THE GOVERNMENT OF MAURITIUS
MINISTRY OF ENERGY, WATER RESOURCES AND POSTAL SERVICES
CENTRAL WATER AUTHORITY

THE DETAILED DESIGN ON THE PORT LOUIS WATER SUPPLY PROJECT IN MAURITIUS

FINAL REPORT (1)

DESIGN REPORT FOR LOT-I

MARCH 1991

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

In response to a request from the Government of Mauritius, the Japanese Government decided to conduct a Detailed Design on the Port Louis Water Supply Project and entrusted the works to the Japan International Cooperation Agency (JICA).

JICA sent to Mauritius a study team headed by Mr. Norizo Fujita of Nippon Koei Co.,Ltd, composed of members from Nippon Koei Co.,Ltd. and Nihon Suido Consultants Co.,Ltd. from May to November,1990.

The team held discussions with concerned officials of the Government of Mauritius, and conducted field surveys. After the team returned to japan , further studies were made and the present report was prepared.

I hope that this report will contribute to the promotion of the project and to the enhancement of friendly relations between our two countries.

I wish to express my sincere appreciation to the officials concerned of the Government of Mauritius for their close cooperation extended to the team.

March,1991

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation Agency

March,1991

Mr.Yanagiya Kensuke
President
Japan International
Cooperation Agency
Tokyo

Dear Sir,

LETTER OF TRANSMITTAL

We have the pleasure of submitting to you the Final Report (1) of the Detailed Design on Port Louis Water Supply Project in Mauritius prepared for the implementation of Lot-I work in the Project.

This report is composed of eight (8) volumes consisting of the Summary Report, Design Report for Lot-I, Tender Document (Four (4) Volumes), Cost Estimate for Lot-I and Data Book. The Summary Report summarizes outlines of the detailed design carried out for Lot-I. The Design Report for Lot-I contains the results of the detailed design on Lot-I components. The Tender Documents are composed of (i) Vol.I presenting the instructions to tenderers, conditions of contract, various forms for bonds and agreement,etc.,(ii) Vol.II presenting the general and technical specifications, (iii) Vol.III presenting the form of tender, bill of quantities and schedules of particulars, and (iv) Vol.IV presenting the tender drawings. The Cost Estimate for Lot I contains the unit cost analysis and construction cost estimate for each work item included in Lot-I. The Data Book contains the detailed design calculations, work quantities calculations and field investigation data,etc.

All members of the Study Team wish to express grateful acknowledgement to the personnel of the Advisory Committee, Ministry of Foreign Affairs, Embassy of Japan in Madagascar as well as officials and individuals of Mauritius for their assistance extended to the Study Team.

In conclusion, the Study Team sincerely hopes that the study results would contribute to the further water resource development for water supply to Port Louis and to socio-economic development and well-being in general.

Truly yours,

N. Fujita

Norizo Fujita
Team Leader
The Detailed Design on
Port Louis Water Supply
Project

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CHAPTER I. INTRODUCTION

1.1 Project Location and Description

The State of Mauritius is located at about 900 km to the east of the Madagascar in the Indian Ocean.

The State of Mauritius comprises the islands of Mauritius, Rodrigues, Agalega, and St. Brandon. The total area is 2,040.0 km^2 of which Mauritius island accounts about 91.4% or 1,864.8 km^2 and Rodrigues 5.1% or 104.0 km^2. The island of Mauritius comprise 5 municipal areas and 98 village council areas. Port Louis City, the capital of Mauritius, is one of the five municipalities, and is located at northwestern part of the Mauritius island.

The Grand River North West (GRNW) which is one of the largest rivers in Mauritius originates in the central plateau, runs north-westward and flows into the Indian Ocean in the South of Port Louis City. The water resources for Port Louis Water Supply Project are intended to be developed from the surface water of the basin, and therefore, the project area covers the whole basin of GRNW and Port Louis City. The GRNW basin with a catchment area of 113.21 km^2 (at Municipal Dike) consists of the central plateau of which elevation is more than 300 m and gorge area which was formed by river channel erosion.

The upstream reaches of GRNW are composed of several tributaries named as the Moka river, Profonde river, Cascade river, Terre Rouge river and Plaines Wilhems river. The proposed damsite is located at the downstream of the Terre Rouge river, just upstream of the confluence with the Plaines Wilhems river.

The water to be released from the dam is planned to be taken at the existing Municipal Dike and transmitted to the existing Pailles Treatment Plant located at about 4.0 km downstream of the dam, from which the water will be distributed to Port Louis City.

The general location map of the project area is shown in Fig. 1.1.

1.2 Project Background

The population of Mauritius is about one million and forty-two (42) percent of the population is concentrated in Port Louis City and the neighboring satellite cities. As such, Port Louis City plays a very important role as not only the capital city of Mauritius but also the base of commerce and industry in Mauritius.

The present municipal and industrial water supply is made by utilizing the water resources of GRNW basin. Major water supply facilities are the intake weir called the Municipal Dike, pipelines from the Municipal Dike, Pailles Treatment plant and water distribution system from the Pailles Treatment Plant.

However, the above present water supply system involves various problems. Major problems in the present system are as follows:

(1) The water delivery system is already very old and thereby, water loss due to leakage in the system is remarkable, reaching about 45% of the water volume treated.

(2) The river run-off has a large seasonal fluctuation. On the other hand, the present system has no storage function to regulate the seasonal fluctuation, which causes a severe water shortage in the dry season from July to November every year.

(3) The present water treatment plant is not provided with a sufficient capacity to treat the muddy water. Then, the water supply is frequently forced to be stopped during floodings.

Such being the situation, the Government of Mauritius requested a technical assistance from the Government of Japan for a study on the project to ensure a stable water supply to Port Louis City. The government of Japan, in response to the request of the Government of Mauritius, agreed to make a Feasibility Study on the Port Louis Water Supply Project, and JICA (Japan International Cooperation Agency), the official agency responsible for the implementation of the technical cooperation programmes of the Government of Japan, was appointed to undertake the Feasibility Study in close cooperation with the authorities concerned in Mauritius.

The Feasibility Study was conducted by the JICA Study Team during the period from March, 1988 to July 1989. The Feasibility Study revealed that the project be feasible technically and economically, and recommended to implement the project as early as possible.

In accordance with the recommendation of the Feasibility Study, the Government of Mauritius decided to conduct the Detailed Design of the project, and requested its technical and financial assistance to the Government of Japan.

Thus, the Detailed Design of the project was commenced by the JICA Study Team from March 1990. Since then, the review of the Feasibility Study, various additional field investigations and basic design, etc. of which results are summarized in the Basic Design Report prepared in October, 1990.

Following the Basic Design, the detailed design works for Lot-I have been executed. This Design Report summarizes the results of detailed design carried out for Lot-I.

1.3 Objective of the Project

The objective of the project is to develop the most suitable and economical scheme to improve the water supply system of Port Louis City by harnessing the water resources in the basin of GRNW in order to meet the water demand of Port Louis City at the medium term up to year 2010 and long term up to year 2030.

1.4 Objective of the Detailed Design

The objectives of the Detailed Design are as follows:

(1) To prepare the detailed design with drawings for the structures relevant to the Project including dam and its related facilities, transmission pipeline and treatment plants as well as detailed construction schedule and cost estimate,

(2) To prepare the detailed implementation schedule of the Project,

(3) To prepare the tender documents including general and detailed technical specification for various works as well as the contract document, and

(4) To extend transfer of knowledge to the Mauritius counterpart through the Study.

1.5 Division of Lots and Components in Lot-I

Aiming to facilitate and efficiently implement the Project works, the Project works are divided into the following three (3) Lots:

- Lot-I : Preparatory works and diversion tunnel
- Lot-II : Dam and related facilities, and repair work of the existing Municipal Dike
- Lot-III : Intake at the existing Municipal Dike, water transmission pipeline and treatment facilities

Major components to be included in Lot-I are as follows:

- Diversion tunnel (without the plug work and pipes and valves installation which will be done by Lot-II)
- Diversion gate
- Preparatory works such as the haul road from quarry site to damsite, main access roads in damsite, access road from the water treatment plant to the existing Municipal Dike, buildings including offices, quarters, utility buildings, repair shop and laboratory, etc., water and electric power supply systems, and aggregate and concrete plants, etc.

1.6 Organization

CWA (Central Water Authority) under the Ministry of Energy, Water Resources and Postal Services will be the executing agency of the Project implementation.

During the construction, the construction works will be managed and supervised by CWA with assistance of a consultant.

1.7 Principal Features of the Project

The principal features of the Project based on the basic design are as follows:

(1) Reservoir

Catchment area	54.9 km^2
Annual basin rainfall	2,400 mm
Gross storage capacity	6.7 x 10^6 m^3
Effective storage capacity	6.3 x 10^6 m^3
Flood water level	El. 193.5 m
High water level	El. 189 m
Low water level	El. 139 m
Surface area	30 ha
Mean runoff	1.8 m^3/s
Design flood	1,890 m^3/s
Return period	(PMF)

(2) Dam

Type	Rockfill
Crest elevation	El. 196 m
Height (from riverbed)	80 m
Crest length	230 m
Embankment volume	1,515 x 10^3 m^3

(3) Spillway

Type	Side channel
Crest elevation of weir	El. 189 m
Width of weir	90 m
Discharge	1,890 m^3/s

(4) River Diversion

Type	Tunnel diversion

(4) River Diversion

Type	Tunnel diversion
Design flood	520 m^3/s
Return period	(20 years)
Discharge in tunnel	520 m^3/s
Number of tunnel	1
Diameter	6.8 m
Tunnel length	499 m
Gate type	Sluice gate

(5) Intake

Type	Selectable intake gate
Discharge	1 m^3/s
Number of gates	3
Dimension of gate	2,100 mm x 2,100 mm
Gate type	Fixed wheel gate

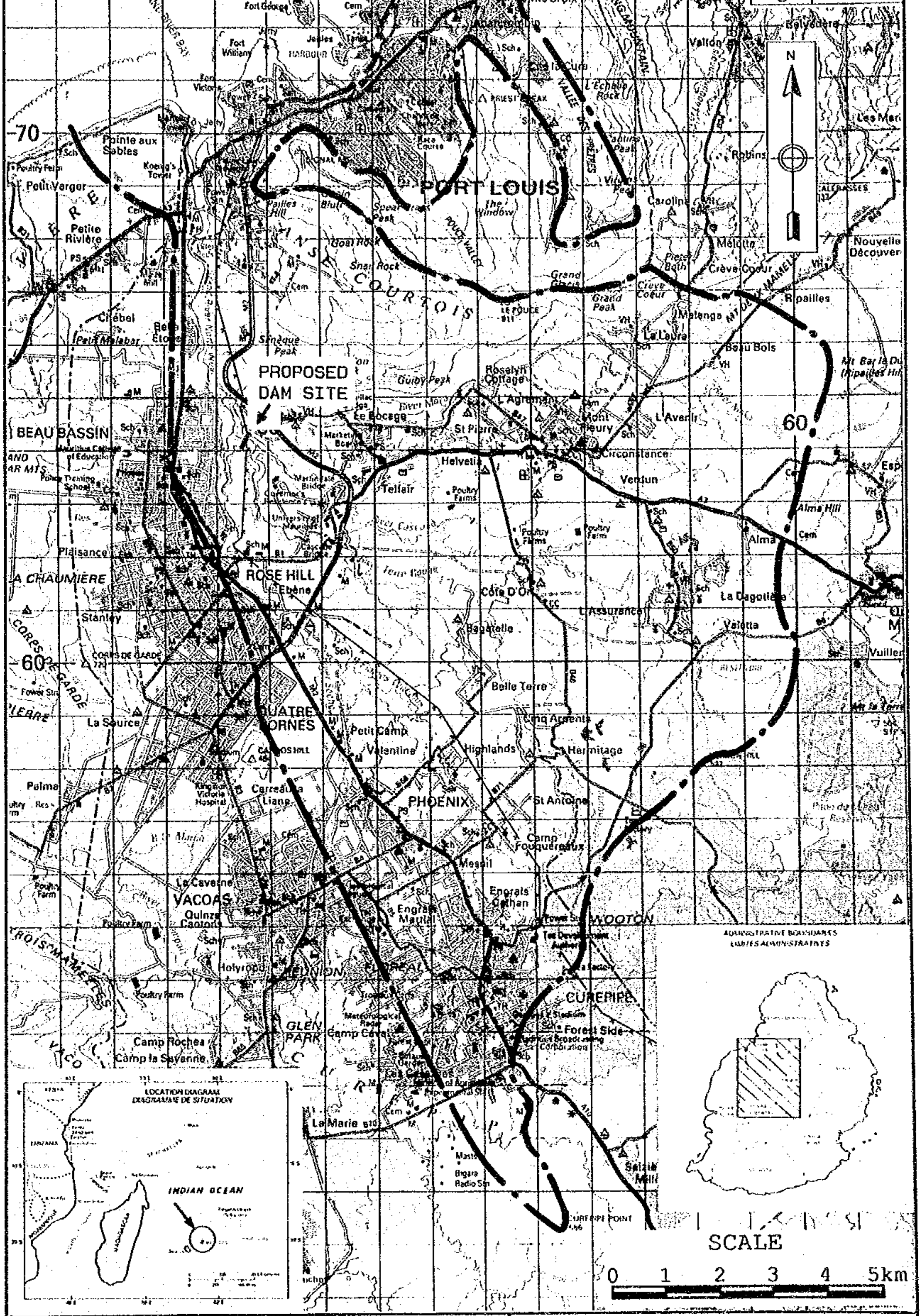
(6) New Transmission Pipeline

Design discharge	660 lit/s
Number of pipeline	1
Diameter	800 mm
Length of pipeline	2,100 m

(7) New Treatment Plant

Type	Rapid sand filtration
Capacity	30,000 m^3/day (First stage)

Fig. 1.1



GENERAL LOCATION MAP OF PROJECT AREA

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

CHAPTER II. SITE CONDITION

2.1 Socio-Economy

According to the population census carried out in 1983, the total population of the country was 1,000,432. The population of Mauritius island was 966,863 or 96.6% of the country's population and population of Rodrigues was 33,082 or 3.3% of the total. Population density of the country was high at 507 persons/km^2 overall and, for Mauritius and Rodrigues islands, 535 persons/km^2 and 339 persons/km^2, respectively.

During the inter-census period of 1972 - 83, the population in Mauritius island grew at an annual average rate of 1.43%. The growth rate in Rodrigues was 2.67% per annum. In Mauritius island, 403,251 people or 41.7% of the total lived in the urban area of five municipalities and the rest in rural areas or villages. Based on the past trends, the total population of the country as of 1988 was estimated at around 1.1 million.

According to 1983 population census, the population of Port Louis City was accounted at 133,702 which corresponded to 33.2% of urban population or 13.8% of the population of Mauritius island. The population density was 3,131 persons/km^2 which is about 6 times higher than that of Mauritius island. The number of households was 29,187 with an average household size of 4.58 persons. The population of Port Louis grew at an annual average rate of 0.97% during 1983 - 85 period.

The gross domestic product (GDP) of Mauritius in 1987 was estimated at Rs. 18,020 million at the current factor cost. Per capita GDP is about Rs. 18,600. Value-added of the manufacturing sector is the biggest with 4,530 million or 25.1% of GDP. Agriculture, hunting, forestry and fishing come next with Rs. 2,495 million or 13.8% of GDP.

2.2 Water Supply Condition

The present Port Louis water supply system consists of the water intake (Municipal Dike), Pailles Treatment Works, three transmission mains to the city, 10 service reservoirs, two booster pumping stations at Plaine Lauzun and Pailles and distribution pipes with a range of diameter 150 mm to 800 mm. The main source of supply is the intake at Municipal Dike on GRNW. In 1980, two additional sources were developed. The Pailles filter beds with slow sand filter beds were first constructed in 1926 and have been expanded in capacity in 1960 and 1981, now having a total filter area of 10,062 m^2. Ten treated water service reservoirs have a combined capacity of 61,000 m^3.

The present distribution system serves an area of approximately 3,900 hectares, equal to 91% of the total area.

The above present water supply system includes various problems. Major problems are (i) no storage function to regulate the seasonal variation of river run-off which causes a severe water shortage in the dry seasons, (ii) the superannuated facilities which cause a remarkable leakage, and (iii) shortage of water treatment capacity, etc., as mentioned in Section 1.2.

According to the flow records for GRNW for 1965 - 1983 period, long-term mean monthly flow is low for the months from July to December. Rainfall and discharge is normally abundant during January - February period due to the visits of cyclones. The Port Louis water supply system, water source of which is mainly dependent on the flow of GRNW, has been suffering a shortage of raw water almost every year.

The most drought condition is recorded in 1983. Recently, the water shortage in 1987/88 was severe and prolonged from August 1987 to February 1988. The whole population and all economic activities in the municipality of Port Louis and its environs were badly affected. During the months of January and February, 1988, piped water was supplied to consumers for only 4 hours in a day, causing serious inconveniences to the inhabitants and disrupting the economy of the town.

2.3 Outline of the Project Area

The Study area consists of Central Plateau whose elevation is more than 300 m, and gorge area which is formed by river channel erosion of GRNW. Central Plateau inclines gently westward to north-westward and high mountain area located in the eastern and south-eastern ridge of the plateau. 66 percent of the plateau is utilized as permanent sugar cane field. Tea trees are planted in relatively high elevation area in southeastern rim of GRNW. These cultivated area is covered with thick or enough vegetation against surface soil erosion occurring in summer season when the basin has intensive rainfall. Natural forest remains in southern and north-eastern rim of the basin. The area is 11 percent of GRNW. There are some urbanized areas such as Rose Hill, Quatre Bornes, Phoenix and Curepipe, which are located along the western side of the GRNW. Proportion of land use is as follows:

	Land Use	Area (km^2)	Percentage (%)
Plateau	Sugar cane field	74.2	66
	Tea plantation	3.8	3
	Forest, etc.	10.5	9
	Urbanized area	22.6	20
Gorge	Forest/Bush	2.1	2
	Total	113.2	100

2.4 Meteo-Hydrology

(1) Precipitation

From the climatical point of view, one year is divided into two seasons. One is the summer season from November to April, and another is the winter season from May to October. 70 percent of annual total rainfall falls in the summer season. Driest month is October when this basin has only 3.5 percent of annual total rainfall on an average. Heaviest rainfall occurs usually in December to March which is caused by cyclone, or by front of Inter Tropical Convergence Zone. Annual average rainfall from 1,400 mm in the north to 3,200 mm in the south.

In and around the Study Area, fifty six rainfall gauging stations are in operation by CWA, Meteorological Office and some sugar estates. These data have been collected and compiled by Meteorological Service. Some of them have quite a long recording duration of about 100 years, such as Alma, Reduit Experimental Station and Vacoas. Automatic recorders are operated at Vacoas and Velle Rive. At the remaining stations, daily total rainfall is read at 8 a.m. every day. Original data in the past twenty five years are still stored in Meteorological Service. On the other hand the previous data are missing or not well preserved.

These available rainfall data are presented in Appendix A, Feasibility Study Report, 1989.

The probable basin rainfall by return year in terms of the point rainfall is analyzed as follows:

Return Year	One-day	Two-day	Three-day
2	272	398	470
5	387	551	632
10	463	661	751
20	536	765	864
50	630	901	1,021
100	701	1,003	1,140
200	771	1,114	1,260
1,000	935	1,381	1,551
10,000	1,168	1,799	1,999

The average basin rainfall by return year is analyzed as follows:

Return Year	One-day	Two-day	Three-day
10	77	168	393
20	84	195	455
100	116	257	596
200	125	291	656
10,000 (PMP)	171	536	993

Fig. 2.4.1 shows the location of rainfall stations in and around GRNW. Fig. 2.4.2 shows the duration of records at each station. The iso-hyetal map of rainfall in and around the basin is given in Fig. 2.4.3.

(2) Evaporation

Evaporation data at three meteorological stations such as Reduit experiment station, Vacoas and Velle Rive are available in/around the GRNW basin. Average annual evaporation reaches 1,694 mm. Maximum and minimum evaporation rate is 5.9 mm/day in January and 3.2 mm/day in June. Evaporation from a wide open water surface is estimated to be 70% of Class A-pan evaporation of corresponding season.

Class-A pan evaporation and Penman's estimate for 20 year average (1961 - 1980) are as follows:

(Unit: mm)

Stations	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sep.	Oct.	Total
<u>Class-A pan</u>													
Vacoas	171	179	176	155	161	133	123	107	107	122	141	164	1,739
Reduit	169	178	182	151	155	129	111	96	104	119	137	163	1,694
Belle Rive	132	142	127	113	115	105	103	89	87	89	101	123	1,326
<u>Penman's Formula</u>													
Curepipe	142	153	158	140	135	110	90	67	78	93	105	129	1,400

(3) Wind

Extreme wind in Mauritius is caused by cyclones. Data of annual highest wind speed of 30 MPH (Miles per Hour) and above over a whole hour recorded during tropical cyclones from 1876 to 1975 are plotted in Fig. 2.4.4. The recorded highest wind speed is 82 miles per hour in one hour average, which was recorded at Mon Desert during Cyclone Gervaise. As seen in Fig. 2.4.4, the recorded highest wind speed corresponds to the magnitude of 100-year return period.

(4) Discharge

Discharge of the GRNW basin is changed according to the mentioned climatical cycle. Mean annual total discharge of GRNW at Municipal Dyke (113.2 km^2) is estimated to be 88 MCM. One eighth of total discharge is presently used for water supply. The rest runs through into the sea as unused water because there is no surface water storage facilities in the basin and the flow volume is changeable and not reliable for perennial use.

There are six gauging stations in operation in GRNW basin as shown in Fig. 2.4.3. Of them, five stations are in tributaries in central plateau. The rest station, W13, is located at Municipal Dyke in GRNW. Conditions of operation and equipment are as follows:

Station	River Name	Recorder	Flow Condition
W03	Plaines Wilhems	A, D	Concrete control section
W04	Terre Rouge	A, D	Concrete control section
W05	Cascade	A, D	Concrete control section
W08	Profonde	A, D	Concrete control section
W10	Moka	A, D	Natural control section
W13	GRNW	A	Concrete control section

A: Automatic recorder D: Daily staff reading

Gauging station on the Profonde river (W08), the Cascade river (W05), the Terre Rouge river (W04), the Plaines Wilhems river (W03) and GRNW at Municipal Dike (W13) have concrete control weir with iron blade, and critical flow occur over the weirs. As for the gauging station on the Moka river (W10), the river bed section about 10 meters downstream of the station is covered with fresh lava and critical flow also occurs over the section. The stations have been well maintained and operated by CWA, Hydrological Section and data have been published as hydrological year book since 1966.

All the surface flow at Municipal Dyke (W13) is abstracted in the driest period of every year, being carried through Municipal Pipeline into Pailes treatment plant, so water level often drops down below the lowest level of the weir crest in the seasons. At station W13, conduct pipe which connects reservoir of Municipal Dyke and the cylinder of a float of the recorder is set at the same level as that of low flow section of the weir. Furthermore, abstracted water volume at Municipal Dyke has not been observed. Therefore recorded data at W13 are, so far, not available especially for dry seasons.

There is another station in the Moka river called W12, but it has not been operated since 1972. Therefore, the data of the station is not used in the study. Table 2.4.1 shows monthly discharge of 6 stations.

(5) Floods

The 10-year, 20-year, 100-year, 200-year and 10,000-year (PMF) probable floods at the damsite are analyzed by the storage function method based on the probable basin rainfall records.

Each flood peak discharge is worked out as follows: Its flood hydrograph is given in Fig. 2.4.6.

Probable Flood Peak Discharge

Return Year	Peak Discharge (m3/s)	Specific Discharge ($m^3/s/km^2$)	Creager's C
10	440	8	17
20	520	9	19
100	1,040	18	37
200	1,200	22	46
10,000 (PMF)	1,890	35	72

(6) Sedimentation

The average annual sediment yield in the basin is analyzed to be 3,949 m^3/year or 72 m^3/km^2/year in terms of the specific sediment yield (0.07 mm in the denudation depth of the basin) on the basis of the available sediment data, and the bed load transport is estimated at 140 m^3/year.

The trap ratio of wash load into the reservoir is estimated to be 70% or 2,764 m^3/year. The total sediment to be stored in the reservoir is, therefore, estimated to be 2,904 m^3/year with the bed load transport of 140 m^3/year.

2.5 Topography

The project area, which consists of basins of the main rivers such as the Plaines Wilhems, the Terre Rouge, the Cascade, the Moka and their tributaries, occupies about 130 sq.km elongating to east in the upstream reaches. These rivers join to be the Ground River North West (G.R.N.W.) directing north-west in the downstream, draining into the Ground River Bay. The rivers flow in very gentle Plaines from the east to the west or the southeast to the northwest, meandering extensively in the upstream reaches.

At the just upstream parts where each river joins to be G.R.N.W. rivers dissect the gentle Plaines deeply and changes to be very steep gorges accompanying rapids and waterfalls. The G.R.N.W. has the width of 50 m to 100 m in general with very steep, mostly vertical, river flanks.

The gentle Plaines are bounded by outstanding high mountain ranges of Anse Courtois of which main mountain peaks are Mount Ory (349 m), Le Pouce (811 m), Ground Peak, Pieter Both, etc. The western part of the gentle Plaines are bounded by Corps de Garde (720 m) and other small hills. The boundary of eastern and southern parts are in the highlands area, and the watersheds of the study area are not very clear in these parts. The eastern part of study area forms the boundary between the project area and the river basins of the Ground River South East and River La Chaux.

The proposed damsite is situated at the downstream of the Terre Rouge river, just upstream of the confluence between the Plaines Wilhems and the Terre Rouge river. The dam abutment forms a deep gorge of about 130 m in height.

The dam abutment of this site is very steep, about 50 deg., on the right abutment and about 35 deg. on the left abutment.

2.6 Geology

(1) Regional Geology

A prominent feature of the project area is the clear topographic distinction between high mountainous area and gentle flat land. The high mountainous area is composed of old volcanic series. General regional geological condition of the project area is indicated in Fig. 2.6.1.

According to the potassium-argon dating by N. McDOUGAL and CHAMALAUN, the main shield volcano composed of the older volcanic series was built between 7.8 and 6.8 m.y. ago in the early Pliocene and lavas of the younger volcanic series were erupted from about 3.5 m.y. ago to less than 0.2 m.y. ago.

The young volcanic series are composed of basaltic lavas and agglomerates, generally dipping to the north to the northwest at low angle around 5 deg. The old lavas dip about 10 deg. from the south to the north or the southeast to the northwest.

The vesicles in the old lavas are generally filled with zeolites. The lavas of this series appears to be dark grayish, and fresh part is very hard with emitting clear sound by hammering. Uniaxial compressive strength of the fresh lavas is more than 1,000 kg/cm ^{2}. Tuff layers and volcanic breccias intercalate with the lava layers. Weathering on the tuff layers or volcanic breccias is developed on the ground surface.

The young volcanic lavas are characteristic with frequently developed vesicular appearance. Volcanic breccias intercalate the lava layers with thickness of about 3 m to 10 m. In the upstream reaches of the Ground River North West (G.R.N.W.), hard lava layers are predominantly observed. Columnar jointed basaltic lava layers of more than 10 m in thickness expose in the river flanks, intercalating with volcanic breccias. In the middle reaches of the G.R.N.W. volcanic breccias are observed predominantly near Municipal Dyke.

(2) Geology of dam site

Bedrock of the dam site is composed of superposed several sub-horizontal sheets of basaltic lava flows of different times, with unconformity planes at each contact of two adjacent lava flows. The lava flows are divided into two groups, that is, Old Lavas and Young Lavas overlying the former. Members of the Young Lavas are glassy vesicular basalts and hypocrystalline less vesicular basaltic rocks, which are occasionally auto-brecciated in parts close to the flow-to-flow contacts. The Old Lava member are slightly altered vesicular basaltic lavas, which are characterised by presence of plagioclase phenocryst.

Each member of the lava flows is more or less weathered, especially in the upper zones and at the bottom. Layer of consolidated lateritic residual soil, which is a product of intensive surface weathering, occurs at the top of each flow. This sort of reddish brown decomposed rock plus old top soil, also moderately consolidated, have been located at several stratigraphic horizons, or at several topographic levels, in the dam site. At least four members are distinguished in the Young Lavas by the help of these consolidated soil layers as markers. The boundary between the Young Lavas and the Old Lavas is also discerned in the same way at the level within 15 meters' height from the river bed. A boundary has been found also in the Old Lavas under the river bed.

In the geological investigations to date, it has been noticed that these layers of decomposed rock or consolidated soil are often watertight or remarkably less previous than other hard rock portions of the bedrock, probably for the lack of open cracks in them. In mechanical aspect, their strength for dam foundation is self-evident by the history of their being under the load of overlying strata, while due consideration should be taken in designing for difference in elasticity and deformation of these soft rocks and of the hard rocks in the other levels.

In the other parts of the dam foundation, the basaltic bedrock is sufficiently hard, though occasionally fractured with closely spaced open joints and cracks. While the Lugeon tests performed so far have obtained Lugeon unit ranging from one to 50 or infrequently more, the test grouting recently made in the site indicates that the seepage potential of the bedrock can be improved with ordinary cement grouting.

The diversion tunnel will be driven at the level of about 129 meters in elevation, where also lies the contact between Young Lavas and Old Lavas. In the surface zone of the Old Lavas, the basalt is intensely weathered and weakened to soft rock for thickness of 7 to 10 meters. Drilling core samples show that the bedrock is composed of basalt lavas, which is intensely weathered in the surface zones near the portals of the tunnel, while it is less weathered or even almost fresh in the middle part of the tunnel through the core of the hill.

The situation of site geology is presented in Fig. 2.6.2 to Fig. 2.6.5.

Results of laboratory tests carried out for various kinds of rocks in damsite are shown in Table 2.6.1.

TABLE 2.4.1 MONTHLY MEAN DISCHARGE (1/6)

Station :W03

River :The Plaines Wilhems River

(Unit : m^3/s)

Hydrological Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Annual Average
1972			0.28	0.34	0.36	0.34	0.37	0.37	0.33	0.29	0.29	0.37	
1973	0.79	0.40	0.44	0.37	1.48	0.38	0.37	0.37	0.38	0.39	0.41	0.34	0.51
1974	0.15	0.17	0.19	0.35	0.38	0.37	0.34	0.37	0.35	0.31	0.37	0.23	0.30
1975	0.16	0.21	0.23	0.27	0.48	0.35	0.57	0.32	0.23	0.34	0.34	0.27	0.31
1976	0.21	0.19	0.15	0.52	0.29	0.36	0.24	0.90	0.28	0.29	0.29	0.28	0.33
1977	0.19	0.19	0.22	0.28	0.30	0.32	0.28	0.28	0.26	0.14	0.14	0.14	0.23
1978	0.10	0.24	1.32	0.14	0.09	0.43	0.17	0.18	0.16	0.16	0.16	0.14	0.27
1979	0.08	0.12	0.31	0.20	0.68	0.22	0.19	0.21	0.21	0.26	0.12	0.14	0.23
1980	0.11	0.97	12.10	1.31	3.01	1.90	0.43	0.19	0.31	0.27	0.23	0.23	1.75
1981	0.18	0.12	0.14	0.18	0.21	0.47	0.23	0.24	0.25	0.22	0.25	0.21	0.23
1982	0.20	0.24	0.26	6.03	0.33	0.26	0.85	0.30	0.54	0.40	0.26	0.27	0.83
1983	0.28	0.62	0.41	0.23	0.21	0.21	0.20	0.18	0.22	0.14	0.09	0.07	0.24
1984	0.13	3.05	0.30	0.29	0.24	0.14	0.13	0.14	0.15	0.17	0.18	0.16	0.42
1985	0.12	0.22	0.72	5.97	0.25	0.47	0.24	0.25	0.25	0.23	0.22	0.25	0.76
1986	0.19	1.14	0.30	0.31	0.32	0.30	0.25	0.23	0.16	0.21	0.15	0.18	0.31
Average	0.21	0.56	1.22	1.17	0.59	0.44	0.32	0.30	0.27	0.25	0.23	0.21	0.48
Maximum	0.79	3.05	12.10	6.03	3.01	1.90	0.85	0.90	0.54	0.40	0.41	0.34	1.75
Minimum	0.08	0.12	0.14	0.14	0.09	0.14	0.13	0.14	0.15	0.14	0.09	0.07	0.23
Var.	0.17	0.76	3.03	1.99	0.75	0.41	0.18	0.18	0.10	0.08	0.09	0.07	0.40

Var. : Standard Deviation

TABLE 2.4.1 MONTHLY MEAN DISCHARGE (2/6)

Station :W04
River :The Terre Rouge River

(Unit : m^3/s)

Hydrological Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Annual Average
1966			0.67	0.39	0.67	0.15	0.09	0.49	0.18	0.17	0.07	0.09	
1967	0.07	0.61	0.95	0.28	0.99	0.67	0.23	0.66	0.68	0.63	0.13	0.14	0.50
1968	1.04	0.93	0.36	2.50	2.20	0.25	0.19	0.18	0.26	0.15	0.08	0.07	0.68
1969	0.07	0.08	0.07	0.48	0.30	0.98	0.23	0.39	0.55	0.44	0.09	0.07	0.31
1970	0.08	0.46	3.22	1.78	3.41	0.56	0.29	0.44	0.41	0.40	0.09	0.09	0.94
1971	0.07	0.07	0.32	0.46	0.26	0.94	0.44	0.14	0.76	0.17	0.10	0.17	0.33
1972	0.09	0.09	0.16	0.98	0.24	0.42	0.24	0.27	0.35	1.15	0.16	0.15	0.36
1973	0.68	0.28	0.88	0.42	1.39	0.30	0.21	0.30	0.49	0.61	0.32	0.09	0.50
1974	0.08	0.08	0.08	0.13	0.22	0.12	0.10	0.16	0.28	0.95	0.11	0.18	0.21
1975	0.14	0.21	0.19	0.67	0.47	0.31	1.09	0.44	0.12	0.09	0.31	0.12	0.35
1976	0.08	0.14	0.16	1.01	0.22	0.52	0.90	0.99	0.12	0.17	0.09	0.09	0.37
1977	0.09	0.10	0.34	0.60	0.38	0.42	0.20	0.16	0.11	0.10	0.09	0.07	0.22
1978	0.11	0.29	1.39	0.10	0.31	1.77	0.25	0.18	0.15	0.13	0.08	0.15	0.41
1979	0.11	0.15	0.50	0.46	1.03	0.43	0.14	0.13	0.09	0.59	0.07	0.11	0.32
1980	0.10	2.36	8.59	1.52	2.06	1.34	0.36	0.21	0.22	0.11	0.14	0.08	1.42
1981	0.08	0.10	0.08	0.10	0.10	1.39	0.21	0.16	0.10	0.12	0.10	0.12	0.22
1982	0.09	0.08	0.15	4.99	0.40	0.24	1.21	0.55	0.82	0.88	0.27	0.34	0.83
1983	0.28	0.68	0.59	0.25	0.16	0.15	0.20	0.13	0.19	0.11	0.09	0.07	0.24
1984	0.09	1.91	1.26	0.28	0.17	0.17	0.15	0.11	0.12	0.17	0.13	0.09	0.39
1985	0.08	0.22	1.33	4.36	0.83	0.66	0.18	0.21	0.29	0.12	0.11	0.13	0.71
1986	0.17	1.59	0.34	0.66	0.88	0.42	0.20	0.14	0.16	0.27	0.12	0.13	0.42
Average	0.18	0.52	1.03	1.07	0.79	0.58	0.34	0.31	0.31	0.36	0.13	0.12	0.48
Maximum	1.04	2.36	8.59	4.99	3.41	1.77	1.21	0.99	0.82	1.15	0.32	0.34	1.42
Minimum	0.07	0.07	0.07	0.10	0.10	0.12	0.09	0.11	0.09	0.09	0.07	0.07	0.21
Var.	0.24	0.65	1.83	1.31	0.83	0.45	0.31	0.22	0.22	0.31	0.07	0.06	0.29

Var. : Standard Deviation

TABLE 2.4.1 MONTHLY MEAN DISCHARGE (3/6)

Station :W05

River :The Cascade River

(Unit : m^3/s)

Hydrological Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Annual Average
1966			0.67	0.47	0.45	0.19	0.12	0.57	0.21	0.37	0.15	0.12	
1967	0.09	0.46	1.02	0.26	1.27	1.15	0.35	0.46	1.08	1.01	0.48	0.44	0.67
1968	0.92	1.09	0.31	3.47	2.65	0.38	0.26	0.29	0.59	0.46	0.22	0.20	0.90
1969	0.19	0.17	0.14	0.48	0.47	1.30	0.47	0.51	0.95	0.52	0.25	0.15	0.47
1970	0.15	0.65	3.24	1.78	4.22	0.63	0.35	0.97	0.30	0.47	0.16	0.16	1.09
1971	0.15	0.15	0.16	0.68	0.23	1.23	0.67	0.48	0.48	0.21	0.16	0.15	0.40
1972	0.21	0.15	0.18	1.31	0.45	1.01	0.42	0.83	0.77	1.52	0.35	0.31	0.63
1973	0.53	0.47	0.76	0.52	1.18	0.51	0.32	0.89	0.71	1.06	0.84	0.42	0.68
1974	0.32	0.21	0.24	0.52	0.55	0.35	0.26	0.49	0.61	1.19	0.43	0.28	0.45
1975	0.26	0.30	0.20	1.05	0.83	0.51	1.33	0.73	0.52	0.44	0.45	0.31	0.58
1976	0.20	0.16	0.13	0.98	0.47	1.24	1.48	1.27	0.52	0.58	0.35	0.33	0.64
1977	0.29	0.39	1.21	0.92	0.44	0.84	0.60	0.39	0.48	0.40	0.28	0.19	0.54
1978	0.17	0.35	1.61	0.36	0.71	2.33	0.74	0.57	0.70	0.52	0.39	0.24	0.72
1979	0.22	0.17	0.56	1.27	1.61	0.71	0.42	0.39	0.33	0.68	0.24	0.20	0.57
1980	0.17	3.08	8.15	1.56	3.38	2.14	0.93	0.64	0.49	0.31	0.29	0.29	1.79
1981	0.28	0.24	0.19	0.17	0.27	2.24	0.64	0.45	0.32	0.32	0.29	0.18	0.47
1982	0.21	0.30	0.52	6.14	0.93	0.54	2.02	0.96	1.12	1.33	0.80	0.84	1.31
1983	0.82	1.19	1.56	0.76	0.41	0.38	0.24	0.25	0.30	0.22	0.15	0.17	0.54
1984	0.17	2.13	2.09	0.72	0.36	0.53	0.45	0.32	0.32	0.37	0.28	0.21	0.66
1985	0.19	0.37	2.48	6.67	1.18	1.25	0.51	0.55	0.99	0.50	0.43	0.25	1.28
1986	0.31	2.06	0.57	0.97	1.06	0.97	0.52	0.36	0.22	0.43	0.21	0.20	0.66
Average	0.29	0.70	1.24	1.48	1.10	0.97	0.62	0.59	0.57	0.62	0.34	0.27	0.73
Maximum	0.92	3.08	8.15	6.67	4.22	2.33	2.02	1.27	1.12	1.52	0.84	0.84	1.79
Minimum	0.09	0.15	0.13	0.17	0.23	0.19	0.12	0.25	0.21	0.21	0.15	0.12	0.40
Var.	0.21	0.79	1.76	1.74	1.04	0.61	0.45	0.26	0.27	0.37	0.18	0.15	0.35

Var. : Standard Deviation

TABLE 2.4.1 MONTHLY MEAN DISCHARGE (4/6)

Station :W08

River :The Profonde River

(Unit : m^3/s)

Hydrological Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Annual Average
1966			0.434	0.217	0.356	0.177	0.133	0.154	0.149	0.149	0.179	0.125	
1967	0.07	0.17	0.61	0.24	0.69	0.63	0.30	0.26	0.41	0.27	0.21	0.18	0.34
1968	0.50	0.49	0.18	0.64	0.62	0.16	0.12	0.13	0.15	0.15	0.15	0.11	0.28
1969	0.07	0.10	0.09	0.26	0.49	0.54	0.29	0.20	0.44	0.42	0.30	0.24	0.29
1970	0.10	0.55	1.68	0.76	1.51	0.30	0.28	0.32	0.37	0.35	0.28	0.24	0.56
1971	0.24	0.13	0.22	0.62	0.29	0.76	0.40	0.23	0.31	0.28	0.25	0.19	0.33
1972	0.18	0.11	0.14	0.49	0.40	0.40	0.22	0.29	0.25	0.73	0.25	0.25	0.31
1973	0.32	0.25	0.35	0.33	0.59	0.27	0.21	0.26	0.21	0.30	0.24	0.14	0.29
1974	0.13	0.12	0.08	0.29	0.35	0.19	0.13	0.12	0.16	0.29	0.14	0.11	0.18
1975	0.16	0.10	0.12	0.50	0.42	0.28	0.37	0.20	0.19	0.20	0.15	0.12	0.23
1976	0.16	0.10	0.15	0.42	0.20	0.34	0.50	0.38	0.23	0.22	0.19	0.18	0.26
1977	0.15	0.26	0.48	0.51	0.25	0.30	0.22	0.17	0.19	0.12	0.15	0.13	0.24
1978	0.11	0.17	0.62	0.15	0.25	0.73	0.26	0.20	0.24	0.18	0.14	0.14	0.27
1979	0.18	0.10	0.18	0.48	0.35	0.27	0.19	0.14	0.11	0.23	0.17	0.18	0.21
1980	0.19	1.37	4.08	0.58	1.75	0.93	0.44	0.28	0.21	0.16	0.12	0.12	0.85
1981	0.11	0.13	0.11	0.10	0.12	0.85	0.31	0.18	0.13	0.12	0.10	0.13	0.20
1982	0.16	0.19	0.22	2.58	0.46	0.26	0.57	0.42	0.35	0.39	0.35	0.31	0.52
1983	0.33	0.44	0.34	0.30	0.20	0.19	0.11	0.07	0.07	0.06	0.05	0.05	0.18
1984	0.12	1.29	0.57	0.37	0.20	0.17	0.12	0.12	0.09	0.12	0.09	0.09	0.28
1985	0.08	0.13	0.83	2.46	0.55	0.45	0.23	0.22	0.25	0.17	0.18	0.15	0.47
1986	0.15	0.84	0.24	0.50	0.50	0.29	0.22	0.14	0.17	0.24	0.16	0.17	0.30
Average	0.18	0.35	0.56	0.61	0.50	0.40	0.27	0.21	0.22	0.24	0.18	0.16	0.32
Maximum	0.50	1.37	4.08	2.58	1.75	0.93	0.57	0.42	0.44	0.73	0.35	0.31	0.85
Minimum	0.07	0.10	0.08	0.10	0.12	0.16	0.11	0.07	0.07	0.06	0.05	0.05	0.18
Var.	0.10	0.38	0.86	0.64	0.40	0.23	0.13	0.09	0.10	0.14	0.07	0.06	0.16

Var. : Standard Deviation

TABLE 2.4.1 MONTHLY MEAN DISCHARGE (5/6)

Station :W10
River :The Moka River

(Unit : m^3/s)

Hydrological Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Annual Average
1966		0.19	0.40	0.32	0.92	0.34	0.15	0.16	0.24	0.20	0.20	0.11	
1967	0.08	0.79	0.81	0.28	1.26	1.09	0.39	0.32	0.81	0.55	0.41	0.27	0.59
1968	0.83	0.80	0.30	1.82	1.33	0.37	0.24	0.27	0.32	0.23	0.23	0.16	0.58
1969	0.11	0.20	0.13	0.39	1.00	1.20	0.64	0.34	0.64	0.48	0.25	0.14	0.46
1970	0.11	1.40	2.66	1.53	2.63	0.64	0.34	0.48	0.35	0.45	0.23	0.11	0.91
1971	0.12	0.08	0.66	0.88	0.42	1.36	0.78	0.36	0.51	0.39	0.24	0.18	0.50
1972	0.24	0.15	0.49	0.89	0.65	1.09	0.61	0.86	0.59	1.55	0.39	0.45	0.66
1973	0.52	0.51	1.01	1.07	1.07	0.50	0.58	0.63	0.63	0.65	0.46	0.23	0.65
1974	0.15	0.21	0.33	1.01	0.86	0.45	0.31	0.41	0.65	1.19	0.43	0.24	0.52
1975	0.15	0.36	0.28	0.72	0.86	0.51	0.97	0.54	0.40	0.32	0.38	0.19	0.47
1976	0.14	0.12	0.22	1.02	0.67	0.94	1.70	1.11	0.49	0.51	0.30	0.27	0.62
1977	0.18	0.34	0.94	0.82	0.33	0.73	0.60	0.39	0.47	0.37	0.21	0.17	0.46
1978	0.13	0.32	1.56	0.46	0.64	1.81	0.58	0.45	0.82	0.65	0.37	0.24	0.67
1979	0.17	0.18	0.73	1.23	0.86	0.56	0.55	0.42	0.29	0.73	0.30	0.18	0.52
1980	0.16	2.70	12.02	0.99	7.86	1.10	0.78	0.47	0.38	0.28	0.22	0.19	2.26
1981	0.16	0.13	0.14	0.17	0.38	2.07	0.58	0.28	0.23	0.23	0.24	0.18	0.40
1982	0.33	0.42	0.61	6.77	0.69	0.35	1.23	0.68	0.74	0.78	0.68	0.61	1.16
1983	0.71	0.86	1.01	0.60	0.35	0.22	0.18	0.17	0.20	0.18	0.14	0.14	0.40
1984	0.19	3.14	1.33	0.64	0.37	0.37	0.26	0.27	0.28	0.43	0.31	0.24	0.65
1985	0.19	0.44	2.43	6.09	0.82	0.70	0.34	0.56	0.62	0.51	0.45	0.31	1.12
1986	0.24	1.96	0.57	1.05	0.92	0.46	0.56	0.34	0.26	0.51	0.28	0.23	0.61
Average	0.24	0.73	1.36	1.37	1.18	0.80	0.59	0.45	0.47	0.53	0.32	0.23	0.69
Maximum	0.83	3.14	12.02	6.77	7.86	2.07	1.70	1.11	0.82	1.55	0.68	0.61	2.26
Minimum	0.08	0.08	0.13	0.17	0.33	0.22	0.15	0.16	0.20	0.18	0.14	0.11	0.40
Var.	0.20	0.84	2.48	1.69	1.57	0.49	0.36	0.22	0.19	0.32	0.12	0.11	0.41

Var. : Standard Deviation

TABLE 2.4.1 MONTHLY MEAN DISCHARGE (6/6)

Station :W12
River :The Moka River

(Unit : m^3/s)

Hydrological Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul	Aug.	Sep.	Oct.	Annual Average
1966	0.57	0.34	1.54	0.40	0.60	0.59	0.18	0.24	0.23	0.20	0.19	0.13	0.43
1967	0.10	2.98	1.62	0.54	0.40	1.08	0.52	0.31	0.50	0.43	0.38	0.44	0.77
1968	0.41	1.20	0.39	2.01	1.61	0.44	0.24	0.30	0.25	0.31	0.23	0.16	0.63
1969	0.11	0.16	0.11	0.38	0.42	0.84	0.96	0.38	0.59	0.57	0.31	0.17	0.42
1970	0.09	0.61	1.43	1.57	2.91	1.32	0.32	0.36	0.34	0.34	0.27	0.16	0.81
1971	0.08	0.08	0.36	1.05	0.37	0.47	0.54	0.49	0.30	0.32	0.27	0.15	0.37
1972	0.19	0.19	0.31	1.12	0.66	0.61	0.40	0.77	0.52	1.13	0.49	0.32	0.56
1973	0.39	0.47	0.81	0.71	1.29	0.31	0.35	0.49	0.51	0.60	0.40	0.17	0.54
1974	0.07	0.13	0.37	0.97	0.98	0.52	0.25	0.45	0.44	0.60	0.42	0.15	0.45
1975	0.06	0.27	0.30	1.61	1.00	0.59	0.53	0.44	0.48	0.34	0.37	0.19	0.52
1976	0.21	0.15	0.20	1.08	0.59	0.48	0.88	0.75	0.48	0.46	0.30	0.19	0.48
1977	0.26	0.34	0.79	1.63	0.41	0.57	0.47	0.36	0.39	0.29	0.18	0.16	0.49
1978	0.07	0.22	0.55	0.44	0.50	1.04	0.61	0.32	0.35	0.33	0.28	0.20	0.41
1979	0.14	0.17	0.38	0.90	0.55	0.44	0.57	0.38	0.23	0.31	0.31	0.16	0.38
1980	0.14	0.71	2.40	1.74	2.41	1.61	0.49	0.26	0.13	0.24	0.15	0.16	0.87
Average	0.19	0.53	0.77	1.08	0.98	0.73	0.49	0.42	0.38	0.43	0.30	0.19	0.54
Maximum	0.57	2.98	2.40	2.01	2.91	1.61	0.96	0.77	0.59	1.13	0.49	0.44	0.87
Minimum	0.06	0.08	0.11	0.38	0.37	0.31	0.18	0.24	0.13	0.20	0.15	0.13	0.37
Var.	0.15	0.71	0.65	0.51	0.75	0.36	0.21	0.15	0.13	0.22	0.09	0.08	0.15

Var. : Standard Deviation

Table 2.6.1 RESULTS OF LABORATORY TESTS FOR ROCKS IN DAMSITE

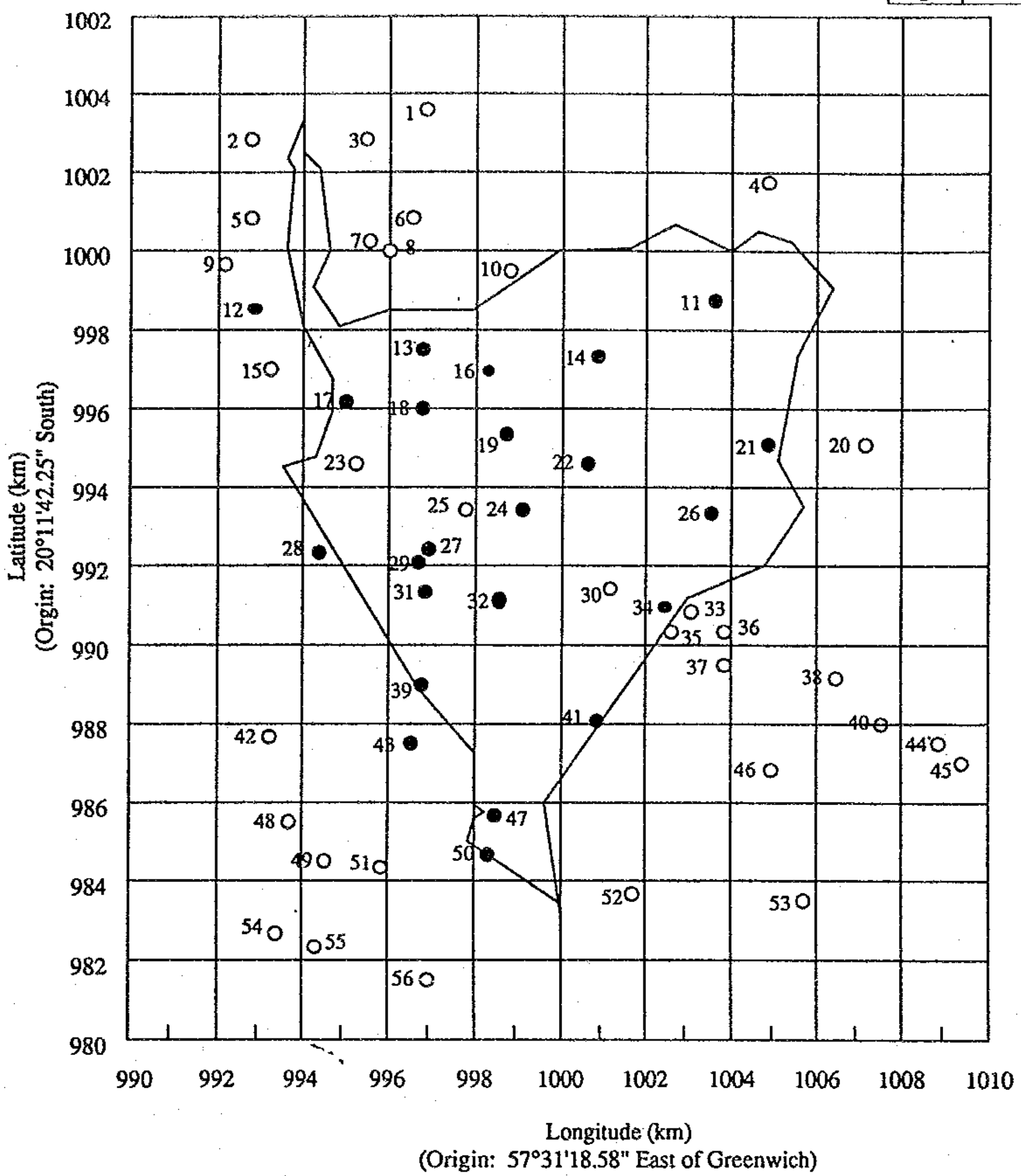
Sample No.	Type of Rocks	Length (mm) L	Diameter (mm) D	Unit Wt of intact sample (g/cm³)	Moisture Content (%)	Specific gravity (g/cm³)	Water Absorption (%)	Measured Compressive Strength (MN/m²)	Corrected Compressive Strength (MN/m²)	Remarks
1.	Old lava (Doleritic)	103.3	51.6	2.563	1.9	2.570	2.3	36.3	36.3	
2.	Old lava (Felty)	104.4	51.8	2.893	1.5	2.862	2.1	137.3	137.4	
3.	Young lava (Massive)	102.0	51.7	2.807	0.4	2.813	1.1	118.8	118.6	
4.	Young lava (Massive)	103.8	51.0	2.738	1.3	2.708	1.7	98.6	98.8	
5.	Young lava (Vessicular)	101.7	51.4	2.320	2.8	2.371	6.4	26.4	26.4	
6.	Young lava (Vessicular)	101.0	49.5	2.202	1.1	2.275	4.1	28.5	28.6	Dam Foundations
7.	Young lava (Vessicular)	101.8	51.5	2.319	0.8	2.563	3.6	26.5	26.5	
8.	Flow Breccia	91.8	51.4	1.400	20.0	2.698	-*	2.46	2.43	
9.	Old Residual Soil	95.0	50.5	2.000	34.9	2.754	-*	0.55	0.55	
10.	Weathered Basalt (leached)	91.2	51.5	1.306	11.0	2.698	-*	1.92	1.89	
11.	Weathered Basalt (Moderate)	75.0	50.8	1.471	14.9	2.703	-*	2.42	2.32	

Note: * The water absorption value cannot be determined for samples Nos. 8 to 11 because the materials were of highly weathered nature and the specific gravities were conducted as for soil materials.

Corrected compressive strengths have been calculated using formula (ASTM D2938), $C_c = C_m / (0.88 + 0.24\ (d/h))$ where,

C_m = measured compressive strength
C_c = corrected c/s of an equivalent L/D = 2 specimen
d = test core diameter
h = test core height

Fig. 2.4.1



No.	Station	No.	Station	No.	Station
1	Line Barracks	21	Alma	39	Vacoas
2	Pte. aux Sables	22	Cote d'Or	40	Dubreuil Factory
4	Industries	23	Ebene	41	Wooton
5	Richelieu	24	Bagatelle (H)	42	Holyrood
6	Les Guibies	25	Camoene	43	Reunion
7	Pailles	26	Valetta	44	Dubreuil (3E,3N,3W)
9	Les Rosieres	27	Mauriffods (Trianon)	45	La Pipe
10	Montagne (MDA)	28	Quatre Bornes	46	Chartreuse
11	Beau Bois (MDA)	29	Trianon	47	Curepipe Garden
12	Chebel	30	Hermitage	48	Moon
13	Bagatelle	31	Phoenix	49	Henrietta
14	Mon Desert Alma	32	Highlands	50	Curepipe experi. St.
15	Barkly	33	Belle Rive (1N)	51	La Marie
16	Minissy (MDA)	34	Belle Rive (SIRI)	52	XVI Mile
17	Bega	35	Belle Rive (2N	53	Bananes
18	Reduit Experi. St.	36	Belle Rive (1E)	54	Tamarin (Res.)
19	Minissy (H)	37	Belle Rive (2E)	55	Bonnefin
20	Bonne Veine	38	Piton du Milieu	56	Good End

Legend

- ○ Rainfall Station
- ● - (Selected)
- — Boundary of GRNW Basin

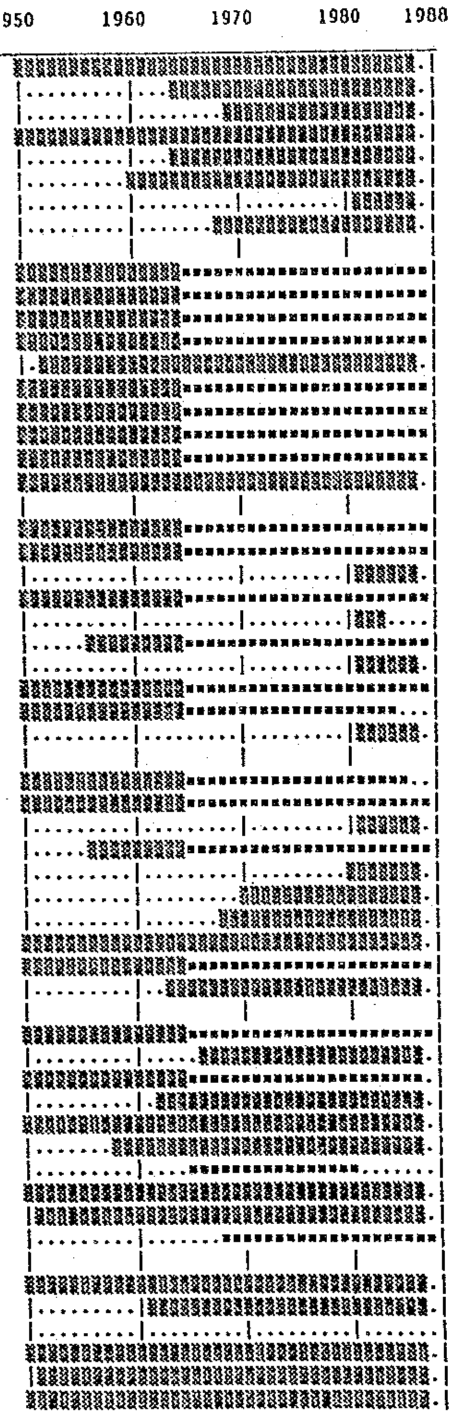
LOCATION OF RAINFALL STATIONS IN AND AROUND GRNW

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.4.2

No.	(1) Ident	(2) Ident		Name of Station	Lat. (km)	Long. (km)
1	BB112			Line Barracks	97	295
2	DD101			Pte. aux Sables	102	272
4	BB313			Industries	107	341
5	DD203			Richelieu	113	272
6	BB214			Les Guibles	114	293
7	BB215			Pailles	116	288
9	DD204			Les Rosieres	118	268
10	119305			Montagne (MDA)	119	305
11	FF301	W18.MDA	(**)	Beau Bois (MDA)	124	335
12	DD306	V6.MED	(**)	Chebel	125	270
13	DD308	W4.MDA	(**)	Bagatelle (MDA)	131	292
14	FF302	W12.MDA	(**)	Mon Desert Alma	133	316
15	134274			Barkly	134	274
16	FF304	W6.MDA	(**)	Minissy (M)	143	306
17	DD312	W1.H	(**)	Bega	137	285
18	DD314	-	(**)	Le Reduit Experi. St.	139	293
19	FF303	W7.H	(**)	Minissy (MDA)	135	302
20	EE301	E15.MDA		Bonne Veine	143	353
21	FF405	W19.MDA	(**)	Alma	144	340
22	FF306	W11.H	(**)	Cote d'Or	147	315
23	147285			Ebene	147	285
24	FF307	W9.H	(**)	Bagatelle (H)	151	308
25	1532301			Camoene	152	301
26	FF408	W17.MDA	(**)	Valetta	152	334
27	FF310		(**)	Maurifoods (Trianon)	159	295
28	155288		(**)	Quatre Bornes	155	288
29	DD317	W3.MDA	(**)	Trianon	150	291
30	FF411	W14.H		Hermitage	161	318
31	FF312		(**)	Phoenix	164	295
32	FF313	W8.H	(**)	Highlands	164	305
33	FF414	E18.CWA		Belle Rive (1N)	166	333
34	FF415	W15.SIR	(**)	Belle Rive (SIRI)	168	326
35	EE403	W16.CWA		Belle Rive (2N)	170	330
36	EE404	E17.CWA		Belle Rive (1E)	170	335
37	EE406	E16.CWA		Belle Rive (2E)	174	335
38	EE307	-		Piton du Milieu	175	346
39	FF316	.MET	(**)	Vacoas	176	294
40	EE310	E14.DUB		Dubreuil Factory	188	348
41	FF418	-	(**)	Hooton	182	316
42	FF320	T6.MED		Holyrood	186	275
43	FF319	T10.MED	(**)	Reunion	184	293
44	EE308	E13.CWA		Dubreuil (3N)	184	361
45	EE311	E12.CWA		La Pipe	188	368
46	EE309			Chartreuse	188	341
47	194304		(**)	Curpipe Gardens	194	304
48	FF321	T7.MED		Moon	192	276
49	FF422	T8.MED		Henrietta	199	281
50	FF423		(**)	Curepipe experi. St.	200	303
51	FF424	T9.MED		La Marie	201	290
52	EE412	G4.CWA		XVI Mile	205	323
53	220333	G3.RB		Bananes	220	333
54	FF425	S9.CEB		Tamarin (Res.)	211	275
55	FF426	S7.MED		Bonnefin	214	283
56	FF427	T11.CWA		Good End	216	294



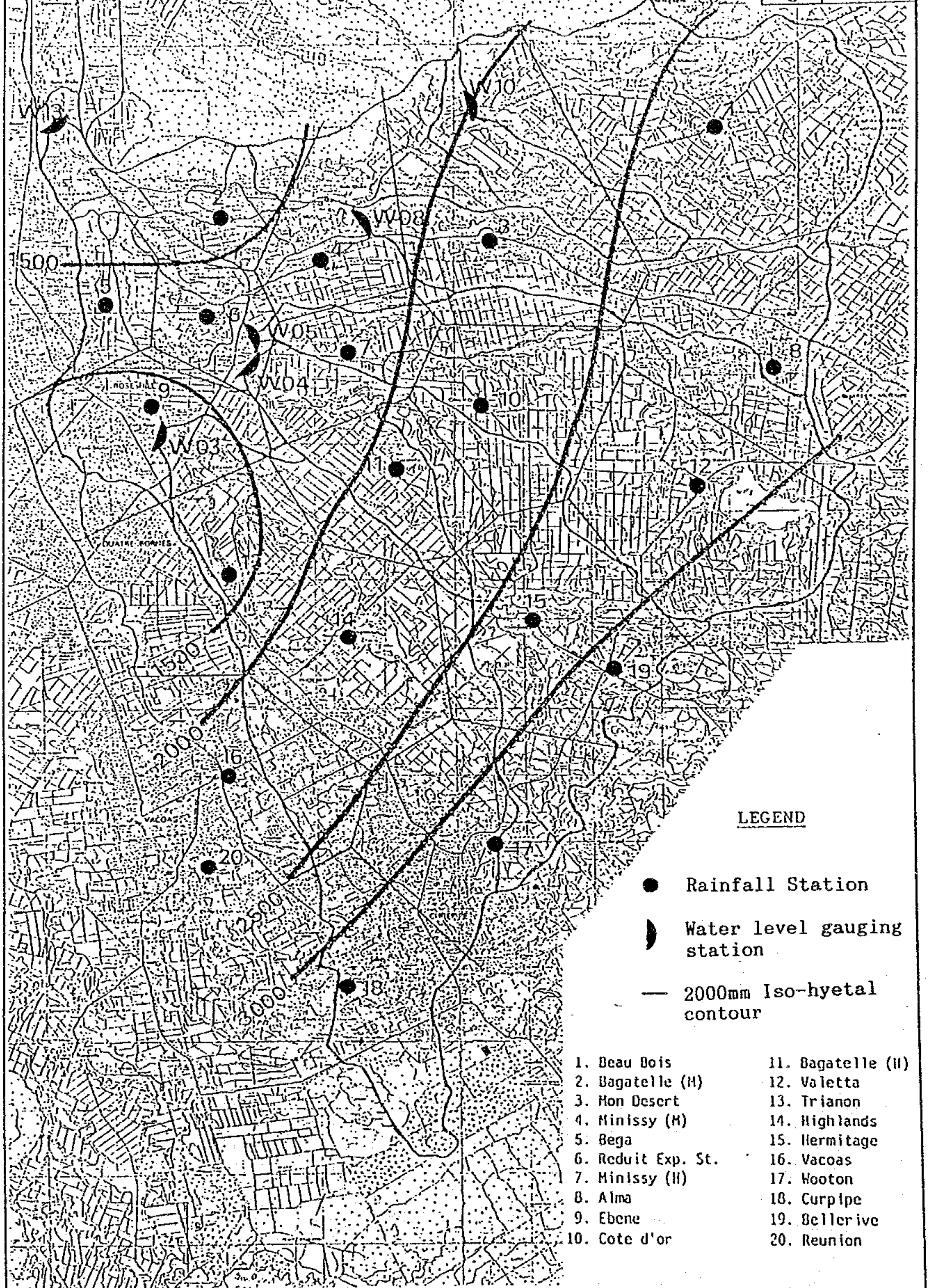
Ref: (1) Identifier by Meteorological Service
(2) Identifier by Hydrological section, CWA
(**) Selected stations for daily dase analysis (1965-1987)
▓▓ Period in which monthly data are available
** Period in which daily data are available

DURATION OF RECORD OF RAINFALL STATIONS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.4.3

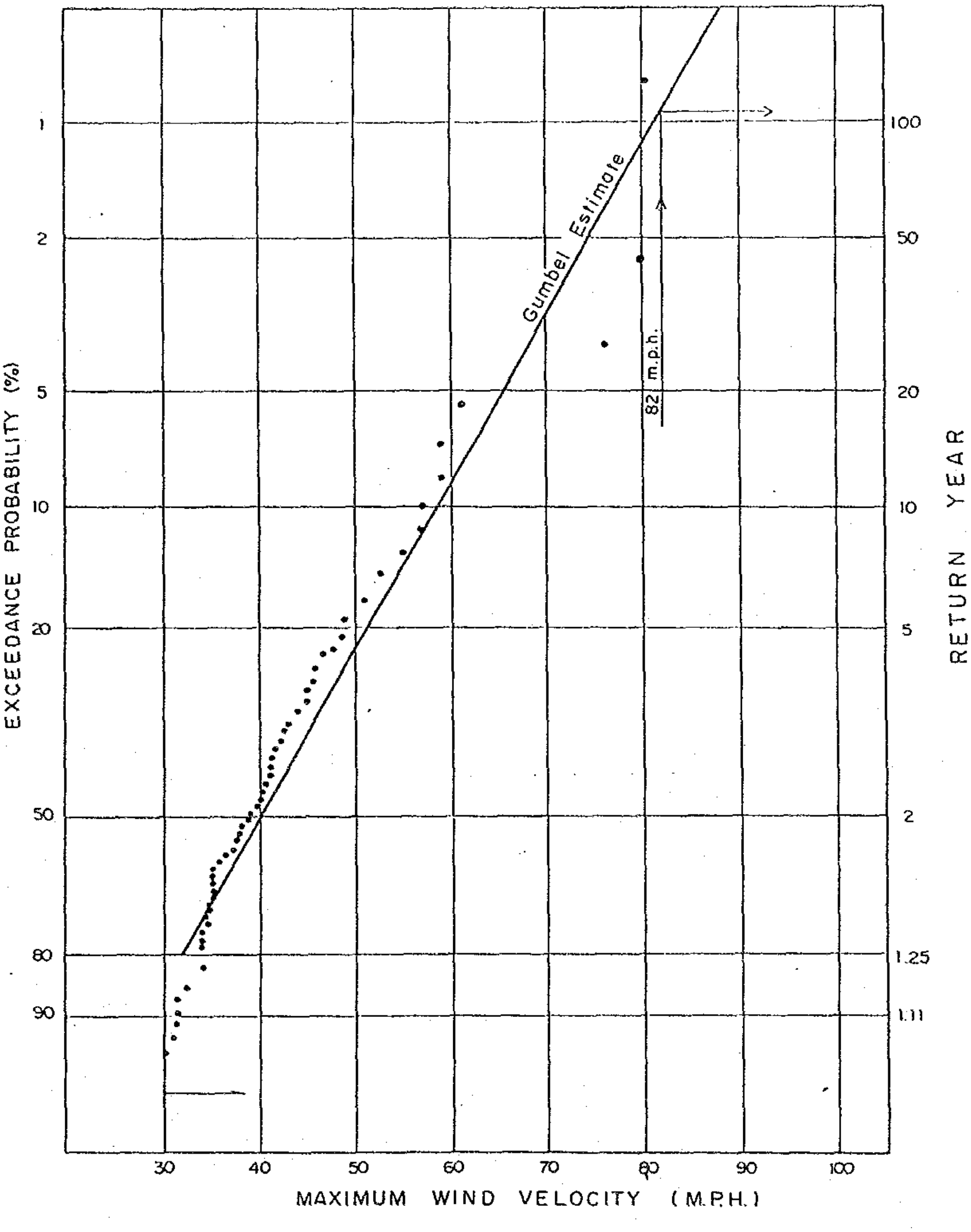


LOCATION MAP OF WATER LEVEL GAUGING STATION AND SELECTED RAINFALL STATION

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

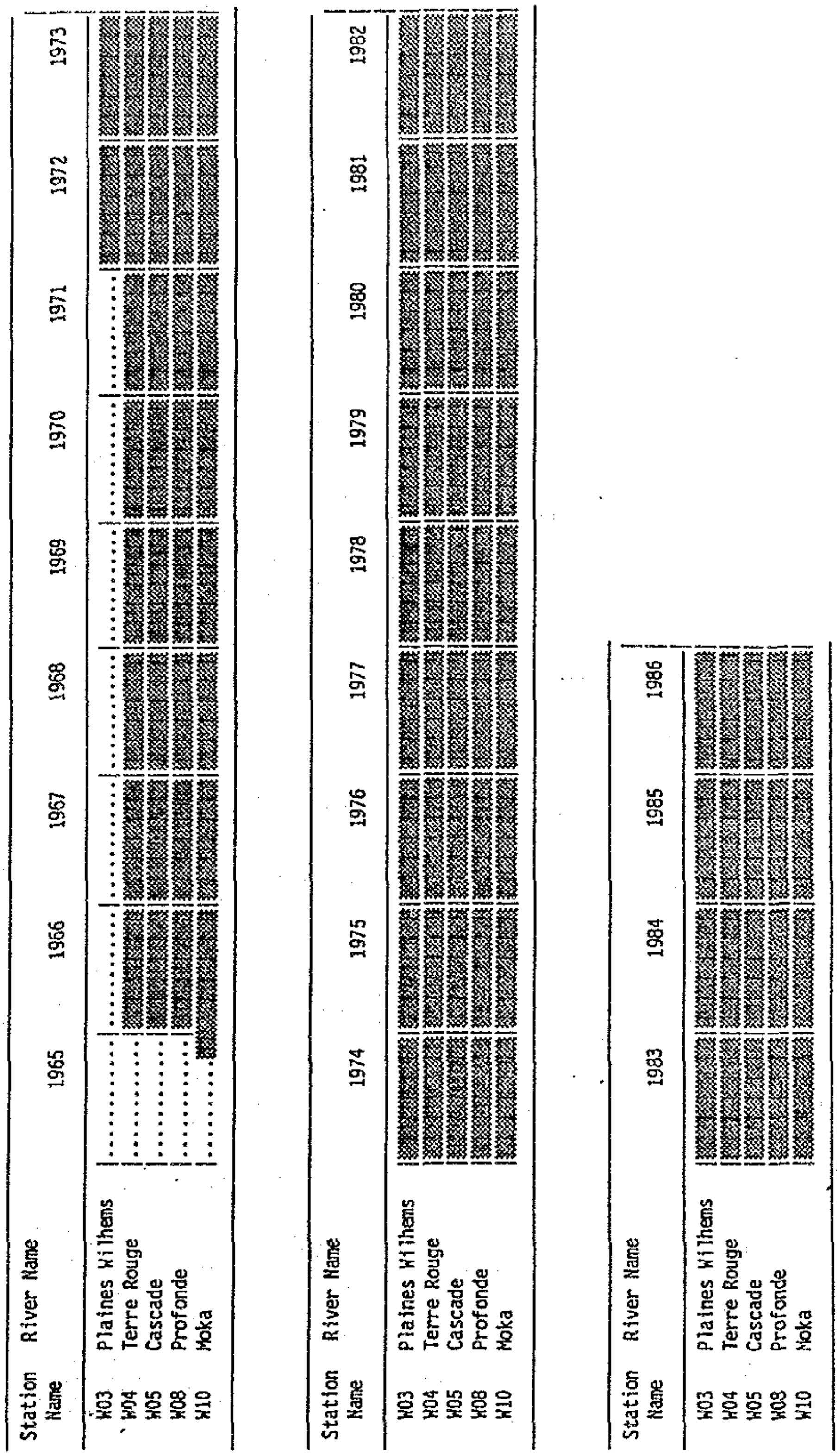
Fig. 2.4.4



Annual highest wind speed of 30 MPH and above over a whole hour

PROBABILITY OF ANNUAL MAXIMUM WIND SPEED	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.4.5

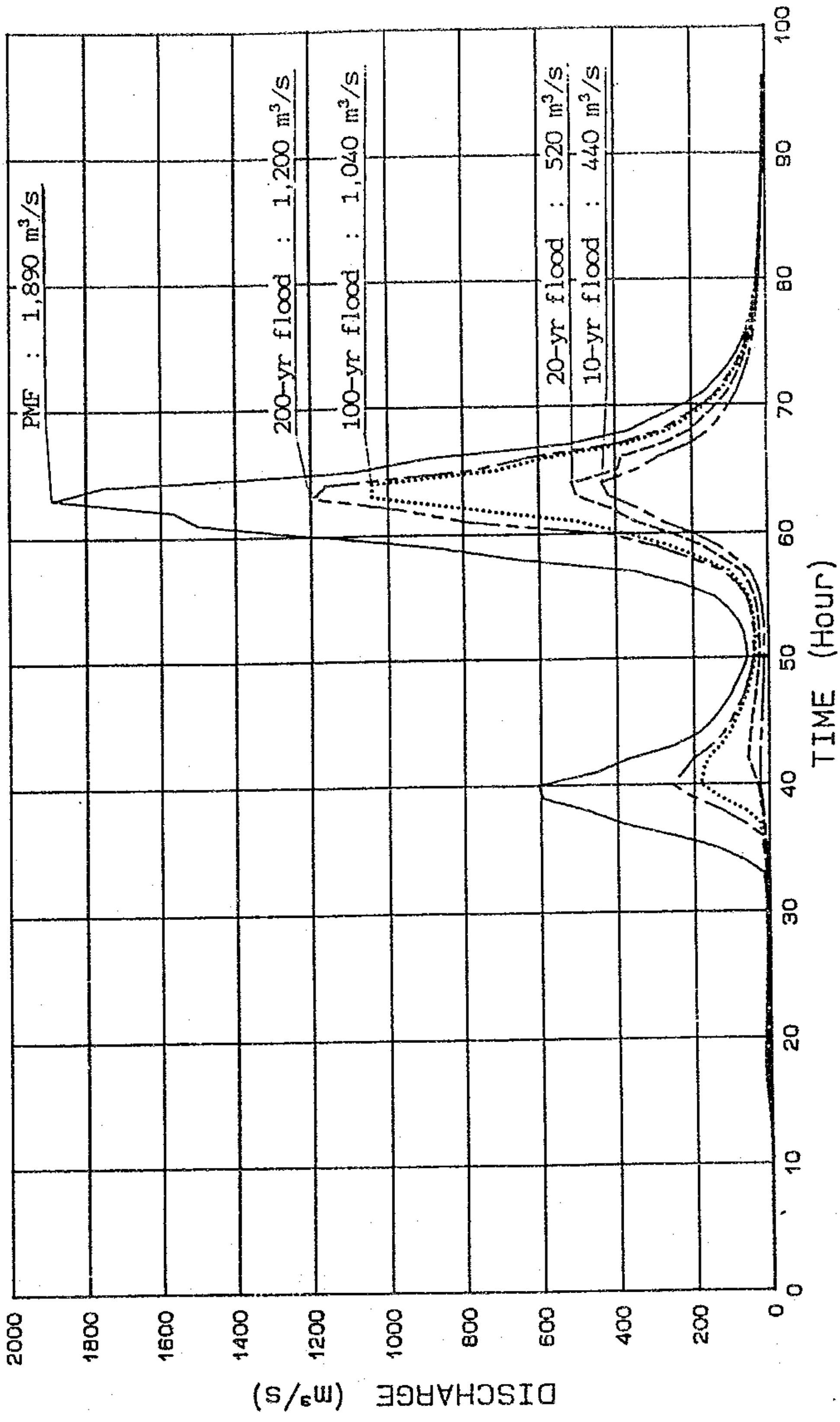


DURATION OF RECORD OF WATER LEVEL GAUGING STATIONS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.4.6



PROBABLE FLOOD HYDROGRAPH

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

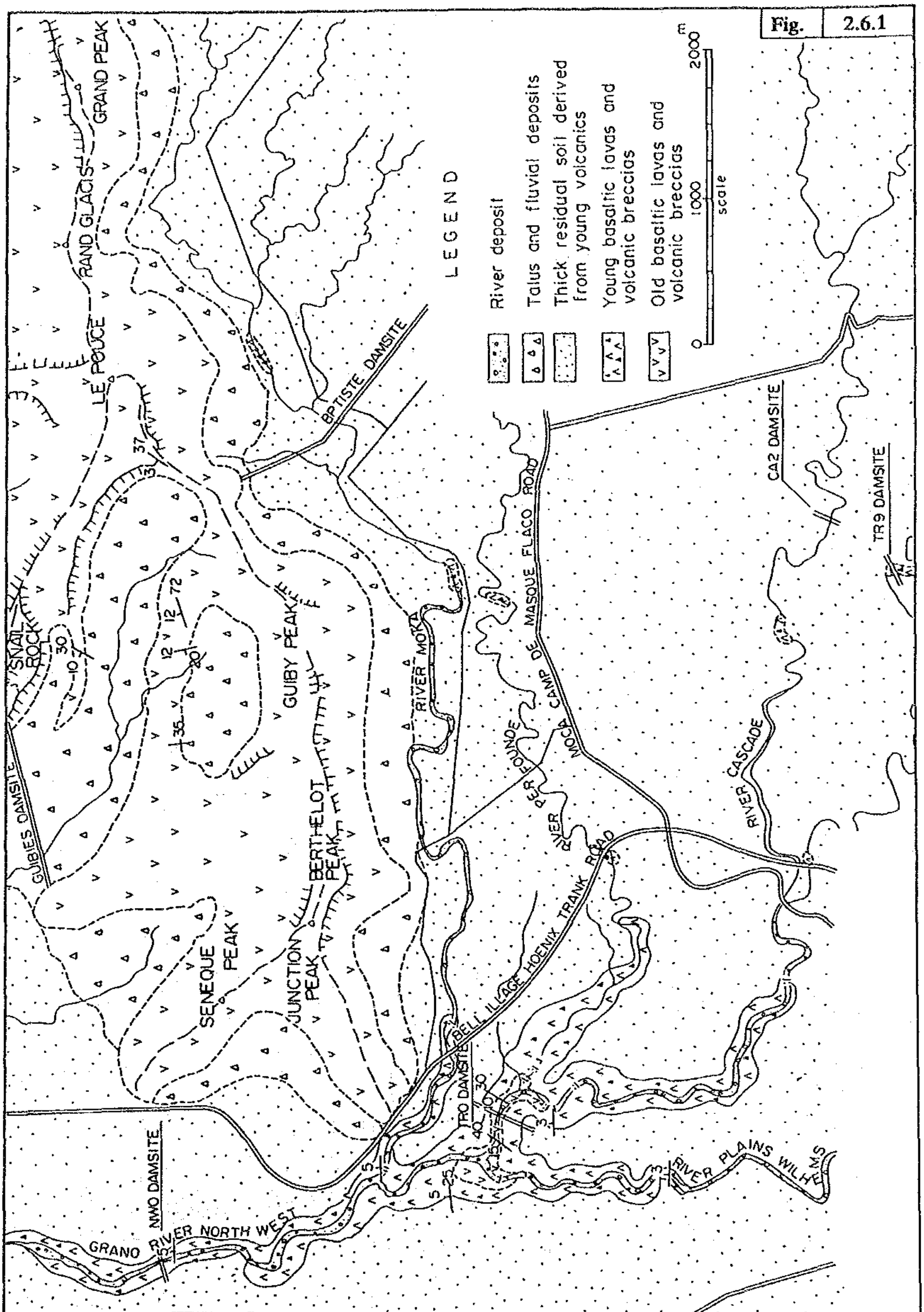


Fig. 2.6.1

REGIONAL GEOLOGICAL MAP

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

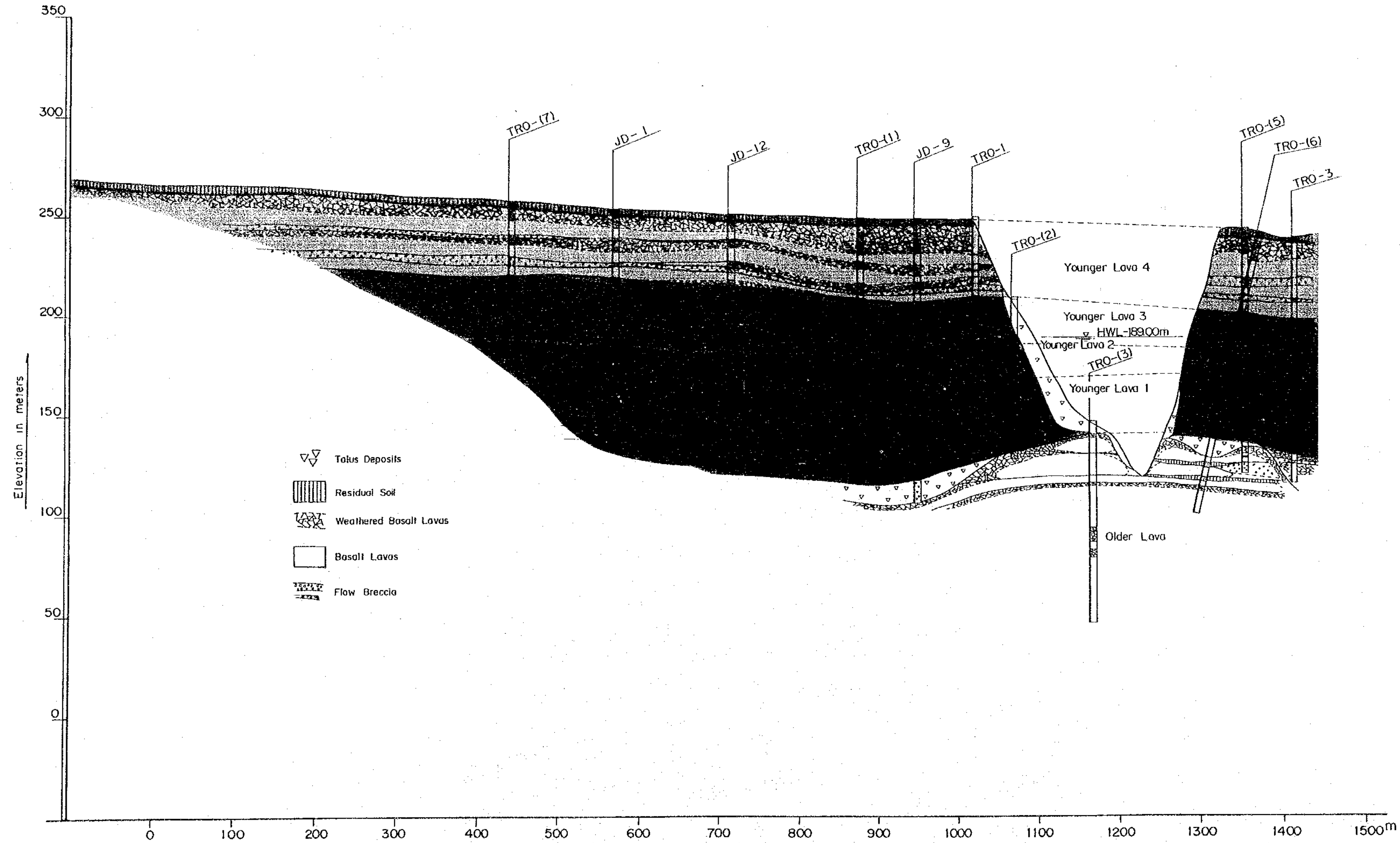
Fig. 2.6.2

GENERAL
GEOLOGICAL PLAN AT DAM SITE

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

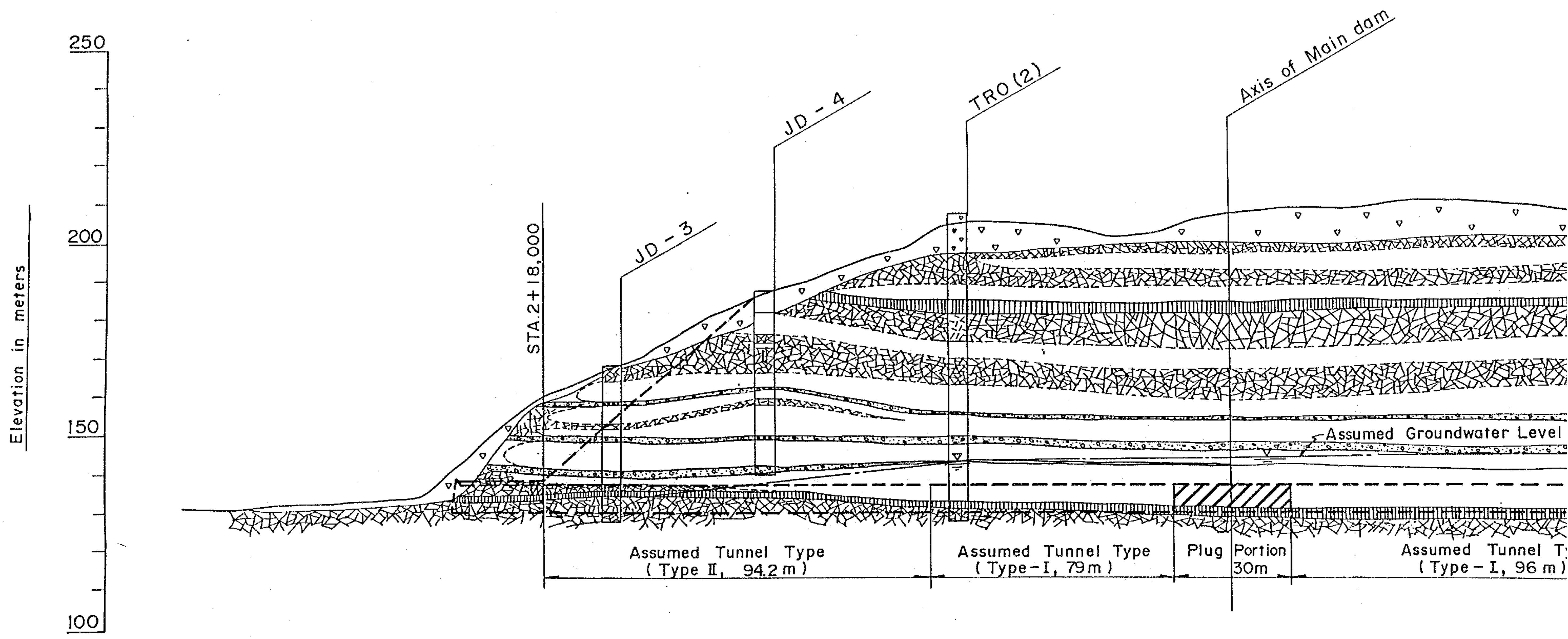
Fig. 2.6.3



GENERAL
GEOLOGICAL SECTION AT
DAM SITE

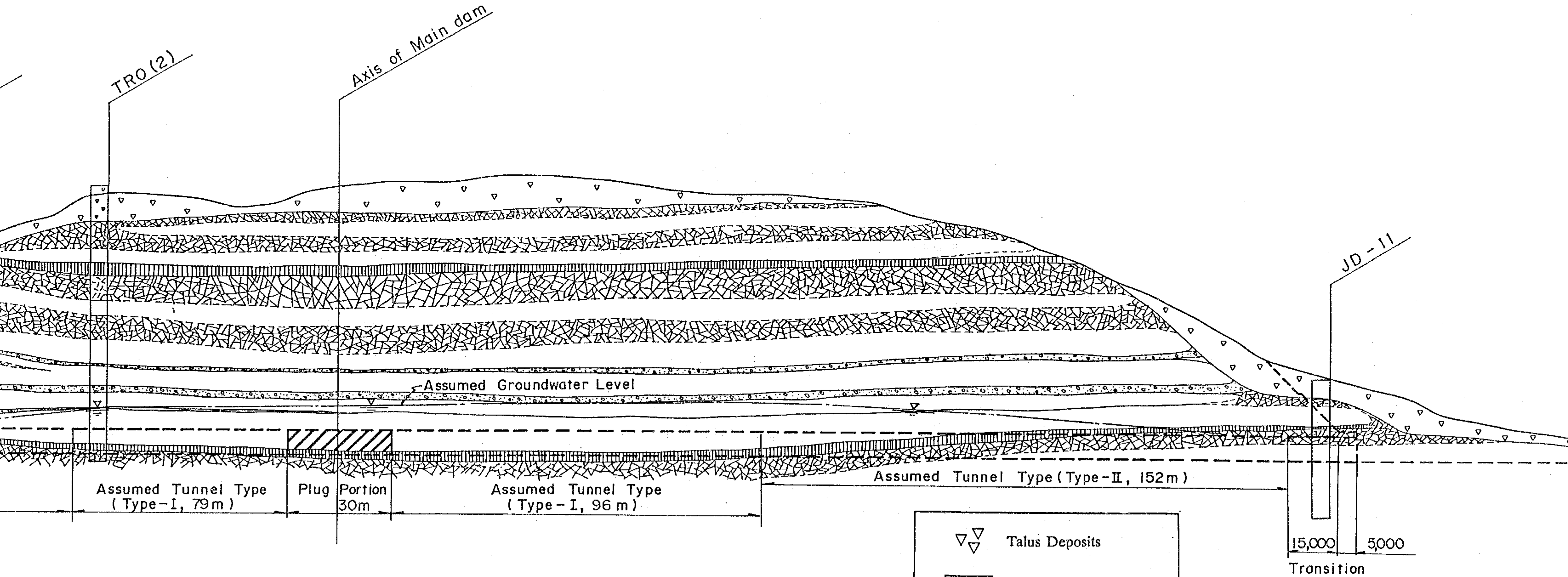
GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

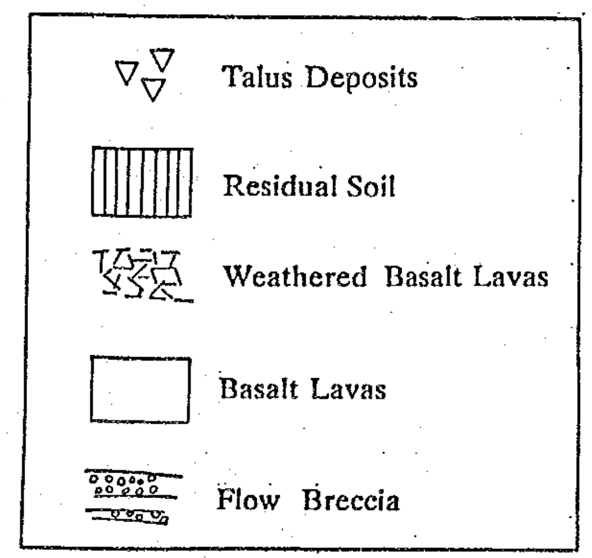
JAPAN INTERNATIONAL COOPERATION AGENCY



Notes : Application of the tunnel type shown on this drawing is only for refrence.

See notes on Dwg. NO. D-001.





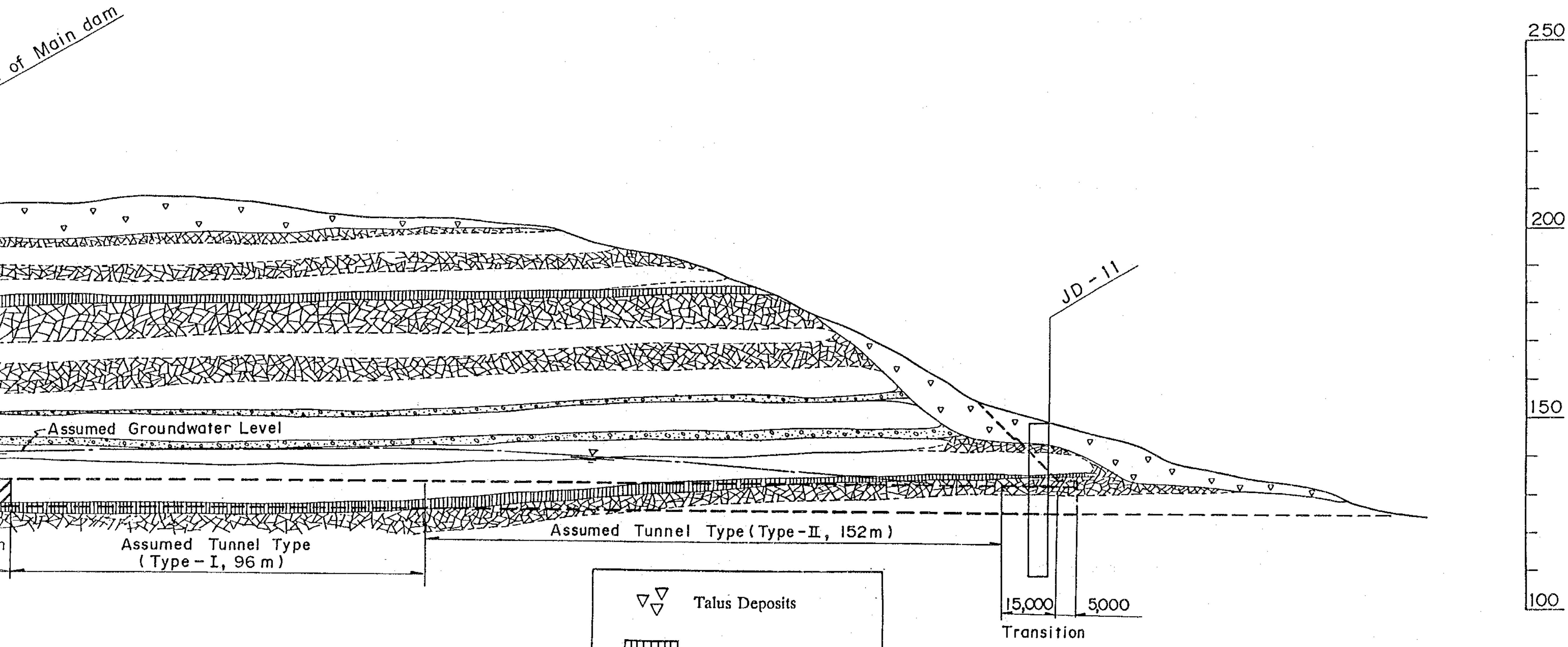
Notes : Application of the tunnel type shown on this drawing is only for refrence.
See notes on Dwg. NO. D-001.

SCALE 0

(1:

GENERAL
GEOLOGICAL PROFILE, DIVERSION TUNNEL

Fig. 2.6.4



shown on this

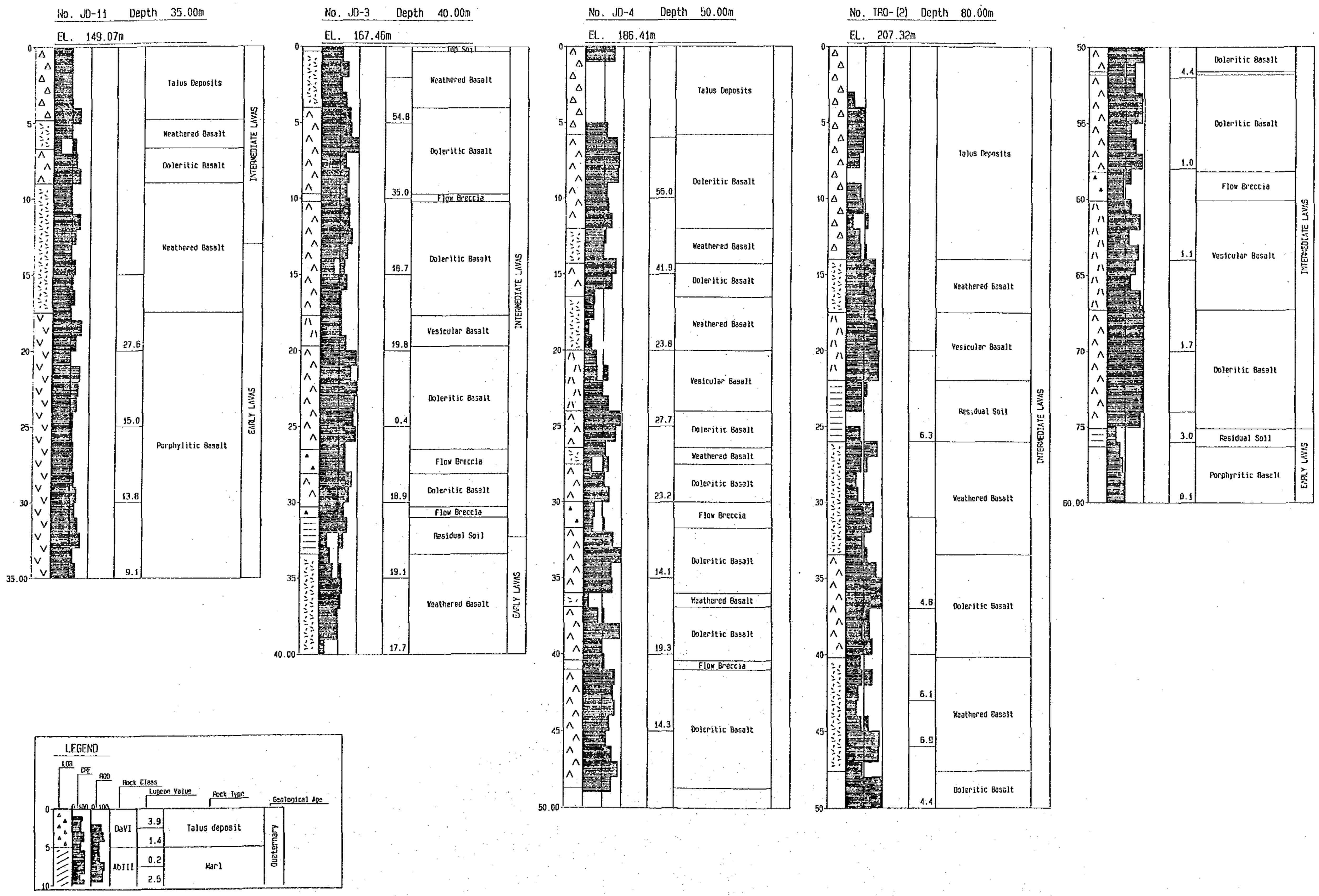
▽▽	Talus Deposits
	Residual Soil
	Weathered Basalt Lavas
	Basalt Lavas
	Flow Breccia

SCALE 0 50 100m
(1 : 1,000)

GENERAL
GEOLOGICAL PROFILE, DIVERSION TUNNEL

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT
JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.6.5



GENERAL
SUMMARY OF BORING LOGS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

CHAPTER III. DESIGN FOR LOT-I COMPONENTS

3.1 General

The Project is intended to be implemented by dividing into three (3) Lots in view of an effective implementation of the Project works.

The components included in Lot-I are (i) the diversion tunnel, and (ii) preparatory works such as the haul road, access roads, various buildings, water and power supply systems, aggregate plant and concrete batcher plants, etc., which will be implemented in advance of Lot-II and Lot-III works. It is noted that the cofferdams including the primary cofferdam are included in Lot-II in view that the cofferdams form a part of main dam. This Chapter III presents design concepts on the above Lot-I components.

3.2 River Diversion Scheme

3.2.1 Layout

The river diversion system consists of upstream and downstream cofferdams and a diversion tunnel. Other kinds of river diversion system are not conceivable in the case of the Project in view of the damsite conditions.

It is more desirable from the economic aspect that the diversion tunnel be located in the left bank of the damsite: that is, the topographic condition of the damsite makes the necessary length of diversion tunnel shorter. Further, taking into consideration that the present dam design considers a future expansion, the diversion tunnel outlet should be located in the neighbouring Plaines Wilhem river. By this arrangement of tunnel layout, the tunnel length is not affected by the future expansion of dam in mind.

River diversion is designed taking into consideration the followings:

1) utilization for water supply and river outlet
2) hydrological characteristics of the basin
3) topographic and geological conditions
4) dam type
5) other appurtenant facilities
6) flood damage by shortage of design capacity of diversion tunnel

7) construction period of cofferdam and main dam
8) future implementation of hydropower project

River water will be diverted through the diversion tunnel during the construction period from the topographic condition of gorge. A lane of diversion tunnel will be constructed under the left bank because tunnel length is 75 m shorter than that under the right bank. The length of the tunnel is 499 m. The size of the diversion tunnel is determined in combination of the height of upper cofferdam which should be constructed in one dry season for 7 months from May to November. However, the cofferdam will be constructed as a part of main dam, that is, the size of cofferdam does not influence on the project cost. Therefore, it is economical to select smallest size of the diversion tunnel. As explained in the Section 3.3.3, the diameter is determined at 6.8 m finally to satisfy the necessary conditions that the velocity of the tunnel should be less than 15 m/s. The design flood of 520 m^3/s is released by pressure flow.

Diversion inlet is located at left bank toe and the invert level is determined at El. 129 m, which is river bed elevation of the Terre Rouge river, to avoid dead water. The river profile and cross section near the inlet is shown in Fig. 3.2.1. Diversion outlet is located at the right bank of the Plaines Wilhems river. The river profile and cross section near the outlet is shown in Fig. 3.2.2. The location of the outlet was studied in four alternatives and the alternative 1 was selected from the following reasons: (ref. Fig. 3.2.3)

1) water can be discharged much more than other alternatives by free flow,
2) supercritical slope is adopted to avoid back water from downstream while alternative 3 and 4 are subcritical slope,
3) cofferdam volume is the smallest in the alternatives, and
4) the cost difference is very minor among the alternatives.

The outlet invert level is set at El. 125 m. The curve with a radius of 200 m is horizontally aligned based on the standard that the radius should be longer than 10 times tunnel diameter and with consideration of concrete lining work by a sliding form. The design of the river diversion are shown in Figs. 3.2.22 to 3.2.33.

The slope of the diversion tunnel is 1/112.8 which is supercritical slope up to the discharge of 350 m^3/s. The supercritical slope is adopted in order to release discharge as much as possible by small flow section. The section of circular type is selected because of high pressure tunnel. The edge of entrance portion is rounded to reduce entrance loss.

3.2.2 Determination of Diversion Design Flood

The standard mentions that river diversion is generally designed against recorded maximum flood or 10 to 20 year probable flood for rockfill dam. Damage by flood overtopping dam has not been recorded for design discharge exceeding 20 year probable flood since 1958. In the south of Japan where typhoon attacks frequently, 20 year probable flood is adopted as design discharge of river diversion in many projects of fill type dam. Specific discharge in the range of catchment area between 20 km^2 and 100 km^2 is 10 $m^3/s/km^2$ at most.

In this project, the catchment area at the damsite is 54.9 km^2 and the specific discharge of 20 year probable flood, 520 m^3/s, is 9.5 $m^3/s/km^2$. Recorded maximum flood, which was caused by the cyclone Gervaise in 6 February 1975, is estimated at 450 m^3/s at the damsite in 26 years from 1965 to 1990. It satisfies the design standards in the south of Japan mentioned above. Flood characteristics in Mauritius is similar to that in the south of Japan, that is, extraordinary flood is brought about by storm rainfall or cyclone (typhoon).

Therefore, 20 year probable flood of 520 m^3/s is adopted as the design discharge of river diversion. In the second rainy season of dam construction, it would be safe against 30 year probable flood of 670 m^3/s since the dam embankment can be arranged to reach up to the crest El. 165 m in the upstream portion. Main dam will be completed before the third rainy season.

In order to evaluate this proposed river diversion scheme, risk of damage by the flood exceeding some design floods was studied. The analysis is as follows:

1) Unit cost of dam works (Cd) and diversion works (Ct) is estimated as follows:

$$Cd = \text{cost of dam works/dam volume}$$
$$= (20{,}700 \times 13.7 + 71{,}300) \times 1{,}000/1{,}485{,}000 = \text{Rs. } 240/m^3$$

$$Ct = \text{cost of diversion works/(tunnel length x area)}$$
$$= (3{,}900 \times 13.7 + 19{,}500) \times 1{,}000/(470 \times 32.1536) = \text{Rs. } 4{,}826/m^3$$

2) Three (3) cases of 5 year, 10 year and 20 year probable floods as the design flood of the diversion tunnel are taken up for evaluating the proposed river diversion scheme. The relation among the design flood, tunnel diameter and cofferdam is as shown below.

Return Period (year)	Design Flood (m^3/s)	Tunnel Diameter (m)	Cofferdam Crest El. (m)	Cofferdam Volume (1,000 m^3)
5	200	4.4	156.4	129
10	440	6.2	157.1	136
20	520	6.8	155.5	119

3) The cost of diversion tunnel of which the length is 499 m and cofferdam for respective design flood is as shown below.

Return Period (year)	Design Flood (m^3/s)	Tunnel Diameter (m)	Diversion Cost (Rs. 1,000)	Cofferdam Cost (Rs. 1,000)
5	200	4.4	37,772	30,960
10	440	6.2	74,998	32,640
20	520	6.8	90,216	28,560

4) Assuming that two third of dam embankment is completed in the second year of the dam construction, the accumulated cost of dam including the cofferdam for the second year and third year, that is completion year, is estimated as follows:

Second year : 1,485,000 m^3 x 2/3 x Rs. 240/m^3 = Rs. 237,600,000

Third year : 1,485,000 m^3 x Rs. 240/m^3 = Rs. 356,400,000

5) It is expected that the main dam and appurtenant facilities will be completed for 3 years, 3 dry seasons and 2 rainy seasons. The probability (P) exceeding probable flood during construction period is expressed as follows:

$$P = 1 - (1 - 1/T)^N$$

T : recurrence period of flood

N : construction period (year)

P (%)

N	T 2	5	10	20	25	50	100
1	50.0	20.0	10.0	5.0	4.0	2.0	1.0
2	75.0	36.0	19.0	9.7	7.8	4.0	2.0
3	87.5	48.8	27.1	14.3	11.5	5.9	3.0
4	93.7	59.0	34.4	18.5	15.1	7.8	4.0

6) Assuming that cost of dam works excluding diversion works is lost entirely by the excess flood in stead of no account about other damage, expected value of the flood damage (Vd) is estimated as follows:

Vd = exceedance probability x cost of dam works
= 0.05 x 28,560,000 = Rs. 1,428,000 for example of 20 year flood.

The expenditure of diversion and dam works and the expected value of the damage in the respective year are derived as shown below.

Unit: Rs. 1,000

Year	Return Period	Alt. 1 Cost	Alt. 1 Damage	Return Period	Alt. 2 Cost	Alt. 2 Damage	Return Period	Alt. 3 Cost	Alt. 3 Damage
1	5	68,732	6,192	10	107,638	3,264	20	118,776	1,428
2	8	206,640	55,598	22	204,960	21,146	30	209,040	15,682
3	10,000	118,800	35	10,000	118,800	35	10,000	118,800	35
Total		394,172	61,825		429,888	24,445		446,616	17,145
		455,997			454,333			463,761	

Return period means that of the flood against which the safety is secured by the progress of dam works. The cost of first year consists of those of cofferdam and diversion tunnel.

The study for evaluating the proposed river diversion scheme indicates that the most economical point appears at Alt. 2 (that is, a diversion scheme to handle 10 year probable flood). However, considering such other aspects that this kind of trouble should be minimized, the proposed scheme (Alt. 3) is evaluated to be justifiable.

3.2.3 Determination of Tunnel Diameter and Cofferdam Height

(1) Hydraulic analysis

Design discharge of the diversion tunnel is 520 m^3/s which is equivalent to 20 year probable flood. This discharge will be diverted to the Plaines Wilhems river through the diversion tunnel as pressure flow. Hydraulic calculation of the diversion tunnel at a reservoir water stage was made under the following condition:

Shape	:	circle
Diameter	:	6.8 m
Length of tunnel	:	499m
Inlet invert level	:	El. 129 m
Outlet invert level	:	El. 125 m
Slope	:	1/112.8
Manning's roughness coefficient, n	:	0.014 (lining concrete)

(i) Free flow

The slope of diversion tunnel is 1/112.8. This is the critical slope of the discharge of 350 m3/s and the critical depth is 6.3 m (D/r = 1.85). The discharge less than critical discharge is controlled at the inlet and the water depth along the tunnel decreases to uniform flow water depth from the critical water depth at the inlet. The discharge is calculated by the equation:

$$Q = \sqrt{gA_c^3/T_c}$$

$$A_c = D^2/4\ (s - \sin s \cos s)$$
$$T_c = D \sin s$$

(ii) Pressure flow

Total head is determined by the equation:

$$H_T = he + hf + hv$$
$$= (fe + f + 1)\frac{Q^2}{2gA^2}$$

where fe = entrance loss coefficient, 0.2 for rounded inlet

f = friction loss coefficient, $124.5\ n^2L/D^{4/3}$

Therefore the discharge in pressure flow is calculated by the equation:

$$Q = A\sqrt{2gH_T/(0.2 + 124.5\, n^2 L/D^{4/3} + 1)}$$

The reservoir water level is derived by the equation:

$$WL = \text{Outlet invert level (El. 125 m)} + \text{Diameter (6.8 m)} + H_T$$

Discharge curve of the diversion tunnel is shown in Fig. 3.2.4 in case of free flow and pressure flow. Reservoir water level at free flow is derived by adding velocity head to the inlet water level.

Hydraulic conditions in the river diversion system for the design flood are shown in Fig. 3.2.6 and 3.2.7.

(2) Selection of tunnel diameter and cofferdam height

The cofferdam consists of primary cofferdam and main cofferdam. The primary cofferdam will be embanked at the confluence of the Terre Rouge river and the Profonde river. It will be embanked as fill type with slopes of 1:2.0 for the both sides. The height of primary cofferdam is determined at 5 m to divert discharge in dry season for the main cofferdam works. The crest elevation of the primary cofferdam is El. 134 m.

The main cofferdam will be constructed as a part of the main dam. It should be completed before rainy season. The volume curve of cofferdam with construction period is presented in Fig. 3.2.5. The relationship between the cofferdam and diversion tunnel is as shown below:

Diameter	(m)	6.8	7.0	7.2	7.4	8.0
Flood water level	(m)	154.5	151.9	149.7	147.8	142.7
Velocity	(m/s)	14.3	13.5	12.8	12.1	11.1
Crest El. of cofferdam	(m)	155.5	152.9	150.7	148.8	143.7
Height	(m)	28.5	25.9	23.7	21.8	16.7
Cofferdam volume	(m^3)	119,000	98,000	82,000	71,000	35,000
Construction period	(month)	4	3.2	2.8	2.5	1.3

Note: Crest El. is derived adding 1 m to flood water level as freeboard. Diameter of 8 m is free flow type.

From the economical point of view, the diameter of the diversion tunnel and crest elevation of main cofferdam are determined at 6.8 m and El. 155.5 m respectively. The cofferdam is rockfill type with center core. The side slopes of the upstream side and downstream side are 1:2.3 and 1:1.8. The cofferdam slopes of the upstream side or both sides should be reinforced by some appropriate method against overtop of the extraordinary flood exceeding the design flood.

The downstream cofferdam should be constructed to prevent backwater. It will be constructed before the conjunction of the Terre Rouge river and the Plaines Wilhems river. The height is estimated at 5 m taking into account the water depth for 20 year probable flood.

3.2.4 Structural Design

3.2.4.1 Design Concepts

Major design concepts in the structural design of the diversion tunnel are as follows:

(a) The excavation for diversion tunnel inlet is closely related to the excavation work for the water intake which will be done by Lot-II. It is not desirable that the secondary excavation in the tunnel inlet portion be made for the water intake by Lot-II. Then, the excavation in the tunnel inlet portion by Lot-I is made not to cause the mentioned secondary excavation by Lot-II.

(b) As for the excavation above El. 196 m, the excavation will overlap with that of the spillway by Lot-II. Therefore, the portion above El. 196 m is excavated with a temporary cutting slope of 1 to 0.5 without surface protection.

The cutting slope of the portion below El. 196 m should be permanent one. Thus, the excavation is made with the cutting slope of 1:1.0 and 2 m wide berm at an interval of 10 m in height. This cutting surface has a slope of 1 to 1.2 (about 40 degree) in average which coincides with the necessary slope for the water intake structure. The surface protection is considered necessary, provided with the shotcrete (5 cm x 2) as well as grouted anchor bars of D25.

Here, it is noted that the part where the water intake structure is to be constructed will be protected without anchor bars, since some foundation excavation for the structure will be required by Lot-II.

(c) The transition in the tunnel inlet is rearranged to be provided in the inlet portal (not in the tunnel portion) in consideration of more complicated excavation and support works in the case that it is provided in the tunnel portion.

(d) The standard mentions that the radius of curvature in tunnel should be more than 10 times tunnel diameter for ensuring a smooth hydraulic condition (D = 6.8 m).

However, the radius of curvature at the curve of the tunnel is provided with R = 200 m, which is much more than 10 times tunnel diameter, in view that the tunnel lining will be made by using a sliding form.

(e) The standard mentions that it is desirable to provide a length more than 20 times tunnel diameter for the distance from the end of tunnel curve to the outlet.

It is difficult to satisfy the above standard in the case of the Project. The present design provides 45 m for the above distance.

Instead, a concrete slab of 10 m long and 1.0 m in thickness is provided in immediate downstream of the outlet portal to prevent possible erosion due to the shorter length from the end of the curve as mentioned.

(f) The tunnel outlet is aligned to meet the Plaines Wilhems river with a large angle. It is considered necessary to protect the opposite bank from the erosion. The opposite bank is provided with gabion protection for the above purpose.

(g) The tunnel invert is set at El. 129.0 m. The level of the approach channel is lowered by 0.3 m to El. 128.7 m. The intention is to protect the tunnel lining from damages due to boulders to discharge into the tunnel. Further, the approach channel in front of the inlet will be protected from possible erosion with a concrete slab.

(h) A detailed examination for necessary plug length finds that its length can be reduced from 45 m in the basic design to 30 m as follows:

In the equation to examine the necessary length of plug (Refer to Section 4.4.6 of Basic Design Report), an actual shearing strength of about $t = 200\ t/m^2$ can be applied, obtaining a calculation result of about 11.0 m based on the above equation.

However, the standard specifies that a length of 0.3 to 0.5 times maximum water head should be provided in consideration of a complete stoppage of seepage water, resulting in the plug length of 30 m (78 m x 0.4 ≒ 30 m).

(i) The keys will be provided in the plug portion so that the concrete lining is not necessary to be broken for plug concreting.

(j) Drains holes will be provided in the lining of the tunnel downstream of the plug so that the lining will not be subject to the external water pressure.

(k) The curtain grouting is provided for the water stop by two lines at the dam axis.

(l) The consolidation grouting is provided for the tunnel upstream of the plug as well as the plug portion. The tunnel downstream of the plug is, in principle, considered not to require the consolidation grouting.

3.2.4.2 Structural Analysis

(1) General descriptions

The structural analyses are largely divided into three of the tunnel, inlet portal and outlet portal portions.

The whole tunnel will be lined with concrete. Two types of tunnel lining are applied in accordance with geological conditions: that is, Type I (50 cm in lining thickness) will be applied for $C_M \sim C_H$ class of rock. Type II (80 cm in lining thickness) will be applied for $C_L \sim C_M$ class of rock. The diversion tunnel is a permanent structure which is intended to be used for the waterway for water supply, and therefore, all the concrete linings shall be the reinforced concrete structure. Principal features of both tunnel types are given in Table 3.2.1.

Design loads for the analyses are as shown in Table 3.2.2.

The maximum internal and external loads which will determine the design of the structures are the water pressure to act after impounding the reservoir (water level in the dam expansion scheme). As for the tunnel upstream of the plug where the internal and external water pressures are balanced in the normal condition, the following extreme loading conditions have to be taken into consideration: (i) the external water pressure due to the remaining groundwater when the reservoir water level will suddenly drawdown, and (ii) the internal water pressure to act to the lining before the external water pressure will act to the lining. These extreme cases of loadings are rarely caused temporarily, and therefore, an allowable stress increased by 65% is applied in accordance with the standard. With regard to the loadings during the river diversion, particular examinations are considered unnecessary since its loading is very small as compared with the mentioned extreme loading cases even though the increase of the allowable stress is taken into consideration in the extreme cases.

In consideration that the internal and external water pressure will be balanced in the usual condition, application of consolidation grout pressure of 2 kg/cm^2 is considered sufficient. The grout pressure of backfill grout is 2kg/cm^2 at maximum which is same as or less than the consolidation grout pressure. Hence, a particular examination for backfill grout pressure is omitted. In consideration that the grout pressure is also very tentative, the allowable stress is increased to 210 t/m^2 in concrete compressive stress and 18 kg/cm^2 in concrete shearing.

Concrete and steel properties applied are presented in Table 3.2.3.

Rock loads will be supported with the tunnel supports erected during tunnel excavation. Thus, no rock loads are considered to be imposed on the concrete lining. Dead loads are also neglected because it is negligibly small compared with others.

The Otto-Frey-Bear's theory is applied for analyzing the tunnel structure against the internal and external loadings. The cylindrical shall theory is applied for examination against the grout pressure.

As for the inlet portal and outlet portal, the structures are examined by the frame structural analyses.

(2) Analysis for tunnel

Structural calculation is made on the basis of the following methods and assumption.

Marks used in the calculation

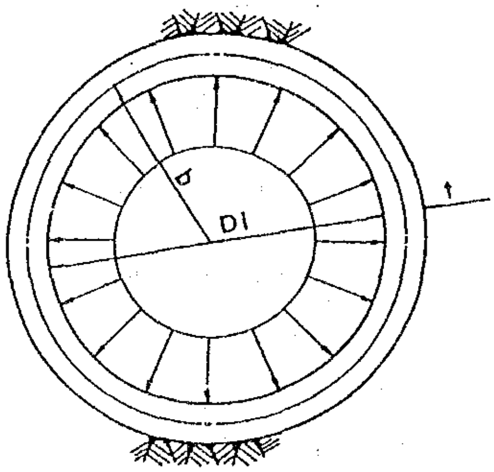
- D_i : internal diameter of tunnel
- t : thickness of concrete lining
- b : radius of neutral axis of concrete lining (= r + t/z)
- c : radius of external axis of concrete lining (= r + t)

P_i : internal water pressure
P_e : external water pressure
P_g : Grouting pressure
E_r : Elastic modulus of rock
E_s : Elastic modulus of reinforcement bar
E_c : Elastic modulus of concrete
n_c : Poisson's ratio of concrete
n_r : Poisson's ratio of rock
m_c : Poisson's number of concrete
m_r : Poisson's number of rock
A_s : sectional area of reinforcement bar

a) Stress due to internal water pressure

For the calculation of stress due to internal water pressure, Otto-Frey-Bear's method is applied with the under conditions:

- Rock around tunnel lining exists infinitely,
- Concrete and rock are homogeneous and isotropic, and
- boundary between concrete lining and rock is continuous.



When the lining concrete is not reinforced by steel bar, according to Otto-Frey-Bear's method,

$$\sigma_t^c = Pi \times \left\{ \frac{\lambda(\frac{1}{2} + \frac{t}{Di})^2 - \frac{1}{4}}{\frac{t}{Di}(1 + \frac{t}{Di})} + \left(\frac{1}{4(\frac{b}{Di})^2} \times \frac{(\lambda - 1)(\frac{1}{2} + \frac{t}{Di})^2}{\frac{t'}{Di}(1 + \frac{t}{Di})} \right) \right\}$$

where,

σ_t^c = stress in concrete in tangential direction

$$\lambda = \frac{Pc}{Pi} = \left\{\frac{1}{2 \times \frac{t}{Di} \times (1 + \frac{t}{Di})}\right\} / \left\{\frac{m_r' + 1}{m_r'(\frac{E_r'}{E_c'})} + \frac{(m_c' - 1)(\frac{1}{2} + \frac{t}{Di})^2 + \frac{1}{4}(m_c' + 1)}{m_c' \times \frac{t}{Di}(1 + \frac{t}{Di})}\right\}$$

where,

$$m_r' = m_r - 1$$
$$m_c' = m_c - 1$$
$$E_r' = \frac{m_r^2}{m_r^2 - 1}$$
$$E_c' = \frac{m_c^2}{m_c^2 - 1} E_c$$
$$b = (Di + t)/2$$

When the lining concrete is reinforced by steel bar, stress analysis is conducted by extending the above equations.

$$\sigma_t^s = Pi \frac{1}{2(\frac{As}{Di})}\left[\frac{1}{1 + 4.6E_s(\frac{As}{Di})\left\{\frac{\log 2}{E_c'} + \frac{m_r' + 1}{2.3m_r' \times E_r'} + \frac{1}{E_c'}\log(0.5 + \frac{t}{Di})\right\}^{-1}}\right]$$

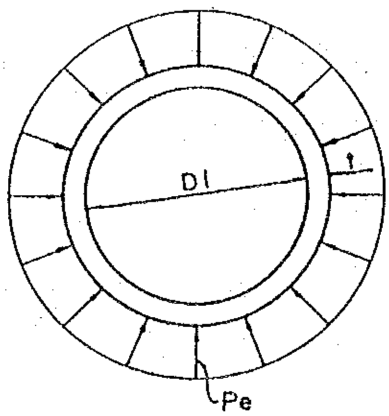
where;

σ_t^s = tensile stress in reinforced bar in tangential direction

b) Stress due to external water pressure

External water pressure is assumed to act as an uniform load along the concrete lining.

The compressive stress which occurs in tunnel lining is obtained from Otto-Frey-Bear's method.



$$\sigma_c = Pe\left[\frac{X}{X - \frac{m_c' - 1}{m_c' + 1} \times \frac{i}{j^2} \times Y} + \frac{Y}{\frac{m_c' + 1}{m_c' - 1} \times X - \frac{i}{j^2} \times Y}\right]$$

where;

$$X = \frac{1}{2Es \times \frac{As}{Di}} \times \frac{m_c' \times E_c'}{m_c' + 1} + 1$$

$$Y = \frac{1}{2Es \times \frac{As}{Di}} \times \frac{m_c' \times E_c'}{m_c' - 1} - 1$$

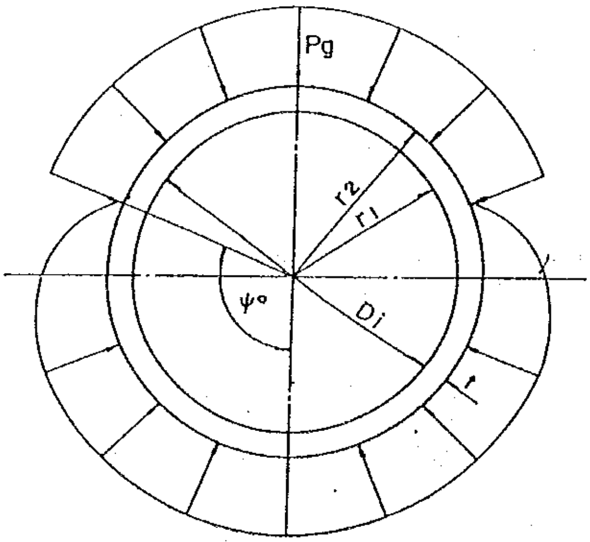
$$j = 1 + 2 \times \frac{t}{Di}$$

σ_c = compressive stress in concrete

c) Stress due to grouting pressure

The theory of thin cylindrical shell is applied to calculate the stress due to grouting pressure and calculation is executed as follows.

This calculation is obtained from the following equation by trial and error method.



The angle of boundary point is expressed by ψ°

$$\{\frac{3\bar{a}^2}{\eta} - \frac{b}{a}(\eta - 3) + \alpha(\eta - 3)(\psi_o - \pi)\} \sin\psi_o \cos\psi_o +$$

$$2\{\frac{\bar{a}}{\eta}(2 - \eta) - \bar{b}\,\bar{\eta} + (\eta - 1)(\psi_o - \pi)\} \sin^2\psi_o +$$

$$\{\bar{b}\pi\alpha - \frac{\bar{a}^2}{\eta} - 2\eta + \bar{a}\beta(\psi_o - \pi)\}(\psi_o - \pi) = 0$$

where;

$$\bar{a} = \sqrt{\eta^2 - 1}$$

$$\eta^2 = 1 + \frac{3Er}{4Ec} \times \frac{1 + \frac{t}{Di}}{\frac{1}{2} + \frac{t}{Di}} \left\{1 + \frac{1}{\frac{t}{Di}}\right\}^3 \times \frac{1 - \nu c^2}{1 + \nu r}$$

$$\alpha = \sqrt{\frac{\eta - 1}{2}}$$

$$\beta = \sqrt{\frac{\eta + 1}{2}}$$

$$\bar{b} = 1 + \frac{1}{3}\{\frac{t}{Di} / (1 + \frac{t}{Di})\}^2 \times \eta^2$$

$$R = \{\alpha(3 - \eta) \sin\psi_o \cos\psi_o - 2\{1 + (\eta - 1)\sin^2\psi_o + \bar{a}\beta(\psi_o - \pi)\}\}\eta$$

$$Q_1 = \alpha(3 + \eta) \sin\psi_o \cos\psi_o + 2\sin^2\psi_o - \bar{a}\beta(\psi_o - \pi)$$

$$Q_2 = \beta(\eta - 3) \sin\psi_o \cos\psi_o + 2\bar{a}\sin^2\psi_o \cos\psi_o - \bar{a}\alpha(\psi_o - \pi)$$

$$\sigma_{1,2} = \frac{N}{t} \pm \frac{6M\psi}{t_2}$$

$$N\psi = \frac{Pg\cdot Di}{2}\{1 - \frac{(\beta Q_2{}^2 - \alpha Q1)}{R \sin\psi_o}\} \cos\psi$$

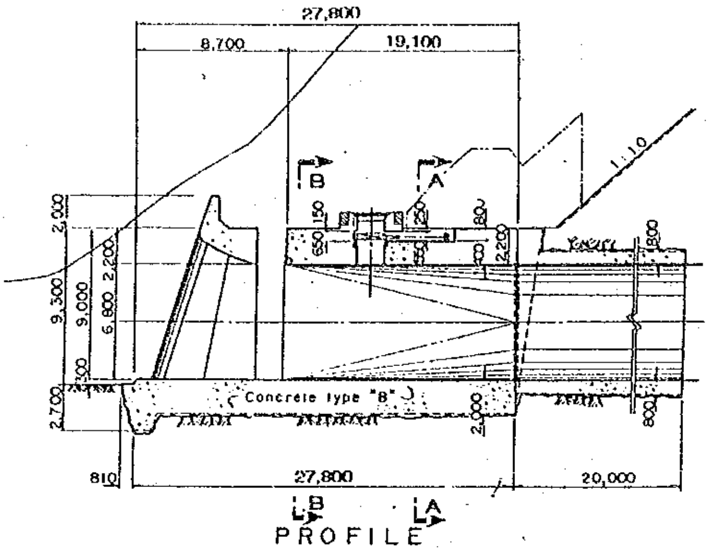
$$M\psi = -\frac{Pg\cdot Di^2}{4}\{-\frac{1}{a}(\frac{\eta^2}{R} Q_2 + \bar{a}) + \frac{\beta Q_2{}^2 - \alpha Q1}{R \sin\psi_o} \times \cos\psi\}$$

d) Result

Table 3.2.5 presents results of the tunnel analysis for internal and external pressures by the Otto-Frey-Bear's theory. Table 3.2.6 presents results of the tunnel analysis for grout pressure by the theory of thin cylindrical shell.

Reinforcement bar arrangements determined through the above analysis and stress in each case are summarized in Table 3.2.4.

(3) Analysis for inlet portal



Sections to be analyzed are shown above (Section A-A and Section B-B).

(a) Combination of loads

Case-1: during flood

Water pressure, dead load, earth pressure and reaction ($= \frac{V^2}{2g}$)

Case-2: Just before opening of bulkhead gate of river outlet

Water pressure (up to El. 189.0 of spillway crest), dead load, earth pressure and reaction

Case-3: during gate installation (Section B only)

Gate load, dead load, earth pressure and reaction

(b) Load

i) Section A-A: Case 1

Water pressure: Pw= Pex (total static head) - Pin = $\frac{V^2}{2g}$

$= \frac{(520/44.04)^2}{19.6} = 7.1\ t/m^2$

Dead load : Dead loads to be considered are the ones under water.

Upper slab	$2.2 \times 1.4 = 3.1\ t/m^2$
Side wall	$2.0 \times 1.4 = 2.8\ t/m^2$

Earth pressure: Ka γ_s H

Top:	$0.5 \times 1.1 \times 1.1 = 0.6\ t/m^2$
Bottom:	$0.5 \times 1.1 \times 9.0 = 5.0\ t/m^2$

Reaction : $(11.0 \times 10.8 - 2.0 \times 10.8 - 44.04) \times 1.4/10.8 = 6.9\ t/m^2$

ii) Section A-A: Case 2

Water pressure:

Upper slab	$189.0 - 138.0 = 51.0\ t/m^2$
Side wall (top)	$189.0 - 136.9 = 52.1\ t/m^2$
Side wall (bottom)	$189.0 - 128.0 = 61.0\ t/m^2$
Bottom slab	$189.0 - 127.0 = 62.0\ t/m^2$

Dead load :

Upper slab	$2.2 \times 2.4 = 5.3\ t/m^2$
Side wall	$2.0 \times 2.4 = 4.8\ t/m^2$

Earth pressure:

Top:	$0.6\ t/m^2$
Bottom:	$5.0\ t/m^2$

Reaction : Dead load of upper slab & side wall buoyancy

$$\frac{(11.0 \times 10.8 - 2.0 \times 10.8 - 44.04) \times 2.4 - 11.0 \times 10.8}{10.8}$$

$= 0.8\ t/m^2$

iii) Section B-B: Case 1

Water pressure: $Pw = \frac{(520/6.8^2)^2}{19.6} = 6.5\ t/m^2$

Dead load :

Upper slab	$2.2 \times 1.4 = 3.1\ t/m^2$
Side wall	$2.0 \times 1.4 = 2.8\ t/m^2$

Earth pressure: Top: 0.6 t/m^2
Bottom: 5.0 t/m^2

Reaction : (11.0 x 10.8 - 2.0 x 10.8 - 6.8^2) x 1.4/10.8 = 6.6 t/m^2

iv) Section B-B: Case 2

Water pressure: Upper slab 51.0 t/m^2
Side (top) 52.1 t/m^2
Side (bottom) 61.0 t/m^2
Bottom slab 62.0 t/m^2

Dead load : Upper slab 5.3 t/m^2
Side wall 4.8 t/m^2

Earth pressure: Top: 0.6 t/m^2
Bottom: 5.0 t/m^2

Reaction : $\frac{(11.0 \times 10.8 - 2.0 \times 10.8 - 6.8^2) \times 2.4 - 11.0 \times 10.8}{10.8}$

= 0.3 t/m^2

v) Section B-B: Case 3

Gate load : $\frac{\text{Gate weight}}{\text{Bottom area}}$
$= \frac{50}{7.5 \times 1.1} \doteqdot 6.0$ t/m^2

Dead load : Upper slab 2.2 x 2.4 = 5.3 t/m^2
Side wall 2.0 x 2.4 = 4.8 t/m^2

Earth pressure: Top: 0.5 x 1.95 x 1.1 = 1.1 t/m^2
Bottom: 0.5 x 1.95 x 9.0 = 8.8 t/m^2

Reaction : $\frac{(10.8 \times 2.2 + 6.8 \times 2.0 \times 2) \times 2.4}{10.8} + 6.0 = 17.3$ t/m^2

(c) Allowable stress

Case	Increment Ratio	Concrete		Re-bar
		σ_{ca} (kg/cm^2)	τa (kg/cm^2)	σ_{sa} (kg/cm^2)
1	50%	105	12.8	2,700
2	50%	105	12.8	2,700
3	30%	91	11.1	2,340

(d) Analysis

SECTION A-A:

The section and dimension of Section A-A are shown in Fig. 3.2.9. The loading diagrams are given in Fig. 3.2.10. The structural analyses for Section A-A are made in Table 3.2.7. Its results are all shown in the bending moment, shearing force and axial force diagrams in Fig. 3.2.12 and 3.2.13.

As understood from the analysis results, the minimum reinforcement arrangement with D19 @300 will be sufficient for Section A-A.

An examination on stirrups are made as follows:

$$\tau = \frac{Q}{B \cdot j \cdot d}$$

B : width (cm)
j : 0.875
d : effective height (cm)

Mem	Spot	B (cm)	d (cm)	Q (ton)	τ (kg/cm^2)	τa (kg/cm^2)
2	2	100	240	188.6	8.98	12.8
6	6	100	170	164.9	11.09	12.8
13	14	100	240	188.6	8.98	12.8
15	15	100	240	188.7	8.99	12.8
17	1	100	240	188.1	8.96	12.8

Stirrups are not required in calculation, but 2-D16 @300 is applied to the corners of the structure for caution.

SECTION B-B:

The section and dimension of Section B-B are shown in Fig. 3.2.9. The loading diagrams are as seen in Fig. 3.2.11. The structural analyses for Section B-B are made in Table 3.2.8. Its results are presented in the bending moment, shearing force and axial force diagrams in Fig. 3.2.14 to 3.2.16.

Table 3.2.9 presents the calculations of internal stress in the reinforced concrete for the reinforcement arrangement provided based on the structural analyses.

Followings are examinations on the necessary stirrups:

Mem	Spot	B (cm)	S (cm)	τ (kg/cm^2)	A_V (cm)	Application
1 9	2 9	100	30	14.9	9.4	4-D19 = 11.46
10 12	11 12	100	30	14.6	9.1	4-D19

$$A_V = \frac{(\tau - \tau_a/2) \cdot B \cdot S}{\sigma_{sa}}$$

A_V : required area of stirrup (cm^2)

S : pitch of stirrup (cm)

Based on the above, the stirrups of 4-D19 are provided in Section B-B.

FOUNDATION STRENGTH:

As seen in fig. 2.6.4, the tunnel inlet is expected to be situated in the weathered basalt which has a bearing strength of about 200 ton/m^2 on the basis of the compressive strength test carried out for the weathered basalt.

On the other hand, the loading to act to the foundation of the tunnel inlet will be about 60 ton/m^2 at maximum as seen in the loading diagrams shown in Fig. 3.2.10 and 3.2.11. As such, the foundation of the tunnel inlet is considered to have a sufficient bearing strength against the loading.

(4) Analysis for outlet transition

(a) Combination of load

Case-1: During diversion
Dead load + water pressure (Ground WL. - Tunnel center)

Case-2: Grouting condition
Dead load + Backfill grout pressure

(b) Load

Dead load : $1.0 \times 2.4 = 2.4\ t/m^2$

Water pressure : G.WL - Tunnel Center
$140 - 128.4 = 11.6\ t/m^2$

Grout pressure : $2\ kg/cm^2 = 20\ t/m^2$ (backfill grout)

Reaction :

Case-1 $R = (3.9 \times \pi + 3.4 \times 2) \times 1.0 \times 2.4/8.8$
$\fallingdotseq 5.2\ t/m$

Case-2 $R = 5.2 +$
$2 \times (2.019 \times 20 \times \cos 15° + 2.019/2 \times 20 \times \cos 45°)/8.8$
$= 17.3\ t/m$

(c) Allowable stress

Case	Increment Ratio	Concrete		Re-bar
		σ_{ca} (kg/cm^2)	τ_a (kg/cm^2)	σ_{sa} (kg/cm^2)
1	30%	91	11.1	2,340
2	strength	210	18.0	3,000

(d) Analysis

The section, dimension and loading diagram for the outlet transition portion are shown in Fig. 3.2.17. Structural analysis for the outlet transition portion is given in Table 3.2.10. Fig. 3.2.18 and 3.2.19 present its results by the bending moment, shearing force and axial force diagrams. Table 3.2.11 presents the calculation of internal stress in the reinforced concrete for the reinforcement arrangement provided based on the structural analysis.

(5) Analysis for outlet portal

(a) Combination of load

Dead load + earth pressure + surcharge load

Dead load : $1.0 \times 2.4 = 2.4\ t/m^2$

Earth pressure	:	$Ka \cdot \gamma \cdot H$
		$= 0.5 \times 1.95H$
Surcharge load	:	1.0 t/m2
Earth pressure due to surcharge		$0.5 \times 1.0 = 0.5$ t/m^2

Water pressure is neglected because it will be drained through weep holes provided in wing wall.

Above condition is considered to be normal condition. So allowable stresses are:

$$\sigma_{ca} = 70 \text{ kg/cm}^2$$
$$\sigma_{sa} = 1{,}800 \text{ kg/cm}^2$$
$$\tau_a = 8.5 \text{ kg/cm}^2$$

(b) Analysis

The section, dimension and loading diagram for the outlet portal are shown in Fig. 3.2.20. The structural analysis is made in Table 3.2.12. Fig. 3.2.21 shows the bending moment, shearing force and axial force diagrams based on the structural analysis made in Table 3.2.12. Table 3.2.13 presents the calculation of internal stress of reinforced concrete for the reinforcement arrangement provided on the basis of the structural analysis.

The tunnel outlet is expected to be founded on the hard basalt as seen in Fig. 2.6.4, which have a sufficient bearing strength against the loadings.

(6) Analysis for steel support

As mentioned, the load of rock to be loosened by tunnel excavation works is assumed to be supported by the steel support. Hence, the safety of steel support to be installed in examined herein.

The examination is made by checking the stress to be given to the steel support in the following steel support arrangements:

Tunnel Type	Steel Support Dimension (H-shape steel, mm)	Interval of Steel Support (m)
Type I	200 x 200 x 12	1.5
Type II	200 x 200 x 12	1.0

Loading conditions are based on those proposed by Terzaghi: that is, Terzaghi proposes the following rock loads in accordance with geological conditions in the tunnel excavation work.

Geological Condition	Height of rock to act as load (m)
CL ~ CM class	0.25 B ~ 0.35 (B+H)
CM ~ CH class	0 ~ 0.25 B

where, B : Width of tunnel excavation (= 7.8 m)
H : Height of tunnel excavation (= 7.938 m)

Thus, the rock load to act to the steel support is assumed as follows:

Tunnel Type	Steel support interval (m)	Height of rock to act as load (m)	Rock load (ton/each•m)
Type I	1.5	0.25 B = 1.95	7.31
Type II	1.0	0.35 (B+H) = 5.5	13.75

Note: In the above calculation of rock load, the unit weight of rock is assumed to be r=2.5 ton/m^3.

The stress in the steel support is calculated by the following equation:

$$\delta = T/A \pm M/Z$$

where, T: Axial force (ton)
A: Sectional area of H-shape steel (= 71.53 cm^2)
M: Bending moment (t•m)
Z: Section modules of H-shape steel (= 498 cm^2)

The structural model is shown in Fig. 3.2.34. The axial force (Ti) at each point of the steel support and maximum bending moment (Mmax) are calculated by the method of Procter and White in Table 3.2.14 for the tunnel Type-I and Table 3.2.15 for the tunnel Type-II, respectively.

Based on the calculated maximum axial force and bending moment, the maximum stress in the steel support is found as follows:

Tunnel Type	Max. Axial Force (t)	Max. Bending Moment (t•m)	Rock load (ton/each•m)	
			δ max (kg/cm^2)	δ min (kg/cm^2)
Type I	24.12	0.57	452	223
Type-II	50.98	1.21	956	470

As seen above, the stress in the steel support is sufficiently less than the allowable stress of 1,800 kg/cm^2. The sufficient allowance given to the steel support design is considered reasonable in consideration of uncertainties involved in the assumptions of loading.

3.3 Diversion Gate

3.3.1 General Description

One complete set of slide type diversion gate with guide frames and hoist tower is provided on the diversion tunnel inlet structure for closing the diversion tunnel.

The gate consists of a gate leaf, guide frames and a steel structure of hoist tower for assembling the gate leaf and for operation of the gate.

The gate is operated by a temporarily installed wire rope winch through rope sheaves provided on the hoist tower.

Slide type gate is selected for the gate by the following reasons:

(1) Less operation force due to low head at operation
(2) Simple structure which makes the fabrication easy and minimize possible troubles
(3) Low construction cost

The diversion gate leaf and guide frames are designed under the condition of reservoir water level of El. 189.0 m which is the spillway overflow crest elevation. The above design condition is based on the consideration that the reservoir water level rise after closing the diversion gate will be very rapid due to a small reservoir storage capacity and that the diversion gate structure should withstand the full reservoir water pressure for the necessary works in the diversion tunnel after the gate closure.

A steel hoist tower is provided on the concrete slab of the diversion tunnel inlet.

The allowable stresses for the diversion gate are applied 1.5 times of the allowable stresses for the permanent hydromechanical structures such as intake gates etc., since the diversion gate is installed as a temporary equipment for construction of this project.

The design data of the diversion gate are summarized below.

Type	:	Steel slide gate
Quantity	:	1 set
Clear Span	:	6.80 m
Clear height	:	6.80 m
Design water level	:	El. 189.00 m
Sill elevation	:	El. 129.00 m
Design head of gate	:	60.00 m
Water seal	:	4 edges rubber seal on downstream face of gate
Type of hoist	:	Steel hoist tower with winch
Operation	:	Temporary winch

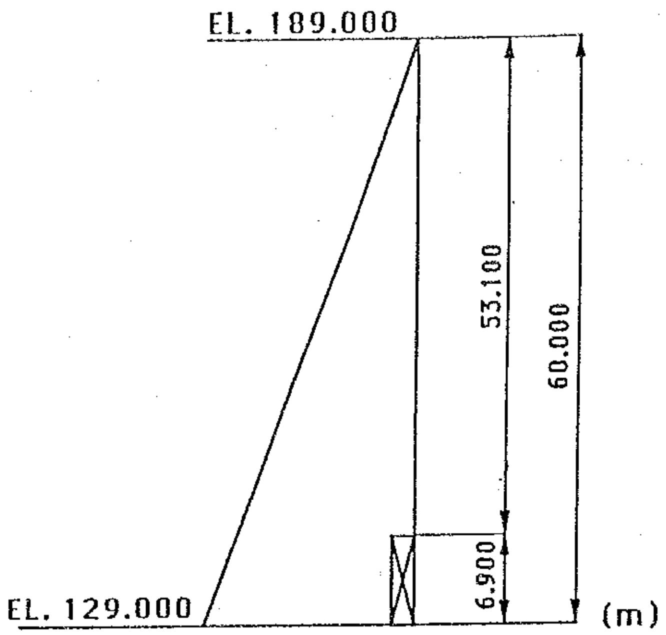
Design of diversion gate is shown in Fig. 3.3.1 to 3.3.2. Structural analyses of the diversion gate structure are made hereunder.

3.3.2 Structural Analysis of Diversion Gate

(1) Design conditions

Type	:	Sluice gate
Quantity	:	1 set
Clear span	:	6.8 m
Clear height	:	6.8 m
Max. design head	:	60 m (El. 189 - El. 129)
Max. deflection	:	1/800 of supporting span
Sealing method	:	4 edges rubber seal at downstream face of gate
Corrosion allowance	:	Not considered
Closing operation	:	Handled by temporarily installed winch

(2) Total hydraulic load



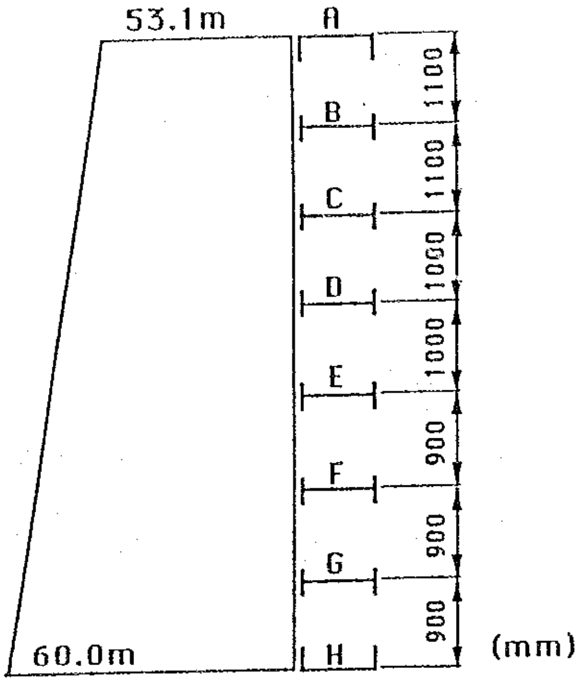
$Pt = 0.5 \times (H2^2 - H1^2) \times B \times Gw = 2732$ (ton)

where,

Pt	=	Total hydraulic load	
H1	=	Design head at gate top	53.100 (m)
H2	=	Design head at gate bottom	60.000 (m)
B	=	Sealing span	7.000 (m)
Gw	=	Specific gravity of water	1.000 (ft/m^3)

(3) Main horizontal beams

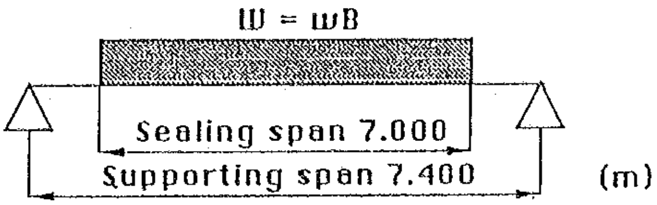
(a) Arrangement and reaction force (w) of main horizontal beams



A-beam	0.5 x (53.100 + 53.650) x 0.550 = 29.356 (t/m)
B-beam	0.5 x (53.650 + 54.750) x 1.100 = 59.620 (t/m)
C-beam	0.5 x (54.750 + 55.800) x 1.050 = 58.039 (t/m)
D-beam	0.5 x (55.800 + 56.800) x 1.000 = 56.300 (t/m)
E-beam	0.5 x (56.800 + 57.750) x 0.950 = 54.411 (t/m)
F-beam	0.5 x (57.750 + 58.650) x 0.900 = 52.380 (t/m)
G-beam	0.5 x (58.650 + 59.550) x 0.900 = 53.190 (t/m)
H-beam	0.5 x (59.550 + 60.000) x 0.450 = 26.899 (t/m)

The strength calculation has to be made for A-beam and B-beam which will be subject to the maximum load in each type of beam.

(b) Bending moment and shearing force



$Mm = W \times (2 \times L - b)/8$

$Sm = W/2$

where,

Mm	=	Max. bending moment	
Sm	=	Max. shearing force	
W	=	Water pressure load on each beam (ton)	
B	=	Sealing span	7.000 (m)
L	=	Supporting span	7.400 (m)

A-beam

W = 29.356 x 7.000 = 205.492 (ton)

Mm = 205.492 x (2 x 7.400 - 7.000)/8 = 200.355 (ton-m)

= 20035500 (kg-cm)

Sm = 205.492/2 = 102.746 (ton)

= 102746 (kg)

B-beam

W = 59.620 x 7.000 = 417.340 (ton)

Mm = 417.340 x (2 x 7.400 - 7.000)/8 = 406.907 (ton-m)

= 40690700 (kg-cm)

Sm = 417.340/2 = 208.670 (ton)

= 208670 (kg)

(c) Bending stress and shearing stress

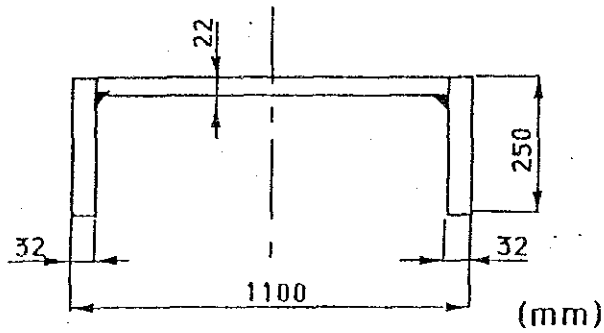
$\sigma m = Mm/Z$

$\tau m = Sm/Aw$

where,

σm	=	Max. bending stress
τm	=	Max. shearing stress
I	=	Moment of inertia (cm^4)
Z	=	Modulus of section (cm^3)
Aw	=	Area of web (cm^2)

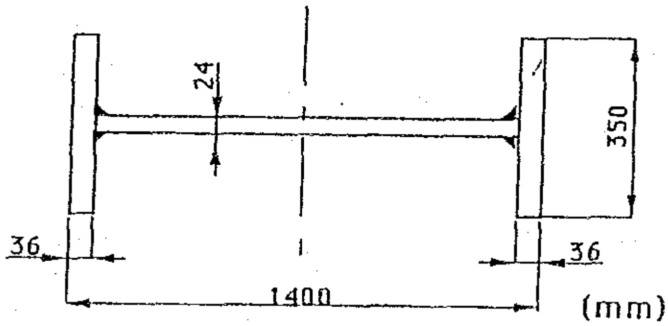
A-beam



I = 660241 (cm^4)
Z = 12004 (cm^3)
Aw = 228 (cm^2)

σm = 20035500/12004 = 1669 (kg/cm^2) < 2700 (kg/cm^2)
τm = 102746/228 = 451 (kg/cm^2) < 1575 (kg/cm^2)

B-beam



I = 1640793 (cm^4)
Z = 23440 (cm^3)
Aw = 264 (cm^2)

$\sigma m = 40690700/23440 = 1736\ (kg/cm^2) < \ 2700\ (kg/cm^2)$
$\tau m = 208670/264 = 790\ (kg/cm^2) < \ 1575\ (kg/cm^2)$

(d) Deflection

$$\delta m = W \times (L^3 - L \times B^2/2 + B^3/8) / (48 \times E \times 1)$$

where,

δm = Max. deflection
W = Water pressure load on each beam (kg)
E = Young's modulus (kg/cm^2)

A-beam

$$\delta m = \frac{205492 \times (740^3 - 740 \times 700^2/2 + 700^3/8)}{(48 \times 2.1 \times 10^6 \times 660241)} = 0.824\ (cm)$$

$\delta m/L = 0.824/740 = 1/898 < 1/800$

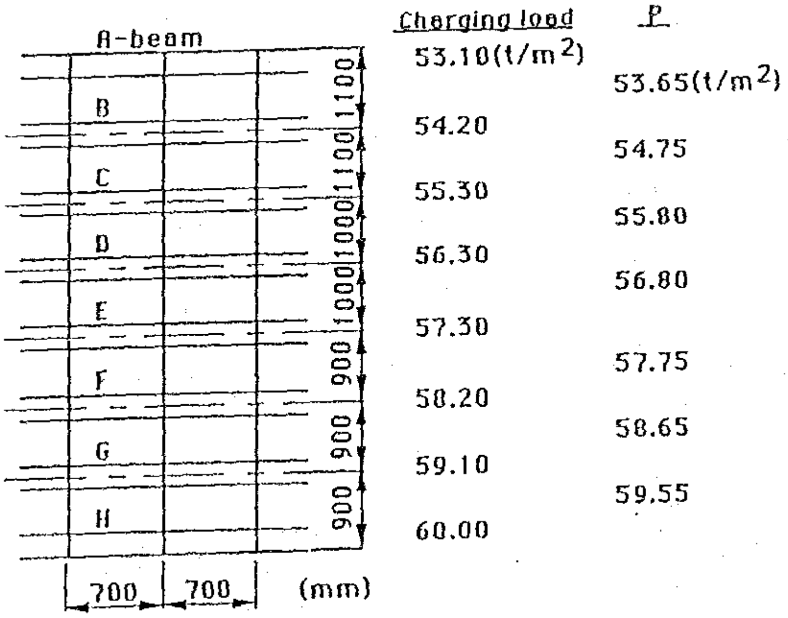
B-beam

$$\delta m = \frac{417340 \times (740^3 - 740 \times 700^2/2 + 700^3/8)}{(48 \times 2.1 \times 10^6 \times 1640793)} = 0.673\ (cm)$$

$\delta m/L = 0.673/740 = 1/1100 < 1/800$

(4) Vertical girders

(a) Bending moment and shearing force

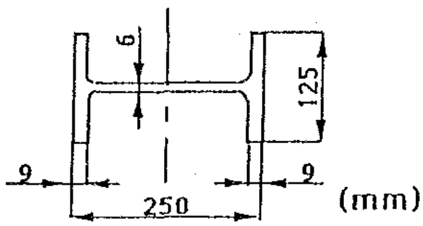


Mm = Max. bending moment
Sm = Max. shearing force

No.	P (kg/cm^2)	a (cm)	b (cm)	Mm (kg-cm)	Sm (kg)
1	5.365	70	110	491345	14083
2	5.475	70	110	501419	14372
3	5.580	70	110	408503	12695
4	5.680	70	110	415823	12922
5	5.775	70	90	326769	11117
6	5.865	70	90	331861	11290
7	5.955	70	90	336954	11463

(b) Bending stress and shearing stress

(b-1)



Z = Modulus of section 324.0 (cm^3)

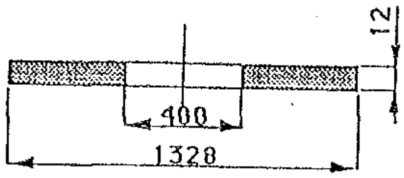
Aw = Area of web 13.9 (cm^2)

No.	Bending stress (kg/cm^2)	Shearing stress (kg/cm^2)
1	1516	1013
2	1548	1034
3	1261	913
4	1283	930
5	1009	800
6	1024	812
7	1040	825

Allowable bending stress: 2700 (kg/cm^2)

Allowable shearing stress: 1575 (kg/cm^2)

(b-2)



(mm)

Z = Modulus of section 3430.8 (cm^3)

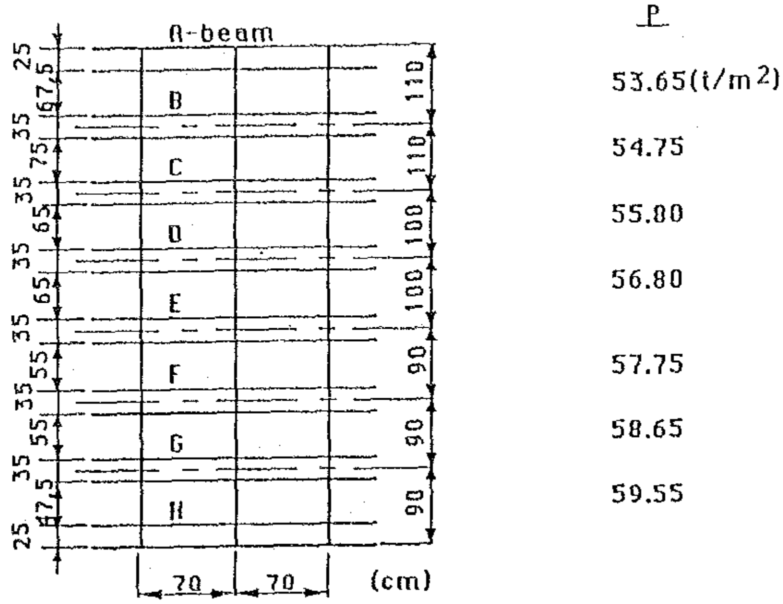
Aw = Area of web 111.4 (cm^2)

No.	Bending stress (kg/cm^2)	Shearing stress (kg/cm^2)
1	143	126
2	146	129
3	119	114
4	121	116
5	95	100
6	97	101
7	98	103

Allowable bending stress: 2700 (kg/cm^2)

Allowable shearing stress: 1575 (kg/cm^2)

(5) Skin plate



$\sigma = (k \times a^2 \times p/t^2)/100$

where,

σ = Bending stress (kg/cm^2)

k = Coefficient by "b/a"

a = Short span of plate (cm)

b = Long span of plate (cm)

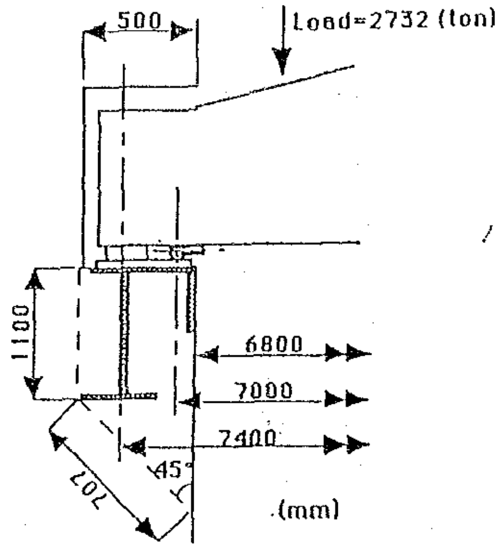
p = Water pressure (kg/cm^2)

t = Thickness of skin plate (cm)

No.	a	b	b/a	k	p	t	σ
1	67.5	70	1.04	32.6	5.365	2.3	1506
2	70	75	1.07	34.0	5.475	2.3	1724
3	65	70	1.08	34.4	5.580	2.3	1533
4	65	70	1.08	34.4	5.680	2.3	1561
5	55	70	1.27	40.6	5.775	2.3	1341
6	55	70	1.27	40.6	5.865	2.3	1362
7	47.5	70	1.47	45.0	5.955	2.3	1143

Allowable bending stress: 2700 (kg/cm^2)

(6) Guide frame



(a) Shearing area of concrete section

$A1 = (110.0 + 70.7) \times 690.0 = 124683\ (cm^2)$

$A2 = (110.0 \times 50.0 + 0.5 \times 50.0 \times 50.0) \times 2 = 13500\ (cm^2)$

$A = A1 + A2 = 124683 + 13500 = 138183\ (cm^2)$

(b) Shearing stress of concrete

$\tau = load/2A$

$= 2732000/(2 \times 138183) = 9.89\ (kg/cm^2) < 12.75\ (kg/cm^2)$

(7) Weight of the gate leaf

		Weight/unit	Q'ty	Weight
(a)	Main horizontal beams			
	1100 x 250 x 22 x 32 x 7400	2254 (kg)	2	4508 (kg)
	1400 x 350 x 24 x 36 x 7400	3316 (kg)	6	19896 (kg)
(b)	Skin plate			
	6950 x 7600 x 23	9537 (kg)	1	9537 (kg)

(c)	Vertical beams			
	250 x 125 x 6 x 9 x 6950	206 (kg)	5	1030 (kg)
	1328 x 12 x 6950 - 400 x 400 x 12 x 7	764 (kg)	4	3056 (kg)
(d)	Side beams			
	1100 x 200 x 22 x 32 x 6950	1942 (kg)	2	3884 (kg)
		Total		41911 (kg)
				x 1.2 ≒ 51 (ton)

(8) Strength of the steel supporting structure

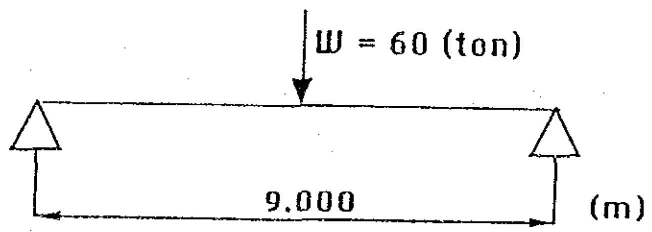
(a) Operation load

$W = w + F = 59.4 \fallingdotseq 60$ (ton)

where,

W	=	Total operation load (ton)	
w	=	Weight of gate leaf	51 (ton)
F	=	Friction load due to guide frame (2m head)	8.4 (ton)

(b) Beam



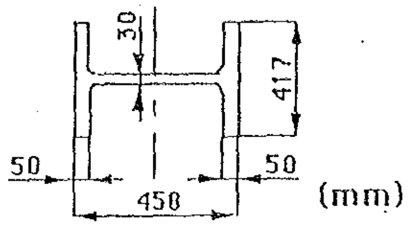
$Mm = WL/4 = 13500000$ (kg-cm)

$Sm = W/2 = 30000$ (ton)

where,

W	=	Total operation load (ton)	60000 (kg)
L	=	Beam span	900 (cm)

(b) Beam section

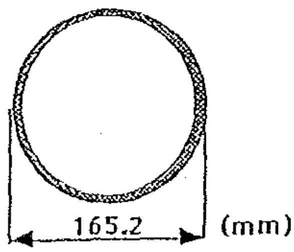


$Z = 8170\ (cm^3)$

$Aw = 107.4\ (cm^2)$

$\sigma m = 13500000/8170 = 1652\ (kg/cm^2) < 2700\ (kg/cm^2)$

(c) Main posts



Steel pipe: Dia. 165.2 (mm) x Thickness 5 (mm)

$Pk = \pi^2 EI/kL^2 = 5795\ (kg/cm^2)$

$Pa = (W + w)/2A = 1265\ (kg/cm^2)$

$Pa < Pk$

where,

Pk	=	Critical backling pressure (kg/cm2)	
Pa	=	Acting load by gate leaf (kg/cm2)	
E	=	Young's modulus (kg/cm2)	
I	=	Moment of inertia	808 (cm4)
L	=	Post height	850 (cm)
k	=	Safety factor	4
W	=	Total operation load	60000 (kg)
w	=	Beam weight	3735 (kg)
A	=	Sectional area of main post	25.2 (cm^2)

3.4 Preparatory Works

3.4.1 Haul Road

The quarry site for extracting dam rock materials and concrete aggregates is selected at the Mt. Ory about 1.0 km north from the damsite as seen in Fig. 3.1.1.

The main trunk road of Mauritius passes near the quarry site and damsite. However, the use of this trunk road for hauling the rock materials will seriously disturb the heavy traffic in the trunk road, and therefore, a separate haul road is planned to be constructed from the quarry site to the damsite.

As seen in Fig. 3.4.1, the haul road is provided in parallel with the existing trunk road from the quarry site to the Moka river. To reach the damsite, the haul road requires to cross the trunk road. Since two-level crossing with the trunk road is considered essential, the haul road utilizes the existing bridge of the trunk road which crosses the Moka river: that is, the haul road passes beneath the existing bridge of the trunk road, which is considered the most economical way for two-level crossing with the trunk.

A submergible bridge, which will make the traffic of dump trucks possible except the time of flooding, is provided in the Moka river beneath the existing bridge of the trunk road. The haul road is constructed along the left bank of the Moka river after the above submergible bridge, and then, reaches the damsite. The total length of the haul road is measured at about 1,370 m. It is provided with a width of 9 m so that two dump truck can pass each other.

The design of the haul road is difficult to strictly follow the design standard of a permanent road due to topographic conditions. Further, since the haul road is considered to be a temporary structure for dam construction works, its design is not always necessary to strictly follow the design standard of the permanent road.

Therefore, the design of the haul road is made so as to be practical for the dam construction works as follows:

(1) Lane:

- Number of lanes : 2 lanes
- Width of one lane : 4 m for one lane

(2) Shoulder:

The road has shoulders of 0.5 m width at both sides. The shoulder shall be constructed selected gravel materials.

(3) Ditch:

A ditch with 0.5 m in width is provided along the road to drain the rain. From an economical point of view, the ditch is, in principle, provided at one side of the road, and then, the road surface is provided with a cant of 2.0% to facilitate the drainage to the ditch.

(4) Radius of curvature:

A radius of curvature more than 15.0 m is provided at the curves of road based on the design speed of 20 km/hr. of dump trucks.

(5) Cant:

Based on the design speed of 20 km/hour of dump trucks, the following necessary cants in compliance with the radius of curvature are provided at curves of the haul road:

Radius of Curvature (m)	Cant (%)
$220 \leq R$	2.0
$150 \leq R < 220$	3.0
$110 \leq R < 150$	4.0
$80 \leq R < 110$	5.0
$R < 80$	6.0

R: Radius of curvature (m)

(6) Slope of the road:

The maximum slope of the road is limited to 10% which is considered to allow a smooth transportation of rock materials by dump trucks.

(7) Cutting slope of the road:

In consideration that the haul road is a temporary structure, the following cutting slope is applied for the excavation of the haul road in accordance with the standard:

Materials	Cutting Slope
Earth material	1 : 1.0
Weathered rock	1 : 0.5
Fresh rock	1 : 0.3

The design of berms to be provided on the cutting slope follows the following standard: that is,

- A berm shall be provided at the interval of 15 m height.
- The first, third, and fifth berms shall be 1.5 m wide without ditch.
- The second and fourth berms shall be 2.0 m wide with ditch.

(8) Road embankment:

The haul road includes a part of embankment which is made with materials excavated in the construction of the haul road. The above excavated materials will be composed mainly of earth and weathered rock.

In consideration that the embankment material will include much earth material, a road embankment slope with earth material of 1 to 2.0 is provided.

(9) Pavement:

Some pavement on the haul road is considered necessary for a smooth transportation work of rock materials. With consideration that the road is temporary, a simple pavement consisting of a base coarse of 150 mm in thickness and surface coarse of 50 mm in thickness is applied.

Details of the design made for the haul road are as seen in Fig. 3.4.2 to Fig. 3.4.4.

3.4.2 Access Roads in Damsite

Access roads in damsite will consist of the road from the end of haul road at damsite to river bed, road to go to the dam crest in the right bank, roads to go to the inlet and outlet of the diversion tunnel near the river bed in the left bank, road from the outlet of diversion tunnel to spillway and various branch access roads necessary for construction works of dam and related structures.

Out of these access roads, the followings are included in Lot-I works:

- The road from the end of haul road at damsite to river bed
- The submergible bridge to cross the river
- The roads to go to the inlet and outlet of diversion tunnel

The road to go to the dam crest in the right bank, the road from the outlet of diversion tunnel to spillway and various branch access roads necessary for construction works are closely related to the dam and spillway excavation works in Lot-II. Therefore, those should be included in Lot-II works.

Details of the design made for the access roads to be included in Lot-I are given in Fig. 3.4.5 and Fig. 3.4.2. The design concepts are mostly same as those mentioned in the haul road. The access road from the end of haul road at damsite to river bed partly includes the permanent road (the road to go to dam crest in the right bank), for which the cutting slope will be coated with the shotcrete.

3.4.3 Access Road from Water Treatment Facilities to Municipal Dike

An access road from the existing Pailles water treatment facilities to the Municipal Dike is planned to be constructed for the installation work of the additional water transmission pipeline and future maintenance works of the pipeline.

The total length of the above access road is measured at about 1,800 m. The road is provided with one lane of 4 m wide. It has the shoulder of 0.5 m wide in both sides. Thus, the total width of the road is 5.0 m which will sufficiently make the pipe installation or future maintenance works possible. All other design concepts are same as those mentioned in Section 3.4.1 "Haul Road".

The road is constructed along the existing pipeline because the additional pipeline is installed along the existing one. It has to cross the river two times. To save the cost, the crossings with the river are made by submergible bridges which will be available except flooding times.

The design of the road is shown in Fig. 3.4.6 to 3.4.8.

3.4.4 Aggregate Plant

The Project considers to produce the concrete aggregates and dam filter materials, etc. from the quarry rock, requiring the installation of a suitable aggregate plant.

Necessary capacity of the aggregates plant is determined at 90 ton/hour through the following examinations:

(1) Required capacity for concrete aggregate production

The Project works require concreting works for the diversion tunnel, spillway, intake structure, and water treatment facilities, etc.

Required concrete volume and construction period for each structure are as follows:

Works	Concrete Volume (m^3)	Construction Period
D. tunnel	11,300	Feb. 1993 ~ Feb. 1994 (13 months)
Spillway	51,000	Dec. 1994 ~Jan. 1996 (14 months)
Intake	4,900	Feb. 1996 ~Aug. 1996 (6 months)
Plug & valve chamber	3,000	Jul. 1996 ~Sept. 1996 (3 months)
W. Treatment facilities	2,205	Dec. 1994 ~1996 (18 months)

As seen above, the diversion tunnel, intake and plug & valve chamber works will not overlap with the spillway concreting work.

The water treatment facilities and spillway works will overlap with each other, and the required capacity for concrete aggregate production should take these concreting works into consideration: this is, the concreting works for the diversion tunnel, intake and plug & valve chamber, etc. is not taken into account for determining the required capacity of aggregate plant.

Concrete volumes by each class of concrete for the above structures are approximately estimated as follows:

	Structure	Concrete Type	Concrete Volume (m^3)
a.	Spillway	C	49,000
		E	1,300
c.	W. Treatment Facilities	A	470
		B	1,435
		D	300

The aggregate requirement by each size is estimated by applying a standard requirement per m^3 for each type of concrete as follows:

Class of Concrete	Required Compressive Strength (kg/cm^2)	Aggregate Requirement (ton/m^3)			
		80 ~ 40 mm	40 ~ 20 mm	20 ~ 5 mm	5 mm under
A	210	-	-	0.889	0.977
B	180	-	-	0.892	1.023
C	180	-	0.548	0.593	0.836
D	140	-	0.548	0.593	0.908
E	120	0.436	0.303	0.450	0.851

Thus, the total aggregate requirement in ton is summarized as follows:

Concrete Class	Volume (m^3)	Aggregate Requirement (ton)				
		80 ~ 40 mm	40 ~ 20 mm	20 ~ 5 mm	5 mm under	Total
A	470			417.8	459.2	877.0
B	1,435			1,280.0	1,468.0	2,748.0
C	49,700		27,235.6	29,472.1	41,549.2	98,256.9
D	300		164.4	177.9	272.4	614.7
E	1,300	566.8	333.9	585.0	1,106.3	2,652.0
Total		566.8	27,793.9	31,932.8	44,855.1	105,148.6
Hourly Production (t/hr)		0.2	9.9	11.4	16.0	37.5

The hourly production capacity for the concrete aggregate is calculated on the assumption of the workable days of 20 days/month, working hours of 10 hours/day, working period of 14 months in accordance with the construction work schedule of the spillway: that is, it is calculated as follows:

$$\text{Hourly Production Capacity (For concrete aggregate)} = \frac{\text{Aggregate Requirement}}{14 \text{ months} \times 20 \text{ days} \times 10 \text{ hr.}} \times 1.3$$

where, C (allowance coefficient) is assumed to be 1.0 in consideration that the work schedule will have a sufficient time to produce and stock the aggregates before the commencement of the spillway concreting work.

(2) Required capacity for filter material production

Required dam filter material is estimated at 100,000 m^3 in total. The volume by each size is estimated to be 60,000 m^3 for 5 mm under, 22,000 m^3 for 20 ~ 5 mm size, 10,000 m^3 for 40 ~ 20 mm size and 8,000 m^3 for 80 ~ 40 mm size, respectively.

Thus, the requirement for filter material in ton is summarized as follows:

	Volume (m^3)	Filter Material Requirement (ton)				
		80 ~ 40 mm	40 ~ 20 mm	20 ~ 5 mm	5 mm under	Total
	60,000	-	-	-	120,000	120,000
	22,000	-	-	44,000	-	44,000
	10,000	-	20,000	-	-	20,000
	8,000	16,000	-	-	-	16,000
Total	100,000	16,000	20,000	44,000	120,000	200,000
Hourly Production (t/hr)		4	5	11	30	50

The hourly production capacity for dam filter material is calculated on the assumption of the workable days of 20 days/month, working hours of 10 hours/day and working period of 20 months in accordance with the dam embankment schedule: that is, it is calculated as follows:

$$\text{Hourly Production Capacity (For filter material)} = \frac{\text{Filter Requirement}}{20\ \text{months} \times 20\ \text{days} \times 10\ \text{hr.}}$$

(3) Necessary capacity of plant

As mentioned above, the concrete aggregate production requires the capacity of 37.5 ton/hr. The dam filter material production requires the capacity of 50 ton/hr. According to the construction schedule, both of the concrete aggregate and filter material productions will mostly overlap.

Thus, the aggregate plant is planned to have the hourly production capacity of 90 ton/hr. The sand is also forced to be produced from the quarry rock, which requires to be equipped with the rod mill.

The flow and plan of the aggregate plant are given in Fig. 3.4.9. The examination on the location for the aggregate plant as well as the batcher plant selects an area near the haul road on the right bank of the damsite with consideration that the area would be most convenient for the transportation of quarry rock and concrete.