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 (2)

DESIGN REPORT

FOR

LOT II : CIVIL WORKS(DAM AND APPURTENANT STRUCTURES INCLUDING CLOSURES OF DIVERSION TUNNEL)

MARCH 1992

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

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

JICA sent to Mauritius a study team headed by Mr. Norizo FUJITA, Nippon Koei Co.,Ltd., and composed of members from Nippon Koei Co.,Ltd. and Nihon Suido Consultants Co.,Ltd., for four times from May 1990 to December 1991.

The team held discussions with the officials concerned of the Government of Mauritius, and conducted field surveys at the study area. 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 , 1992

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation Agency

March,1992

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 (2) of the Detailed Design on Port Louis Water Supply Project in Mauritius prepared for the implementation of Lot-II work in the Project.

This report is composed of ten (10) volumes consisting of the Summary Report, Design Report for Lot-II (Three (3) volumes of English and Japanese version and Appendix), Tender Document (Four (4) Volumes), Cost Estimate for Lot-II and Data Book. The Summary Report summarizes outlines of the detailed design carried out for Lot-II. The Design Report for Lot-II contains the results of the detailed design on Lot-II 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 II contains the unit cost analysis and construction cost estimate for each work item included in Lot-II. 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 have been executed. The detailed design works for Lot-I were finished in March, 1991. The detailed design works for Lot-II and Lot-III have been continued. This Design Report summarizes results of the detailed design carried out for Lot-II. Results of detailed design for Lot-III are presented separately.

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-II

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-II are as follows:

- Cofferdam (the cofferdam forms a part of the main dam)
- Main dam
- Spillway structure
- Water intake and water supply facilities
- River outlet facilities
- Repair work of the existing Municipal Dike

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 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	84 m
Crest length	250 m
Embankment volume	1,548 x 10^3 m^3

(3) Spillway

Type	Side channel
Crest elevation of weir	El. 189 m
Width of weir	92 m
Discharge	1,890 m^3/s

(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

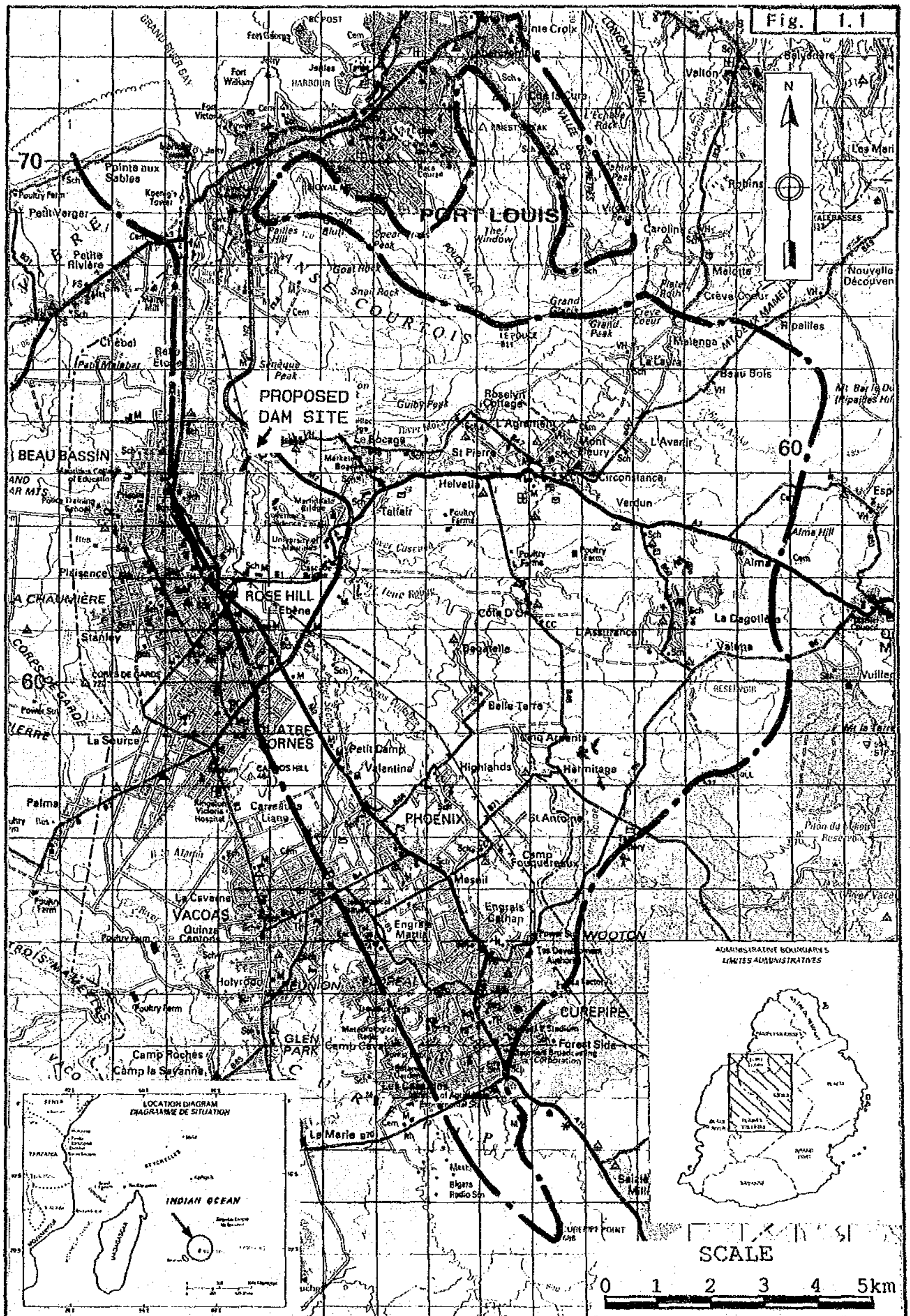
(6) New Transmission Pipeline

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)

FIGURES



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
Class-A pan													
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
Penman's Formula													
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

The river Terre Rouge for the dam and reservoir forms a gorge dissected in the Late Lavas. Top of the lava plateau is at elevation 240 metres to 250 metres in the contemplated dam site, and the river bed is at elevation 120 metres. At the dam site, the river flows westward through a 30 metre wide channel. The river bank rises with steep slopes of 1 vertical to 1 or 1.25 horizontal up to the flat tops of the plateau. A 40 metre wide terrace is formed at elevation 140 to 150 metres on the left bank.

A gorge of Plains Wilhems joins the Terre Rouge from the left bank side at about 350 metres downstream of the dam site. With the Terre Rouge on the east to north and the Plains Wilhems on the west, the plateau on the left bank is as narrow as 400 metres to even 250 metres in the narrowest portion at the contemplated high water level of the reservoir, i.e. elevation 189 metres. When the reservoir is built up in the Terre Rouge valley, this thin plateau has to form a water-tight barrier against the empty Plains Wilhems. The bottoms of these two valleys are nearly at the same level on cross sections right angle to the axis of the plateau.

The bedrock is composed of a number of basalt lava flows, with thicknesses varying from a few to more than 10 metres, which are stratified sub-horizontal. The basalt is generally characterized by coarse plagioclase phenocrysts of about one millimetre in size among basic minerals, and hard and solid if it is fresh. It is often the case that the basalt is vesicular bearing many pores near the upper and lower boundaries of each flow. It is also frequently observed that the basalt is intensively weathered at the top into soft deteriorated rocks and dense or compact residual soil, namely "hard clay" in this report. Sometimes, signs of volcanic ash and old surface soils are seen in the hard clay. A sort of auto-brecciated lavas, mingled with soil or volcanic ash are also observed in the boundary zones between two lava flows, as called flow breccias in this report.

A unit of lava flow is, accordingly, represented by a schematic sequence, in ascending order, of flow breccia, vesicular basalt, non-vesicular massive basalt, weathered vesicular basalt, highly weathered basalt and hard clay, though all the real lava flows do not always include all these members in their cycles.

Stratigraphic correlations, with the boundary features as marker beds, among all the drilling core samples of the dam site indicates wide horizontal development and continuity of each flow though some of thin basalt beds may pinch out and do not endure for a long distance.

Encountered in lower horizons, or in levels lower than elevation 140 metres, are a 20 metre thick porphyritic basalt with relatively large plagioclase phenocrysts of 2 to 3 millimetres and the underlying cycles of fine grained basalt flows. The latter shows an aphanitic texrure, that has too fine grained component minerals to be visible to naked eyes and appears to have no discrete crystalline units. Hence the name of glassy basalt in this report. It is also characterized by inclusion of fluorite and opal in pores.

Considering dominant existence of the similar sort of aphanitic basalt in Mount Ory, a contemplated quarry site, which obviously consists of the Older Volcanic Series for its geomorphological situation, the "glassy basalt" below the river bed of the dam site is deemed to fall under the same old series. In the other hand, the coarse basalts forming the dam abutments may be classified into the Late Lavas in the Younger Volcanic Series.

The above classification of the dam site geology in terms of the theoretical stratigraphy of Mauritious, however, may not be well proved. For the engineering geological purpose of this report, a tentative terminology of Old Lavas and Young Lavas will be applied.

A layer of intermingled basalt and hard clay, named the pyroclastic flow, which has been encountered by a few drill holes near the level of the river bed, is deemed to mark the base of the Young Lavas. An unconformity plane as the boundary of the Old Lavas and the Young Lavas is located between the pyroclastic flow and the underlying porphyritic basalt. Accordingly, the stratigraphic sequence of the dam site is schematically tabulated as shown below:

Volcano-stratigraphic Sequence
of Basaltic Lava Flow at the Dam Site

Classification	Strata		Thickness (m)
Young Lavas	Young Lava 4	(YL4)	40
	Young Lava 3	(YL3)	20
	Young Lava 2	(YL2)	20
	Young Lava 1	(YL1)	40
	Pyroclastic Flow	(YL0)	0 - 10
	Unconformity		
Old Lavas	Old Lava 3 (porphylitic basalt)	(OL3)	5 - 20
	Old Lava 2 (glassy basalt 2)	(OL2)	20
	Old Lava 1 (glassy basalt 1)	(OL1)	7

No major fault has been in the dam site and the reservoir area.

With the sub-horizontal bedding of the lava flows, the stratigraphy of the reservoir area is essentially similar to that of the dam site. Its major part is situated in the Young Lavas, because of the raised river bed higher than in the dam site.

The situation of site geology is presented in Fig. 2.6.2 to Fig. 2.6.3.

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

The detailed geological field investigations in the damsite included (i) core borings with permeability tests, (ii) test groutings, (iii) test aditing and (iv) in-situ rock tests in the test adits, etc.

Results of the above geological investigations are reported in detail in Appendix of this Design Report: that is, Appendix, Design Report for Lot-II, Final Report (2).

TABLES

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/cm^3)	Moisture Content (%)	Specific gravity (g/cm^3)	Water Absorption (%)	Measured Compressive Strength (MN/m^2)	Corrected Compressive Strength (MN/m^2)	Remarks
1.	Old lava (Doleritic)	103.3	51.6	2.563	1.9	2.570	2.3	36.3	36.3	Dam Foundations
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	
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	

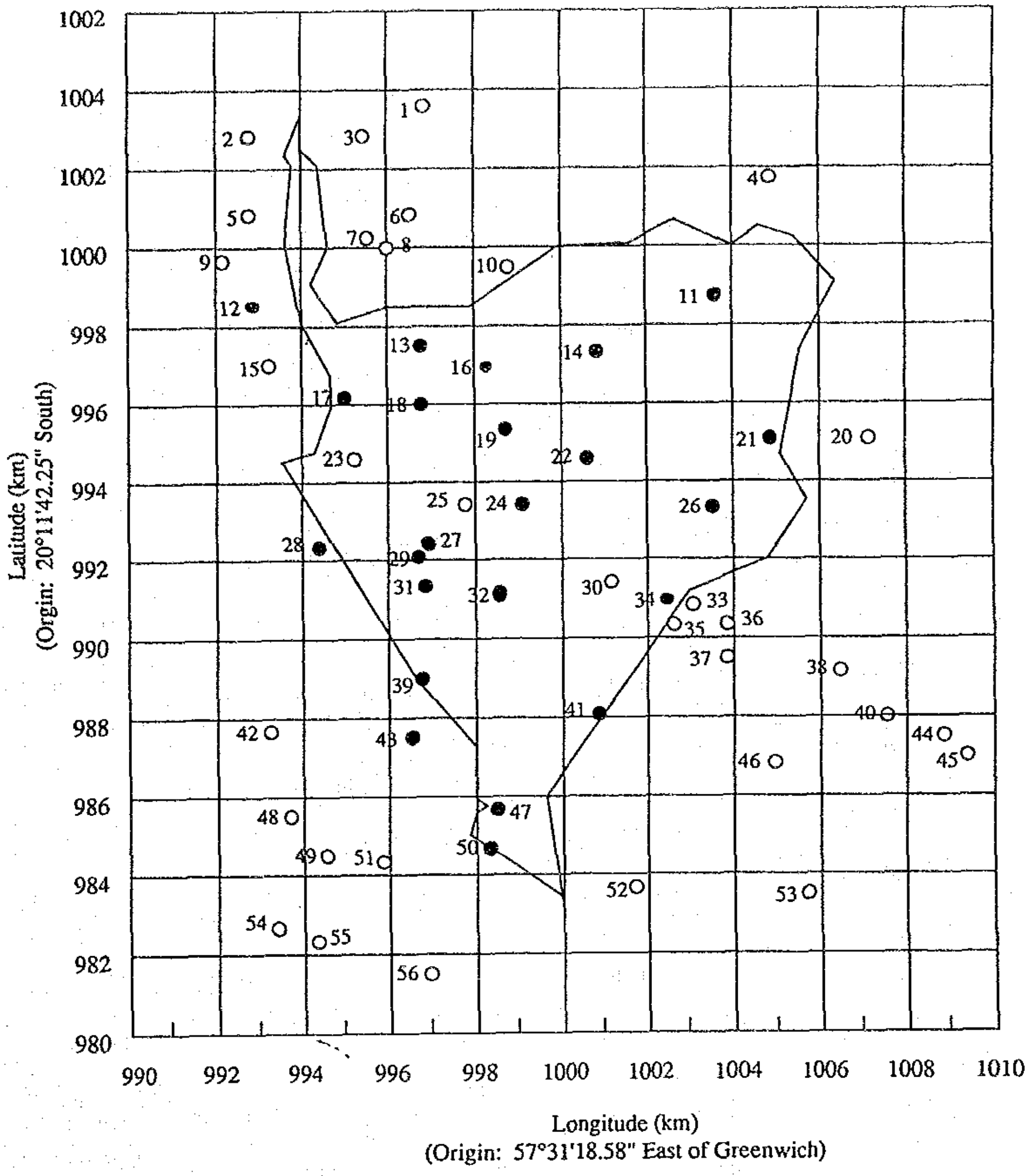
Note: * The water absorption value cannot be determined for samples Nos. 8 to 11 because the materials were of highly weathered nature and the specific gravities were conducted as for soil materials.

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

FIGURES

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	Quare 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)

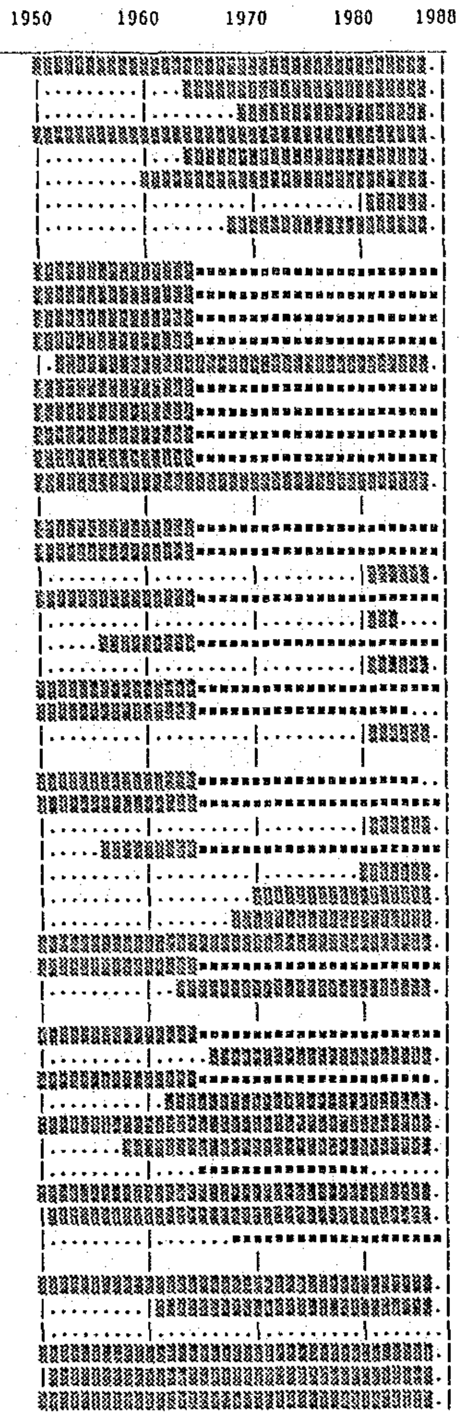
— Boundary of GRNW Basin

LOCATION OF RAINFALL STATIONS IN AND AROUND GRNW

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

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 Guibies	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.M	(**)	Bega	137	285
18	DD314	-	(**)	Le Reduit Experi. St.	139	293
19	FF303	W7.M	(**)	Minissy (MDA)	135	302
20	EE301	E15.MDA		Bonne Veine	143	353
21	FF405	W19.MDA	(**)	Alma	144	340
22	FF306	W11.M	(**)	Cote d'Or	147	315
23	147285			Ebene	147	285
24	FF307	W9.M	(**)	Bagatelle (M)	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.M		Hermitage	161	318
31	FF312		(**)	Phoenix	164	295
32	FF313	W8.M	(**)	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	-	(**)	Wooton	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



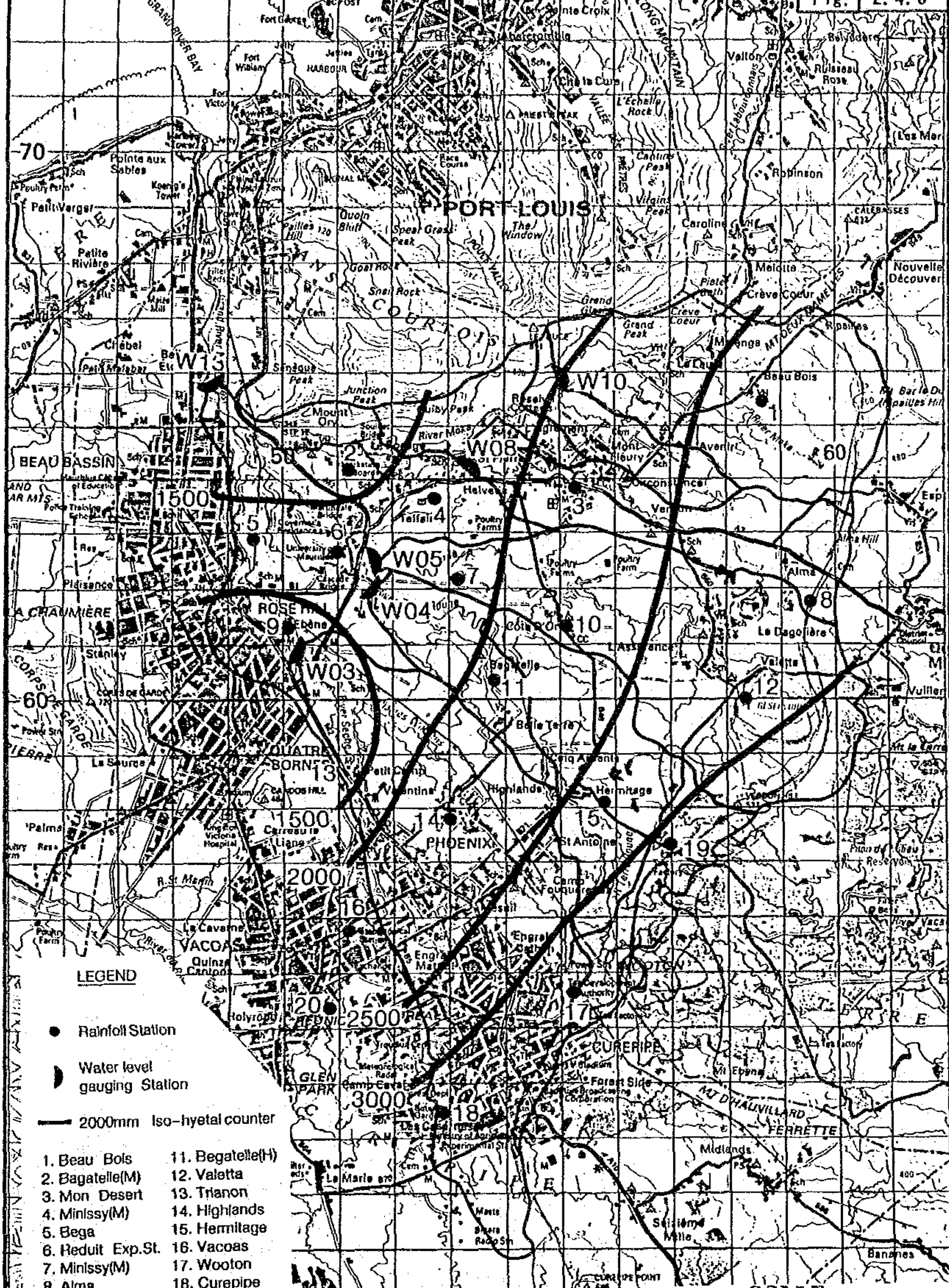
Ref: (1) Identifier by Meteorological Service
(2) Identifier by Hydrological section, CWA
(**) Selected stations for daily dase analysis (1965-1987)
▓▓ Period in which monthly data are available
** Period in which daily data are available

DURATION OF RECORD OF RAINFALL STATIONS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.4.3

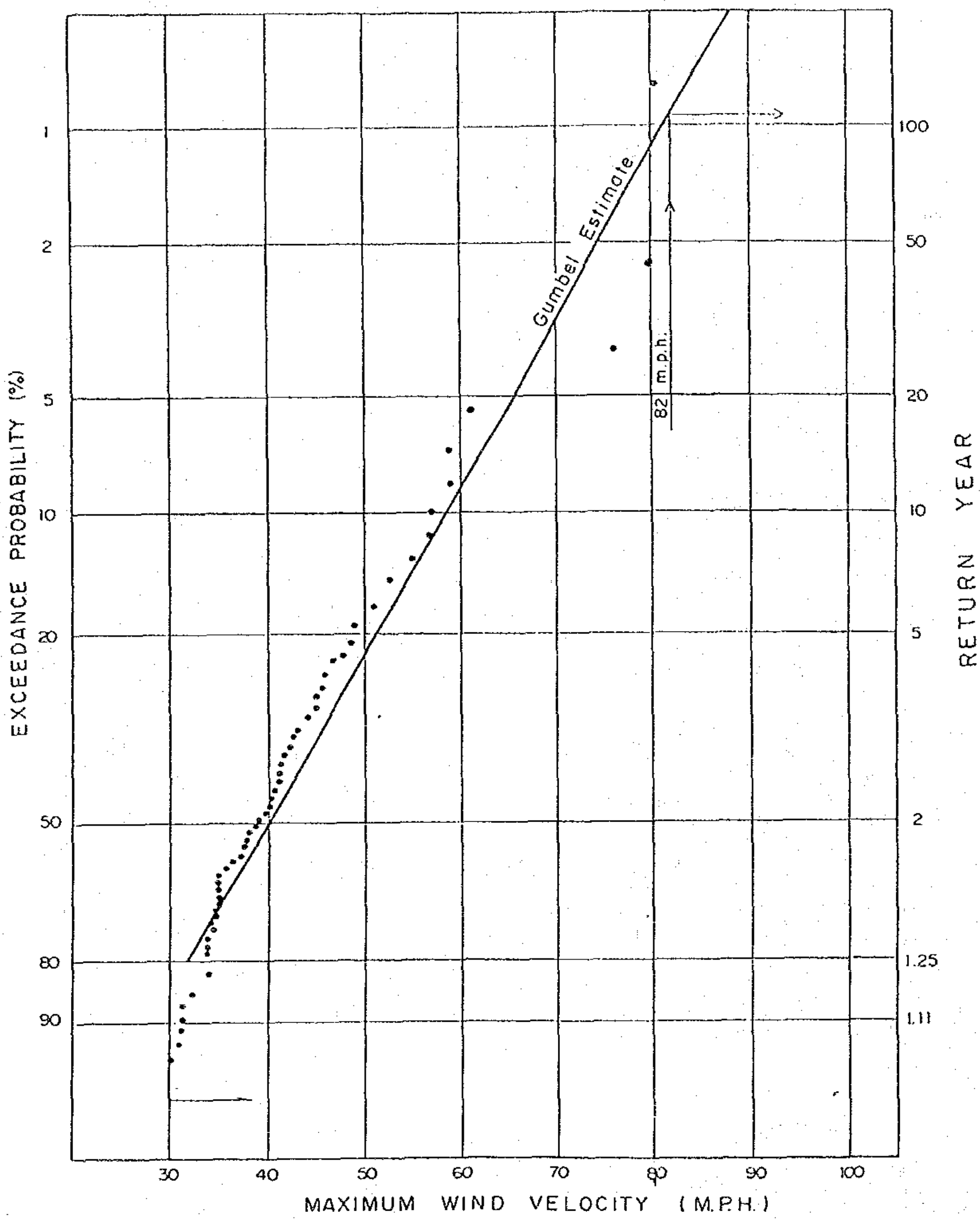


LOCATION MAP OF WATER LEVEL GAUGING STATION AND SELECTED RAINFALL STATION

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

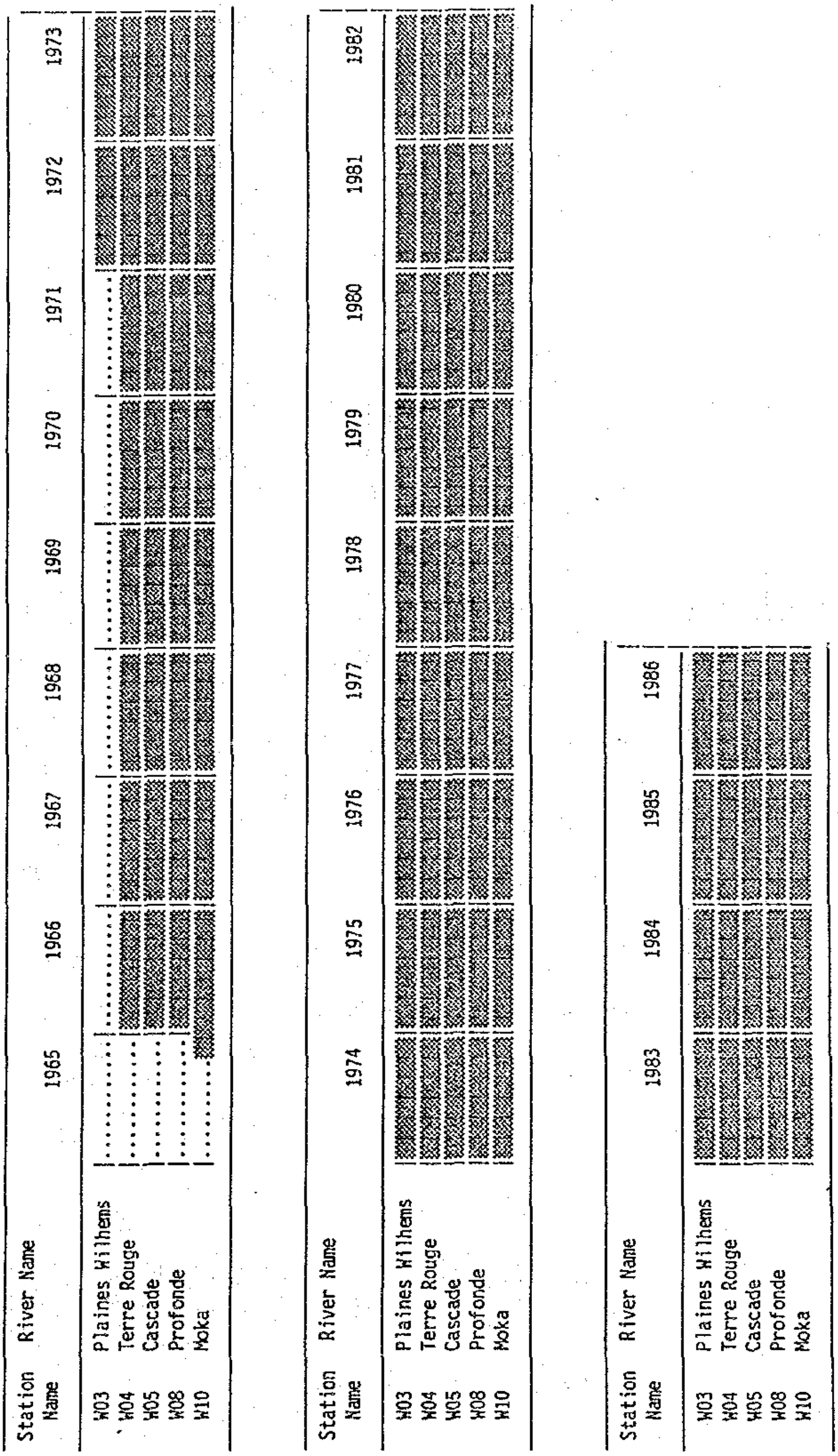
JAPAN INTERNATIONAL COOPERATION AGENCY

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

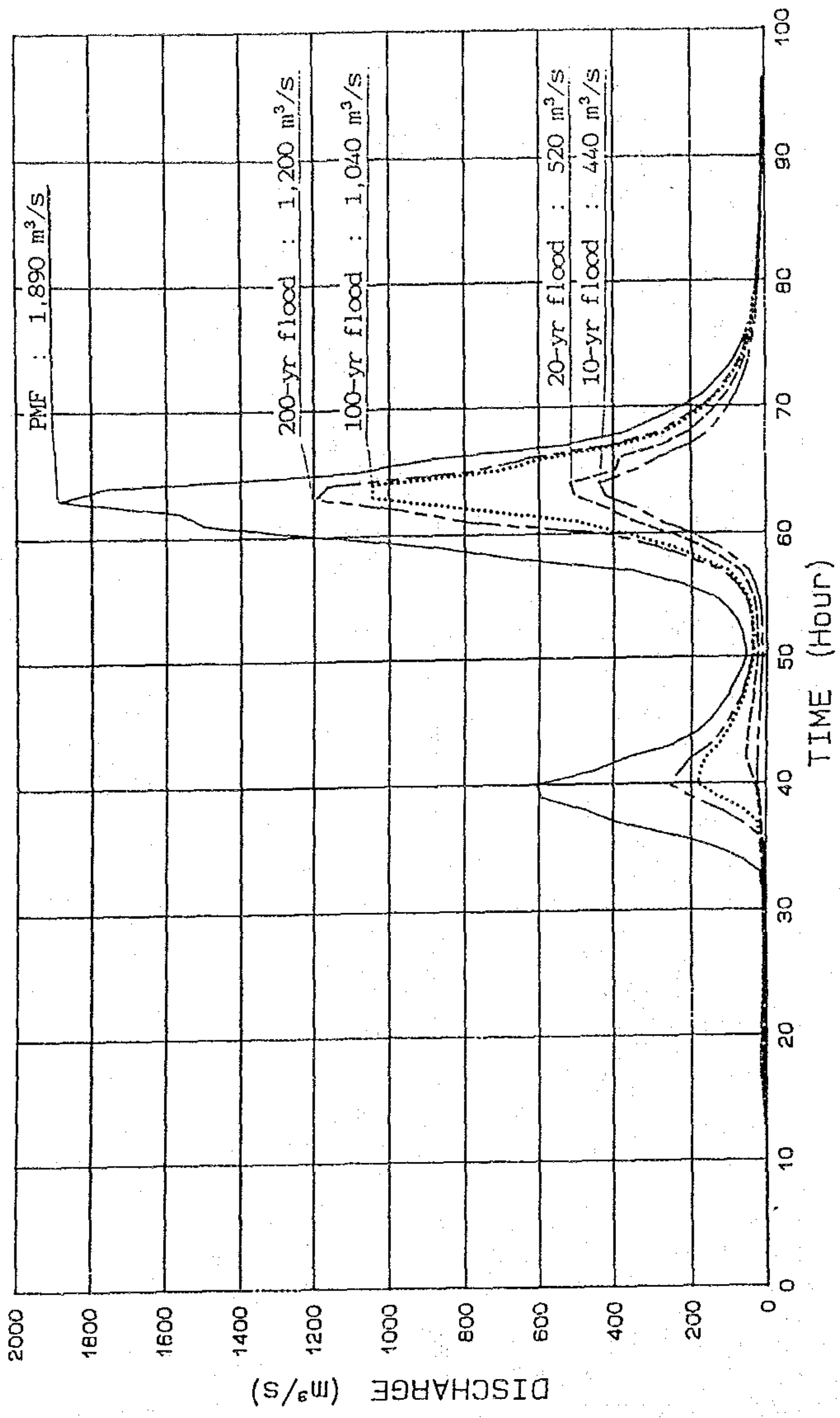


DURATION OF RECORD OF WATER LEVEL GAUGING STATIONS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.4.6

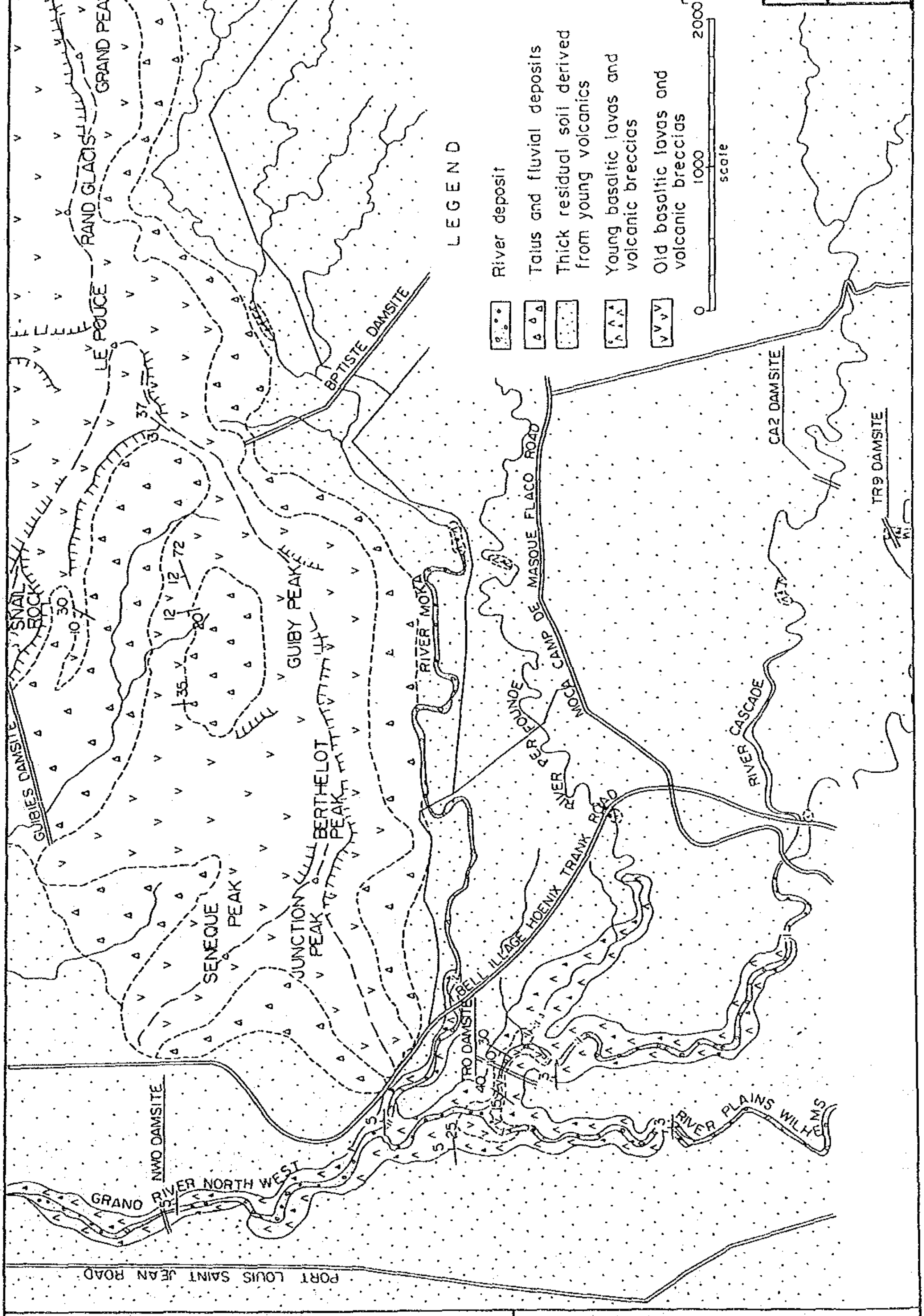


PROBABLE FLOOD HYDROGRAPH

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig. 2.6.1

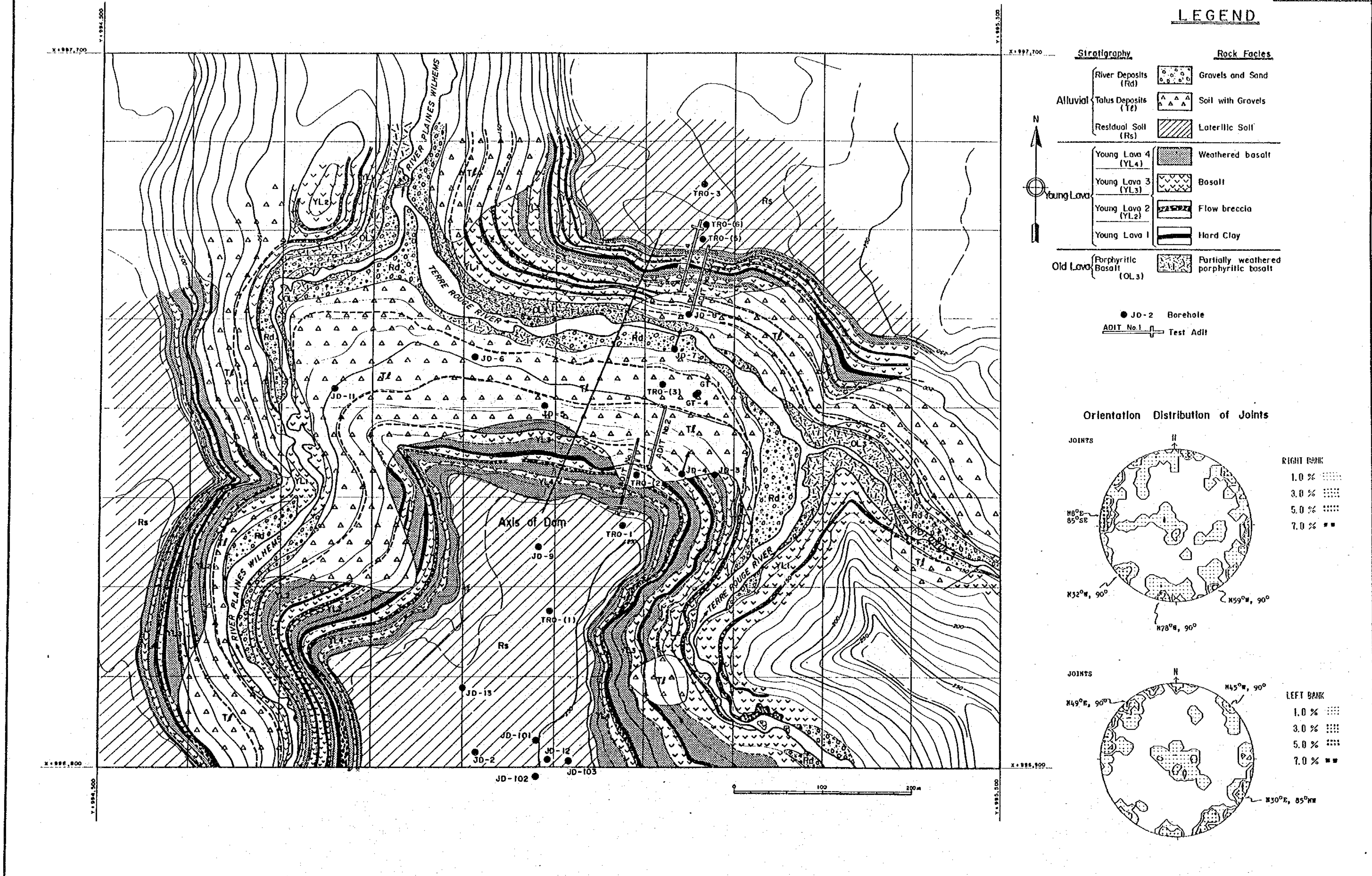


REGIONAL GEOLOGICAL MAP

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig 2.6.2

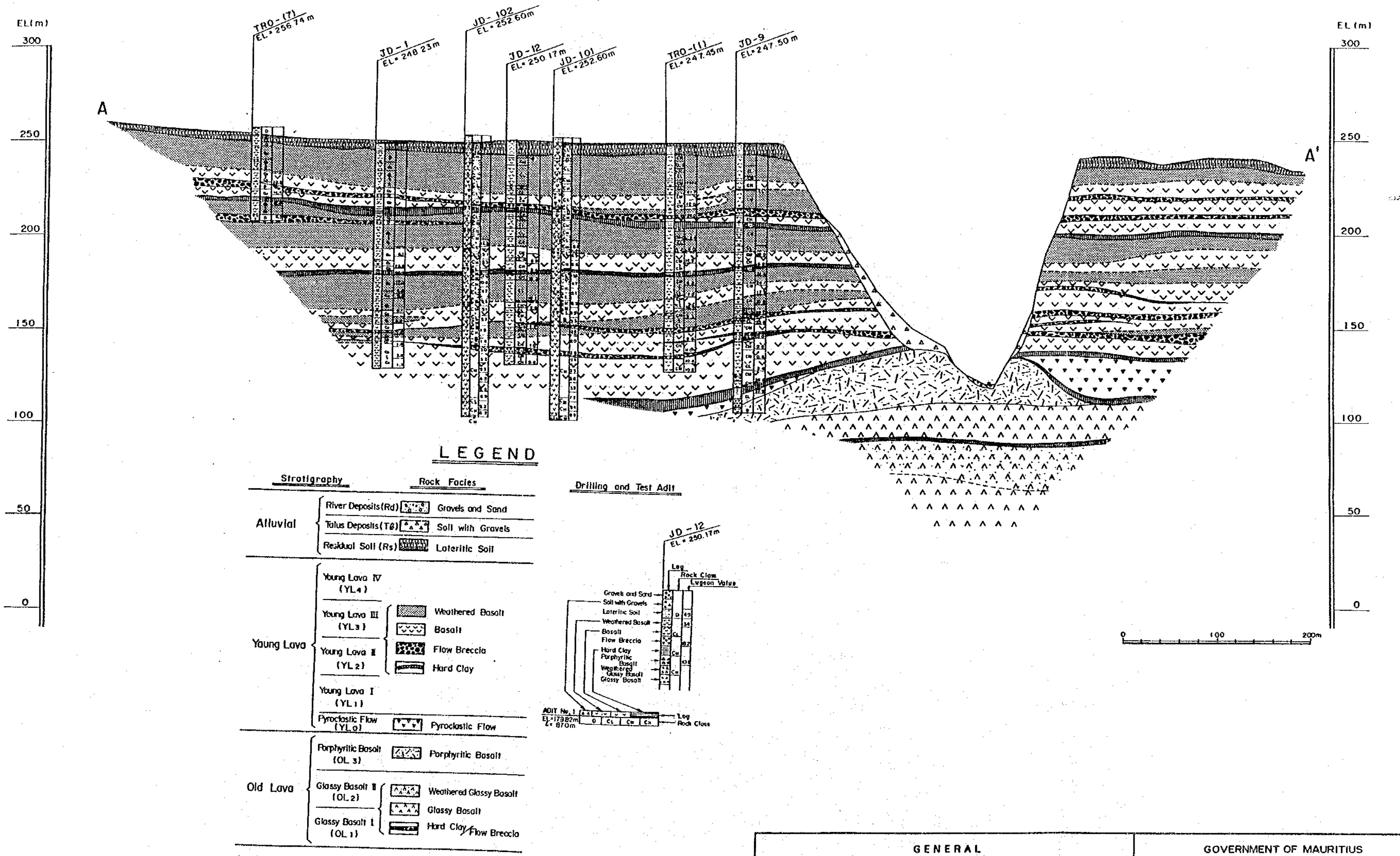


GENERAL
GEOLOGICAL MAP OF THE DAMSITE

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

Fig 2. 6. 3



GENERAL
GEOLOGICAL PROFILE ALONG
THE DAM AXIS

GOVERNMENT OF MAURITIUS
PORT LOUIS WATER SUPPLY PROJECT

JAPAN INTERNATIONAL COOPERATION AGENCY

CHAPTER III. DAM DESIGN

3.1 Damsite and Type of Dam

The proposed damsite is situated at about 6 km from the estuary of G.R.N.W (the Ground River Bay). Just downstream of the TRO dam site G.R.N.W starts after the confluence between the Terre Rouge river and the Plaines Wilhems. The damsite is situated in the straight section between two confluences; one is the starting point of G.R.N.W at the downstream site and the other is between the Terre Rouge river and the Profonde river. The proposed dam will be embanked in a deep gorge with the difference of height of 130 m; the elevation of top of the steep river bank is about 250 m and the riverbed at 120 m.

The most desirable dam type has been examined through the Feasibility Study and reviewal studies in the Basic Design.

The examination on the dam type included comparative studies on various dam types such as the concrete gravity dam, earthfill dam, rockfill dam (with center core and inclined core), upstream impervious membrane type dam, etc., and the study finally selected the rockfill type dam with center core as most suitable from both technical and economic aspects.

The above comparative study is detailed in the Basic Design Report. Following is a summary of the comparative study:

(i) The concrete gravity dam will be much costlier due to the weak foundation strength.

(ii) The project area has difficulty to obtain such a large amount of earth material required for the homogeneous earthfill type dam, resulting in a costlier scheme.

(iii) In the case of the upstream impervious membrane type, a technical problem was pointed out: that is, the upstream impervious membrane tends to be subject to damages due to settlement of dam which will inevitably be caused. The membrane is to be supported at one thin line (at one point in a section). Especially in the case that the foundation to support the membrane is not homogeneous mechanically including highly weathered portions like the proposed damsite, the membrane will have a high possibility to cause differential settlement, leading to a fatal damage of the dam.

Furthermore, it was also revealed that its cost will not be less than that of the rockfill dam with the usual impervious core.

(iv) In the comparison between dams with center core and inclined core, the dam with center core was finally selected in the following views:

- The behavior of the impervious core against dam settlement is much more favorable in the dam with center core, meaning that the dam with center core has higher safety.

- The dam with center core has a slight economic merit since its dam slope can be steeper than the dam with inclined core.

- Although the dam with inclined core has a merit that it has a flexibility in the construction schedule, the dam with center core was finally recommended in view of two merits as mentioned above.

3.2 Reservoir Capacity and Dam Height

The Stage-Storage-Area curve of the reservoir is given in Fig. 3.2.1.

The effective reservoir storage required to meet the water supply requirement is found to be 6.3 MCM. L.W.L and H.W.L of the reservoir are determined at EL. 139.0 m and EL. 189.0 m, taking the above required effective storage into consideration. The determination of the above L.W.L and H.W.L also, considers the necessary dead storage to accommodate the sediments for 100 years and the estimated evaporation loss from the reservoir surface.

The determination of dam crest level is related to the spillway crest length to be provided, since the dam has to be provided with the necessary free board. The most economical dam crest level in relation to the spillway crest length should be determined. Thus, the relation between the dam crest level and spillway crest length is obtained through flood routings for several assumed spillway crest lengths. The probable maximum flood (PMF) of which peak discharge is worked out at 1,890 m^3/s is applied for the flood routings.

The relationship between the spillway crest length and the cost of dam and spillway is given in Fig. 3.2.2 which indicates the most economical combined cost at the spillway crest length of 90 m. (It is noted that the spillway crest length is finally adjusted into 92 m as a result of the spillway model test.) Hence, the design flood water level is EL. 193.5 m.

Then, the dam crest level is determined at EL. 196.0 m as follows: The sum of 0.7 m in wave height and 1.0 m for necessary allowance is less than 2.0 m which is the specified minimum requirement of free board for the dam. Thus, the free board of 2.0 m (up to the top of the core above the flood water level) is provided. Assuming a surfacing of 0.5 m thick above the top of the core, the dam crest level comes to EL. 196.0 m.

3.3 Zoning of Dam

The dam is a zoned rockfill dam with impervious center core. The maximum height of dam above core foundation is measured at 84 m. The crest length is 230 m. A width of 10 m is provided at the dam crest EL. 196 m in consideration that the dam crest may be used as a traffic road in future. The dam stability analysis determined the upstream dam slope of 1 to 2.3 and downstream dam slope of 1 to 1.8, respectively.

The dam is composed of the cofferdam and the main dam as seen in Fig. 3.3.3. The cofferdam forms the upstream part of the dam, having the impervious core, filter and rock zones in it. The main dam which is also zoned into the impervious core, filter and rock zones will be embanked in immediate downstream of the cofferdam.

As seen in Fig. 3.3.3, the dam embankment of about 1.4 million m^3 in total is divided into five (5) zones: those are (i) the central impervious core zone of 0.24 million m^3 in volume, (ii) fine filter zone of 0.069 million m^3 in volume, (iii) coarse filter zone of 0.063 million m^3 in volume, (iv) pervious rock zone of 0.99 m^3 in volume and (v) riprup in the upstream slope surface of 0.017 million m^3 in volume.

The impervious core is provided with the thickness not less than 30% of water depth on the basis of the standard saying that generally, in the case of dams of thickness equal to 30 to 50 % of water depth, the structure is considered stable even under extremely adverse conditions. As for the thickness of filter, the standard mentions that its minimum thickness should be 2.4 to 3.0 m and that it is desirable to provide a thickness of 3.6 to 4.2 m. Taking into consideration the properties of available core material for the project which is classified into MH soil and in due consideration of the above standard, the double filter consisting of fine and coarse filters, each having 3.0 m thickness, is provided.

3.4 Excavation of Dam Foundation

Fig. 3.4.1 shows the excavation plan in the dam foundation. Fig. 3.3.4 shows a profile along the dam axis.

The dam excavation plan is prepared with the following considerations:

(i) The impervious core and filter zones should be founded on a sound foundation to keep sufficient water tightness and to avoid any differential settlement. Thus, the top soil and highly weathered portions should be excavated to expose a sound foundation.

(ii) In the rock zone, its foundation should not include any organic materials such as grass on roots of tree. Therefore, the surface excavation with 2.0 to 3.0 m thickness is made for the rock zone to eliminate all the organic materials.

(iii) Some talus deposits exist in the rock zone foundation. The talus deposits do not have to be excluded unless it effects on the dam stability. Hence, some talus deposits remain in the rock zone foundation based on the dam stability analysis.

(iv) The damsite geology consists of the horizontal alternate layers of sound basalt lava and highly weathered portion. As such, the surfaces of the highly weathered layer are forced to expose on the foundation of impervious core and filter zones. Some treatments are required for the above highly weathered layers on the foundation to avoid possible differential settlement and damage on the impervious core.

(v) The slope of core foundation will not be steeper than 1 to 0.7 (55° with horizontal) to secure a sufficient contact with the foundation in due consideration of the standard for abutment slope saying that commonly, adopted slopes are within the range of about 45° (1 to 1.0) to 70° (1 to 0.36) with the horizontal. The excavation plan of the impervious core zone in the right abutment which has a very steep slope is made to meet the above requirement.

3.5 Dam Materials

(1) Impervious Core Material

The proposed borrow area for the impervious core material is located on the right bank upstream of the damsite as seen in Fig. 3.5.1. This borrow area has been selected through investigations on several conceivable alternatives in the Feasibility Study stage.

The Detailed Design works carried out additional investigations for the selected borrow area, including test pittings, samplings, and laboratory tests such as (1) specific gravity, (2) moisture content, (3) grain size analysis, (4) liquid limit, (5) plastic limit, (6) soluble salt determination, (7) moisture-density relations, (8) permeability, (9) triaxial shear, (10) consolidation, etc.

The test results are summarized in Table 3.5.1 to 3.5.3.

Based on the test results, the design values of the impervious core material is determined as follows:

Design Values of Impervious Core Material

- Specific gravity 2.88
- Natural moisture content 40%
- Dry density .... 1.23 t/m^3
- Wet density .... 1.72 t/m^3
- Saturated density .... 1.80 t/m^3
- Submerged density .... 0.80 t/m^3
- Coefficient of permeability .... 1×10^{-5} cm/sec
- Cohesion c' .... 0 t/m^2
- Internal friction angle ø' .... 30 degree

The above design values are obtained as follows:

Specific gravity (Gs):

The specific gravity is obtained as the average of test results shown in Table 3.5.1.

Natural moisture content (Wn):

The natural moisture content is also obtained as the average of test results shown in Table 3.5.1.

Dry density (γd):

The dry density is obtained as 95% of average of test results of maximum dry density shown in Table 3.5.1.

Wet density (γt):

The wet density is obtained by the following equation:

$$\gamma_t = \gamma_d\,(1 + W_n/100)$$

Saturated density (γsat):

The saturated density is obtained by the following equation:

$$\gamma_{sat} = \gamma_d\,(1 + \gamma_w/\gamma_d - 1/G_s)$$
$$\gamma_w = 1.0\ t/m^3$$

Submerged density (γsub):

The submerged density is calculated by the following equation:

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$
$$\gamma_w = 1.0\ t/m^3$$

Coefficient of permeability (k):

As seen in Table 3.5.1, the coefficient of permeability less than 1×10^{-6} cm/sec is indicted in the laboratory test. However, the design value of the coefficient of permeability for impervious core is conservatively determined to be 1×10^{-5} cm/sec.

Mechanical strength (c', ø'):

The mechanical strength, the cohesion and internal friction angle, is determined conservatively at the design value slightly less than test results shown in Table 3.5.1.

Gradation curves of the impervious core material are given in Fig. 3.5.3. Particle size of the material is very fine.

Soluble salt is tested to be 175 parts/million in average in the range from 100 to 250 parts/million, and the material belongs to a non-dispersive soil in consideration of soluble salt values of 2,000 to 2,500 parts/million shown in dispersive soils.

Further, the tests revealed that the embankment of core material should be undertaken under the following conditions:

As the content of the particles is more than 90%, trafficability of the earth material will be remarkably bad during rainy seasons. For this reason, construction period of the earth material should be limited to only dry seasons, during which field moisture content will be easily adjusted in field, because the natural moisture content of the earth material will be close to its OMC.

For quality control, the earth material is usually compacted at dry density of more than 95% of $\gamma dmax$ which is maximum dry density. Required range of moisture content to attain sufficient compaction effect or dry density more than 95% of $\gamma dmax$ will be OMC to OMC + 3% in field. This is because over compaction may occur and it will be impossible to obtain sufficient trafficability when the material is compacted at moisture content of wet side more than OMC + 3% and embankment of the fine-grained soils like the earth material used in the project tends to be stiff and brittle when compacted at a moisture content several percent below OMC whereas the compacted soils are capable of appreciable deformation without occurrence of crack when compacted at a few percent above OMC.

(2) Filter and Rock Materials

The filter material is planned to be taken from the same quarry site as rock material shown in Fig. 3.5.1. The laboratory test results for filter material are given in Table 3.5.4. Specific gravity of the material on oven dry condition is 2.891 in average. Specific gravity of the material on saturated surface dry condition is 2.913 in average. Absorption is tested to be 0.75% in average, ranging from 0.65 to 0.93%. Loss in weight of soundness test by using sodium sulfate is 2.3% in average. Loss in weight of abrasion by the Los Angeles machine is 16.5% in average. As such, the test results indicate that the material properties will sufficiently be suitable for the filter material.

As mentioned, the soil material available for the impervious core belongs to MH soil with very fine particles which relatively tends to cause a concentrated water leakage resulting in the piping. For preventing the concentrated leak and piping of core material, special care should be paid for the design of filter. Hence, the double filter consisting of the fine and coarse filters is designed to be provided. In due consideration of sizes of core material and rock zone material, the following gradations are recommended for each of fine and coarse filters.

Fine filter

Grain size (mm)	Percentage (%)
0.075	0
0.2	15
1.2	50
1.7	60
3.5	85
5.0	100

Coarse filter

Grain size (mm)	Percentage (%)
0.075	0
3.4	15
5.0	20
24.5	50
36.0	60
70.0	85
100.0	100

The range of gradation in each filter is as shown in Fig. 3.5.4 together with those of impervious core and rock material.

Determination of the above range of gradation is based on the following standard criteria of the filter design:

(a) Filter material shall be cohesionless, and its fine content, which is gravimetric percentage of particles less than 0.075 mm, shall be less than 5%.

(b) Fine filter material with D_{15} size of 0.5 mm or smaller are conservative filters for most fine-graded soils in nature.

(c) $D_{15}/d_{15} > 5$

where D_{15} 15% diameter of soil particle of filter material

d_{15} 15% diameter of soil particle of material protected by filter

(d) $D_{15}/d_{85} < 5$

where D_{15} 15% diameter of soil particle of filter material

d_{85} 85% diameter of soil particle of material protected by filter

(e) Grading curves of recommendable fine and coarse filter materials are desirable to be parallel with core and recommendable fine filter materials, respectively. Grading curve of recommendable rock material is also desirable to be parallel with recommendable coarse filter material.

(f) Grading curves of upper and lower limits are desirable to be parallel with each recommendable grading curve.

Da to Dh in Fig. 3.5.4 are determined in accordance with the above criteria as follows:

- Da

  Da is 0.075 on the basis of criteria (a), and percentage passing of Da is 5%.

- Db

  Db is 0.5 mm on the basis of criteria (b), and percentage passing of Db is 15%.

- Dc, De, Df

  Dc to Df are determined on the basis of criteria (c) and (d). Dc to Df are 1.0 mm, 17.5 mm and 17.0 mm, respectively. However, Dc and De are influenced by the ranges of fine filter and rock materials. Dc and De are decided to be about 2.0 mm and 8.0 mm, respectively. And percentage passing of Dc to Df is 15%.

- Dg

  Dg is controlled by Dh and criteria (f). Dg is assumed to be 70 mm, and percentage passing of Dg is 15%.

- Dh

  Dh is controlled by spreading thickness of rock zone in field. In general, rolling thickness of rock zone is less than 1 m. Dh should be 1.0 m, and percentage passing of Dh is 100%.

Design values of filter and rock materials are determined as shown in Table 3.5.6. Determination of the above design values is based on the followings: that is, Table 3.5.4 and 3.5.5 indicate test results for filter and rock material, respectively. Tests for filter material were made on fresh hard rock. On the other hand, rock material was tested by using the cores of boring performed in the proposed quarry site which include some weathered parts and were somewhat disturbed by drilling works. Therefore, relatively higher quality is shown in test results of filter material. Thus, design values are determined in due consideration of the above both test results as follows:

<u>Specific gravity (Gs)</u>:

The design value for specific gravity of filter and rock materials is provided with the average value of test results in Table 3.5.4 and test results in Table 3.5.5 excluding those on weathered rocks.

<u>Water absorption (W)</u>:

The water absorption of filter and rock materials is taken conservatively at 2.0 % which is the average of test results in Table 3.5.5 excluding those on weathered rocks. For a conservative sake, test results in Table 3.5.4 are not taken into account.

<u>Dry density (γd)</u>:

Determination of dry densities is based on various past examples as follows:

Dry densities of actual dams in Japan ranges from 1.6 to 2.3 t/m^3. The data of dry densities mostly indicate values larger than 1.9 t/m^3, concentrating on about 2.1 t/m^3. Hence, the design value of dry density for rock material is determined to be 2.1 t/m^3, implying that the compaction energy to attain the dry density not less than 2.1 t/m^3 should be given in rock embankment works.

The design value of dry density for fine filter is determined to be 1.9 t/m^3 in consideration that the gradation of filter tends to be poorer than that of rock material. Then, the dry density of coarse filter is taken at 2.0 t/m^3 which is the average of fine filter and rock material.

<u>Wet density (γt)</u>:

The wet density is calculated by the following equation:

$$\gamma_t = \gamma_d (1 + W/100)$$

The wet densities of 1.94 t/m^3, 2.04 t/m^3 and 2.14 t/m^3 are obtained for fine filter, coarse filter and rock material, respectively.

Submerged density (γ_{sub}):

The submerged density is calculated by the following equation:

$$\gamma_{sub} = \gamma_d (1 + \gamma_w/\gamma_d - 1/Gs) - \gamma_w$$
$$\gamma_w = 1.0\ t/m^3$$

The submerged densities are calculated at 1.23 t/m^3, 1.30 t/m^3 and 1.37 t/m^3 for fine filter, coarse filter and rock material, respectively.

Coefficient of permeability (k):

The design value of permeability coefficient is based on the Creager's values: Creager proposes the permeability coefficients in accordance with the gradation. Based on the above Creager's values and in due consideration that the uniformity coefficient assumed by Creager is smaller than the filter and rock materials of the project, the coefficients of permeability are determined to be 1×10^{-3} cm/sec, 1×10^{-2} cm/sec and 1×10^{-1} cm/sec for fine filter, coarse filter and rock material, respectively.

Mechanical strength (c' and ø'):

Determination of design values for mechanical strength such as cohesion (c') and internal friction angle (ø') is based on results of the large scale direct shear test carried out under consolidated-drained condition. As seen Fig. 3.5.5 and 3.5.6, the cohesion and internal friction angle of rock material are tested at 0.14 kg/cm^2 and 42 degree, respectively. Based on this test result, the design values of cohesion and internal friction angle for rock material are determined conservatively at 0 kg/cm^2 and 40 degree, respectively.

Considering that the dry density of filter material will be smaller than that of rock material, less values are given to filter material as follows: Cohesion of 0 kg/cm^2 and internal friction angle of 36 degree for fine filter, and cohesion of 0 kg/cm^2 and internal friction angle of 38 degree for coarse filter.

3.6 Dam Stability Analysis

3.6.1 Method of Dam Stability Analysis

The dam stability is analyzed by the slip circular method which examines trially the safety against slip along various assumed failure circles.

The factor of safety is given by the following equation:

$$Fs = \frac{\Sigma\{C \times L + (N - U - Ne) \times \tan \Phi\}}{\Sigma (T + Te)}$$

where,

Fs	:	Factor of safety
N	:	normal force acting on slip circle of each slice
T	:	tangential force acting on slip circle of each slice
U	:	pore pressure acting on slip circle of each slice
Ne	:	normal force of earthquake load acting on slip circle of each slice
Te	:	tangential force of earthquake load acting on slip circle of each slice
Φ	:	angle of internal friction of materials on slip circle of each slice
C	:	cohesion of materials on slip circle of each slice
L	:	arc length of slip circle of each slice
Σ	:	Summation for all slices

The factor of safety without earthquake force is obtained by substitution Ne = 0 and Te = 0 into the above equation. The normal force (N) and tangential force (T) are defined as:

$$N = W \cos\theta + Pn = W \cos\theta + P \sin\theta$$

$$T = W \sin\theta + Pt = W \sin\theta + P\cos\theta$$

where,

θ	:	angle between horizontal and the tangent of the arc at slice
W	:	total weight of slice (embankment and water)

$$W = (D1\ wet\ .h1 + D1\ sat\ .h2 + D2\ sat\ .h3) * \Delta X$$

or $$W = (D2\ wet\ .h4 + D2\ sat\ .h5) * \Delta X$$

in which

ΔX	:	width of a slice
Di wet	:	wet density of material (i)
Di sat	:	saturated density of material (i)

hi : height of a slice of material (i)

P : differential water pressure acting on both sides of a slice

$$P = Dw\,(h2 + h3 - \Delta X/2 \times \tan\theta) \times \Delta X \times \tan\theta$$
$$= (h2 + h3) \times \Delta X/2 \times \tan\theta \qquad (Dw = 1.0\ t/m^3)$$

The normal and tangential forces due to earthquake are defined as follows:

$$Ne = W \times k \times \sin\theta$$
$$Te = W \times k \times \cos\theta$$

where,

k : horizontal seismic coefficient

Besides the above, the safety should be checked for plane surface sliding of cohesionless materials.

The factor of safety against plane surface sliding of cohesionless materials is given by the following equations. These equations are derived from the slip circular equation assuming a radius of circle to be infinite; that is, infinite slope.

(a) Submerged slopes (upstream slopes)

$$Fs = \frac{m - k \times R}{1 + k \times R \times m} \times \tan\phi$$

(b) Dry slopes (downstream slopes)

$$Fs = \frac{m - k}{1 + k \times m} \times \tan\phi$$

where,

Fs : factor of safety

m : slope gradient (1 : m)

k : horizontal seismic coefficient

ϕ : internal friction angle

R : Dsat/Dsub

D_{sat} : saturated density of material

D_{sub} : submerged density of material (Dsat - 1.0)

3.6.2 Design Value

The design values for dam stability analysis are determined on the basis of the field investigation results on construction materials.

Although Section 3.5 "Dam Materials" of this report presents explanations on determination of design values, those are reproduced below:

Design Values of Dam Materials

Item		Earth	Filter		Rock
			Fine	Coarse	
Specific gravity		2.88			
(Oven dry condition)			2.87	2.87	2.87
Natural moisture content	(%)	40.00			
Water absorption			2.00	2.00	2.00
Dry density	(tf/m^3)	1.23	1.90	2.00	2.10
Wet density	(tf/m^3)	1.72	1.94	2.04	2.14
Saturated density	(tf/m^3)	1.80	2.23	2.30	2.37
Submerged density	(tf/m^3)	0.80	1.23	1.30	1.37
Coefficient of permeability	(cm/sec)	1×10^{-5}	1×10^{-3}	1×10^{-2}	1×10^{-1}
Strength parameter (effective stress analysis)					
Cohesion c'	(tf/m^3)	0.00	0.00	0.00	0.00
Internal friction angle (ø)	(degree)	30.00	36.00	38.00	40.00

The dam design considers that some talus deposits be left unexcavated in the rock zone of dam on the basis of dam stability analysis results. Design values for the talus deposits are conservatively assumed to be same as those of the fine filter material. Further, design values for coarse filter are also assumed to be same as those of fine filter for a conservative sake.

As for the seismic coefficient, the seismic coefficient of $k = 0.05$ is taken into consideration in the analysis in accordance with the standard which states that $k = 0.05$ should be taken into consideration even in areas having no earthquakes.

The safety factor which the dam has to secure is more than 1.2 in accordance with the standard.

3.6.3 Examined Cases

(1) Present Scheme and Future Expansion Scheme

The present project design considers to make it possible to expand the dam height if required in future.

Thus, the dam stability and dam slope should be examined in due consideration of both the present and expanded schemes. Further, the safety of the main cofferdam should be confirmed.

Hence, the dam stability is examined for the following three (3) cases:

(a):	Present scheme	(Dam crest	EL. 196.0 m)
(b):	Expanded scheme	(Dam crest	EL. 215.0 m)
(c):	Main cofferdam	(Dam crest	EL. 155.5 m)

(2) Loading Conditions

The dam will be subject to various loading conditions for which the safety of dam should be ensured.

In accordance with the standard, the dam stability is examined for the following loading conditions:

(a) Reservoir normal high water level with the full seismic load.
(b) *Rapid drawdown of water level with the full seismic load.*
(c) Immediately after completion with a half of seismic load.
(d) Design flood water level without seismic load.

(3) Dam Sections

Talus deposits will remain in the upstream rock zone foundation of left bank and in the downstream rock zone foundation of right bank where the critical sliding failure circles appear.

Therefore, the following two dam sections are taken into consideration in the analysis:

(a) Left bank dam section which include talus deposits for determination of upstream dam slope.

(b) Right bank dam section which include talus deposits for determination of downstream dam slope.

3.6.4 Results of Stability Analysis

The computer program calculated the factor of safety for various assumed failure circles in respective cases and indicated the minimum value of the factor of safety in respective cases, which are summarized in Table 3.6.1 to Table 3.6.4. The failure circle along which the minimum value of safety factor appeared is shown in Fig. 3.6.1 to Fig. 3.6.5.

As seen, the factor of safety more than 1.2 is surely secured in the upstream dam slope of 1 to 2.3 and downstream dam slope of 1 to 1.8.

Under the above slope of dam, the safety for plane surface sliding is also secured as follows:

Submerged slope (upstream slope)

$$Fs = \frac{m - k \cdot R}{1 + k \cdot R \cdot m} \times \tan \phi$$

$$= \frac{2.3 - 0.05 \times \frac{2.37}{1.37}}{1 + 0.05 \times \frac{2.37}{1.37} \times 2.3} \times \tan 40°$$

$$= \frac{2.213}{1.199} \times 0.839 = 1.548 > 1.2$$

Dry slope (downstream slope)

$$Fs = \frac{m - k}{1 + k \cdot m} \times \tan \phi$$

$$= \frac{2.3 - 0.05}{1 + 0.05 \times 2.3} \times \tan 40°$$

$$= 1.693 > 1.2$$

3.7 Examination on Soft Layers in Dam Foundation

3.7.1 General

The geology of both abutments of the dam site consists of alternate layers of hard basalt layer and weathered soft layer.

Fig. 3.7.1 shows situations of hard basalt layers weathered soft layers, and talus deposits to be removed. The impervious core to be embanked on such alternate layers may cause differential settlements, resulting in occurrence of cracks in the impervious core.

Such being the case, analyses of deformation and stress by the Finite Element Method are made to provide countermeasures if necessary.

The analyses are made with a model as shown in Fig. 3.7.2. Elastic moduli used in the analyses are assumed on the basis of the in-situ rock tests carried out in the test adits as follows:

	Elastic Modulus (kg/cm^2)
Hard basalt	55,000
Soft layer	2,000
Core material	500

The unit weight of 1.8 t/m^3 which is the saturated density of core material is assumed for the impervious core material and core material depth of about 80 m is considered as loading on the foundation, taking the future expansion of dam into consideration. The unit weight of the hard basalt and soft layer is disregarded with consideration that it provides negligibly small effect on the result of analysis.

3.7.2 Result of Analyses

In order to clarify the effect due to the soft layer, the analyses of deformation and stress are made for the following two cases:

CASE (i): Case that the foundation consists only of the hard basalt; that is, the foundation is wholly provided with elastic modulus of $E = 55,000\ kg/cm^2$.

CASE (ii): Case that the foundation is intercalated with the soft layer as shown in Fig. 3.7.2.

Fig. 3.7.3 and Fig. 3.7.4 show stress distributions for CASE (i) and CASE (ii), respectively.

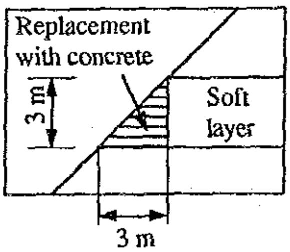
Deformations are calculated as shown below:

Point No.	Deformation (m)	
	CASE (i)	CASE (ii)
34	-2.188×10^{-4}	-2.09×10^{-4}
40	-4.873×10^{-4}	-5.50×10^{-3}
46	-7.466×10^{-4}	-1.03×10^{-2}
52	-9.746×10^{-4}	-1.06×10^{-2}
58	-1.166×10^{-3}	-7.05×10^{-3}
64	-1.321×10^{-3}	-4.21×10^{-3}

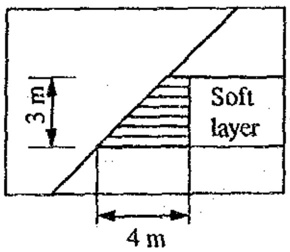
As seen in the above, a remarkable effect on deformations due to the intercalated soft layer is recognized.

As seen in Fig. 3.7.3 which shows the case that the foundation is assumed to consist wholly of the hard basalt, no tensions arise in the impervious core and foundation. However, in the case that the foundation is intercalated with the soft layer as shown in Fig. 3.7.4, tensions occur in the core and upper hard basalt layer due to deformations of the soft layer, suggesting that there is possibility of damage on the hard basalt layer and serious occurrence of cracks in the impervious core and that some countermeasure is required.

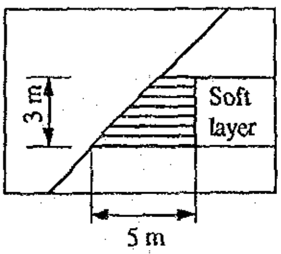
Therefore, examinations are made to find a suitable and effective countermeasure; that is, the following three (3) cases of replacement with concrete are examined with stress analyses by the Finite Element Method.



CASE (A)



CASE (B)



CASE (C)

Results of stress analyses for the above three (3) cases are given in Fig. 3.7.5, Fig. 3.7.6 and Fig. 3.7.7, respectively.

As seen in the above figures, tensions still occur in the upper hard layer in CASE (A) and CASE (B). The tension disappears in CASE (C).

The above analyses reveal that the replacement with concrete up to a depth of about 1.7 times thickness of the soft layer will be the efficient countermeasure to ensure the safety. Based on the above, the design of dam foundation treatment considers to replace with concrete up to the depth of 2.0 times soft layer thickness for the soft layers of core zone. The replacement will be made with lean concrete.

3.8 Leakage Analysis through Dam Body

3.8.1 General

The dam site topography forms a steep slope of about 50 to 60 degree in right abutment and relatively gentle slope of about 35 degree in left abutment. Although the hard basalt is exposed on the river bed, top soils or talus deposits cover the abutments.

The impervious core will be embanked on the foundation where the top soils, talus deposits or highly weathered portions underlain will be removed and the intercalated soft layers will be replaced with concrete. To avoid an excessive leakage and piping through the foundation of the impervious core, the curtain grouting is planned to be carried out in the foundation under the core by two lanes with rout holes at two meters interval.

The excessive leakage and piping under the foundation are considered to be avoided by the foundation treatment as mentioned. However, the leakage through the foundation is one of the most important factors for securing the safety of dam, and therefore, analyses on the leakage through the dam body and its foundation are made to confirm the matter.

3.8.2 Leakage Analysis

The leakage is analyzed by the Finite Element Method. Permeability coefficients of elements required in the seepage analysis by the Finite Element Method are assumed as follows:

Assumed Permeability Coefficient

Items	Permeability Coefficient (cm/sec)
- Impervious core:	
Horizontal	1×10^{-5}
Vertical	1×10^{-6}
- Grout curtain	5×10^{-5}
- Foundation below core	1×10^{-4}
- Foundation deeper than the grout curtain	1×10^{-5}

The above assumptions are based on geological investigations and material investigations carried out. As for improvement of the permeability coefficient by curtain grouting, a relatively conservative assumption is given for a safety sake: that is, the permeability coefficient is assumed to be improved from 1×10^{-4} cm/sec to 5×10^{-5} cm/sec by the curtain grouting.

The analyses are two-dimensionally carried out for five (5) dam sections as shown in Fig. 3.8.1, based on which total leakage through the dam and its foundation is obtained.

Models for analyses by the Finite Element Method for the above five (5) dam sections are as seen in Fig. 3.8.2 to Fig. 3.8.6. Seepage flows obtained by the analyses are as seen in Fig. 3.8.7 to Fig. 3.8.11.

(1) Examination for piping

The piping is a phenomenon that material particles discharge out due to a large seepage pressure, forming a large flow passage and resulting in a damage on the dam.

Possibility of the piping is examined by Justin's method. Justin obtains the upper limit of seepage flow velocity against particle sizes of soil material. Particles of soil material begin to move when the seepage flow velocity exceeds the above upper limit which is called as "the critical velocity of flow", causing the piping as mentioned.

The critical velocity of flow is obtained as follows:

$$Vc = \left(\frac{2}{3}(Gs - 1) \cdot d \cdot g\right)^{0.5}$$

where,

Vc : Critical velocity of flow (cm/sec)

Gs : Specific gravity of soil particle (2.6)

d : Diameter of soil particle
(d_{10} = 0.0001 mm = 0.00001 cm)
g : Acceleration of gravity (980 cm/sec^2)

Hence, the critical velocity of flow is calculated at 1.02×10^{-1} cm/sec as follows:

$$
\begin{aligned}
Vc &= \left(\frac{2}{3}(Gs - 1) \cdot d \cdot g\right)^{0.5} \\
&= \left(\frac{2}{3}(2.6 - 1) \times 0.00001 \times 980\right)^{0.5} \\
&= 1.02 \times 10^{-1} \text{ (cm/sec)}
\end{aligned}
$$

On the other hand, the maximum flow velocity (Va cm/sec) is calculated for each of five (5) dam sections by F.E.M seepage analysis as follows:

Maximum Seepage Velocity

Section No. (Station No.)	Element No.	Maximum Seepage Velocity, Va (cm/sec)
I (4)	52	0.13×10^{-3}
II (8)	48	0.15×10^{-3}
III (12)	41	0.24×10^{-3}
IV (16)	49	0.23×10^{-3}
V (20)	56	0.22×10^{-3}

A criteria specify that the safety factor of 100 times should be ensured: that is, F = Vc/Va should not be less than 100. F is calculated as follows:

Section No. (Station No.)	Element No.	Vc (cm/sec)	Va (cm/sec)	F = Vc/Va
I (4)	52	1.02×10^{-1}	0.13×10^{-3}	785
II (8)	48	1.02×10^{-1}	0.15×10^{-3}	680
III (12)	41	1.02×10^{-1}	0.24×10^{-3}	425
IV (16)	49	1.02×10^{-1}	0.23×10^{-3}	443
V (20)	56	1.02×10^{-1}	0.22×10^{-3}	464

As seen, the values of F above exceed sufficiently beyond 100, suggesting that a sufficient safety against piping is secured.

(2) Examination on seepage quantity

The total seepage quantity is assessed based on the seepage calculated for the five (5) dam sections shown in Fig. 3.8.1.

The total seepage quantity through the dam body and its foundation is calculated at 86.2 x 10^{-4} m^3/sec as follows:

Total Seepage through Dam and Foundation

Section No. (Station No.)	Width (m)	Unit Seepage (l/sec/m) Core	Unit Seepage (l/sec/m) Foundation	Total Seepage (m3/sec) Core	Total Seepage (m3/sec) Foundation
I (4)	30	1.4 x 10^{-2}	0.8 x 10^{-2}	4.2 x 10^{-4}	2.4 x 10^{-4}
II (8)	50	2.2x 10^{-2}	1.1 x 10^{-2}	11.0 x 10^{-4}	5.5 x 10^{-4}
III (12)	30	4.2 x 10^{-2}	0.8 x 10^{-2}	12.6 x 10^{-4}	2.4 x 10^{-4}
IV (16)	50	3.9 x 10^{-2}	1.3 x 10^{-2}	19.5 x 10^{-4}	6.5 x 10^{-4}
V (20)	45	3.4 x 10^{-2}	1.5 x 10^{-2}	15.3 x 10^{-4}	6.8 x 10^{-4}
Total	200			62.6 x 10^{-4}	23.6 x 10^{-4}

The total seepage quantity of 86.2 x 10^{-4} m^3/sec through the dam and its foundation corresponds to about 0.5% of the mean annual river inflow of about 1.8 m3/sec, which is sufficiently within the acceptable seepage quantity in consideration that the standard mentions that the total leakage volume should be less than 1% of mean annual river inflow.

The standard also mentions that the daily seepage volume through the dam and foundation should be less than 0.05% of the gross storage capacity of reservoir. This standard is also met as follows:

Daily seepage	745 m^3/day
Gross storage of reservoir	6.7 x 10^6 m^3
Percentage of daily seepage	0.01%

3.9 Leakage Analysis through Left Bank

3.9.1 General

Fig. 3.9.1 shows a general plan of the dam site and surrounding areas. As seen, the left bank of the dam site forms a relatively thin plateau located between the Terre Rouge river where the dam is planned to be constructed and the Plaines Wilhem river flowing in the south of the Terre Rouge river.

As a result of the geological investigations for this thin plateau, the geology of the plateau is found to have permeability coefficients of 10^{-4} cm/sec as a whole. The geological investigations also revealed that large openings such as the lava tunnel may not exist there. However, the geological investigations discovered a portion where a low piezometric head is shown, implying that a localized zone with relatively higher permeability exists with a limited horizontal width, indicating necessity of rim grouting along the left bank plateau.

Thus, the design considers to execute the rim grouting along the left bank plateau. The rim grouting will be carried out by the split-spacing method which will make it possible to find parts with the high permeability and efficiently concentrate the grouting there.

Although the rim grouting as mentioned will sufficiently improve the parts with relatively high permeability, examinations on the leakage through the left bank after impounding of reservoir are considered necessary because of its thin plateau.

The examinations are made in this section hereunder.

3.9.2 Analysis of Seepage

The geological section of damsite and left bank is shown in Fig. 3.9.2. The low piezometric head is discovered near the boring hole No. JD-12 shown in Fig. 3.9.2. The zone with relatively high permeability is considered to exist at EL. 130 m to EL. 140 m near the boring hole No. JD-12 in the direction from the Terre Rough river to the Plaines Wilhem river. However, the analysis is made, assuming these zones with the higher permeability will be all improved by the rim grouting.

The two-dimensional F.E.M. seepage analyses are made for three (3) typical sections as shown in Fig. 3.9.1, i.e. Section A-A, B-B and C-C. The geological sections of Section A-A, B-B and C-C are as seen in Fig. 3.9.3.

Models for the F.E.M. seepage analysis for the above three (3) sections are given in Fig. 3.9.4 to Fig. 3.9.6. In the models, the permeability coefficient of each element is assumed as follows:

Assumed Permeability Coefficients

Items	Assumed Permeability Coefficient (cm/sec)
- Original left bank:	1×10^{-4}
- Rim grout curtain:	7×10^{-5}
- Old lava below rim grout curtain:	1×10^{-5}

In the above, the permeability coefficient of 7×10^{-5} cm/sec is conservatively given to the rim grout curtain, although the degree of improvement by the rim grout is considered to be more effective.

The F.E.M. seepage analyses for three (3) typical sections have resulted in the followings:

Section	Seepage Quantity (m^3/day/m)
A - A	0.24
B - B	0.35
C - C	0.64

As seen above, the seepage increase in accordance with that of hydraulic gradient, and therefore, assuming that little seepage arise upstream more than Section A-A, the total seepage quantity is approximately assessed at 185 m^3/day as follows:

$$V = 450 \text{ m} \times (0.24 + 0.35 + 0.64) / 3 = 185 \text{ m}^3\text{/day}$$

This daily seepage quantity corresponds to 0.003% of the gross storage capacity of reservoir.

The sum of daily seepage quantities from the dam, its foundation and left bank will be 930 m^3/day which corresponds to 0.014% of the gross storage capacity of reservoir, which is sufficiently less than the standard allowable seepage quantity, 0.05 % of the reservoir gross storage per day.

The maximum flow velocity is calculated to be 2.04 cm/day at Element No. 108 in Section C - C. This maximum flow velocity corresponds to 0.24×10^{-4} cm/sec which is much less than those in the dam, suggesting that the safety against piping more sufficient than the dam will be secured.