paint	5B6/8	260
	Stoplog facility	2	Chlorinated rubber paint	5B6/8	76

Item		Classification Number	Paint	Color or Munsell Color	Coating Area (m^2)
Gantry Crane	Gantry crane	3	Alkyd resin paint	2.5Y8/16	
	Rails & fisher plates	-	No painting	-	-
Penstock	Interior surface & outside surface of exposed in the gallery	1	Tar-epoxy resin paint	Black	1550
	Exposed parts outside of dam & Access ladder	2	Chlorinated rubber paint	5B6/8	320

B. Classification of Paint

(1) Classification 1

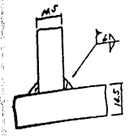
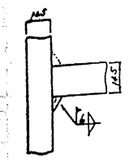
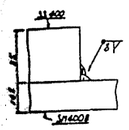
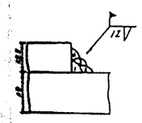
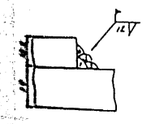
Coat	Paint	Thickness	Applied at	Remark
Primer	Zinc-rich primer	0.02 mm	Mill or shop	Thickness of total dry coat not less than 0.42 mm
First	Tar epoxy paint	0.02 mm	Shop or fieldshop	
Finishing	Same as above	0.20 mm	Shop or fieldshop	

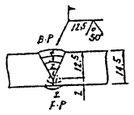
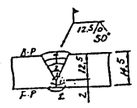
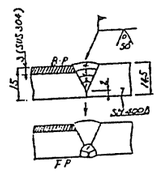
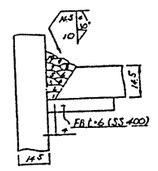
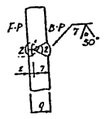
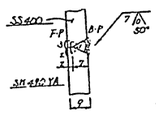
(2) Classification 2

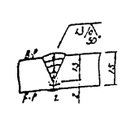
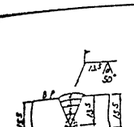
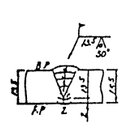
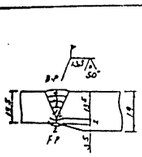
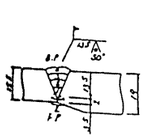
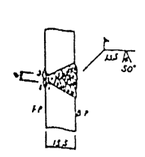
Coat	Paint	Thickness	Applied at	Remark
Primer	Zinc-rich primer	0.02 mm	Mill or shop	Thickness of total dry coat not less than 0.16 mm
First	Chlorinated rubber paint under coat	0.04 mm	Shop	
Second	Same as above	0.04 mm	Shop	
Third	Chlorinated rubber paint (intermediate coat)	0.03 mm	Shop or after installation	
Finishing	Same as above	0.03 mm	Shop or fieldshop	

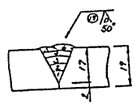
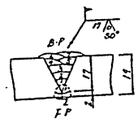
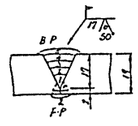
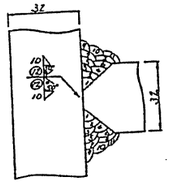
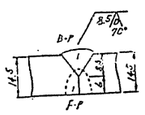
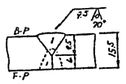
(3) Classification 3

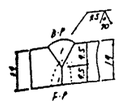
Coat	Paint	Thickness	Applied at	Remark
Primer	Wash primer	0.15 mm	Mill or shop	Thickness of total dry coat not less than 0.135 mm
First	Red lead anti corrosive paint	0.35 mm	Shop	
Second	Same as above	0.35 mm	Shop	
Third	Alkyd resin paint (intermediate coat)	0.25 mm	Shop or after installation	
Finishing	Same as above	0.25 mm	Shop or fieldshop	

Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
				No.	Size (mm)	Brand	Amps
(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	JIS SM400 BX JIS SM400 B	SMAW	Horizontal	1	5.0	LB-52	230
	JIS SM400 BX JIS SM400B	SMAW	Vertical	1	5.0	LB-52	160
	JIS SM400 BX JIS SS400	SMAW	Horizontal	1 2	6.0 6.0	LB-52 LB-52	300 300
	JIS SM400 BX JIS SM400B	SMAW	Horizontal	1 2 3 4	6.0 6.0 6.0 6.0	LB-52 LB-52 LB-52 LB-52	300 300 300 300
	JIS SM400 BX JIS SM400B	SMAW	Verical	1 2 3 4	4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52	140 140 140 140

Application	Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
Field joint of penstock pipe		JIS SM400 BX JIS SM400 B	SMAW	Flat	1 2 3 4 1 2	4.0 4.0 5.0 5.0 4.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 180 230
Ditto		JIS SM400 BX JIS SM400 B	SMAW	Vertical Over head	1 2 3 4 1 2	4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 180 230
Expansion joint of penstock		JIS SUS CLUDX JIS SM400 B	SMAW	FLAT	1 2 3 4 1 2	4.0 4.0 5.0 5.0 4.0 5.0	LB-52 LB-52 NC-39 NC-39 LB-52 LB-52	140 140 140 140 140 140
Ring girder of penstock		JIS SM400 BX JIS SM400 B	SMAW	Flat	1 2 3 4 5 6 7 8 9 10	4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 180 230 230 230 230 230
Ring of penstock etc.		JIS SS400 X JIS SS400	SMAW	Horizontal	1 2 1 2	4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52	170 170 170 170
Ditto		JIS SS400 X JIS SM490 YA	SMAW	Horizontal	1 2 3 1 2 3	4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	170 170 170 170 170 170

Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
	JIS SM400 BX JIS SUS400 B	SMAW	Flat	1 2 3 4 1 2	4.0 4.0 5.0 5.0 4.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 180 230
	SM400 BX SM400 B	SMAW	Flat	1 2 3 4 1 2	4.0 4.0 5.0 5.0 4.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 180 230
	SM 400 BX SM 400 B	SMAW	Vertical Overhead	1 2 3 4 1 2	4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	140 140 140 140 140 140
	SM 400 BX SM 400 B	SMAW	Flat	1 2 3 4 1 2	4.0 4.0 5.0 5.0 4.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	180 180 230 230 180 230
	SM 400 BX SM 400 B	SMAW	Vertical Overhead	1 2 3 4 1 2	4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	140 140 140 140 140 140
	SM 400 BX SM 400 B	SMAW	Horizontal	1 2 3 4 5 6 7 8 9 10 1 2 3	4.0 4.0 4.0 4.0 4.0 4.0 4.0 4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	170 170 170 170 170 170 170 170 170 170 170 170 170

Application	Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
Expansion joint of penstock		SM 400 BX SM 400 B	SMAW	Flat	1 2 3	4.0 4.0	LB-52	
Field joint of penstock pipe		SM 400 BX SM 400 B	SMAW	Flat	1 2 3 4 5 6 7 1 2	4.0 4.0 5.0 5.0 5.0 5.0 5.0 4.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	
Ditto		SM 400 BX SM 400 B	SMAW	Vertical Overhead	1 2 3 4 5 1 2	4.0 4.0 4.0 4.0 4.0 4.0 4.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	
Rocker shoe of penstock		SM 400 BX SM 400 B	SMAW	Flat	1 2 3 4 5 6 7 8 9 10 11 1 2 3 4 5 6 7 8 9 10 11	4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0 4.0 4.0 4.0 4.0 5.0 5.0 5.0 5.0 5.0 5.0 5.0	LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52 LB-52	
Shop joint of penstock pipe		SM 400 BX SM 400 B	SAW	Flat	1 1	4.8 4.8	US-36 US-36	
Ditto		SM 400 BX SM 400 B	SAW	Flat	1 1	4.8 4.8	US-36 US-36	

Joint Detail	Base Metal	Process	Position	Pass	Electrode		Welding Current AC
 This joint shall be taken the qualification tests.	SM 400 BX SM 400 B	SAW	Flat	1 1	4.8 4.8	US-36 US-36	800 800

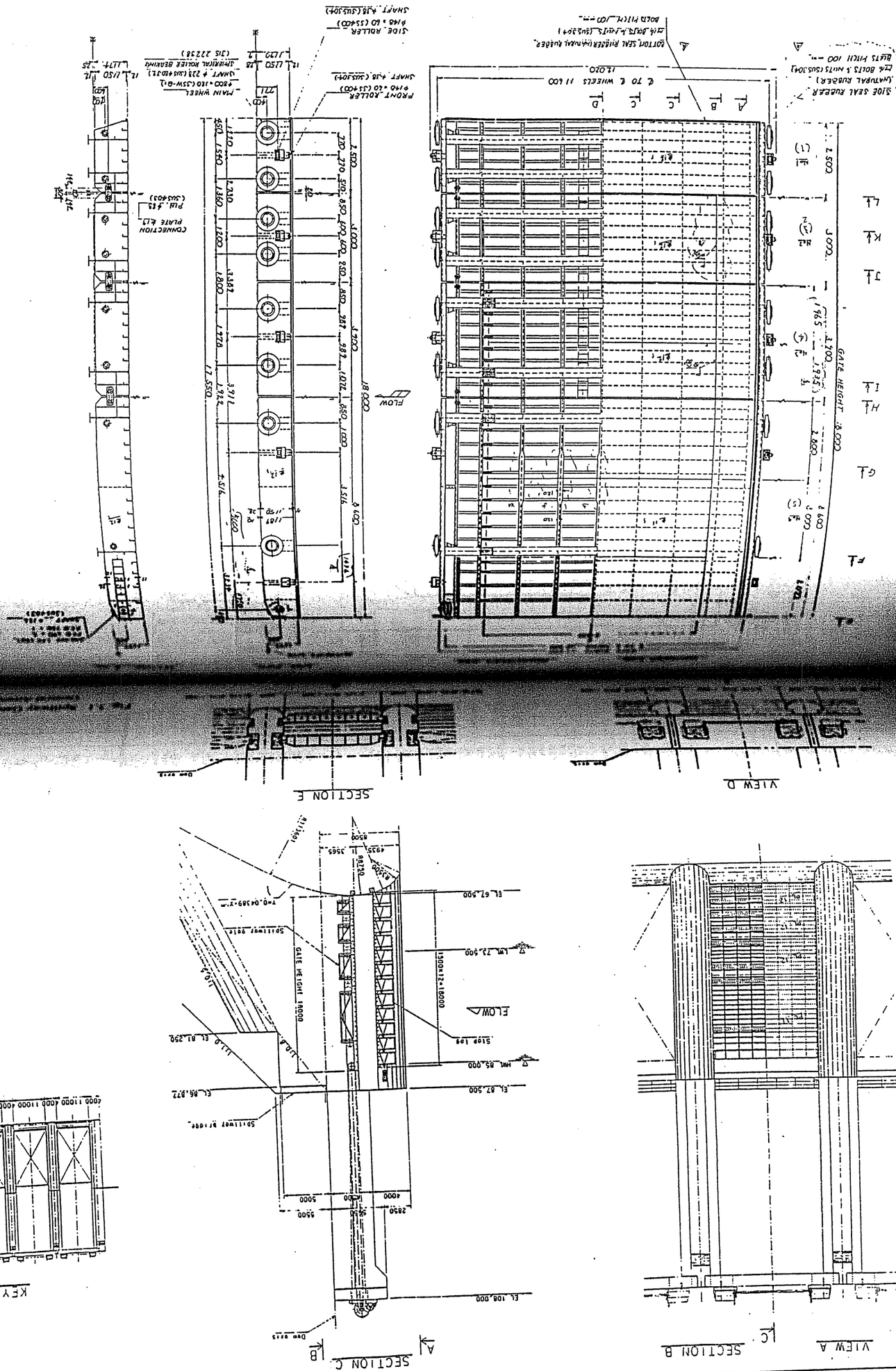


Fig. 5.2 Spillway gate, Detail Drawing of Gate Leaf

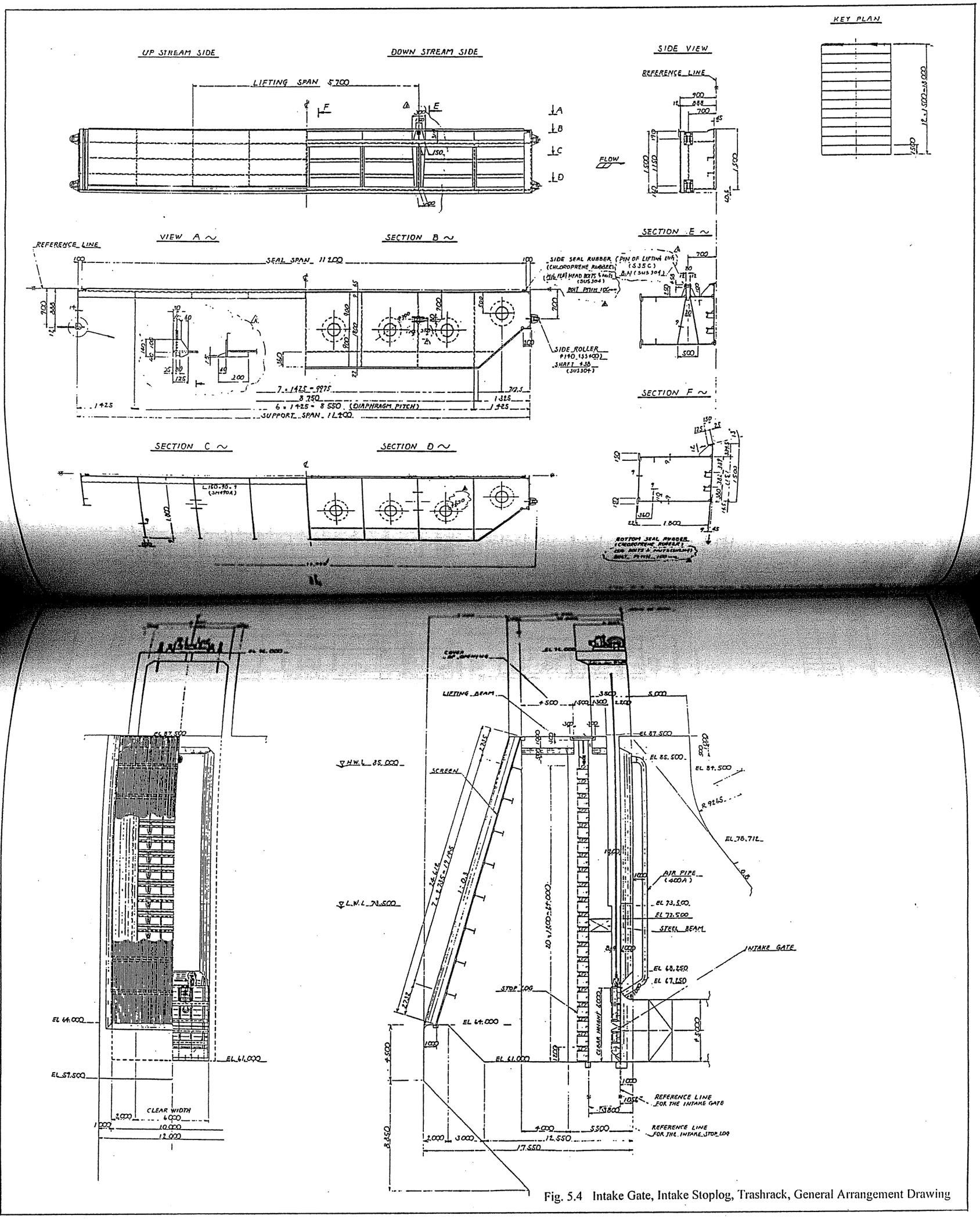


Fig. 5.4 Intake Gate, Intake Stoplog, Trashrack, General Arrangement Drawing

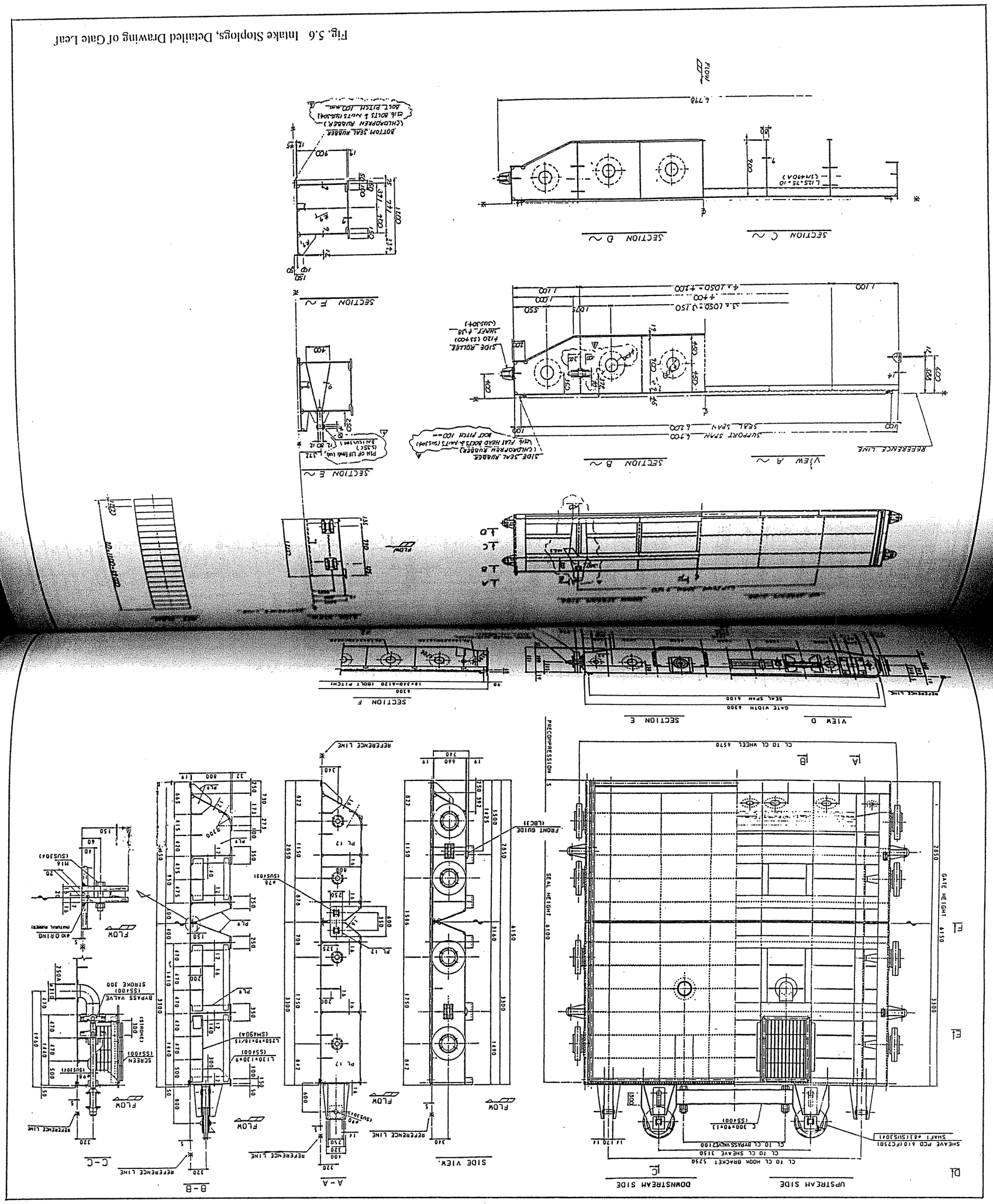


Fig. 5.6 Intake Stoplogs, Detailed Drawing of Gate Leaf

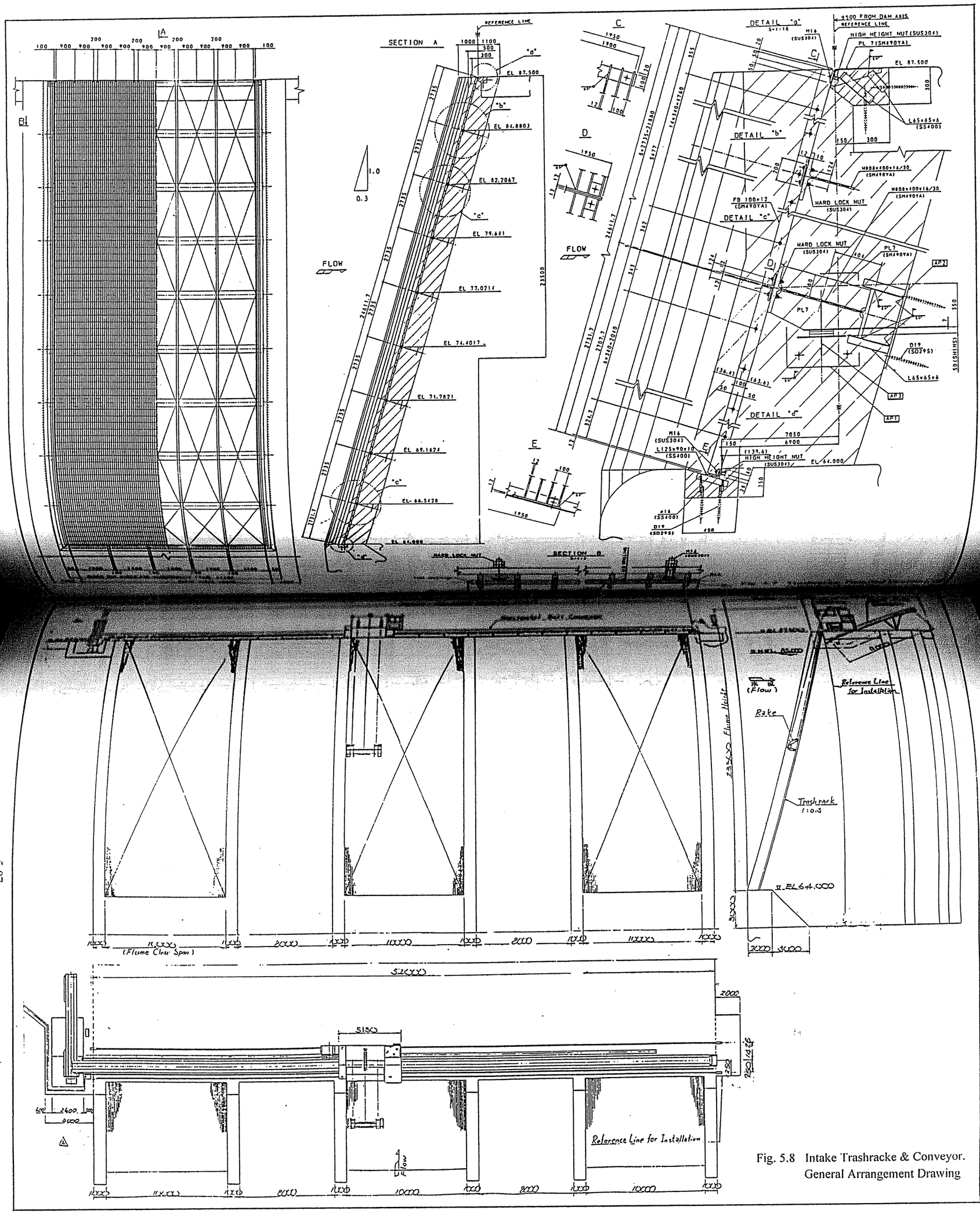


Fig. 5.8 Intake Trashracke & Conveyor. General Arrangement Drawing

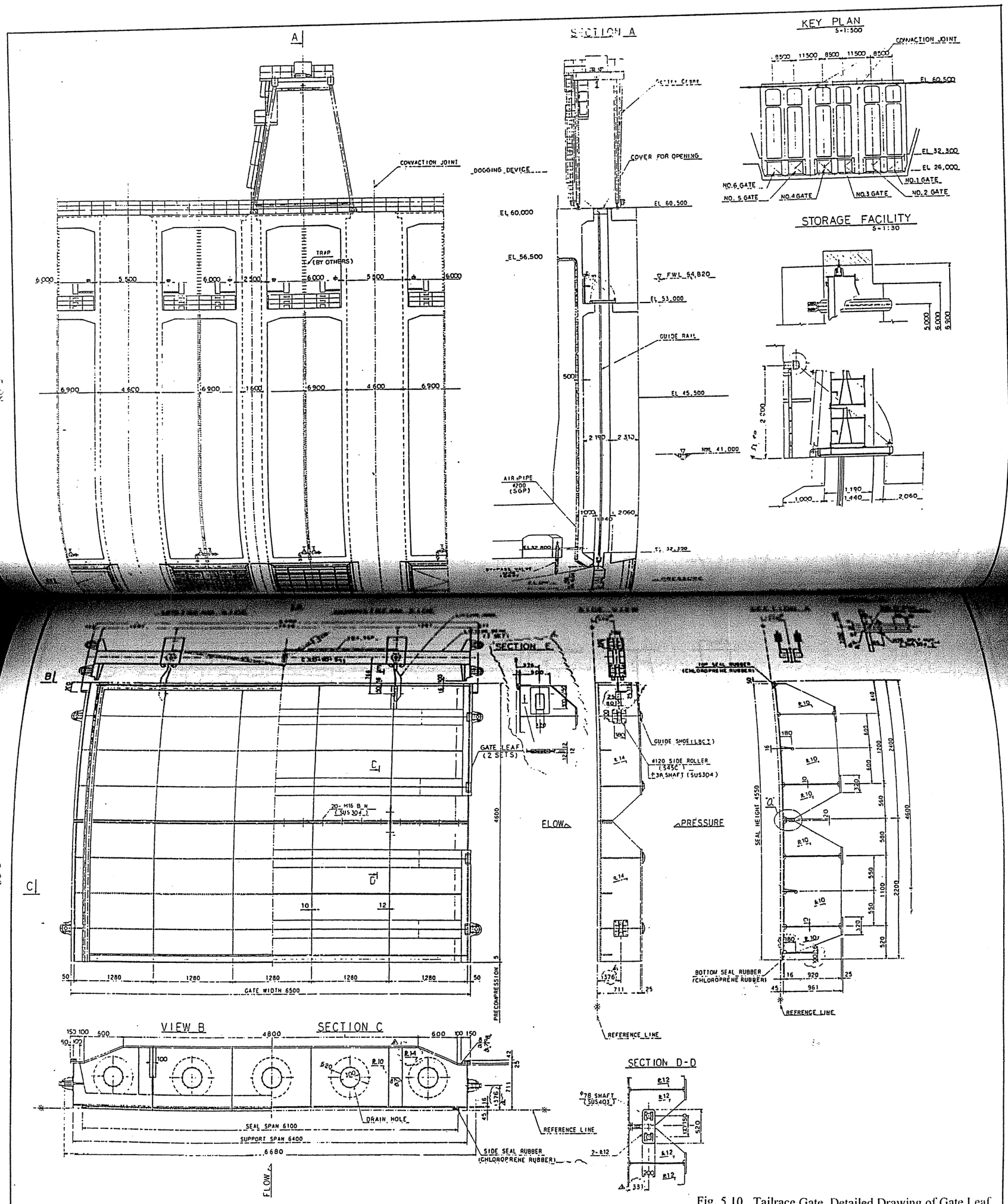
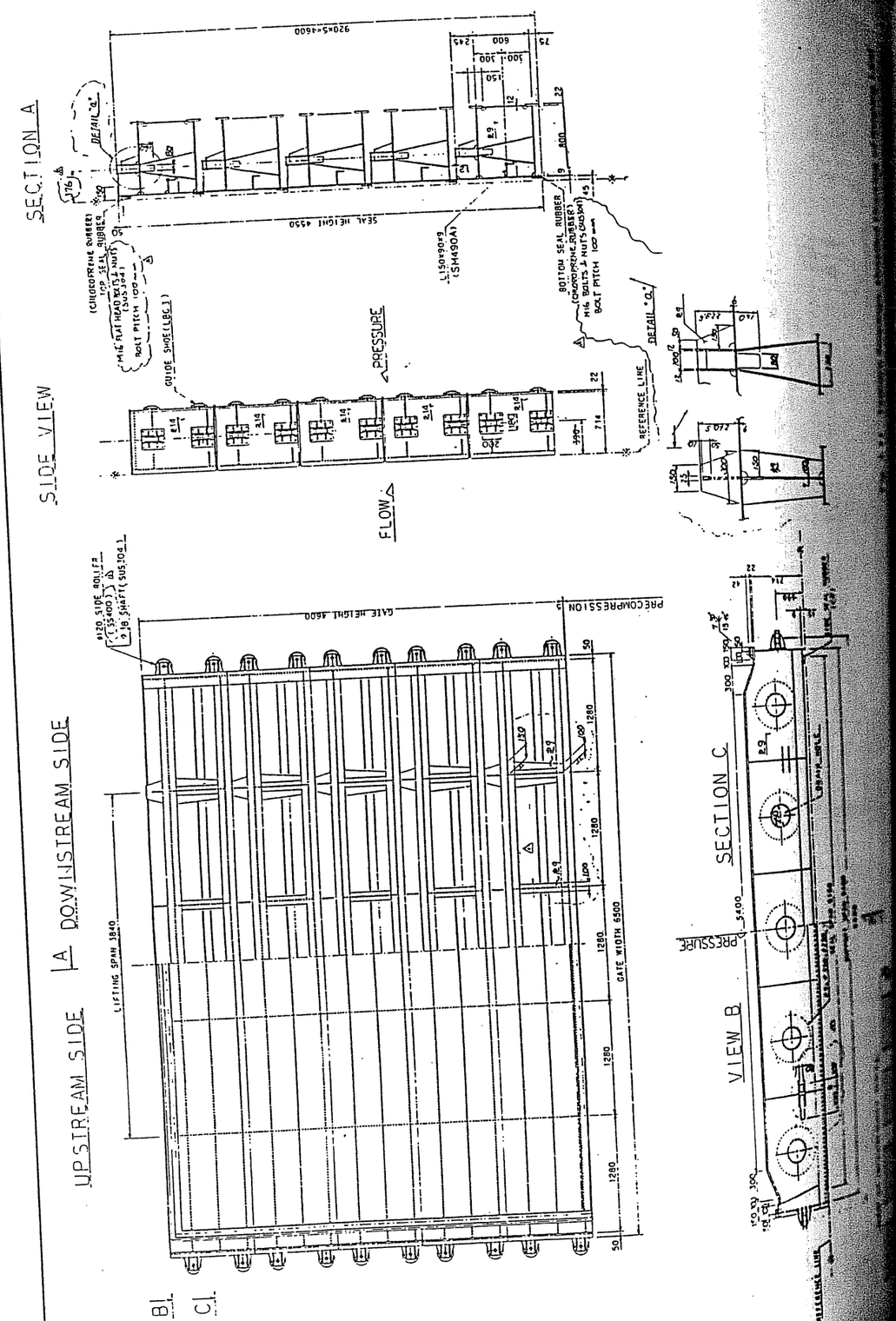


Fig. 5.10 Tailrace Gate, Detailed Drawing of Gate Leaf



5-100

Fig. 5.12 Flow Chart for Manufacturing and Installation of Guide Frame

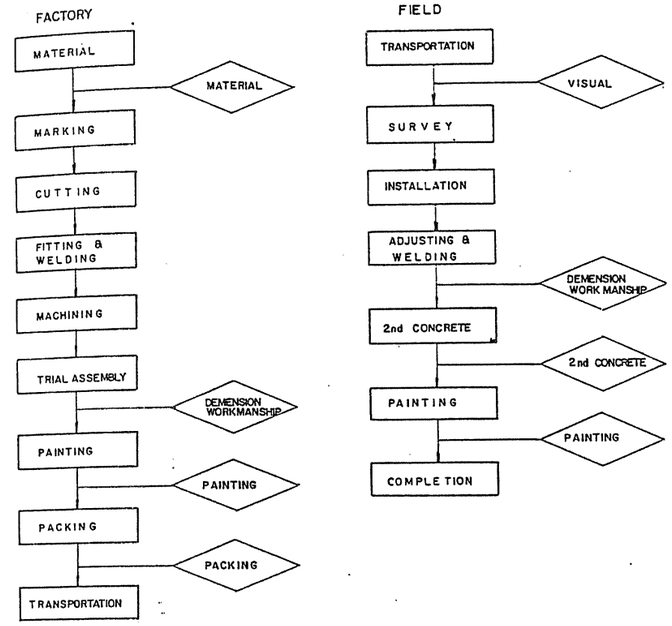
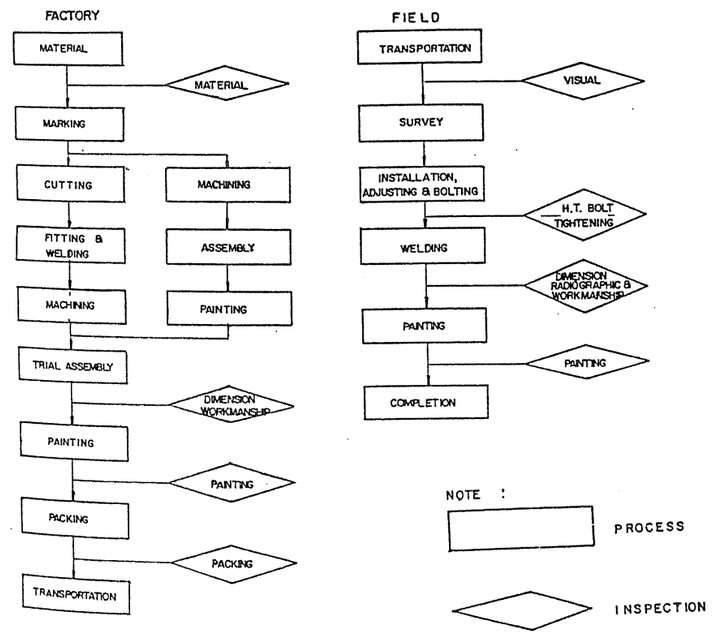
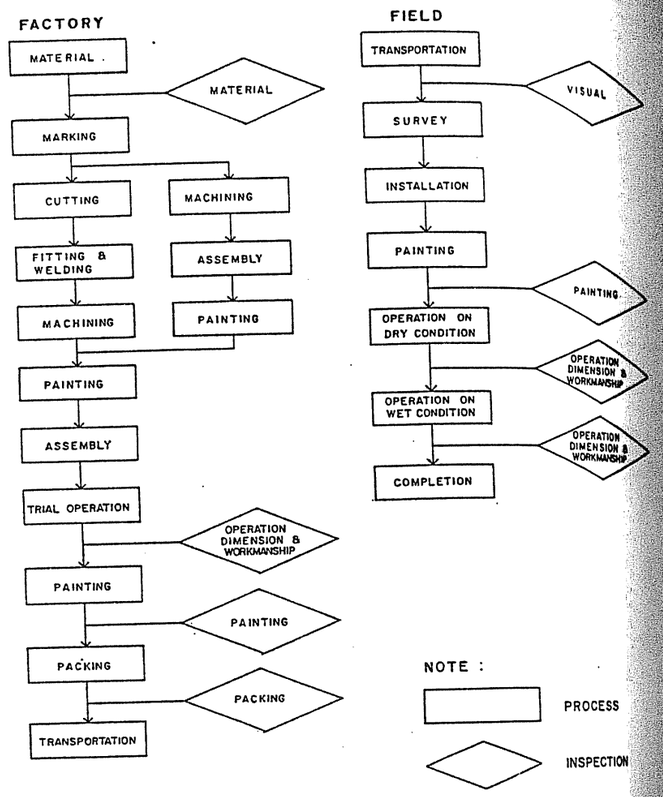


Fig. 5.13 Flow Chart for Manufacturing and Installation of Gate Leaves



5-101

Fig. 5.14 Flow Chart for Manufacturing and Installation of Hoist



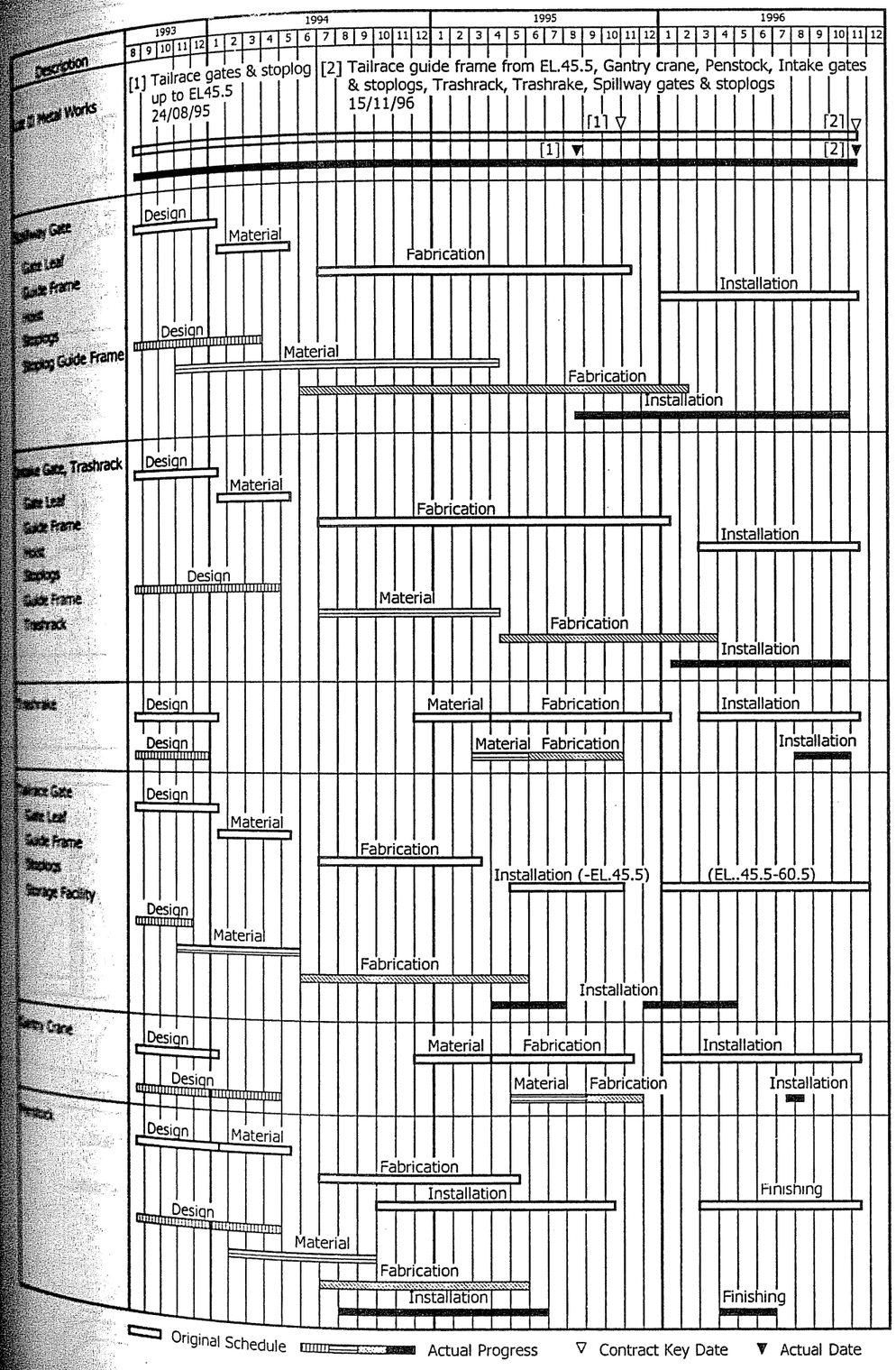


Fig. 5.15 Original Schedule and Actual Progress

Fig. 5.16 Spillway Gate Installation Procedure

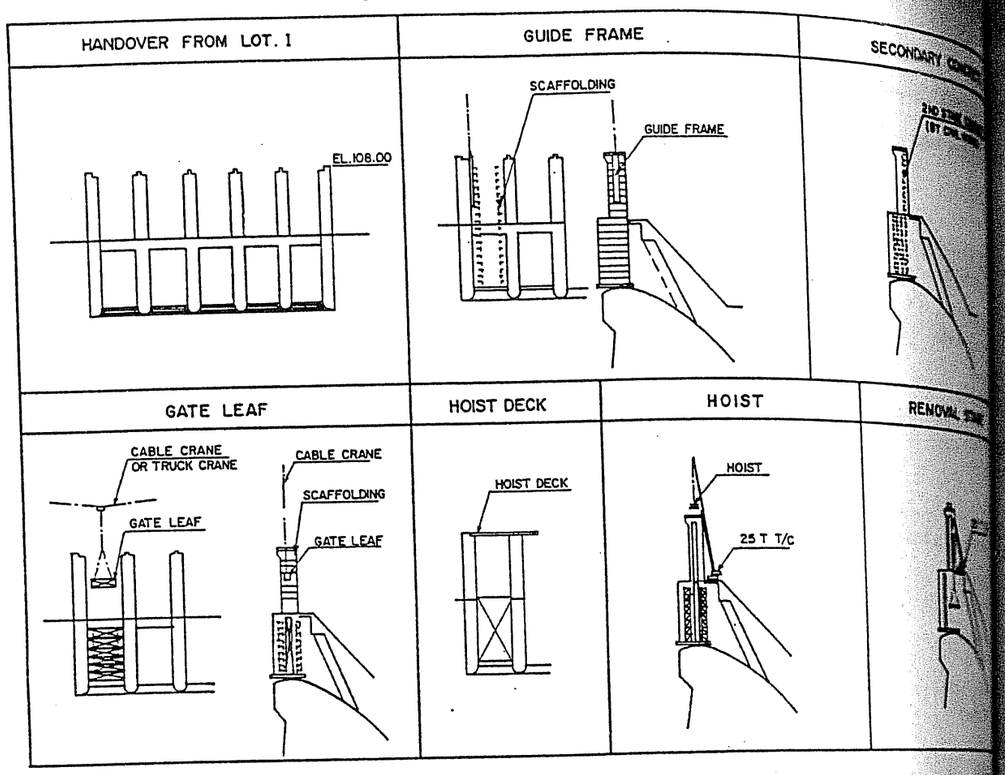


Fig. 5.17 Intake Section Installation Procedure

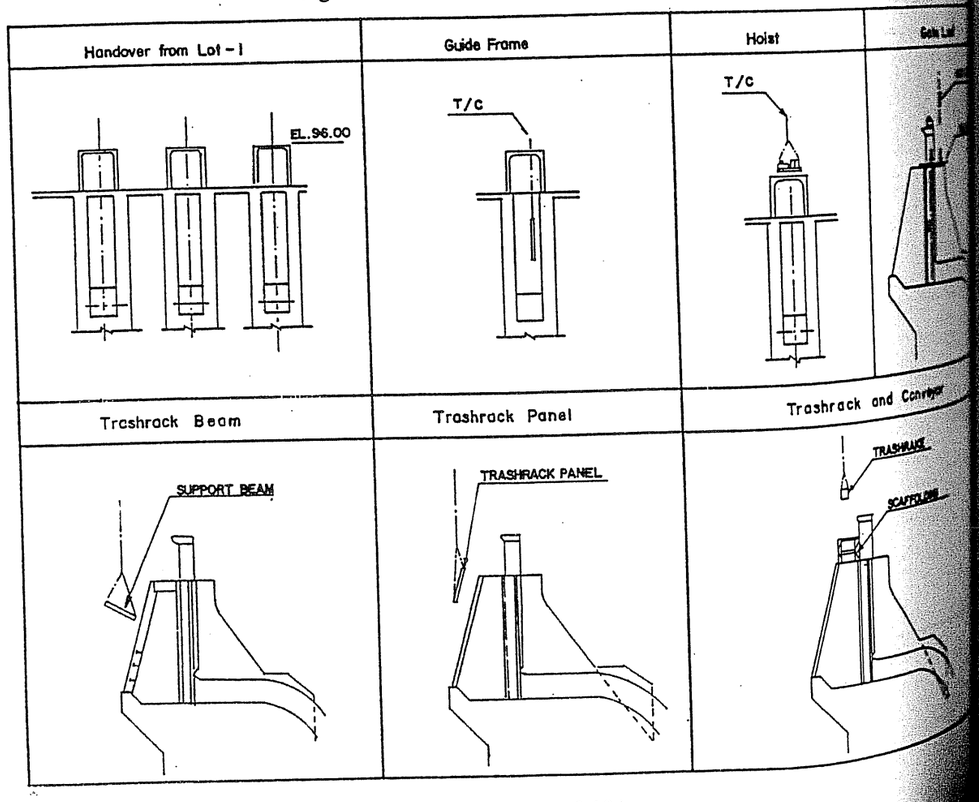


Fig. 5.18 Tailrace Gate Installation Procedure

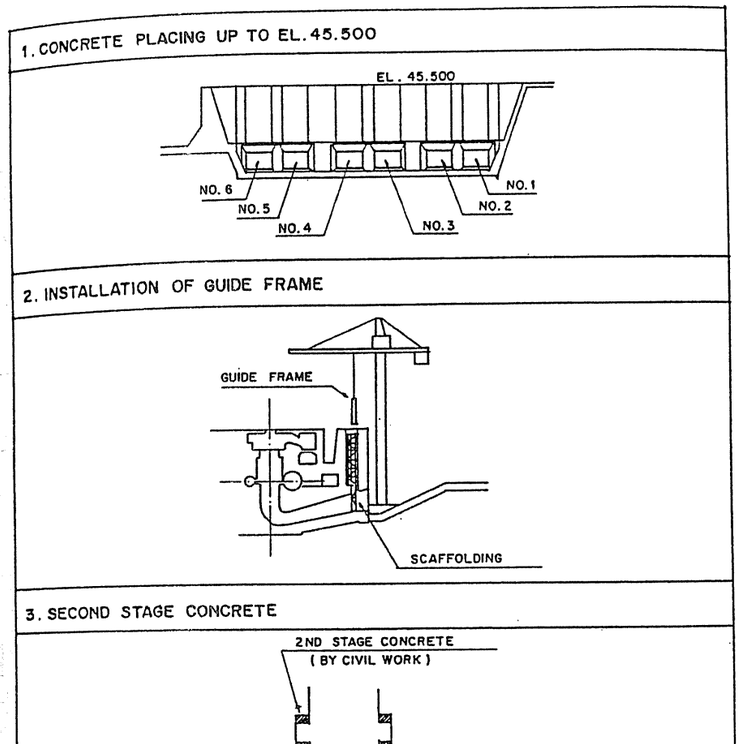
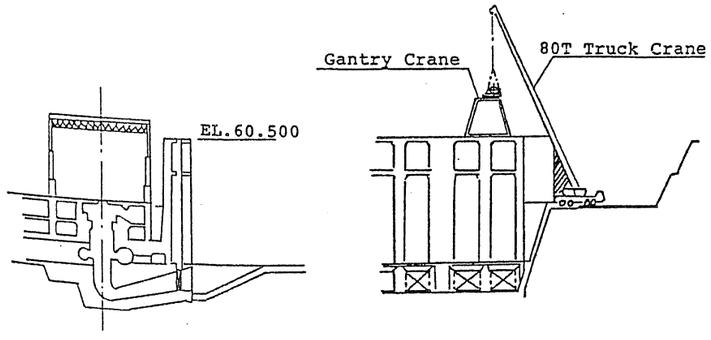
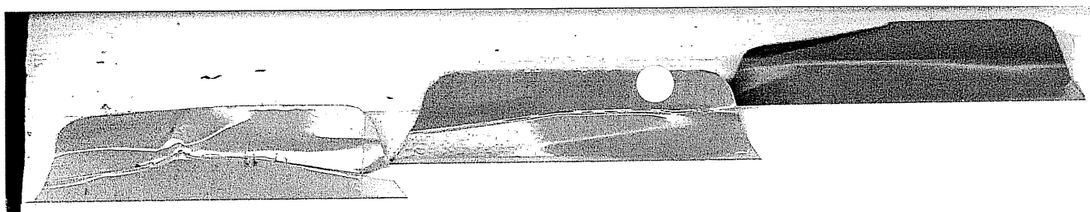
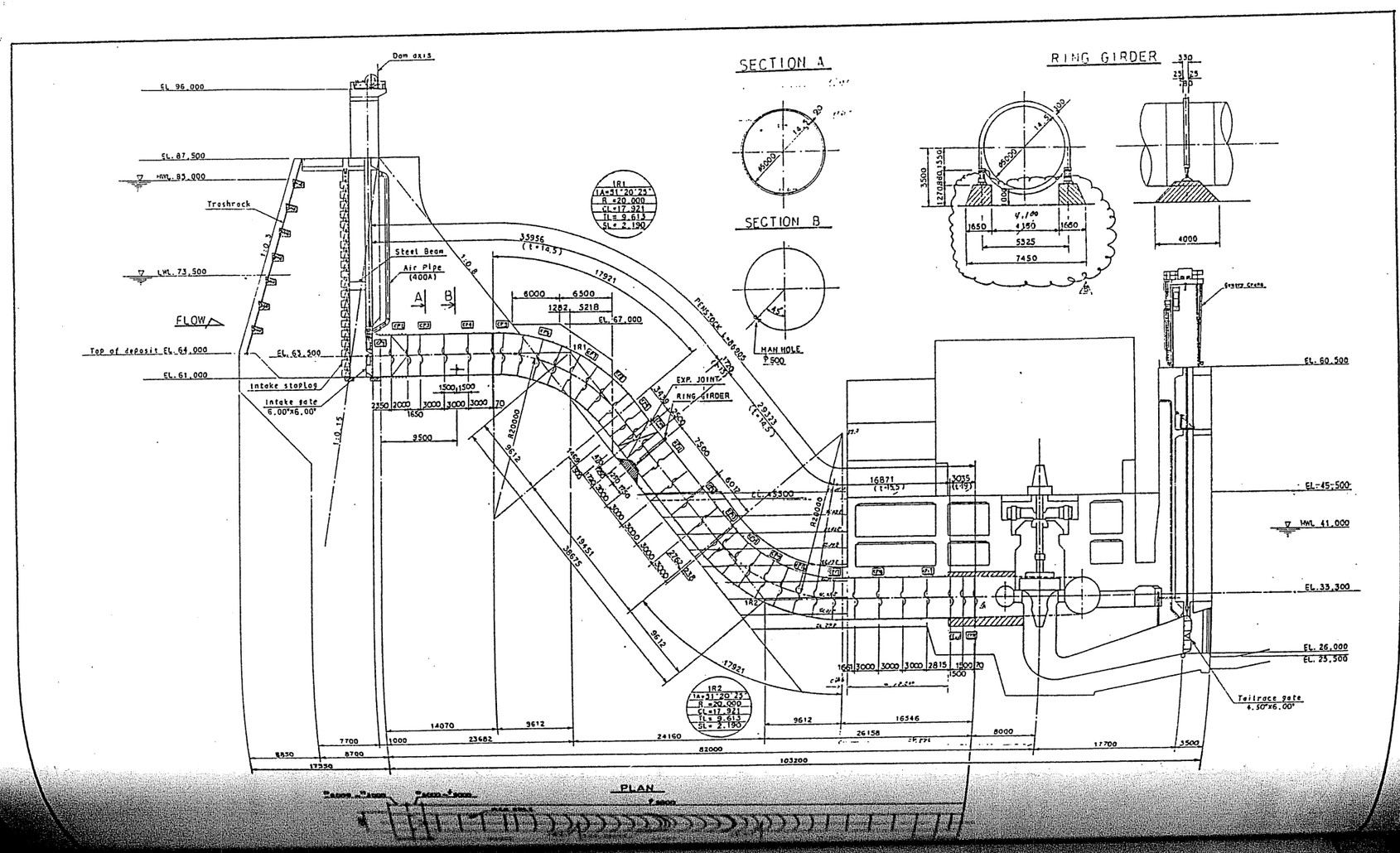


Fig. 5.19 Gantry Crane Installation Procedure







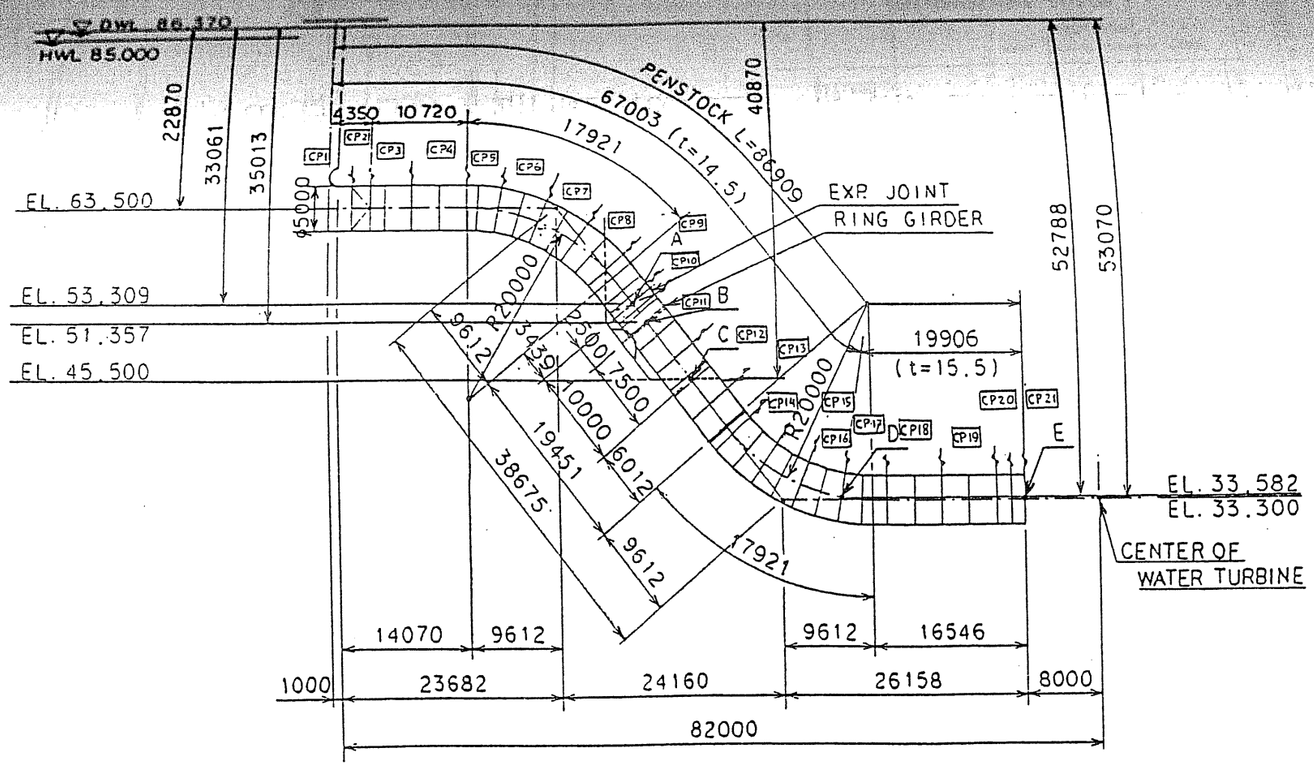


Fig. 5.21 Arrangement of Penstock

Fig. 5.22 Design Head Diagram

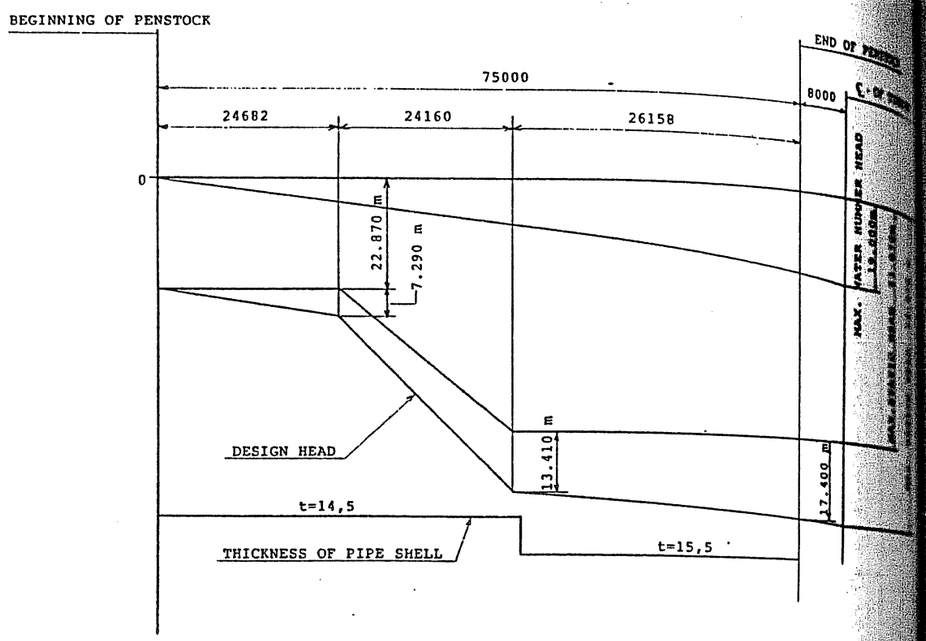
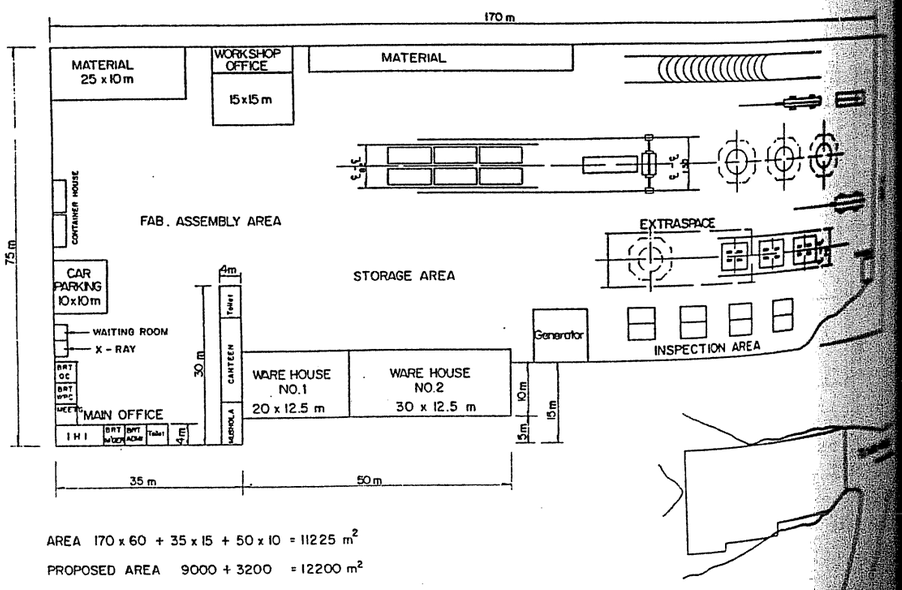


Fig. 5.23 Metal Works (Lot II) Camp & Workshop Area



AREA 170 x 60 + 35 x 15 + 50 x 10 = 11225 m^2

PROPOSED AREA 9000 + 3200 = 12200 m^2

Fig. 5.24 Flow Chart of Penstock Unit Pipe

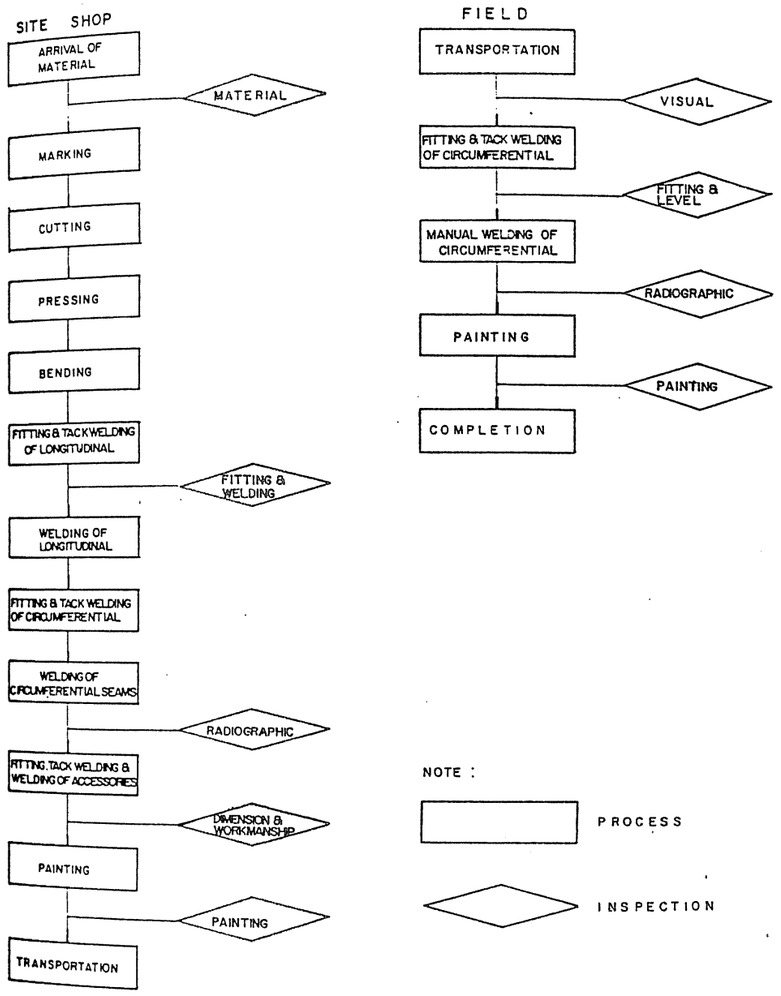


Fig. 5.25 Fabrication Method of Penstock

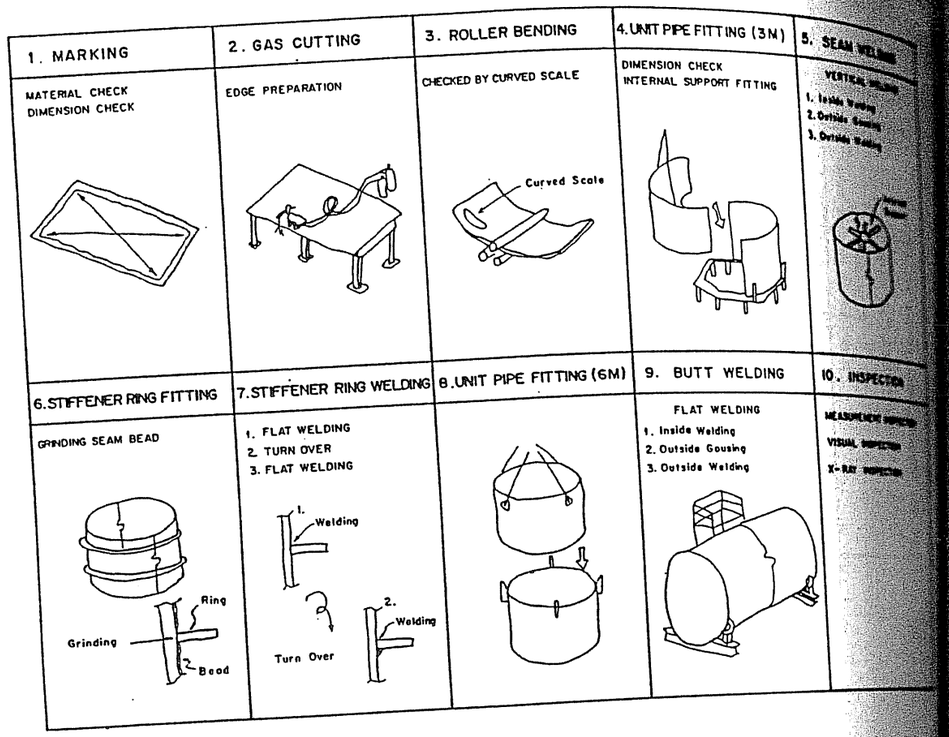


Fig. 5.26 Transportation of Penstock Activities

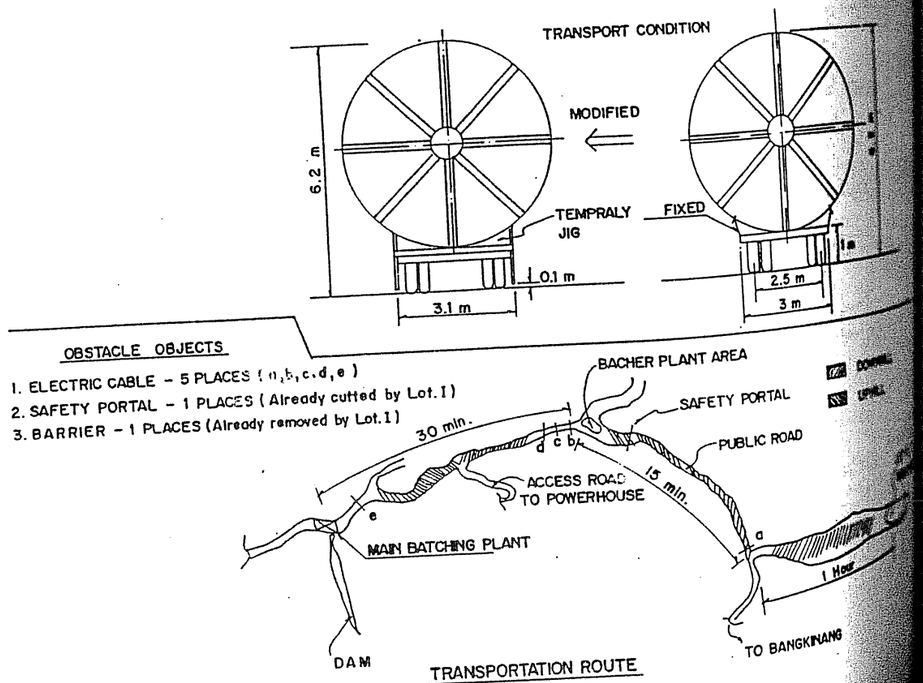
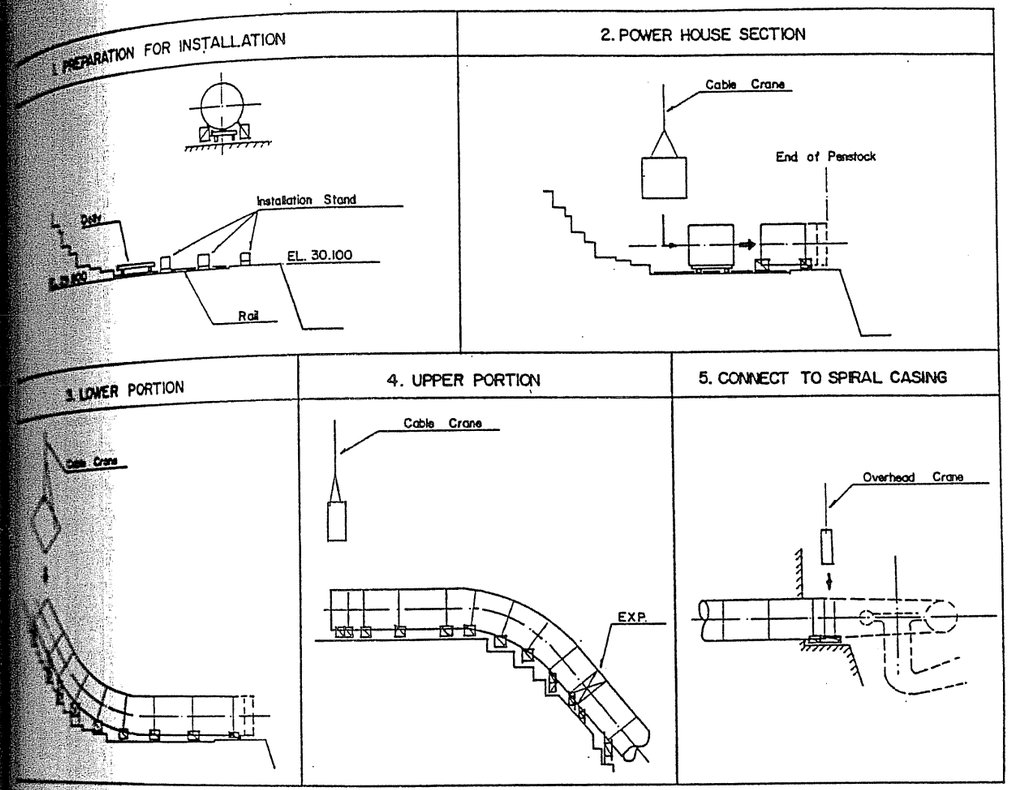


Fig. 5.27 Installation Method of Penstock



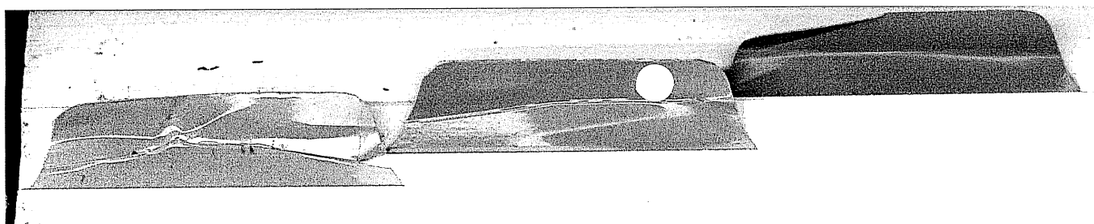


Fig. 5.28 Erection Works Schedule

NO. ITEM	DESCRIPTION	TOTAL QTY	TOTAL WEIGHT (T)	TOTAL INDEX	1994 M	J	J	A	S	O	N	D	1995 J	F	M	A	M	J	J	A	S	O	N	D	1996 J	F	M	A	M	J	J	A	S	O	N
1.	SPILLWAY GATES :			26.19	PREPARATION																														
1.1	GATE LEAF	3	383.723	16.93																					8.07	1.78	1.78	1.78	1.76	1.76					
1.2	GUIDE FRAME	5	60.365	7.74													1.44	0.63	1.26	1.26	1.26	0.63	1.26													
1.3	HOIST & ELECT.EQP.	5	75.711	3.52																									1.47	0.52	0.51	0.51	0.51		
2.	SPILLWAY STOPLOG :			5.27																															
2.1	STOPLOG (12 BLOCKS)	1	88.453	3.80																			2.10	0.85	0.85										
2.2	GUIDE FRAME	5	16.909	1.47													0.41	0.16	0.30	0.30	0.30														
3.	INTAKE GATES :			7.61																															
3.1	GATE LEAF	3	77.649	3.28																											1.77	0.38	0.75	0.38		
3.2	G. FRAME & ST. BEAM	3	28.067	2.69																					0.84	0.69	0.69	0.67							
3.3	HOIST &ELECT. EQP.	3	39.909	1.64																									0.84	0.40	0.40				
4.	INTAKE STOPLOG			4.00																															
4.1	STOPLOG (20 BLOCKS)	1	62.14	2.43																									1.43	0.25	0.50	0.25			
4.2	GUIDE FRAME	3	16.341	1.57																		0.37	0.60	0.60											
5.	TRASHRACK :	1	134.08	6.84																		1.54	0.94	0.94		1.54	0.94	0.94							
6.	TRASHRAKE :	1	20.531	0.57																									0.19	0.19	0.19				
7.	TAILRACE GATES :			2.30																															
7.1	GATE LEAF & LFT.B.	2	23.441	1.14														0.72	0.42																
7.2	GUIDE FRAME	6	21.330	1.16														0.67				0.25	0.24												
8.	TAILRACE STOPLOG :			3.23																															
8.1	STOPLOG (5 BLOCKS)	4	48.494	2.46														0.81	0.83	0.82															
8.2	STORAGE FACILITY	6	14.720	0.77														0.31					0.23	0.23											
9.	GANTRY CRANE :	1	18.780	0.53																							0.14	0.13	0.13	0.13					
10.	PENSTOCK :	3	558.132	39.96														36.6	0.49	0.48	0.48	0.48							0.48	0.47	0.47				
11.	INSTALLATION STAND	1	67.845	1.50														1.39												0.07	0 04				
---- PLANNING :	MONTHLY							3.49	4.48	4.70	4.70	4.70	4.69	4.69	4.69	4.87	0.20	1.15	2.53	2.66	2.04	2.04	2.54	5.36	1.07	3.32	4.15	3.52	5.62	3.54	5.44	1.95	1.14		
	ACCUMUL.		1,756.620	100.00					7.78	12.67	17.37	22.07	26.76	31.45	36.14	41.01	41.21	42.96	44.89	44.75	49.79	51.83	54.37	59.75	78.20	74.64	78.79	82.31	87.93	91.47	96.91	98.86	100.0		
_____ ACTUAL :	MONTHLY				0.74	0.83	5.41	8.17	6.97	2.06	1.80	2.05	3.92	2.57	2.10	3.35	4.17	2.88	3.09	1.84	1.62	1.05	2.78	5.09	3.90	1.06	2.65	7.39	2.59	8.04	5.06	20.00	1.80		
	ACCUMUL.				1.57	2.39	7.80	15.98	22.95	25.01	26.81	28.86	32.78	35.35	37.45	40.80	44.97	47.85	50.94	52.78	54.40	55.45	58.23	63.32	67.22	68.28	70.93	78.32	80.91	88.95	94.01	114.01	115.81		

Fig. 5.29 Monthly Progress Payment

NO. ITEM	DESCRIPTION	QTY	WEIGHT (TON)	INDEX TR.	INS.	TOC	1994 M	J	J	A	S	O	N	D	1995 J	F	M	A	M	J	J	A	S	O	N	D	1996 J	F	M	A	M	J	J	A	S	O	N
1.	SPILLWAY GATES						PREPARATION																														
1.1	GATE LEAF	3	383.723	5.68	6.20	1.69																			5.65	5	1.25	1.25	1.23	1.22						
1.2	GUIDE FRAME	5	60.365	1.01	4.41	0.77													1.01	0.45	0.88	0.88	0.88	0.44	0.88												
1.3	HOIST & ELECT.EQP.	5	75.711	1.03	1.43	0.35																							1.03	0.36	0.36	0.36	0.35		
2.	SPILLWAY STOPLOG :																																			
2.1	STOPLOG (12 BLOCKS)	1	88.453	1.47	1.19	0.38																			1.47	0.60	0.59								
2.2	GUIDE FRAME	5	16.909	0.29	0.74	0.15													0.29	0.11	0.21	0.21	0.21												
3.	INTAKE GATES :																																			
3.1	GATE LEAF	3	77.649	1.24	1.06	0.33																									1.24	0.29	0.40	0.39		
3.2	G. FRAME & ST. BEAM	3	28.067	0.45	1.43	0.27																			0.45	0.49	0.49	0.49							
3.3	HOIST &ELECT. EQP.	3	39.909	0.59	0.56	0.16																							0.59	0.28	0.28				
4.	INTAKE STOPLOG																																			
4.1	STOPLOG (20 BLOCKS)	1	62.140	1.00	0.70	0.24																												1.00	0.18	0.85	0.17
4.2	GUIDE FRAME	3	16.341	0.26	0.84	0.16																			0.26	0.42	0.42								
5.	TRASHRACK :	1	134.080	2.16	2.63	0.68																			1.08	0.66	0.66		1.08	0.66	0.65				
6.	TRASHRAKE :	1	20.531	–	0.40	0.08																										0.14	0.13	0.13		
7.	TAILRACE GATES :																																			
7.1	GATE LEAF & LFT.B.	2	23.441	0.38	0.42	0.11													0.50	0.30																
7.2	GUIDE FRAME	6	21.330	0.34	0.47	0.12													0.47						0.17	0.17									
8.	TAILRACE STOPLOG :																																			
8.1	STOPLOG (5 BLOCKS)	4	48.494	0.78	0.94	0.25													0.57	0.58	0.57															
8.2	STORAGE FACILITY	6	14.720	0.24	0.30	0.08													0.22						0.16	0.16									
9.	GANTRY CRANE :	1	18.780	–	0.37	0.05																						0.10	0.09	0.09	0.09				
10.	PENSTOCK :	3	558.132	–	27.97	4.00													25.63	0.34	0.34	0.33	0.33									0.34	0.33	0.33		
11.	INSTALLATION STAND	1	67.845	–	1.05	0.15													0.97														0.05	0.03		
---- PLANNING :	MONTHLY									2.44	3.14	3.29	3.29	3.29	3.29	3.28	8.41	0.14	0.80	1.77	2.01	1.42	1.43	1.78	3.77	8.09	2.33	2.90	2.47	3.93	2.48	3.81	1.36	0.80	
	ACCUMUL.		1,756.620	70.00		10.00					5.58	8.87	12.16	15.45	18.73	22.02	25.30	28.71	28.85	29.65	31.42	33.43	34.85	36.28	36.06	41.83	49.92	52.25	55.15	57.62	61.55	64.03	67.84	69.20	70.0	
_____ ACTUAL :	MONTHLY						0.52	0.58	1.69	5.72	4.87	1.40	1.12	1.42	2.11	1.80	1.53	2.35	2.92	1.99	1.47	1.07	1.28	1.13	1.95	3.55	3.92	0.76	2.00	5.17	1.88	5.63	3.96	3.43	1.12	
	ACCUMUL.											14.76		17.32		21.23		25.11		30.02		32.56		34.97		40.47		45.15		52.32		59.83		66.82		

D : Sumo2 2 P : Fig 29

PART 6

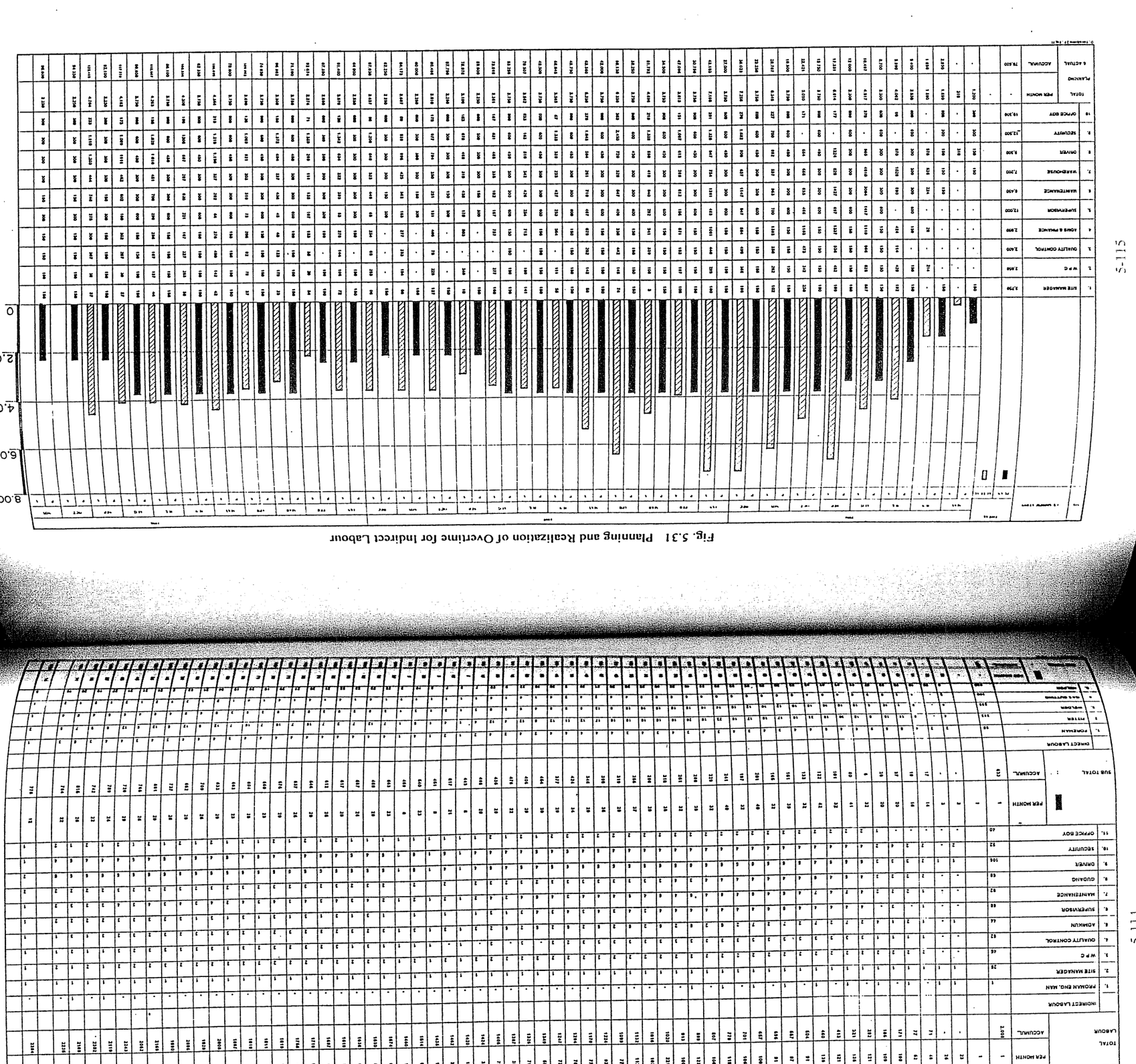


Fig. 5.30 Planning and Actual Utilization of Direct & Indirect Labour

Fig. 5.31 Planning and Realization of Overtime for Indirect Labour

Fig. 5.32 Planning and Realization of Overtime for Direct Labour

