THE GOVERNMENT OF MAURITIUS

THE FEASIBILITY STUDY
ON
PORT LOUIS WATER SUPPLY PROJECT
IN MAURITIUS

MAIN REPORT

JUNE 1989

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

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

JICA sent to Mauritius a survey team headed by Norizo Fujita of Nippon Koei Co.,Ltd, composed of members from Nihon Suido Consultants Co.,Ltd. from April to July, and from October to December ,1989.

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

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

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

June, 1989

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation Agency

June, 1989

Yanagiya Kensuke
President
Japan International
Cooperation Agency
Tokyo

Dear Sir,

LETTER OF TRANSMITTAL

We have the pleasure of submitting to you a Final Report of The Feasibility Study on Port Louis Water Supply Project prepared for the consideration by the Government of Mauritius in implementing water resources development for water supply to Port Louis City.

This report consists of three volumes. The Main Report contains the results of the pre-feasibility level study on alternative schemes for water supply to Port Louis and feasibility study on optimum development plan. The plan indicates that it is the time to expedite water supply to Port Louis to attain socio-economic development in the country. Supporting Report (I) contains hydrological and geological studies to support the plan presented in the Main Report. Supporting Report (II) contains studies on construction materials and comparative study on alternative schemes.

All members of the Study Team wish to express grateful acknowledgment to the personnel of the Advisory Committee, Ministry of Foreign Affairs, Embassy to 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 future water resource development for water supply to Port Louis and to socio-economic development and well-being in general and .

Yours, sincerely

Norizo Fujita
Team Leader
The Feasibility Study on
Port Louis Water Supply Project

SUMMARY

INTRODUCTION

1. Mauritius is a volcanic island of 1,860 km^2 and located about 900 km to the east of the Madagascar in the Indean Ocean. The population of Mauritius is about one million and 42 percent of the population is concentrated in Port Louis City, the capital city of Mauritius, and the neighboring satellite cities. Port Louis City plays an important role as a center of commerce and industry in the country.

2. Municipal and industrial water for Port Louis City originates from the Grand River North West (GRNW) basin. The main water supply facilities are an intake weir called the Municipal Dike, water treatment facilities installed at Pailles, water transmission facilities from the intake to the water treatment facilities at Pailles and water distribution facilities from the water treatment plant, etc.

3. The above present water supply system has the following problems: (i) part of the delivery system is now very old, and the water loss due to leakage from the system is 40 to 45 % of the water volume treated. (ii) the GRNW basin does not have sufficient storage to regulate the seasonal fluctuation of run-off, and as a result, Port Louis City is subject to severe water shortage in the drought season from July to November every year.

4. Accordingly, the Government of Mauritius requested technical assistance from the Government of Japan for a study on a project to cope with the seasonal fluctuation of available water and to provide a stable water supply to Port Louis City.

In response to the above request, the Government of Japan agreed to carry out 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 programme of the Government of Japan, was appointed to undertake the Feasibility Study in cooperation with the Government of Mauritius.

5. JICA dispatched the JICA Study Team for the Feasibility Study from the beginning of April, 1988. Since then, the JICA Study Team has made investigations and studies on the Project in close cooperation with the Central Water Authority (CWA)/Ministry of Energy, Water Resources and Postal Service, the counterpart agencies of the Government of Mauritius.

6. The Feasibility Study was divided into Phase I and Phase II. Phase I resulted in the selection of the optimum project scheme through a comparative study on the conceivable alternative schemes. Phase II has made the plan formulation of the water supply project for Port Louis City and has examined project feasibility through more detailed investigation on the selected project scheme.

7. This Final Report summarizes all the results of the investigations and feasibility study made in Phases I and II as well as the conclusions and recommendations reached through the study.

SITE CONDITIONS

Socio - Economy

8. Mauritius comprises the islands of Mauritius, Rodrigues, Agalega and St. Brandon. The total area is 2,040 km^2 of which Mauritius island accounts for about 91 %, or 1860 km^2. The island of Mauritius comprises 5 municipal areas and 98 village council areas. Port Louis, the capital city of Mauritius, is one of the five municipalities. The total population of the country which is

1,000,432, according to the last population census in Mauritius island grew at the annual average rate of 1.43 % during the inter-census period of 1973 to 1983. Based on the past trend, the total population of the country in 1988 is estimated at around 1.1 million.

Gross domestic product (GDP) of Mauritius in 1987 was estimated at Rs. 18,020 million at 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.

9. Port Louis is the capital of Mauritius with 42.7 km^2 of area. It is located in the northwestern part of the island of Mauritius and to the northeast of the river mouth of the Grand River North West. Administratively Port Louis is designated as a municipality among 5 municipalities that are urbanised areas in the island of Mauritius and also constitutes one district by itself. It comprises 6 wards and 19 localities.

According to the 1983 Population Census, the population of Port Louis was enumerated at 133,702, which accounted for 33.2 % of the urban population or 13.8 % of the population of the island of Mauritius. Out of the total, 66,132 persons were male, accounting for 49.5 %. Population density was estimated at 3,131 persons per km^2 which is about 6 times higher than that of the island as a whole. The number of households was 29,187 with an average household size 4.58 persons. According to the population estimates made by the Central Statistical Office based on 1983 Population Census results, the population of Port Louis grew at the average annual rate of 0.97 % during 1983-85 period, which is slightly higher than 0.91 % for the island as a whole and is identical with that for the country.

According to the 1983 Population Census, 61,387 people were employed in Port Louis district, of which 46,240 were males and 15,147 were females. Out of the total, 28,930 people or 47.1 % were the residents of Port Louis district and the rest were commuters from outside. Of the Port Louis residents, 5,805 were working outside the Port Louis district. The total number of employed residents in Port Louis district were, therefore, estimated at 34,735.

Existing Water Supply Facilities

10. 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 Palain 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 main problems of the present system are:

(i) The system has no storage capacity to regulate the seasonal fluctuation of run-off,

(ii) The transmission and distribution system is now very old with leakage losses amounting to 45 %.

(iii) The capacity of the existing filter beds is not sufficient, making it impossible to treat the muddy water.

Water Demand Forecast

11. The water demand projection study carried out indicates the followings:

(i) The most likely total population to be served by the Port Louis Water supply system in the year 2010 and 2030 is projected to be 162,494 and 176,838 respectively.

(ii) The water requirement (the water production requirement taking into account the leakage loss) will increase as follows:

Year	Water Demand at Production Level
1988	62,040 m^3/day (0.718 m^3/s)
1990	60,250 " (0.697 ")
2000	71,210 " (0.824 ")
2010	78,569 " (0.909 ")
2030	82,490 " (0.954 ")

Note: It is noted that the above requirement is worked out on the assumption that the leakage loss will be decreased from 46 % in 1988 down to 35 % in 1990, 30 % in 2000, 30 % in 2010 and 25 % in 2030 respectively under the Leakage Control Program which is now in hand.

12. The above demand does not include the necessary river maintenance flow of 0.05 m^3/s and the loss at the treatment plant of 5 %.

Taking the above into account, the total water supply requirement is as follows:

Year	Total Water Supply Requirement
1988	0.80 m^3/s
1990	0.78 "
2000	0.92 "
2010	1.00 "
2030	1.05 "

Water Balance

13. The water demand - supply balance in future has been examined based on the results of the water demand projection above. The water deficits in respective years worked out through the simulation by using the available hydrological data are as follows:

Year	Water Supply Deficit (m^3/s)	Necessary Effective Storage Capacity of Reservoir (MCM)
1988	0.063(5,443 m^3/d)	1.99
1990	0.060(5,184 ")	1.89
2000	0.124(10,714 ")	3.91
2010	0.170(14,688 ")	5.36
2030	0.199(17,194 ")	6.28

Note: The above water supply deficit and requirement of reservoir storage capacity occur in the worst recorded drought year (1983).

Meteo-Hydrology

14. From the climatic point of view, the year may be divided into two seasons. One is the summer season from November to March, and another is the winter season from April to October. 65 to 75 percent of annual total rainfall falls in the summer season. The driest month is October when this basin has only 2.5 to 3.6 percent of annual total rainfall. The heaviest rainfall occurs usually in January to February and is caused by cyclones, or by fronts of the Inter Tropical Convergence Zone. The discharge of the GRNW basin

varies according to the above-mentioned climatic cycle. The mean annual total discharge is 84 MCM. Most of this discharge runs through into the sea without use because the flow is highly concentrated during storms.

15. Annual rainfall at five water level gauging stations, W03 (Plaines Wilhems river), W04 (Terre Rouge river), W05 (Cascade river), W08 (Profonde river) and W10 (Moka river) range from 2,300 mm to 2,550 mm in average.

The probable rainfall by return periods in GRNW basin is analyzed as follows:

Probable Rainfall by return periods

Return Period	One-day	Two-day	Three-day
10000	1168	1799	1999
1000	935	1381	1551
200	771	1114	1260
100	701	1003	1140
50	630	901	1021
20	536	765	864
10	463	661	864
5	387	551	632
2	272	398	470

16. The low flow analysis based on the river discharge data reveals the reliability of available discharges in the five tributaries as follows:

Unit: m^3/s

Station		Reliability			
		Average	80 %	90 %	95%
W03	Plaines Wilhems	0.48	0.170	0.137	0.120
W04	Terre Rouge	0.48	0.102	0.087	0.077
W05	Cascade	0.73	0.247	0.182	0.157
W08	Profonde	0.32	0.136	0.109	0.091
W010	Moka	0.69	0.230	0.171	0.145

TRO damsite selected through the comparative study is located on the Terre Rouge river immediately upstream of the confluence with the Plaines Wilhems river, and would collect the discharges from three (3) tributaries of the Terre Rouge, Cascade and Profonde rivers with 54.9 km^2 in total catchment area.

The reliability of available discharges at this damsite and from the residual basin is analyzed as follows:

Unit: m^3/s

Station	Reliability			
	Average	80 %	90 %	95%
TRO damsite	1.56	0.66	0.58	0.51
Residual basin	1.23	0.31	0.20	0.15

17. The high flow analysis works out the probable flood peaks by return periods at TRO damsite as follows:

Probable Flood Peak

Return Period (Year)	Flood Peak Discharge (m^3/s)
10	455
20	536
100	718
200	796
10,000	1,596

The analysis of the probable maximum precipitation indicates that the probable maximum precipitation is nearly equal to the 10,000-year probable precipitation, and therefore, 10,000-year flood is taken as PMF, the extraordinary flood for spillway design.

18. The average annual sediment yield at TRO damsite is assessed to be 3,949 m^3 or 0.07 mm/year in the denudation depth of the basin.

Geology

19. Mauritius island is a shield volcano formed by volcanic activity at the end of the Miocene or early Pliocene in the Tertiary period. Therefore, most of the project area is covered by volcanic deposits such as lava, basalt, etc. 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 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 and to the northwest at a low angle of around 5 degree. The young volcanic lavas are characterized with frequently developed vesicular appearance. Volcanic breccias intercalate the lava layers with a thickness of about 3 m to 10 m. The hard old volcanic series underlies the young volcanic series, dipping about 10 degree to 15 degree from the south to the north or the southeast to the northwest.

20. The selected damsite has an unsymmetrical triangular shape with the steeper slope on the right bank.Several layers of the young volcanic lavas are seen in the river bank slopes. Along the riverbed, there are continuous outcrops of the old volcanic series overlain by the young volcanic rocks in the lower slopes of the river bank.

21. The boring investigations carried out at the damsite show that,

(i) The weathering is developed irregularly without any close relationship with depth from the ground surface.

(ii) However, the permeability of the foundation rocks is quite low range, indicating permeability coefficients of 10^{-4} cm/s to 10^{-6} cm/s. Foundation treatment can be performed by normal cement grouting.

(iii) The rather thick talus deposits and scree deposits developed on the left abutment and to some extent the young volcanic rocks underlying these deposits are highly weathered. These talus deposits, scree deposits and highly weathered young volcanic rocks should be removed for the foundation of the impervious core of the rockfill dam, and therefore, the excavation depth of cut-off trench will be 10 m to 15 m on the left abutment. The excavation depth of cut-off trench in the right abutment will be about 5 m on average.

(iv) Drilled core recovery is very high, indicating nearly 100 % core recovery. There are no abnormal features in groundwater tables observed in drilled boreholes. Consequently no significant development of cavities and spaces are expected in the area.

22. The seismic exploration carried out at the damsite confirms that there are no geological discontinuities or faults in the damsite.

23. The water tightness of the damsite and reservoir area with or without the continuous openings has been investigated and examined in detail. All these investigations suggest that sufficient water tightness will be secured.

Construction Materials

24. A rocky mountain about 1 km from the damsite is selected as the rock quarry site. Investigation of this quarry site reveals that the materials are satisfactory qualitatively and quantitatively. The concrete aggregates will have to be produced from the above quarry rock. The quality and quantity are also confirmed as satisfactory for concrete aggregates.

25. The clayey soil materials distributed in 0.8 to 1.2 km upstream of the damsite are selected for the borrow area for impervious core materials through investigations on several conceivable borrow areas. The soil properties satisfy the criteria for core materials of a rockfill dam. The available quantity is also sufficient.

PLAN FORMULATION

Selection of Optimum Scheme

26. The optimum project scheme is found out through a comparative study of the conceivable alternative schemes.

Although various damsites had been identified in the basin by previous studies or reconnaissance by the JICA Study Team, the following six(6) schemes were finally selected as the conceivable alternative schemes.

(i) G1 (Bocage-Guibies), (ii) MO4 (Baptiste), (iii) TRO, (iv) NWO, (v) TR9, (vi) CA2

27. The comparative study on the above alternative schemes was made through comprehensive evaluation from the economic, technical and social aspects. TRO scheme was selected as the optimum scheme.

28. The following summarizes the evaluation based on the comparative study:

G1 (Bocage-Guibies) scheme will not be the optimum scheme because of its high project cost mainly due to unfavourable geological conditions in the dam foundation, although this scheme has been taken up in the past studies as one of the most promising schemes. NWO scheme or the staged construction of several small schemes will not be as advantageous economically as TRO or MO4 (Baptiste) schemes. Besides, these schemes do not have any particular advantage from technical or other aspects. Thus, these schemes are also rejected.

MO4 (Baptiste) and TRO schemes are comparable with each other from the economic aspect, indicating the least project cost. It is also difficult to destinguish any superiority or inferiority from the technical aspect between both schemes. However, MO4 (Baptiste) scheme whose reservoir would submerge a large area of sugarcane lands would require a solution to the severe social constraints. MO4 (Baptiste) scheme is also expected to be subject to eutrophication of its reservoir, and therefore requiring a solution to deterioration of its water quality. TRO scheme, however, woul have no such social constraints.

The degree of eutrophication in the reservoir will also be much less than MO4 (Baptiste) scheme. As such, it is concluded and recommended that the TRO scheme should be selected as the optimum water storage scheme for further detailed investigations and studies.

Design

29. The objective of the project is to improve the water supply of Port Louis at the least cost. Therefore, the project includes only components essential for the single purpose of water supply. The necessary project components are as follows:

(i) Dam and appurtenant facilities such as the spillway, diversion tunnel, intake and river outlet
(ii) Water transmission facilities
(iii) Water treatment facilities

30. The height of dam is based on the necessary gross storage of 6.7 MCM consisting of the required effective storage capacity of 6.3 MCM worked out by a water balance study, necessary dead storage for sedimentation assessed at 0.3 MCM for 100 years, and the estimated evaporation loss of 0.1 MCM.

H.W.L and L.W.L are determined at EL.189.0 m and EL.139.0 m respectively so as to satisfy the above required storages. The dam would be provided with a freeboard of 6.0 m above H.W.L.
With this freeboard, the probable maximum flood of 1,596 m^3/s at flood peaks, which is taken as the extraordinary flood, can be handled with an allowance of 1.2 m.

The dam type is determined to be one of rockfill of the center core type as most desirable economically and technically through examination of other types such as concrete gravity and homogeneous earthfill, etc. The dam would have an upstream slope of 1:2.5 and downstream slope of 1:2.0 for which a stability analysis confirms its safety.

Talus deposits and scree deposits cover the damsite. Some young volcanic rocks underlying these deposits are highly weathered and should be removed for the foundation of the impervious core. The cut-off trench for the impevious core will be excavated down to 10 to 15 m depth in the left abutment and about 5 m depth in the right abutment.

31. The spillway would be located on the left bank with a crest length of 80 m, based on the results of comparative studies on the cost of spillway and dam.

The design of the spillway is made in accordance with the standard in Japan as follows: that is, the spillway is designed for 1.2 times the 200-year probable flood peak (950 m^3/s) without considering the retardation effect of the reservoir. The necessary freeboard is then decided by taking into consideration an extraordinary flood, wave height and allowance in accordance with dam type.

The probable maximum flood corresponding to 10,000-year probable flood is taken as the extraordinary flood mentioned above.

The spillway would have to be of the side channel type to suit the topographic conditions and for reasons of economy. Furthermore, the spillway is not gated, in the interests of safety.

32. The river diversion system consisting of a diversion tunnel and cofferdams is designed to have the capacity to handle the diversion design flood of a 20-year probable flood, 540 m^3/s, in accordance with the design standard.

The crest elevation of upstream cofferdam and tunnel diameter would be EL.149 m and 6.4 m respectively, which will be the most economical, satisfying the following necessary conditions: (i) maximum velocity in the tunnel less than 15 m/s, (ii) volume of coffer dam to be constructable during one dry season.

The river diversion system also considers to safely handle 100-year probable flood in the second year by embanking the cofferdam up to

EL.161 m before the beginning of the second rainy season.

33. The intake for water supply would be located on the left bank connected to the diversion tunnel nearby. The diversion tunnel would be utilized to release the water for water supply. The intake has five (5) gates to select the water from the optimum depth for the water quality conditions in the reservoir.

The river outlet facility is prepared for the emergency release of water. The facility consists of a concrete inlet tower, steel pipe and hollow jet valve, having the capacity to empty the reservoir in a few days.

34. In planning the water transmission facilities it is proposed that the existing pipelines should be fully utilized for saving of cost. Nevertheless an additional pipeline capacity more than 659 l/s will become necessary for meeting the demand in 2030. The necessary diameter of pipe for the above is calculated at 800 mm. A total of 2,100 m of 800 mm diameter pipe will be required. The additional pipeline would be installed in parallel with the existing pipelines from the existing Municipal Dyke to the Pailles treatment Plant, which has been revealed to be the cheapest way.

35. The design of water treatment facilities will also make full use of the existing plant which has a capacity of 60,000 m^3/day. Further, installation would be stepwise in accordance with the growth of demand to lessen the initial investment cost. An additional treatment capacity of 30,000 m^3/day would be installed at the end of 1994, and another 10,000 m^3/s in 2005 when the capacity will become short.Thus, the first stage new treatment facilities are designed to have the capacity of 30,000 m^3/day.

The water treatment facilities will consist of the following processes; the water receiving, chemical mixing, flocculation, sedimentation, filtration and disinfection. Taking into consideration the characteristics of the raw water quality, the conventional rapid sand filtration system would be applied.

Construction Schedule

36. The construction plan is prepared based on the preliminary design of the project components. It will take 46 months from the contract award to Lot 1 (Diversion Tunnel). In addition to the above construction period, 14 months are necessary for the supplemental investigation, detailed design, tendering and contract award, etc.
Assuming that the supplementary investigations and detailed design can be commenced in January 1990, the project will be completed by the end of 1994.

37. It is assumed that the project will be executed under the following international competitive contracts and local contracts:

International competitive contracts

Lot 1. Construction of diversion tunnel (Excluding the diversion closure work to be included in Lot.2)

Lot 2. Construction of dam and appurtenant facilities, and supply and installation of metal works except diversion tunnel

Lot 3. Supply and installation of water transmission pipeline and water treatment facilities

Local contracts

Lot 1. Preparatory works at site undertaken by the Government

Lot 2. Construction of permanent access road and relocation works of the existing roads in the project area

For effective implementation of construction work, construction of the diversion tunnel is recommended to be started from May 1991 in advance under a separate contract.

Cost Estimate

38. The total construction cost is estimated at US$ 69.7 millions (Rs. 954.4 millions) composed of US$ 48.3 millions (Rs. 660.4 millions) in foreign currency and US$ 21.4 millions (Rs. 294 millions) in local currency as follows:

Unit: Rs. 10^6

Items	F.C.	L.C.	Total
A. Dam and Appurtenant Facilities:			
- Direct Cost	472.4	199.7	672.1
- Compensation	-	0.2	0.2
- E/S & Admin.	47.2	25.0	72.2
- Physi. Contingency	52.0	22.5	74.5
Sub-Total	571.6	247.4	819.0
(US$ 10^6)	($41.8)	($18.0)	($59.8)
B. Water Transmission & Treatment Facilities:			
- Direct Cost	73.5	37.6	111.2
- E/S & Admin.	7.3	4.7	12.0
- Physi. Contingency	8.0	4.2	12.2
Sub-Total	88.8	46.6	135.4
(US$ 10^6)	($6.5)	($3.4)	($9.9)
Total	660.4	294.0	954.4
(US$ 10^6)	($48.3)	($21.4)	($69.7)

39. The disbursement of the construction cost in accordance with the implementation schedule will be as follows:

Unit: Rs. 10^6

Year	F.C.	L.C.	Total
1989/90	17.3	8.9	26.2
1990/91	21.0	28.6	49.6
1991/92	93.0	60.2	153.2
1992/93	150.8	43.7	194.5
1993/94	314.3	123.0	437.3
1994/95	64.0	29.6	93.6
Total	660.4	294.0	954.4
(US$ 10^6)	($48.3)	($21.4)	($69.7)

40. The total investment cost of the project including the price contingency (price escalation) and interest during construction is estimated at US$ 88.2 millions (Rs. 1,208.7 millions) comprising US$ 59.5 millions (Rs. 815.4 millions) in foreign currency and US$ 28.7 millions (Rs. 393.3 millions) in local currency as follows:

Unit: Rs. 10^6

Items	F.C.	L.C.	Total
A. Dam and Appurtenant Facilities			
- Construction cost	571.6	247.4	819.0
- Price Contingency(Price Escalation)	85.6	82.2	167.8
- Interest during Construction	52.6	-	52.6
Sub-total	709.8	329.6	1,039.4
B. Water Transmission & Treatment Facilities			
- Construction cost	88.8	46.6	135.4
- Price Contingency(Price Escalation)	12.7	17.1	29.8
- Interest during Construction	4.1	-	4.1
Sub-total	105.6	63.7	169.3
Total Investment Cost	815.4	393.3	1,208.7
(US$ 10^6)	($59.5)	($28.7)	($88.2)

Note: The interest rate is assumed to be 2.9 % per annum.

Economic Evaluation

41. The economic evaluation is made by assessing the economic internal rate of return (E.I.R.R.) on the basis of the economic cost and benefit.

The economic cost is calculated at Rs.901,480 x 10^3 in total, based on the conversion factor (the ratio of economic cost to financial cost) which has been assessed at 0.82 and is applied for the local currency portion of the construction cost.

The economic benefit is composed of the benefit by the domestic water supply,and non-domestic and government use.

Each benefit is estimated as follows:

(i) Benefit by Domestic Water Supply

(Unit : Rs. 10^3/year)

Year	1990	2000	2010	2030
Benefit	777	2,443	11,152	15,972

(ii) Benefit by Non-Domestic and Government Use

(Unit : Rs. 10^3/year)

Year	1990	2000	2010	2030
Benefit	9,551	30,935	155,602	241,245

42. E.I.R.R. is calculated at 8.7 % which is considered enough high for a water supply project. Thus, the project is evaluated to be reasonably viable economically.

The economic benefit assessed above does not include various intangible and indirect benefits which are not possible to be quantified. Taking into account the above , the project is evaluated sufficiently justifiable economically.

Financial Evaluation

43. The financial evaluation is made in terms of F.I.R.R. and loan repayability. The revenue of the Project is calculated on the basis of projected incremental volume of water supply by category and water tariff by category i.e., domestic, non-domestic and government uses. The water tariff is assumed to be revised every three years based on the assumed rate of increase of the consumer price index (CPI), 7.2 % per annum.

44. FIRR is calculated at 6.8 % which can be considered as a reasonably high value for a water supply project.

45. Loan repayability is assessed under the following conditions:

Local portion of the construction cost:

To be funded by the Government or financed by CWA through its own funds (internal reserves, depreciation or others)

Foreign portion of the construction cost:

To be financed by a loan with the following conditions:

o Repayment period	30 years
o Grace period	6 years
o Interest rate	2.90 %

Annual net revenue will go into black from the 13th year after the commencement of the Project. Accumulated surplus go into black in the 24th year, i.e., the last year, of the repayment period, indicating that the assumed loan is repayable. However, since the deficit period is rather long, it is desirable that the Government should extend financial assistance to CWA, such as by an interest-free loan for example, until the annual net revenue turns into surplus.

ENVIRONMENTAL ASSESSMENT

46. The environmental investigation revealed the following:

In Mauritius, the urban population has increased and manufacturing industries have developed in recent years. However, the construction of sewerage facilities is retarded in Maritius as compared with advanced countries. Inadequate treatment of industrial wastes as well as of city sewage is causing pollution of public water areas such as the sea, rivers, etc. and is giving rise to serious problems. The greatest problems of water pollution in Mauritius include the damage to the river and coastal fisheries done by effluent discharges from factories.

The anticipated environmental impacts of the Project are mainly those of construction of the dam and reservoir. The following items are conceivable as environmental impacts due to dam and reservoir construction: a) Physical resources; b) Ecological resources; c) Human use values and d) Quality of life values. However, no severe adverse environmental effects are expected to be caused by the proposed Project, since no residential houses or agricultural lands exist in the reservoir area, nor will there be any effects on the existing irrigation system, etc. It is recommended to minimize the deterioration of the natural environment due to the construction works.

47. The water quality tests carried out on the river water and existing reservoir and lake water in Mauritius suggest that a water quality problem may occur due to eutrophication of reservoir, especially in shallow water. In Phase II, a detailed study was carried out on water quality projections in the future for consideration of necessary countermeasures. According to the results, for example in the case of river water, the value of consumption of potassium permanganate changed remarkably through the year. In the case of a reservoir like Champagne reservoir the number of phytoplankton

increased compared to those in the dry season, so that the value of consumption of potassium Permanganate increased.

Considering various problems, especially the low alkalinity in the rainy season, prechlorination treatment, alkali treatment, and deodorization treatment by powdered activated carbon or granuar activated carbon will be required.

CONCLUSIONS AND RECOMMENDATION

48. It has been revealed through the Feasibility Study that the Project will be justifiable technically, economically and financially.

Since water supply shortages occur in Port Louis City almost every year have to be solved urgently, the Project should be promoted for implementation as soon as possible.

Prior to the implementation, the Project requires detailed design, preparation of tender documents, tendering and contracting, etc. It is strongly recommended to proceed with the necessary procedures for executing these works as soon as possible.

PRINCIPAL FEATURES OF PROJECT

(1) **RESERVOIR**

Catchment area	55 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.192 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	950 m^3/s
Return period	(1.2 x 200 years)
Extraordinary flood	1,596 m^3/s
Return period	(PMF)

(2) **DAM**

Type	Rockfill
Crest elevation	El.195 m
Height (from river bed)	75 m
Crest length	230 m
Embankment volume	1,485 x 10^3 m^3

(3) **SPILLWAY**

Type	Side channel
Crest elevation of weir	El.189 m
Width of weir	80 m
Discharge	950 m^3/s

(4) **RIVER DIVERSION**

Type	Tunnel diversion
Design flood	540 m^3/s
Return period	(20 years)
Discharge in tunnel	500 m^3/s
Number of tunnel	1
Diameter	6.4 m
Length	375 m
Gate type	Roller gate

(5) **INTAKE**

Type	Selectable intake gate
Discharge	1 m^3/s
Number of gates	5
Dimension of gate	800 mm x 800 mm
Gate type	Sluice gate

(6) **NEW TRANSMISSION PIPELINE**

Design discharge	660 l/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/d

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1. INTRODUCTION

1.1 Project Background

Mauritius is a volcanic island of 1,860 km^2 and located at about 900 km to the east of the Madagascar in the Indian Ocean. The population of Mauritius is about one million persons and 42 percent of the population is concentrated in Port Louis city and the neighboring satellite cities. Port Louis city plays an important role as not only as the capital city of the Mauritius but also the base of commerce and industry in the country.

Municipal and industrial water supply for Port Louis city, commenced in 1790, is the responsibility of the Central Water Authority (CWA) in the Ministry of Energy and Internal Communications, and, using the water resources of the Grand River North West Basin. The chief water supply facilities are an intake weir called the Municipal Dyke and water treatment facilities at Pailles. Since the commencement of supply, these facilities has been enlarged gradually to meet the water demand of Port Louis.

However, since part of the delivery system is now very old, the water losses due to leakage from the system are estimated to be 40 - 45 percent of the water volume treated. There are also problems such as a substantial increase in water demand due to the influx of population to Port Louis city. As far as the basin of the GRNW is concerned, present storage does not meet the seasonal deficit of water supply and irrigation water requirements for sugarcane. Thus, the above-mentioned water leakage and seasonal fluctuation of runoff cause severe water shortages in the dry season from July to November every year.

Previous studies were made in the 1970's and several damsite for water storage were selected. These studies varied from preliminary reconnaissance studies by geophysical methods to feasibility studies.

The following sites were studied:

Site	Study	Type of Study
Riviere Baptiste	GOM-FAO	Electrical resistivity
Hermitage	GOM-FAO	- ditto -
Cote d'Or	GOM-FAO	- ditto -
Soreze	CEB-CEBTP	Core-borings
Baptiste-Guibies	CWA	Core-borings, pits, pumping tests
Bocage-Guibies	CWA	- ditto -
Bagatelle	CWA	Desk study

Of these, three schemes in above list are considered relatively suitable damsites by CWA. They are,

Soreze scheme	(1970)
Baptiste-Guibies scheme	(1973)
Bocage-Guibies scheme	(1979)

However, no scheme has been realized yet. As for the water leakage problem, the Leakage Reduction Project with UK assistance aims to reduce losses to 30 percent in two years.

Accordingly, the Government of Mauritius requested technical assistance from the Government of Japan for a study on the Project to deal with the seasonal fluctuation of available water and to ensure a stable water supply to Port Louis.

In response to the request of the Government of Mauritius, the Government of Japan 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.

JICA discussed the Terms of Reference for the Feasibility Study with the Government of Mauritius in October, 1987, and the Scope of Work was agreed and signed in January 1988.

JICA then dispatched a Study Team for the Feasibility Study from the beginning of April, 1988. Since then, the JICA Study Team has made investigations and studies in close cooperation with the Government of Mauritius.

The study was made in two phases. Phase I study had to select the optimum project scheme through a comparative study on conceivable alternative schemes. Phase II study comprised more detailed investigations on the selected scheme, plan formulation and examination of the project's feasibility.

This Report (Draft Final Report) presents the results of all the investigations and studies carried out through Phase I and Phase II.

1.2 Objectives of Project

The objective of the Feasibility Study were to develop the most suitable and economical plan to improve the water supply system or Port Louis by harnessing the water resources in the basin of the Grand River North West in order to meet the water demand of Port Louis City at the medium term up to year 2010 and the long term up to year 2030, and to assess its technical, economic and financial feasibility.

1.3 Relevant Studies

Major studies relevant to the Project are as follows;

(i) Past studies which selected possible reservoir sites and from preliminary reconnaissance studies by geophysical methods to feasibility studies.

Three(3) schemes of Soreze (1970), Baptiste-Guibies (1973) and Bocage-Guibies (1979) are major ones out of the above past studies.

(ii) Master Plan Study for Water Resources for Port Louis which is now ongoing by CWA.

(iii)Leakage Control Project in the Port Louis City water supply system which is now ongoing by CWA.

This Feasibility Study duly referred to these studies and reports in the selection of alternative schemes, planning of additional investigations and studies, etc.

Although Master Plan Study for water resources development for Port Louis City is not completed yet, this Feasibility Study has tried to be consistent with it. Accordingly, all basic data, water demand and its future projections, criteria, conceivable alternative schemes for water resource and problems, etc. which were contemplated by the Master Plan Study have duly been discussed and incorporated into the Feasibility Study.

The Leakage Control Project, also still in progress, is dealing with the planning for leakage reduction in the water supply system, and has set the target and program for leakage reduction, which is closely related to the feasibility Study. The Feasibility Study duly takes the above target and program into consideration.

1.4 Organization of Study

The Feasibility Study was organised as shown in Table 1.1 to comprise three(3) parties: the JICA Study Team, counterpart personnel from MEIC/CWA; and the Japanese Advisory Committee.

The JICA Study Team consisting of the experts in various fields carried out the investigations and study for the Project. The counterpart personnel from MEIC/CWA investigated and studied the Project with the JICA Study Team as well as providing certain specific inputs.

The Japanese Advisory Committee, consisting of Japanese Government officials gave timely advice, joining the discussions on major reports prepared by the JICA Study Team.

1.5 Outline of Project Area

The Project area consists of the area covered by the water delivery system (24.7 km^2) and Grand River North West area of the water resources for the Project (115.3 km^2) as shown in Fig.1.1.

The general conditions of the project area may be summarised as follows:

The Mauritius is volcanic island of 1,860 km^2 and locates at about 900 km to the east of Madagascar in the Indian Ocean between 19° 59' and 20° 39' of south latitude and 57° 18' and 57° 47'of east longitude.

The population of Mauritius is about one million persons of which 42 percent are concentrated in Port Louis city and in the neighboring satellite cities in the northwestern area of the Mauritius.

The project area has a marine climate characterized by a wet season from November to April and a dry season from May to October. The air temperature in Port Louis city ranges from 22° in August in the dry season to 28° in February in wet season. Rainfall is caused by the trade wind from southeastern direction. Annual rainfall ranges from about 1,000 mm at the coast to about 3,000 mm in the mountains and the basin mean annual rainfall of GRNW is estimated at about 2,500 mm. 70 percent of the annual rainfall is concentrated in the wet season. Cyclones also characterizes, and affects the climate of the project area and the whole of Mauritius.

Port Louis city is surrounded by steep mountains of elevation of 300-800 m in the eastern, western and southern parts. The GRNW originates in the Wilhems tableland of El.300-500 m and flows down to the coast through a V shaped gorge developed in its middle reaches. Finally, the GRNW enters the Indian Ocean through a more gently sloping river course of riverbed slope of 1:50 and length of 7 km.

The Mauritius island is a shield volcano formed by volcanic activity at the end of the Miocene or early Pliocene in the Tertiary period. Therefore, most of the project area is covered by the volcanic deposit such as lava, basalt, etc.

TABLE AND FIGURE

Table 1.1 ORGANIZATION OF THE STUDY

MEWRPS/CWA

Mr.R.Kisnah (Chairman)
Mr.D.Soobrah (General Manager)
Mr.E.Seenyen (Deputy General Manager)
Mr.R.Mungra (Chief Engineer)
Mr.N.Luchmaya (Economist)
Mr.V.Proag (Executive Engineer)
Mr.H.Joseph (Chief Finance Officer)
Mr.G.Rogbeer (Head of Commercial Service)
Mr.Ramrekha (Head of Hydrological Section)
Mr.Kauppaymuthoo (Chief Surveyor/ Water Right Administrator)
Mr.Mowlabacus (Hydrological Officer)
Mr.H.Durup (Hydrological Officer)
Mr.N.Dookhony (Economist)
Mr.K.Mandhub (Assistant Surveyor)
Mr.R.Doobory (Surveyor)
Mr.Y.Rughoobur (Geologist)
Mr.N.Navkal (Technical Advisor)

JICA STUDY TEAM

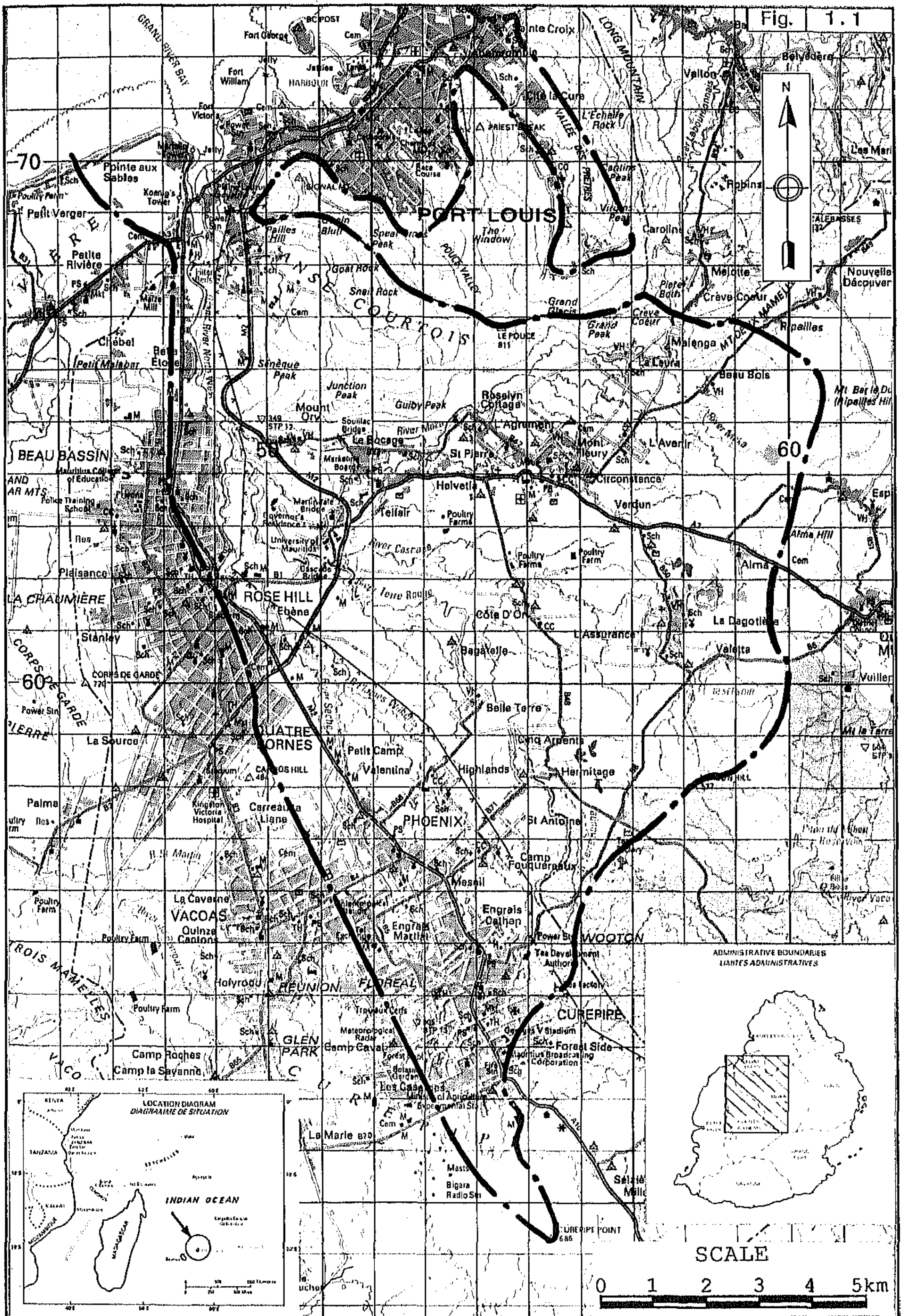
Mr.N.Fujita (Team Leader/Water Resources Planner)
Mr.S.Sasaki (Co-team leader/Water Supply Planner)
Mr.M.Yako (Geologist)
Mr.M.Kuwabara (Hydrologist)
Mr.T.Hirota (Geotechnical/Boring Expert)
Mr.S.Shimoda (Topo. Serveyor)
Mr.G.Kimura (Environmentalist)
Mr.H.Yamazaki (Water Transmission Planner)
Mr.Y.Oyama (Structural Engineer)
Mr.M.Akagawa (Economist)

ADVISORY COMMITTEE

Mr.T.Usami (Chairman)
Kanagawa Water Supply Authority
Mr.I.Yokota (Member)
Water Resources Development Public Cooperation
Mr.K.Umeda (Member)
Ministry of Health and Welfare

JICA COORDINATOR

Mr.H.Takama
Social Development Cooperation Department
Mr.T.Iwata
Social Development Cooperation Department



GENERAL LOCATION MAP OF PROJECT AREA	GOVERNMENT OF MAURITIUS PORT LOUIS WATER SUPPLY PROJECT JAPAN INTERNATIONAL COOPERATION AGENCY

2. SITE CONDITIONS

2.1 Socio-Economy and Project Economy

2.1.1 Socio-Economy in Mauritius

(a) Administrative Division

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 1864.8 km^2 and Rodrigues 5.1% or 104.0 km^2. The island of Mauritius comprises 5 municipal areas and 98 village council areas. Port Louis, the capital of Mauritius, is one of the five municipalities, and is located at northwestern part of the country.

(b) Population

According to the population census carried out in 1983, the total population of the country was 1,000,432, of which male accounted for 49.8%. 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 on Mauritius island grew at an annual average rate of 1.43%. The growth rate of Rodrigues during the same period was higher with 2.67% per annum. On Mauritius island, 403,251 people or 41.7% of the total lived in the urban areas of five municipalities and the rest in rural areas or villages. Based on the past trends, the total population of the country in 1988 was estimated at around 1.1 million.

(c) GDP

The gross domestic product (GDP) of Mauritius in 1987 was estimated at Rs. 18,020 million at current factor cost. With net factor income from the rest of the world of minus Rupees 520 million, gross national product (GNP) was Rs. 17,500 million. Per capita GDP was estimated at about Rs. 18,600. Value-added of the manufacturing sector was the biggest with Rs. 4,530 million or 25.1% of GDP. Agriculture, hunting, forestry and fishing came next with Rs. 2,495 million or 13.8% of GDP. Wholesale and retail trade, restaurants, hotels sector and financing, insurance, real estate and business services sector followed with 13.4% and 13.5% of GDP, respectively. GDP of Mauritius at current factor cost grew sharply during the 1981-87 period at an average annual rate of 12.8%. The manufacturing sector grew the most rapidly among all the sectors at an average annual rate of 22.0%, increasing its share to 25.1% from 15.7% during the period. Of the value-added of manufacturing, the Export Processing Zone (EPZ) accounted for about 54%, being the biggest contributor to GDP surpassing the sugar industry.

(d) Labour force and Employment

As of March 1987, 245,381 people were employed in the private and government sectors, of which males were 161,807 or 65.9% of the total. Of the total, 54,857 or 22.4% were employed in the central and local government services. In the private sector, manufacturing industry contributed the most to employment providing 93,311 jobs or 49.0% of the total in the private sector. Agriculture and the fishing industry came next with 46,381 jobs or 24.3%. EPZ employed as many as 76,819 workers or 40.3% of the total in private sector, being the single biggest industry in terms of employment. The sugar industry followed, employing 40,974 workers.

Based on the 1983 Census results, the unemployment rate in 1986 was estimated by the Ministry of Economic Planning and Development at 13.8%. If the existence of the informal sector and the effect of introducing unemployment benefit provided by the Government are taken into account, however, the real unemployment figure may be much lower at present. The total Mauritian labour force in the country, namely the people of 15 years of age and above excluding those inactive was estimated at 399,200 of which males were 275,000.

(e) Trade and Balance of Payments

Exports have been steadily increasing in recent years at an average annual rate of 18.1% during 1980-86 period and reached Rs. 9,062 million at f.o.b value in 1986. The biggest single export industry was Export Processing Zone, Mauritius (EPZM) with Rs. 4,950.5 million of export earnings or 54.6% of the total. The sugar industry came next with Rs. 3,553.0 million or 39.2%, making together 93.8% of the total. Imports increased steadily during the same period. The average annual rate of increase was, however, much lower at 11.8%, reaching Rs 9,199.0 million at c.i.f value in 1986. The import value of manufactured goods and machinery was the biggest with Rs. 6,146.5 million or 66.8% of total imports. Food, live animals and beverages came next with Rs. 1,215.9 million or 13.2% of the total. Mineral fuels was the third with Rs. 706.6 million or 7.7% of the total.

With rapidly growing exports and relatively the slow increase of imports, the trade deficit had been decreasing steadily with the trade deficit of Rs. 137 million in 1986 compared with Rs. 1,380 million in 1980. In 1986, the balance of payments recorded a surplus of Rs. 1,288 million. Foreign exchange reserves at the end of 1986 were positive with Rs 1,983 million.

(f) Government Revenue and Expenditure

According to the provisional estimates by the Government of Mauritius, total Government revenue was Rs. 5,494 million for the 1986/87 fiscal year, out of which indirect taxes were Rs. 3,541 million or 64.5% of the total and direct taxes were Rs. 759 million or 13.8% of the total. Total expenditure was Rs. 5,918 million comprising Rs. 4,625 million of recurrent expenditure, Rs. 1,164 million of capital expenditure and Rs. 129 million of net lending. Overall budget deficit was estimated at Rs. 424 million or 2.2% of GDP at market prices, much improved from 9.5% in the 1982/83 fiscal year. Debt service ratio, which is the public external debt as a percentage of exports of goods and services, was 7.3% in 1986, much lower than 20% which is considered as the critical level for external debt servicing.

(g) Currency and Prices

Reflecting Mauritius' favorable balance of payments and foreign exchange reserves, the Mauritius rupee has kept stable exchange rates against other foreign currencies in recent years. The exchange rate of rupees as of June 1987 were Rs. 12.652 (buying) and Rs. 13.158 (selling) against US dollar and Rs. 8.793 (buying) and Rs. 8.99 (selling) against Japanese yen.

Consumer price indices (CPI) fluctuated much in early 1980's but kept stable afterwards. The yearly change in CPI from 1985 to 1986 was 1.8% and 1.2% from 1986 to 1987.

(h) Household Income and Expenditure

According to the 1986/87 household budget survey, average household monthly income in the 1986/87 fiscal year was Rs. 3,496 which is 58.0% higher than that in the 1980/81. Considering the inflation rate of 47.5% during the same period, household income improved about 10% in real terms. Over 42% of households had monthly incomes in excess of Rs. 3,000 in the 1986/87 compared with 21% in the 1980/81, indicating certain improvements in income distribution.

41.9% of household expenditure was spent on food and drinks which was 2.5% lower than in 1980/81. Among the expenditure items, education and other services showed the highest increase with 82.4% more expenses on these than in 1980/81 while the average total household expenditure increased by 33.3% during the same period.

2.1.2 Socio-Economy in Port Louis

(a) Administrative Division

Port Louis is the capital of Mauritius and 42.7 km^2 in area. It is located in the northwestern part of the island of Mauritius and to the northeast of the river mouth of the Grand River North West. Administratively Port Louis is designated as a municipality among 5 municipalities that are urbanized areas in the island of Mauritius. It also constitutes one district in itself. It comprises 6 wards and 19 localities.

(b) Population and Employment

According to 1983 Population Census, the population of Port Louis was enumerated at 133,702, which accounted for 33.2% of urban population or 13.8% of the population of the island of Mauritius. Out of the total, 66,132 persons were male, accounting for 49.5%. Population density was estimated at 3,131 persons per km^2 which is about 6 times higher than that of the island of Mauritius. The number of households was 29,187 with an average household size of 4.58 persons. According to the population estimates made by the Central Statistical Office based on 1983 Population Census results, the population of Port Louis grew at the average annual rate of 0.97% during 1983-85 period, which is slightly higher than 0.91% for the island of Mauritius and is identical with that for the country.

According to the 1983 Population Census, 61,387 people were employed in Port Louis district. Out of the total, 28,930 people or 47.1% were the residents of Port Louis district. Of the Port Louis residents, 5,805 were working outside the Port Louis district. The total number of employed residents in Port Louis district were, therefore, estimated at 34,735.

(c) Occurrence of Water Shortage in Port Louis Area

According to the flow records for the Grand River North West (GRNW) for 1965-1983 period, long-term mean monthly flow is low for the months starting from July till December. Rainfall and discharge is normally in abundance 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 last period of shortage was severe and prolonged till February, 1988 starting from August 1987. 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.

To cope with the shortages, CWA has been taking various measures to alleviate the adverse effects. In emergency cases including severe shortage, water is supplied through water main to Pailles treatment plant from the Petite Riviere reservoir of Mare-aux-Vacoas system with 5,000 m^3 per day at the maximum. CWA is advising factories to make production to their full capacities during January-August period and to lower production during the rest of year when water shortage and consequent water supply cuts usually occur. In order to alleviate the serious water shortage in the low pressure area, CWA supplements water supply by water tankers during period of shortage.

Although the seriousness of these shortages can be lessened by the counter measures taken by CWA, water shortages are expected to continue unless fundamental measures are taken to augment water resources.

2.1.3 Summary of Socio-Economic Data

Various analyzed and summarized socio-economic data are attached herewith.

Islandwise the population of Mauritius at the time of 1983 Census is given in Table 2.1.1. Population figures of the Island of Mauritius enumerated at the censuses from 1846 to 1983 are given in Table 2.1.2 together with the average annual increase rate during each intercensal period. Population of the District of Port Louis is given by ward in Table 2.1.3. Population of Mauritius at the end of 1983, 1984 and 1985 which was estimated based on 1983 Census by Central Statistical Office is given in Table 2.1.4.

Mauritian population in the island of Mauritius aged 15 years and above by activity status and sex in 1986 is given in Table 2.1.5. Employment in large establishments by major industrial group for March 1980-March 1987 period is given in Table 2.1.6. Employment for Export Processing Zone, Mauritius (EPZM) is given in Table 2.1.7.

Gross domestic product (GDP) and Gross National Product (GNP) by industrial origin at current factor cost are given in Table 2.1.8. Imports and exports by major commodity group for 1985-June 1987 period are given in Table 2.1.9. Balance of payments for 1981-1986 is given in Table 2.1.10.

Consumer price indices for 1980 through 1987 are given in Table 2.1.11. Foreign exchange rates of Mauritius rupee against sterling pound, Japanese yen and U.S. dollar are given in Table 2.1.12. Expenditures of an average household for the fiscal year 1980/81 through the 1986/87 are given in Table 2.1.13 together with average household income.

Recurrent and capital expenditures of the Government of Mauritius are given in Tables 2.1.14 and 2.1.15, respectively.

2.1.4 Project Economy

(1) Water Tariff

(a) Present water tariff

The water tariff was revised and new tariff has come into operation on the 16th of May, 1988. Under the tariff water rates are charged by the following categories of water users.

(i) Living quarters
(ii) Vegetable and flower growers and livestock producers
(iii) Government departments, parastatal bodies and religious institutions approved by the Minister
(iv) A public fountain
(v) Water supplied to a ship
(vi) A stand pipe
(vii) Enterprise supply
(viii) Other non-domestic consumers
(ix) Concessionary/Acquired enterprise supply

Except categories (iv), (vi), (vii) and a part of category (ix), water charge is calculated and collected according to the water consumption in combination with the minimum fixed charge. The tariff is basically progressive in character. In 1987, the average water charge per m^3 of water sold was Rs. 3.29 based on the previous water tariff.

(b) Water tariff revisions in the past

Water tariff have been revised 8 times since the establishment of CWA in July 1973 in order to reflect the cost of water supply and to improve the financial position of CWA so that CWA may be capable of extending its supply facilities and upgrading its service level as follows:

(i) 1st of November, 1974
(ii) 1st of July, 1979
(iii) 1st of July, 1980
(iv) 1st of December, 1981
(v) 1st of January, 1983
(vi) 1st of January, 1984
(vii) 1st of October, 1984
(viii) 16th of May, 1988

(2) CWA's Financial Position

(a) Financial policy

All the loan agreements with which CWA is concerned, provide that the authority shall generate sufficient revenues to cover:

(i) operating expenses;
(ii) depreciation;
(iii) interest payments on borrowing and repayment of long-term indebtedness to the extent that it does not exceed the depreciation provisions, and
(iv) a surplus for financing a reasonable portion of future expansion.

Specific provision is also made as regards debt service ratio which requires internal cash generation to be not less than 1.5 times the maximum debt service requirement. Besides the above, the IBRD agreement provides for additional measures including: to reach a certain specific "Operating Ratio" and an 8% rate of return on the assets employed and to re-value the assets for the purpose of making depreciation provision. The IBRD requirements are deemed to be applicable as from the fiscal year 1983/84 as regards "Operating Ratio", etc.

(b) Financial results

According to the "Summary of Financial Matters for the year 1985/86, CWA", the net income of the Authority recorded surplus for the first time in its history with Rs. 8.3 million after deducting operating expenses, depreciation, interest payment and others from the revenue as follows:

	Unit: Rs. million
Income	184.5
Expenditure	173.5
Prior year adjustment & exceptional items	2.7
Surplus	8.3

Of the total revenue collectible from potable water supply service, revenue from "Domestic Consumers" is the biggest with 62% share. "Government" followed with 22% and "Non-domestic consumers" 16%. Revenue accrued from irrigation water supply was quite small, about 1.4% of the revenue from potable water supply.

The return on the net re-valued operational assets for the potable water supply service was +6.34% while that for the irrigation service was -8.24%. The overall return was +5.26%.

(3) Household Expenditure on Water Consumption

According to the water sales data for 1987 compiled by CWA, total number of domestic consumers, (subscribers) for CWA water supply was 132,883 on an average consuming 56,504 m^3. Corresponding annual revenue of CWA was Rs 185,841. Average revenue per domestic consumer is estimated at Rs 69.7 per month.

According to the Country Report for Mauritius and three other neighbouring countries (No. 2 1988) published by the Economist Intelligence Unit, UK, monthly income and expenditure of the average household for the 1986/87 fiscal year were Rs. 3,496 and Rs. 2,764 respectively. Proportions of water expenditure to household income and expenditure are 2.0 percent and 2.5 percent respectively, which seems to be in a reasonable range.

(4) Financial Cost

The financial cost (price) of the materials and equipment produced domestically to be used for the Project is identical with market prices. If goods are to be purchased from overseas utilizing any external loan, either bi-lateral or extended from international financial organizations, custom duties and stamp duties on imports as well as sales tax and excise duties have to be excluded. Financial cost at the site is calculated as the sum of c.i.f. value of the goods, port handling charge including storage charge and inland transportation cost between the port and the site. Otherwise all the duties and taxes have to be added to the above cost.

Consumer price indices increased to 125.4 in the first half of 1987 from 106.6 in 1983. The average annual increase rate is estimated at 4.1%. Annual price escalation of some selected construction materials including iron bars and cement was 2.0% on an average from 1983 through 1988 (average of former half). Statutory minimum wage rates applied for construction labor increased by 7.0% annually on an average for 1983-87 period, while basic wage scale per day of manual workers recorded sharp increase with 23.5% for specialized laborers and 21.5% for the unskilled for 1986-87 period.

Future increase of the materials and labor prices will be determined considering the past price trend as well as the price prospects in Mauritius and the advanced economies.

Value of Mauritius rupee remained basically stable against international currencies in recent years. Average exchange rates of rupee against U.S. dollar and Japanese yen from 1984 through the former half of 1988 was as follows:-

Japanese yen 1.0 = Rs 0.105
U.S. $ 1.0 = Rs 13.7

The above rates may be used for conversion purpose in the Study.

(5) Economic Cost

Economic cost of the project is calculated by deducting transfer payments such as taxes and duties from the financial cost.

The unemployment rate was estimated at 13.8% for 1986. The real rate, however, may have been much lower, taking into account the existence of informal sector and introduction of unemployment benefit. Though data is not available for the current unemployment rate, it is very likely that it has been much improved considering the good performance of the country's economy in recent years. The trend is likely to continue in the coming years and prevailing market wage rates of unskilled laborers may be used as economic cost of the country.

The Mauritius rupee kept quite stable against international currencies and shadowing is not necessary. The exchange rates to be used for the financial analysis will also be used for the economic study.

(6) Average Water Revenue for Port Louis System

Future water revenue will be estimated for the purpose of the Study based on the average water charge or average water revenue per m^3 of water sold by category for Port Louis System after May 16, 1988 when the current water tariff became effective. Future increase of the average revenue due to the revision of water tariff will be assumed mainly based on the following considerations:

(i) Rate of inflation including the change of C.P.I.
(ii) Repayment of the loans to be extended to CWA for the implementation of the Project.
(iii) Repayment of the outstanding loans
(iv) Depreciation of the fixed assets
(v) Expansion and upgrading of the water supply facilities
(vi) Capacity-to-pay of the consumers for water bills

It will also be assumed that revisions, if deemed necessary, should be made at 3 to 5 year intervals.

(7) Opportunity Cost of Capital

Principal interest and discount rates in Mauritius in June 1987 are given hereunder:

	Unit: % per annum
Lending	
Bank of Mauritius	
- Bank Rate	10.0
- Rediscount facilities	10.25
Commercial Banks	10.25 - 16.5
Treasury bills (discount rate)	9.0

Deposits

Savings	8.0 - 8.5
Fixed deposits	8.25 - 11.25

Opportunity cost of capital which would be required to be estimated in case economic appraisal is to be conducted. It may be determined for the purpose of the Study, considering the above rates as well as the rate of inflation.

Revenue account of CWA is given for 1980/81-1985/86 period in Table 2.1.16, showing the surplus in the 1985/86 fiscal year for the first time since its establishment.

2.2 Existing Water Supply Facilities

2.2.1 General

The main water supply facilities of Port Louis System at present consist of the water intake/source of the Municipal Dyke, Pailles Treatment Works, three transmission mains to the city, 10 service reservoirs, two booster pumping stations at Plain Lauzun and Pailles and distribution pipelines with a range of main pipe diameters dia. 150 mm to dia. 800 mm.

2.2.2 Source of Supply

The piped water supply system of Port Louis has been in service for about 200 years.

The present source of supply is mainly the Municipal Dyke on Grand River North West. In 1980 two additional sources were developed and pipelines were laid to bring supplementary water supplies to Pailles water works.

These are:

(1) Montebello/Soreze-Pailles pipeline, which brings raw water from the Profonde river and Moka river

(2) Coromandel-Pailles pipeline, which brings clean water from Pierrefonds Via Petite Riviere reservoir to Pailles

Municipal Pipeline:

The raw water for treatment is taken at the Municipal Dike located on the way of the GRNW and transmitted by gravity to the pailles Treatment Plant through the raw water transmission pipeline. The raw water transmission line consists of three pipelines with diameters of 27", 19" and 18" installed in old historical days before 1925 (19" CIP and 18" CIP) and comparatively recently in 1960 (27" RCP). The length of the pipeline is about 2 km. They are presently working, although some leakages occur. The potential carrying capacity of this existing Municipal Pipeline is presently calculated at 622 l/sec.

Montebello/Soreze-Pailles Pipeline:

Montebello/Soreze-Pailles Pipeline consists of the Montebello system and Soreze system.

Montebello system takes the surface water of the profonde river at Martinale bridge (W006). This water runs through Bagatelle canal and pours into the Bagatelle reservoir. Montebello pipeline is constructed from the reservoir to Pailles Treatment Plant. Capacity of the pipe is said to be 283 l/sec. Bagatelle reservoir has a spillway through which surplus water returns to the Profonde river. There exist about 10 l/sec. abstraction for irrigation directly from the pipeline.

Soreze system takes the surface water of the Moka river mainly for the industrial use in Soreze and Montebello areas. The residual flow reaches Pailles Treatment Plant. The intake part consists of Soreze dam

(W002) and open canal (W003, Pailles canal) located at 150 m downstream of Soreze dam. Water through the canal is impounded in a small reservoir having a spillway through which surplus water returns to GRNW. After chlorination, water is carried through 236 mm dia. pipeline. The Pailles canal was originally constructed for irrigation purpose, but CWA has priority to use this water for potable water at present.

Coromandel-Pailles Pipeline:

This pipeline brings water from Pierrefonds to Pailles Treatment Plant via Petite Riviere reservoir where water is chlorinated. Its capacity is estimated at about 10,000 m^3/day. This pipeline is operational in emergency cases only.

2.2.3 Treatment Works

Pailles Treatment Plant/Filter Beds were constructed initially in 1926, then expanded twice in 1960 and 1980. The filter beds area was expanded as follows, in accordance with the past plant expansion:

(1) Old Filter Beds

a. approx. 1,800 m^2 with 2 beds (each has 900 m^2) constructed in 1926.

b. approx. 2,700 m^2 with 4 beds (each has 675 m^2) expanded in 1960,

Sub total a & b approx. 4,500 m^2 (4,482 m^2)

The old filter beds consist of two stage filter, which have roughing filter and conventional slow sand filter.

(2) New Filter Beds

Total filter area is 5,580 m^2, with 6 beds each has 930 m^2, constructed in 1980/81.

The new filter beds are single stage conventional slow sand filter.

(3) Total of old and new filter area = 10,062 m^2

Supposing that the design filter rate of old and new filters is at 5 m/d to 6 m/d, the total production capacity is at around 50,310 m^3/d to 60,372 m^2/d. However, according to the past records the production amount exceeds 58,000 m^3/d and over 60,000 m^3/d which were experienced several times.

Regarding the filter beds operation during the normal period with the sufficient supply of raw water, 5 filter beds with a total filter area of about 4,600 m^2 are operated to meet the demands and 7 filter beds are not in operation for the routine maintenance and kept for stand-by.

While, during the dry months, (November to December), only 2-3 filter beds are in operation due to the decrease of raw water availability at Pailles Treatment Works.

During the dry months, the raw water intake amount from Municipal Dyke is not sufficient and the temporary raw water pumping facility is installed to reinforce the raw water taking from Grand River North West.

The pumped water at about 2,200 m^3/d is directly feeded into the Municipal Pipelines on the way to Pailles Treatment Works.

In times of heavy rainfall, it is reported that the new filter beds, which have single traditional slow sand filter are easily clogged by the heavy silt load in the raw water. On the other hand the old filters with two stage filters dare to be operated even at the time of heavy rainfall.

2.2.4 Distribution System

There are 10 treated water service reservoirs on the hillsides in the service area with a combined capacity of 61,000 m^3 as shown in Table 2.2.2 and Fig.2.2.1.

Regarding the transmission and distribution mains, Table 2.2.1 presents the sizes of pipes and their locations. The existing distribution system serves an area of approximately 3,900 hectares, equal to 91 per cent of the total area within the municipal boundary of Port Louis (4,270 ha) (Ref. Table 2.2.3).

2.2.5 Problem

Major problems in the existing water supply facilities are:

- Deterioration of facilities.
  As Port Louis water supply system has a long history some facilities have deteriorated. Some of the old distribution pipelines in particular are in a very bad condition, and result in severe leakage.

- Lack of raw water storage.
  There is no storage facility for raw water; and when river flow is very low in the dry season raw water to be treated is insufficient to maintain distribution 24 hours a day.

- Treatment facility.
  The current treatment process is based on use of slow sand filtration applied to low turbidity raw water. However, sometimes after heavy rain the raw water from the river has very high turbidity and treatment has to stop.

- Leakage of raw water pipelines.
  The existing raw water conveyance system from the Municipal Dyke to Pailles Treatment Plant consists of three pipelines of 27", 19" and

18" in diameter, which suffer much leakage due to deterioration and make it difficult to transmit enough water.

2.3 Water Demand Projection

2.3.1 Population Projection

Forecasts of population for the planned years 2010 and 2030 of the study have been made based on the 1983 Housing and Population Census and the Digest of Demographic Statistics 1985.

Three (3) series of population projections, i.e. the high, medium and low series population projections, have been made.

The high and medium series of population projections are made on the bases of the past trend of natural population increase in Port Louis. Table 2.3.1 presenting the vital statistics of Port Louis (1975-1985) indicates the natural annual average increase of 1.3 -1.8 percent. Assuming that the population in Port Louis would increase in accordance with the natural increase, the high and medium projections were made by using average annual increase rates of 1.4 and 1.2 percent respectively over the 25-year study period.

On the other hand, the low series of population projection is made in consideration that the city of Port Louis has a restricted potential of physical development and practical limits to residential population growth. In fact, the city of Port Louis physically has no large allowance for the residential population growth. As seen in Table 2.3.1, despite the vital statistics show the natural average population growth of about 2,000 persons per annum, the actual increase of total population in Port Louis is much less than the natural increase. It is considered that some population in the city of Port Louis is emigrating to the surrounding areas due to the limited allowance for development. Since the above is recognized to be the real situation of the city of Port Louis, the population projection in which the mentioned physical limitation is taken into account is considered the most probable case of the future population in the city of Port Louis.

Then, the low series population projection which duly takes the physical limitation into consideration forms the principal bases of the water demand projection in the study. The high and medium estimates are subsequently used as indications of the possible changes of future domestic water demand.

In the low series of population projection, the saturated

population growth method by using the logistic curve, which is a practice to project the population in the case that a saturated condition is assumed and is given by the following equation:

$$Y_x = \frac{k}{1 + 10^{a+bx}}$$

where,

- x : year,
- k : saturated population,
- Y_x : population in year x, and
- a,b: coefficient

In the projection, the saturated population of Port Louis is assumed to be 200,000 which was estimated in the national Physical Development Plan (1977) and is considered reasonable.

Table 2.3.3 and Fig.2.3.1 present the result of projection of future population up to 2030. As seen, the population in the city of Port Louis is projected to increase to 162,494 in 2010 and 176,838 in 2030 respectively on the basis of the low series population projection, the most probable case of population increase.

2.3.2 Water Demand

(1) General

Taking into account CWA's present billing system, the study projects the following three categories: 1) domestic, 2) non-domestic, including industrial, commercial, public sector and seaport 3) government, covering government offices and enterprises.

Future demands of each category are then projected for these demand categories for the study planned years of 1990, 2000, 2010 and 2030.

Past records of Port Louis system total water consumption are presented in Table 2.3.4. Table 2.3.5 presents the said three categories of water sales as recorded for the Port Louis system in the period of 1981/82 - 1986/87. Table 2.3.6 presents the latest monthly record of domestic and total consumption in the year 1988.

(2) *Domestic Water Demand*

The present unit demand (per capita consumption) for the domestic life in the Port Louis system is analyzed and presented on Table 2.3.7. The figures analyzed in 1985 and 1986, i.e. 216 lpcd, 226 lpcd respectively are rather high amount as experienced in developing countries. While those of the year 1987 and 1988 - 6 months average are at 178 lpcd and 180 lpcd respectively.

In this study, the future unit water demand of domestic requirements is projected on the basis of the 1988 analysis at 180 lpcd, thereafter projected to increase up to 200 lpcd in the years 2010 and 2030 in consideration of the experience in developed countries.

The projection of domestic water demand is made applying the above unit demand to the projected population of low series shown in Table 2.3.3. In the projection of domestic water demand, the percentage of the served population is assumed to increase from 92% at present to 100%

in the year of 2000.

The projection of domestic water demand estimated as mentioned is given in Table 2.3.8. As seen, the domestic water demand is projected to increase from 23,184 m^3/day in 1988 to 35,368 m^3/day in 2030.

(3) Non-Domestic Water Demand

The non-domestic water demand is considered to consist of the commerce, industry, education and hospital uses. The water demand for each of above sub-groups is projected as follows:

<u>Commerce</u>

The commercial use is considered to be closely related to the number of employment, i.e. the number of commuters.

According to Bi-Annual Survey of Employment and Earning, March 1987, the total employments in Port Louis or commuters are estimated at around 90,000 to 95,000 in 1988, including the employments in the government. The employments in the government are counted at about 40,000, and therefore, the employments excluding those in the government are estimated at around 50,000 to 55,000 in 1988, based on which the number of employments excluding those in the government are assumed to be 56,000 in 1988.

As for the increase of number of employments, National Physical Development Plan, 1977 estimates to be about 110,000 for the physically maximum number or a saturated number of employments in Port Louis. Then, the number of employments is assumed to increase up to 110,000 towards the year of 2030.

Per Capita consumption is assumed to be about half of the domestic consumption in consideration that its activities are limited in the daytime.

The projection for the commercial consumption estimated as mentioned is shown in Table 2.3.10. As seen in Table 2.3.10, the commercial consumption is projected to increase from 4,480 m^3/day in 1988 to 9,900 m^3/day in 2030.

Industry

Based on the available industrial consumption record (1985-1988), the record indicates that the average consumption jumped from 1,300 m^3/day level in 1985 to 3,500 - 5,000 m^3/day level in 1986 thereafter, although its consumption dropped to 2,000 m^3/day level in 1988.
The decrease of consumption in 1988 to 2,000 m^3/day level is considered partially due to the water shortage, and the actual industrial water demand at present is estimated to be around 3,000 to 3,500 m^3/day.

As for the composition of the present industrial water consumption, the textile industry occupies 40 to 50 % of the total industrial consumption. The remaining 50 to 60 % is consumed by various kinds of industries such as the wearing apparels, retreading, leather product, wood, furniture and paper products, jewellery and related articles, laundry, etc.

The projection of industrial demand has an extreme difficulty for its estimate due to the following:

- The water demand is largely effected by the activity of the textile industry. However, the future tendency of its activity is very difficult to be projected exactly, since its activity depends on the national development plan and policy which are not definite at present.

- The composition of industry in the developing countories tends to largely change in accordance with the economic growth.
  In the case that a definite industrial development plan of the

nation is not available, the analysis on the future industrial water demand and its accurate projection are considered to be nearly impossible.

Such being the case, the projection of the industrial water demand is tried to be made on the following assumptions, although the result is forced to be very approximate:

- The present composition of industrial water consumption would be generally maintained in the same proportion.

- Then, the total industrial water consumption would approximately be proportional to the total number of employments.

The present number of industrial employments is estimated at around 15,000. On the one hand, the saturated number of employments is estimated at about 50,000.
Based on the above mentioned assumptions and present and future number of industrial employments and assuming that the present industrial water demand is 3,300 m^3/day, the total industrial water demand in future (in 2030) is calculated to be an order of 10,000 m^3/day as follows:

- Present consumption:

Use by employment :	15,000 x 100 l/day	= 1,500 m^3/day
	3,300 - 1,500	= 1,800 m^3/day
Total		3,300 m^3/day

According to past experience, industrial water consumption may approximately be assessed by applying the unit consumption rate of 220 l/day/person for the total number of industrial employments. Applying the above empirical practice, the total industrial consumption in future coincides with the above result obtained through a different approach, and therefore, the industrial water

consumption is assumed to increase up to about 11,000 m^3/day towards the year of 2030.
However, as discussed above, this projection is very approximate: The projection may considerably change in accordance with the future nation's policy. It is noted that the planning of the water supply project should take this possibility of change of the demand into consideration.

Education and Hospital Demands

Based on the present student population and the estimated per capita consumption (50 l/capita/day), the future demands of the education sector are projected as follows:

Description	1987/88	1990	2000	2010	2030
No. of students	30,000	30,000	35,000	40,000	50,000
Water Demand (m^3/d)	1,500	1,500	1,750	2,000	2,500

Regarding the hospital demands, the present and future demands are estimated as presented below, concurring with the assumption in the on-going rehabilitation program, and allowing for some future unit consumption increase.

Description	1987/88	1990	2000	2010	2030
Beds (No.)	450/500	600	800	1,000	1,000
qi22 Consump. (l/bed/day)	400	400	400	400	450
Water Demand (m^3/d)	200	240	320	400	450

The result of projection for non-domestic water demand consisting of the commerce, industry, education and hospital uses is as summarized in Table 2.3.11.

(4) Government Demand

The government consumption record in 1988 indicates an average amount of 2,000 to 2,500 m^3/day. The study adopts the past record at 2,500 m^3/day as constant for the future projection, since the nature of consumption is considered not to change much.

(5) Demand Projection Summary

Low Estimate (Most likely estimate)

The present and future water consumption by category of consumer are summarized in Table 2.3.12. It is noted that these projections present the low estimate or most likely set of estimates. As seen in the Table, the total water demand is projected to reach 82,490 m^3/day at production level in 2030.

One of the most significant features of the Table is the average increase rate of the domestic demand is not much at about 0.6 percent per year through the 45-year study period till 2030. While the non-domestic category demand presents its increase steadily at an average increase rate of 2.4 present per year through the study period.

Medium and High Estimates

In order to estimate the possible degree of change in water demand projection, medium and high estimates of water demand are made, considering that the possible change depends mainly on the variation of estimated population in Port Louis.

The medium and high estimates of water demand are prepared as seen in Table 2.3.13 based on the medium and high series increase of population in Port Louis estimated in the previous section 2.3.1. As seen in Table, the medium and high demands at production level are estimated to be 97,520 and 103,290 m^3/day respectively in the year of 2030.

The results of the water demand projections are shown in Fig.2.3.3. Fig.2.3.4 shows the most probable case of water demand projection together with the existing treatment peak capacity and required treatment peak capacity.

2.4 Meteo-Hydrology

2.4.1. General

Hydrological assessments cover: (1) availability of water for the project, (2) preparation of hydrological information for structural designs.

From the climatical point of view, the year is divided into two seasons, the summer season from November to March, and the winter season from April to October. 65 to 75 percent of annual total rainfall fall in the summer season. The driest month is October which has only 2.5 to 3.6 percent of annual total rainfall. The heaviest rainfall occurs usually in January to February. It is caused by cyclone or by front of Inter Tropical Convergence Zone. The discharge of the GRNW basin changes according to the above mentioned climatical cycle. The mean annual total discharge is estimated to be 88 MCM. One eighth of the discharge is used presently for water supply and the rest runs through into the sea unusably because there is not adequate surface water storage facilities to store flood discharge in GRNW basin.t

2.4.2 Rainfall

In the Study Area, four meteorological stations and fifty six rainfall gauging stations are in operation by CWA, the Meteorological Office and some sugar estates. These data have been collected and compiled by the Meteorological service. Some gauging stations such as Alma, Reduit Experimental Station and Vacoas have quite long recording durations of about 100 years. Locations of rainfall stations and meteorological stations are shown in Figs. 2.4.1 and 2.4.2.

a) Rainfall data

In order to investigate rainfall features 20 stations are selected. Daily data of these stations for recent 23 years (1965-1987) and monthly total data for recent 37 years (1950-1987) are collected from the Meteorological services. Location of rainfall stations and isohyetal map of GRNW basin are shown in Fig.2.4.1.

One rainfall station covers about 5 km^2 on an average. This density is enough to estimate basin rainfall even if the rainfall pattern is sporadic.

b) Basin rainfall

The Thiessen polygon network method is applied to estimate area rainfall because rainfall station is well distributed and density of station is appropriate. Fig.2.4.1 shows applied Thiessen Polygon. There is a clear tendency for annual rainfall to increase by elevation of the location. This pattern is also observed in a storm like a cyclone, but the polygonal network can only represent such spatial distribution without allowance for topographic conditions in the area.

Annual basin rainfall above the five water level gauging stations (W03,W04,W05,W08 and W10) range from 2300 mm to 2550 mm on an average. Basin rainfall of the five main tributaries by order of magnitude are: the Plaines Wilhems river, the Terre Rouge river,the Cascade river, the Profonde river and the Moka river.

c) Relationship between area and rainfall (heavy storm)

Area-rainfall depth as a percentage of the point rainfall (area reduction factor) is obtained from the isohyetal map of several storms. This relationship indicates that the area reduction ratio during intensive cyclone in the area is relatively uniform and this ratio decrease gradually by the area as shown in Fig.2.4.3. This is because the rainfall area during an intensive cyclone is much larger than the whole of Mauritius island, and also of the Study area. Daily showers in the summer season by local air convection are quite sporadic and basin rainfall of such kinds is variable. Fig.2.4.3 shows also two different equations: Flecher's and Horton's. Cyclone Gervais (5-7, Feb. 1975) had a wider and more uniform rainfall area than usually estimated by these two equations.

d) Probability of Rainfall

Annual maximum rainfall usually occurs during December to February by cyclone or unstable convergent airmass stimulated by cyclone. The duration of such heavy rainfall depends on the hysteresis of the cyclone and large scale climatical system.

Annual maximum point rainfalls at selected 20 stations and the largest among these stations were selected. Frequency analysis was carried out by applying some probability density functions such as Gumbel,Pearson III, Hazen, and Modified Log-Normal (IWAI). The results are as shown in Table 2.4.1. From the standpoint of safty, the estimated maximum values by return periods are selected. Following is a summary of probable rainfall in GRNW.

Probable Rainfall by return periods (Unit : mm)

Return Period	One-day	Two-day	Three-day
10000	1168	1799	1999
1000	935	1381	1551
200	771	1114	1260
100	701	1003	1140
50	630	901	1021
20	536	765	864
10	463	661	751
5	387	551	632
2	272	398	470

e) Probable Maximum Precipitation (PMP)

PMP is calculated by Cyclonic-adjustment method and statistical method. From the safety of a dam structure, PMP by cyclonic adjustment method, PMP by two statistical formulae and 10000-year probable rainfall are compared. Of these 10000-year one-day precipitation is the largest of these four cases.

Cyclonic- adjustment	Formula (I)	(II)	10000-year
1021 mm	1081 mm	1039 mm	1168 mm

d) Other meteorological factors

- 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 1694 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.

- Dew point temperature

  Dew point temperature during extremely heavy storms are also collected to estimate probable maximum precipitation for dam/spillway design by cyclonic-adjustment method .

- Wind velocity

  Extreme wind is caused by cyclone. One of the severest wind at Mon Desert during Cyclone Gervaise (82 miles per hour in one hour average) is selected to determine design wind velocity which is applied to calculate height of wind waves in Dam reservoir. Data of annual highest wind speed of 30 MPH (miles per hour) and above over a whole hour recorded during tropical cyclones at Pamplemousses from 1876 to 1975 indicate that the selected wind velocity corresponds to about 100 year return period.

2.4.3 Runoff

a) Discharge data

There are six gauging stations in operation in GRNW basin as shown in Fig.2.4.1. Of these, five stations are in tributaries in Central Plateau. The rest station ,W13, is located at Municipal Dyke in GRNW.

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 Dyke (W13) have concrete control weir sections and critical flow occurs over the weirs. As for the gauging station on the Moka river (W10), the river section some 10 meters downstream of the station is covered with fresh lava and critical flow also occurs over this section. These stations are well maintained and operated by CWA, Hydrological section and data have been published as hydrological year books since 1964. As for W13, the conduit pipe connected to the cylinder of a float of the recorder is set to the lowest level of the weir crest. The whole amount of water is usually abstracted at the weir during low flow conditions and the water level falls down below the conduit pipe, therefore data at W13 are not available for low flows.

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 omitted in the study.

b) Rating curve at water level gauging stations

Rating curves currently used for the six water level gauging station in GRNW basin do not cover relatively high water levels which occasionally occur during floods, because the above mentioned concrete sections at the water level gauging stations are all designed for low flow measurement. In order to evaluate flood ,especially extreme flood peak, rating curves for the range of high water level are developed for each station by means of non-uniform flow equations. These newly developed curves are applied for flood analysis.

c) Surface water abstraction

The number of main surface abstraction facilities in the GRNW basin is thirty five and their total entitled volume reaches 1 m^3/sec, excluding the Municipal dyke and return flow to the river channel. CWA has monitored the abstraction conditions and has conducted continual direct measurement. Several automatic recorders have also been installed on canals and operated by CWA. Direct measurement data at twenty six sites of these canals are collected to develop a relationship between river flow and abstraction especially for water balance simulation which is described in section 2.4.4 . Intakes of some canals have sluice gates. In these cases, empirical relations are estimated.

d) Discharge at gauging stations

The GRNW consists of five tributaries which originate on the central plateau some 600 m above mean sea level,draining the water westward or northward. These rivers form steep gulleys after running through the plateau and the river bed reduce their elevation drastically. Because of this geographical feature, the flow of the GRNW is changeable according to rainfall. Reliability of flow in each tributary is shown below.

				(m^3/sec)
Station	average	Reliability		
		80 %	90 %	95 %
W03	0.48	0.170	0.137	0.120
W04	0.48	0.102	0.087	0.077
W05	0.73	0.247	0.182	0.157
W08	0.32	0.136	0.109	0.091
W10	0.69	0.230	0.171	0.145

Average annual run-off ratio changes by basins because the existing flow include effect of artificial abstractions as mentioned above. Table 2.4.2 shows annual average run-off ratio at water level gauging stations.

e) Estimates of additional flow between gauging stations and Municipal dyke

Five gauging stations are located in the Central Plateau. On the other hand, the proposed intake site for the project will be located at Municipal Dyke or its adjacent site. The residual area between gauging stations and Municipal Dyke is 24 km^2 ,or 21 per cent of catchment area of GRNW at Municipal Dyke.

In order to estimate available water at the Municipal Dyke, relationship between flow at each gauging station and flow downstream was established on the basis of actual direct measurement carried by CWA Hydrological section and JICA team several times.

Finally, the relationship between the discharge at gauging stations and discharge downstream are determined, which makes the estimation of discharge at W13 possible. The selected relationships are summarized in Table 2.4.3 and Fig.2.4.5 (Site identifiers such as A-6 are indicated in Fig.2.4.5).

Preceding preliminary hydrological study suggests that there may be noticeable flow loss along GRNW ,especially Soreze area. According to water balance at Municipal Dyke based on direct measurements, flow loss can not be identified and gain is observed to the contrary. There are huge amount of river deposit along the river, therefore some of surface water may flow in the river deposit. This loss reappears again at outcrop of rock foundation such as outcrop at Municipal Dyke.

There exists run-off type hydro-power station at the Terre Rouge river and the power station releases water into the river channel when retardation pond is filled with water. This operation of power generation causes cyclic fluctuation of flow volume of GRNW. Therefore, flow at Municipal Dyke is measured continuously to a corresponding cycle of the power generation operation to erase the artificial effect.

According to direct measurement on November 1, total amount of five tributaries is 570 lit/sec. Abstraction downstream of the five stations reaches 275 lit/sec and rest of flow is 295 lit/sec (570 - 275). On the other hand , the average flow at Municipal Dyke is 549 lit/sec,which means that additional flow between gauging stations and Municipal Dyke is 254 lit/sec (549-295).

Date 1/Nov,1988

W03	W04	W05	W08	W10	Total(1) (a)	Abstraction(2) (b)	W13(3) (c)	Balance (c)-((a)-(b))
79.2	130.4	176.3	114.5	69.6	570.0	275.0	549.1	+254.1

(1) : Total flow at five gauging stations (Average of AM.9:00 - PM.1:00)

(2) : W019,W026,W002,W003 and W006

(3) : Average of AM.10:30 - PM.17:30

It seemed as if there is loss along the river channel. But,if abstractions between gauging stations and Municipal Dyke are taken into consideration, total discharge at Municpal Dyke is much more than those of accumulated flow at five gauging stations,suggesting no loss of water in the river channel.

Release from dam for water supply will occur during dry season. Therefor, leakage through river bed does not affect on the storage capacity and release volume from TRO dam.

2.4.4 Water balance and required storage

a) General

The simulation aimed to find out the desirable effective storage of the reservoir needed to meet the water demand in Port Louis City, i.e. 82,000 m^3/day in 2030, through clarifying the relationship between the effective storage and reliability of water supply, and that between the water demand and the required effective storage. A river system diagram and the location of existing water supply facilities are shown in Fig.2.4.4. Detailed conditions for the simulation are described in Appendix-A.

b) Required Storage Volume

The study on water demand and supply balance was carried out to reveal water deficit condition in the target year 2030 and intermediate years.

The severest drought year 1983 under recent 20 years hydrological condition was adopted as "base drought year" and available discharge in 1983 was applied for the water balance study. The lowest 10-day mean discharge in 1983 is 0.34 m^3/sec and annual average 10-day discharge is 1.5 m^3/sec. The results of water balance are as shown in Fig 2.4.6. The water deficit for the target year 2030 and intermediate years are as follows

Year	Water Demand		Water Deficit
	(m^3/d)	(MCM/y)	(MCM/y)
1990	60,250	22.8	1.8
2000	71,210	26.0	3.7
2010	78,569	28.7	5.2
2030	82,490	30.1	6.1

Required storage volumes for several demands are shown in Fig. 2.4.7. The severest hydrological condition occurred in hydrological year 1983. The required storage volume is determined to meet the water demand in the above severest hydrological condition,which corresponds to about 24-year recurrence drought year as seen in Fig.2.4.8.

Water deficit occurs if the demand is more than 0.45 m^3/sec and the necessary volume increases rapidly especially if demand is more than 0.8 m^3/sec. Required storage volume corresponding to the demand in year 2030 (0.95 m^3/s + maintenance flow of 0.05 m^3/s + 5% for the loss at treatment plant = 1.05 m^3/s) is 6.3 MCM.

The present demand of 62,000 m^3/day (or 0.72 m^3/s) may cause 1.3 MCM deficit under the severest condition, which explains the severeness of water supply condition in 1983-1984.

Table 2.4.4 shows released water, inflow and the required volume in 2030 under hydrological conditions of recent 20 years.

c) Reliability of water supply

In order to check severity of deficit and supply reliability for a certain storage volume, the number of days while deficit occurs and water supply reduction ratio are calculated.

The water supply reduction ratio is calculated as follows:

$$C = (S - S_0)/N / 86400 / Q$$

where, C : Water supply reduction ratio

S : Given storage (m^3)

S_0: Required storage (m^3)

N : Number of days when deficit occur

Q : Water demand (m^3/sec)

Water supply has to be reduced by this ratio if the severity of constraint is controlled to be constant while deficit occurs. Table 2.4.5 presents the reliability of water supply for the demand in 2030 under various given effective storages, showing the number of deficit days and water supply reduction ratios under hydrological conditions of recent 20 years.

Table 2.4.5 presents the water supply reliability under various given storages in the case of the low series water demand projection which is considered the most probable estimate, and indicates that the water supply shortage would be solved with the effective storage of 6.3 MCM completely under hydrological conditions of recent 20 years.

The following discusses what will happen in the event that the water demand will increase in accordance with the high or medium series of projection made in section 2.3.2.

Fig. 2.4.7 also presents the relationship among various water demands, various given effective storages of reservoir and water supply reliability under the hydrological conditions of recent 20 years.
In the high and medium series of projection, the water demand in 2030 is projected to be 103,290 m^3/day and 97,520 m^3/day respectively. The water demand is calculated at 1.31 m^3/sec and 1.24 m^3/sec respectively in terms of necessary discharge at production level taking the loss in the treatment plant (5%) and necessary maintenance flow (0.05 m^3/sec) into consideration as follows:

High series projection :
103,290 m^3/day/24 hrs./3600 sec. x 1.05 + 0.05 m^3/sec = 1.31 m^3/sec.

Medium series projection :
97,520 m^3/day/24 hrs./3600 sec. x 1.05 + 0.05 m^3/sec = 1.24 m^3/sec.

Referring to Fig. 2.4.7, the water supply reliability in the high and medium series of demand projection will decrease to 97.7% (annual

average number of deficit days : 8.4 days) and 98.8% (annual average number of deficit days : 4.4 days) respectively, provided with the effective storage of 6.3 MCM.
If the effective storage is limited to 4.0 MCM, the above water supply reliability will decrease down to 94.8% (annual average number of deficit days : 19 days) and 95.6 % (annual average number of deficit days : 16 days) respectively.

It is considered important in the plan formulation of the project to pay a care so that the project has the flexibility for the possible fluctuation of water demand as much as possible.

d) Capacity of existing facility

In order to assess incremental benefit between with and without the Project, the future supply condition with existing facilities is estimated by the simulation mentioned above.

Existing water supply facilities whose source is GRNW basin do not have any storage capacity to meet deficit during the dry season. Besides this, capacity of the transmission pipeline from Municipal dyke decreases. Therefore, supply by existing transmission facilities depends on these two factors,or, flow condition and capacity itself.

Supply volume under two hydrological conditions such as hydrological year 1983 and recent 22 years with enough transmission capacity and capacity of pipelines are calculated by demands in the feature and summarized below,

m^3/sec

Hydrological condition	Water Supply to the demand in 1988	1990	2000	2010	2030
1983	0.737	0.720	0.796	0.830	0.851
22-year average	0.784	0.766	0.887	0.952	0.990
pipeline capacity	0.905	0.901	0.874	0.785	0.753

Dominant factor on supply volume will be transmission capacity after 2000 year. Details are shown in Table 2.4.6.

2.4.5 Flood Analysis

a) Effective rainfall

Effective rainfall is decided from effective rainfall analysis with 27 hydrographies and corresponding basin rainfalls as shown in Fig.2.4.9, and finally the following equation is deduced,

$$Re = 0.170 * R \quad (Rs < 250 \text{ mm})$$
$$Re = 0.625 * R \quad (250 \text{ mm} < Rs)$$

where, R : Rainfall (mm/hour)
Re: Effective Rainfall (mm/hour)
Rm: Accumulated Rainfall (mm)

Flood runoff ratio increases according to total rainfall depth if rainfall intensity is the same level, because larger part of ground surface become satulated. In order to reflect this tendency on probable Maximum flood (PMF) , runoff ratio when total rainfall depth exceeds 700 mm is esimated to be 80 % which is maximum value of flood runoff ratio in Tercialy or Quarternary mountainousarea,or,

$$Re = 0.80 * R \quad (700 \text{ mm} < Rs)$$

b) Storage Function Method

In order to decide design flood, flood simulation is carried out by means of the "storage function method". The storage function presumes a rainfall storage defined as the balance of rainfall and run-off volume, and computes the discharge in time series from the changing of the storage volume in the basin by means of the equation of continuity of volume and the storage function. For detail see Appendix-A.

In order to determine two parameters (k,p), four extreme storms which have both hydrographies at water level gauging stations and corresponding basin rainfall are selected.

By means of Thiessen polygon, daily basin rainfall are calculated according to area-rainfall allocation . Hourly rainfall data at only two stations are available. But hourly rainfall pattern in the Study area is the same ,so long as heavy rainfall such as cyclone are concerned. Based on this information, hourly rainfall patterns at Vacoas during the corresponding storms are applied for basin rainfalls of four cases.

c) Determination of parameter k,p

Parameters of the model ,k,p are determined so that a simulated flood hydrograph coincides with the actual one. Finally determined parameter sets for each flood is as follows;

Case	k value	p value	Station	Catchment Area
A-1	25.6	0.415	W03	29.7
A-2	41.1	0.415	W04	17.6
A-3	32.7	0.415	W05	8.3
B	24.5	0.415	W13	113.2

K values of A-2 and A-3 are larger because station W04 and W05 are located in the Central Plateau and covered with sugar cane and slope of these catchment is quite gentle. On the other hand, W03 is located in the Plaines Wilhems river which is the most urbanized area. Station W13 is located in GRNW gorge and this catchment area include very steep slope area and this area makes flood peak high,therefore k value is smallest among them. Actual and calculated flood hydrographs are shown in Fig.2.4.10 (1) and (2).

d) Design Flood at TRO damsite

- Simulation model

  Based on the calibration with actual hietgraph and hydrograph, 4 sets of parameters are determined as mentioned above. Proposed dam (TRO) is located at 2km upstream of W13 station and the catchment of the damsite also includes steep slope area in the gorge along river channel, therefore the set of parameter of Case B is applied for design flood at TRO damsite.

- Design rainfall

  According to observed data, annual maximum of one-day, two-day, three-day duration occurred in a sequence of one storm. Therefore three-day series of probable one-day rainfall by return periods are developed as follows,

$$P_{3rd} = P_{1\text{-}day}$$
$$P_{2nd} = P_{2\text{-}day} \times 2 - P_{3rd}$$
$$P_{1st} = P_{3\text{-}day} \times 3 - P_{2nd} - P_{3rd}$$

where; P_{nth} : one-day rainfall of n-th day

$P_{n\text{-}day}$: n-day probable point rainfall of given return period in GRNW:

- Basin rainfall

  Three-day series of probable rainfall is point rainfall which may occur in GRNW. Therefore this values should be modified based on the relation between area and basin rainfall as mentioned in chapter 2. Catchment area of proposed damsite TRO is 54.9 km^2, and area reduction factor of 0.85 is applied as shown in Fig.2.4.3. This ratio is maximum value for the area.

e) Hourly rainfall pattern

24-hour rainfall record at Vacoas whose amount exceed 180 mm for recent 21 years are selected to determine hourly rainfall pattern for design flood as shown in Fig. 2.4.11. Of them, rainfall pattern on 6,Feb. 1975 is considered to be severest on both total volume and intensity and 24-hour disbursement is finally determined as follows. As for three-day series of one-day rainfall , same hourly rainfall pattern is considered to repeat.

Hour	Percentage	Hour	Percentage
1	0.52	13	5.70
2	3.52	14	12.11
3	0.58	15	13.42
4	1.26	16	6.58
5	0.93	17	0.82
6	1.62	18	0.47
7	2.74	19	0.08
8	6.85	20	0.14
9	6.30	21	2.16
10	10.96	22	1.53
11	9.04	23	1.51
12	10.96	24	0.44

Based on the above result design rainfall by return period are estimated as follows,

(mm)

Return Year	1st day	2nd day	3rd day	Purpose
10	77	168	393	-
20	84	195	455	coffer dam design
100	116	257	596	
200	125	291	656	dam/spillway design
10000	171	536	993	free board of dam

f) Design flood

Probable flood peaks and their hydrographs by return periods are developed by using three-day series of probable one-day basin rainfall of some return periods.

The estimated peak discharge of the simulated hydrograph at TRO damsite are 455 m^3/sec at 10-year flood and 796 m^3/sec at 200-year flood, respectively.

Probable floods and their hydrographs are shown in Fig.2.4.12 and summarized below.

Probable Peak Discharge

Return year	Peak discharge (m^3/s)
10	455
20	536
100	718
200	796
10000	1596

2.4.6 Sediment

a) Available data

Sediment data are not enough to confirm the relationship between sediment and discharge. Therefore, data of three experimental areas are used though the areas are relatively small. Relationship between sediment content and daily mean discharge are deduced from observed sediment data and corresponding discharge data as shown below;

$$Q_s = q_s \times (1/r) \times 10^{-6} \times Q_{TRO} ,$$

$$Q_{TRO} = q_{TRO} * Area$$

where,

q_s : daily mean sediment content (mg/l)

q_{TRO} : specific discharge of daily mean at TRO damsite($m^3/sec/km^2$),

Q_s : daily mean sediment yield corresponding to Q_{TRO}(m^3/sec),

Q_{TRO} : daily mean discharge at TRO damsite (m^3/sec),

Area : catchment area (54.9 km^2),

r : dry density (g/cm^3) .

The above equations are simplified with assumption that r is 1.5 ;

$$Q_s = 10.54 \times 10^{-6} \times Q_{TRO}$$

or, $$TQ_s = 0.9107 \times Q_{TRO}^2$$

where, TQ_s : daily total of sediment yield (m^3)

b) Estimated sediment yield

Sedimentation in the TRO dam reservoir is estimated on the basis of the equation mentioned above and daily mean discharge at TRO damsite for recent 20 years. Daily mean discharge at TRO damsite is mentioned in section 2.1 (d). Fig.2.4.13 shows estimated sediment yield and its mass curve. Average annual sediment yield is 3949 m^3 ,or 0.07 mm in depth of whole catchment area of the dam under the hydrological conditions for recent 20 years.

Bed load transport is estimated from present sediment condition in the reservoir of Municipal Dyke and annual average sediment is estimated to be 140 m^3 /year in an average. This figure is quite small because the river bed is quite stable and deposit on the river channel is well balanced along the GRNW.

Total sediment trapped in TRO dam reservoir consists of 1) trapped wash load and 2) bed load transport.

Trap ratio of wash road into the reservoir is set to be 70% of total wash load, or 3949 m^3/year. Total sediment stored in the