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

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

FINAL REPORT (2)

SUMMARY REPORT

FOR

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

MARCH 1992

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

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

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

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

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

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

March , 1992

Kensuke Yanagiya

Kensuke Yanagiya
President
Japan International Cooperation Agency

Introduction and Background

1. Mauritius is a volcanic island of 1,860 km^2 and located about 900 km to the east of the Madagascar in the Indian 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 neighbouring 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 the Feasibility Study of the Port Louis Water Supply Project, and JICA (Japan International Cooperation Agency), the official agency responsible for the implementation of the technical cooperation program of the Government of Japan, was appointed to undertake the Feasibility Study in cooperation with the Government of Mauritius.

5. The Feasibility Study was carried out during the period from March, 1988 to July, 1989. The Feasibility Study made examinations on various conceivable alternatives and revealed the following scheme would be most advantageous.

(1) Water Resources

- Damsite = TRO damsite (immediate downstream of the confluence of the Terre Rouge river and the Profonde river)
- Dam type = Rockfill dam with the center core
- Dam height = 75 m (from river bed)
- Dam crest length = 230 m
- Gross reservoir storage = $6.7 \times 10^6 m^3$
- Effective reservoir storage = $6.3 \times 10^6 m^3$

(2) Raw Water Transmission Facility (Addition):

- Discharge = 660 ltr/sec
- Number of lane = 1 lane
- Diameter of pipe = 800 mm
- Length of pipeline = 2,100 m

(3) Water Treatment Facility (Addition):

- Type - Rapid filtration
- Capacity - 30,000 m^3/day

The Feasibility Study also confirmed technical, economic, financial and social feasibilities of the above scheme, and recommended earlier realisation of the project.

6. Following the Feasibility Study, the Government of Mauritius decided to carry out the Detailed Design, and requested technical and financial assistances to the Government of Japan, who agreed to carry out the Detailed Design in response to the request.

JICA was decided to undertake the Detailed Design Works.

7. The Detailed Design works commenced from March, 1990, and have carried out (i) additional field investigations, (ii) Basic Design, (iii) Detailed Design for Lot-I, (iv) Detailed Design for Lot-II and Lot-III in accordance with the work schedule as shown in Fig.2.

Note: The Lots are divided as follows:-

- Lot-I Diversion tunnel and preparatory works
- Lot-II Dam and appurtenant structures
- Lot-III Raw water transmission pipeline and water treatment facilities

8. Out of the above works, (i) the additional field investigations, (ii) Basic Design and (iii) Detailed Design for Lot-I were finished by March, 1991.

9. Following the above, the Final Report (2) which summarises results of the Detailed Design for Lot-II has been prepared. This report is a summary of the above Final Report (2).

It is noted that the Final Report (2) for Lot-III is separately prepared.

Detailed Design Works for Lot-II

10. The construction works of Lot-II include the following components:-
    - (i) Coffer dam (The coffer dam forms a part of the main dam)
    - (ii) Main dam
    - (iii) Spillway and energy dissipator
    - (iv) Intake facilities (including the tunnel closure work)
    - (v) River outlet facilities
    - (vi) Remedial works of the existing Municipal Dike

11. Followings have been prepared through the Detailed Design Works of Lot-II, which compose the Final Report (2).

    - (1) Design Report (with appendix)
    - (2) Tender Documents:
      - Vol. I - Instruction to Tenderer
        - Form of Tender Bond
        - Form of Performance Bond
        - Form of Advance Payment Bond
        - Form of Agreement

- Letter of Acceptance of Tender
- Conditions of Contract
- Form of Certificates for Source and Origin and Eligibility

Vol.II - General Specifications
- Technical Sepcifications

Vol.III - Form of Tender
- Bill of Quantities
- Schedule of Particulars

Vol.IV - Design Drawings

3. Cost Estimate

4. Design and Works Quantities Calculations

Results of Additional Field Investigations

12. The geological conditions of the damsite have been grasped through core borings (17 points, 1,280 m in total length), seismic explorations (22 lines, 4,045 m in total length), etc. carried out in the Feasibility Study Stage. The above investigations found that the geological conditions of the proposed damsite would be favourable for construction of a rockfill type dam with about 80 m in heights.

In carrying out the Detailed Design, the following additional investigations were executed to reconfirm the geological conditions in detail:

(i) Core borings (damsite)	16 points, 1,519 m in total length
(ii) Permeability tests	The tests were made in the above core boring holes
(iii) Measurement of ground water levels	"do"
(iv) Core borings (quarry site)	3 points, 60 m in total length

(v) Test groutings	one point in damsite
(vi) Test aditings	4 adits (2 adits each in both the abutements) 420 m in total length
(vii) In-situ rock tests (shearing and plate loading tests)	4 sets (One set each in 4 test adits)

Locations of the investigations and geological conditions are as shown in Fig.3 to 5.

13. Major items clarified or confirmed on the basis of the additional investigations carried out are as follows:-

(i) The damsite geology is composed of horizontal alternate layers of very hard basalts and weak layers of highly weathered basalt.

(ii) The core recovery rate is very high with approximately 100%

(iii) Permeability is relatively low, indicating permeability coefficients of 10^{-4} to 10^{-5} cm/sec.

The highly weathered weak zones show rather lower permeability.

Parts showing permeability coefficients of the order of 10^{-4} cm/sec exist in the hard basalt layers, which is considered to be due to cracks in the hard basalt. The usual cement grouting will efficiently improve the permeability up to an order of 10^{-5} cm/sec.

(iv) The grout tests clarified that the cement grout is effective to improve permeability of the foundation. The cement injection is about 50 kg/m in average. The design of curtain grouting with 2 lanes having 2 m hole interval will create a sufficient grout curtain in the foundation.

(v) The geology of internal parts of the dam foundation was visually confirmed through the test adits excavation. Any lava tunnels or openings to cause a large amount of leakage were not found in the test adits, suggesting that such openings

do not exist with a very high probability.

(vi) Mechanical strength of the foundation was confirmed through the in-situ rock tests.

The strength is measured as follows:-

Rock Classification	Elastic modulus (kg.cm^2)	Shear strength (kg/cm^2)	Internal Friction Angle(degree)
Highly weathered rock	1,500	1.2 to 3.1	37 to 48
Consolidated hard clay	3,500	2.1	39
Basalt	55,000	19.0	48

(vii) Presence of a place where a low groundwater level is indicated was found in the left bank of the damsite (about 400 m from the left abutment of the damsite).

However, the reason is considered not to be presence of large openings but to be presence of a relatively higher permeable portion, judging from the fact that the water level in the core boring holes lowered gradually.

14. Investigations on the dam construction materials were carried out for determination of dam design values and for confirmation of available quantities. The investigations carried out are as follows:-

(i) Core material (Earth material)
- Test pits excavation and samplings: 4 points 4 m in depths
- Various laboratory tests

(ii) Rock Material
- Core borings in the quarry site
- Various laboratory tests
- Large scale direct shear test

(iii) Concrete aggregates tests

(iv) Concrete mix proportion tests

15. As a result of material investigations, it was confirmed that available quantities would be sufficient and that the quality is also favourable.

Based on the investigations, the design values and concrete mix proportion are determined as shown in Table 2 and 3 respectively.

Dam Design

16. The dam type is determined to be a rockfill dam with center core through comparative studies on various conceivable dam types.

17. The gross reservoir storage necessary to meet the water demand in the Port Louis city in 2030 is assessed to be 6.7 x $10^6 m^3$. The dam height to acquire this gross reservoir storage is 84 m (EL.196.0 m in dam crest level), based on which the dam is designed.

It is determined that the dam design considers a future dam expansion of 20 m in height with consideration that a possibility of further water resources development should remain for the water demand increase after 2030. Hence, the above dam extension in future is also one of the necessary dam design conditions which should be taken into consideration.

18. Upstream and downstream dam slopes are determined through dam slope stability analyses on the basis of dam materials design values determined through materials investigations.

Determined dam slopes are as follows:-
Dam upstream slope = 1 to 2.3
Dam downstream slope = 1 to 1.8

Loading conditions which determined the above slopes are as follows:-

For Upstream slope:	(i)	After dam expansion
	(ii)	Seismic condition
	(iii)	At the time of rapid dreamdown of reservoir water level

For downstream slope:	(i)	After dam expansion
	(ii)	Seismic condition
	(iii)	High water level of the reservoir

19. The dam consists of five (5) zones: that is, (i) the impervious center core, (ii) fine filter zone, (iii) coarse filter zone, (iv) rock zone and (v) surface riprap zone.
It is noted in the zoning that two filter zones of fine and coarse filters are provided by the following reasons: that is, the available core material belongs to MH soil containing much small particles in which water passages resulting in piping tend to be created, and a particular attention should be paid for the design of the filter to prevent the piping.

20. The dam foundation excavation is based on the following considerations:-

(1) The top sol and highly weathered portions are all excavated in the core and filter zones which should be founded on a sound foundation to avoid differential settlements and to secure impermeability in its foundation.

(2) The right abutment of the dam has a very steep slope which should be made gentler to make the contact between core and its foundation sufficient.

The excavation will be made so that the slope becomes l to 0.7 in due consideration of the standard.

(3) On the rock zone, the excavation will, in principle, be made in a depth of 2 to 3 m so as to eliminate all organic materials such as roots of trees.

Talus deposits exist in upstream of the left bank and in downstream of the right bank. These talus deposits do not necessarily have to be excavated. Some talus deposits will remain the rock zone foundation based on the dam stability analyses.

21. As mentioned, the damsite geology consists of the horizontal alternate layers of hard basalt and highly weathered weak layers, and therefore, the highly weathered weak layers will inevitably appear on the excavated surface of the core foundation. Troubles such as occurrence of cracks in the core due to the differential settlement may possibly arise if the heavy weight of core material is loaded on the above foundation.

In order to clarify effects due to the said alternate layers and to examine an effective countermeasure if necessary, deformation and stress analyses by the Finite Element

Method (F.E.M) were executed.

As a result, it was clarified that tension would arise in the basalt layer above the weak layer due to a cantilever action if no countermeasure is provided. The above suggests possibility of break of the basalt layer and crack occurrence in the impervious core founded on it, and some countermeasure is considered necessary.

22. Replacement of the weak layer with concrete was taken up as the countermeasure. Analyses by F.E.M examined the extent of the above replacement with concrete. The analyses indicated that the safety would be ensured by replacing the weak layer with concrete up to a depth of two times weak layer thickness (depth from the lower edge of the weak layer).

    The design of dam foundation treatment takes the above analysis result into consideration.

23. In the dam foundation treatment, provision of the curtain grouting and blanket grouting aiming at the water stoppage in the foundation is essential. These are designed based on the grout test results as follows:-

    (i) Curtain grout:
    - Two lanes, each having grout holes at 2 m interval
    - Depth of grout holes = 30 to 60 m

    (ii) Blanket grout:
    - Eight lanes in total (Four lanes each in upstream side and downstream side of the grout curtain)
    - Grout holes arrangement (For each of upstream and downstream sides of the curtain)
      * One lane with grout hole depth of 10 m at 2 m interval
      * Three lanes with grout hole depth of 5 m at 3 m interval

24. As mentioned, it is investigated that a part where a low groundwater level is indicated exists in the left bank of the dam site, requiring a countermeasure.

    The design considers that rim groutings along the left bank ridge will be most efficient, and arranges the rim grout holes with the rim grout hole depth of about 60 m (from a grout tunnel to be provided nearly at the dam crest level) at 3 m interval.

    On the actual execution of the rim grouting, the split spacing method will be applied.

The method will find places which require dense rim groutings, omitting some rim groutings in places requiring less groutings.

25. With regard to seepage through the dam and its foundation and relatively thin left bank, examinations on the seepage amount and possibility of the piping have been made through seepage analyses by F.E.M.

As a result of the above analyses, it is revealed that conditions necessary to cause no piping are satisfied and that the leakage amount is also within the standard allowable range.

Details are given in the Design Report.

26. Aiming at the dam safety control, various measuring apparatuses are installed. Those are pore pressure meters, earth pressure meters, settlement measuring apparatuses, surface displacement measuring apparatuses, leakage measuring apparatuses, reservoir water level measuring apparatuses, reference points for sediment monitoring,etc.

The measurement data are sent and displayed in the dam control house with an electrical system.

27. The designs of dam and foundation treatment, etc. are as seen in Fig.3 to 10.

Design of Spillway

28. The Probable Maximum Flood (P.M.F) of 1,890 m^3/sec is applied as the spillway design flood. Although the Basic Design of the spillway to safely handle the above spillway design flood was prepared through hydraulic analyses for the spillway and energy dissipator, the spillway hydraulic model tests were carried out for the purpose of confirmation of the Basic Design or adjustments of the design if required in view of importance of the structure.

29. Detailed results of the hydraulic model tests are given in the Spillway Hydraulic Model Test Report. Major items pointed out in the tests are as follows:-

(l) Necessary modification of the shape in both sides of the entrance.
Contracted flows occur under the Basic Design, reducing the overflow capacity,

and therefore, training walls are required to be provided in both sides of the entrance of spillway.

(2) Modification of overflow crest length:-

The overflow capacity does not meet the requirement even if the above improvement in (1) is made. To meet the requirement, extension of the overflow crest length by 2.0 m is required (from 90 m to 92 m).

(3) Improvement in transition portion:

The transition end sill height of about 4.0 m is desirable from the aspect of the favourable Froude Number. However, the sill height has to be lowered down to 3.0 m in order to keep a complete overflow condition over the overflow weir.

The location of the sill should be shifted to downstream of the bend in the transition to make energy reduction effect by the sill more effective. Besides that, a contracted flow arises at the bend due to shortage of transition length, and thereby, the flow condition in the chuteway is not favourable. Therefore, the length of the transition should be increased as well as providing more smooth curve at the bend.

(4) Depth in the energy dissipater (Height of end sill in the energy dissipator)

Trial tests for various heights of the end sill in the energy dissipator indicated that the best height would be 8.0 m: that is, the most desirable jump occurs in this height of the sill for the 100-year probable flood of 1,040 m^3/sec (the design flood for the energy dissipator).

However, the effect of energy dissipation for the P.M.F of 1,890 m^3/sec is not sufficient.

Although the above is considered to cause no problems in the downstream reaches from the aspect of probability, the side walls of the energy dissipator will be overtopped, which may cause troubles on the dam, requiring some heightening of the side walls.

(5) Provision of chute blocks

Provision of chute blocks near the entrance of the energy dissipator will effectively stabilize the jump and lessen fluctuations of water level in the energy dissipator.

The tests found that provision of six (6) chute blocks, each having 2.0 m in height and 2.5 m in width, would be most effective.

(6) Height of side walls in chuteway

The tests confirmed that 8.0 m in height should be provided for the side walls in the chuteway so that P.M.F (1,890 m^3/sec) can safely be discharged with a free board of 1.5 m.

30. The spillway and energy dissipator are designed on the basis of the mentioned test results. Fig.11 shows the Basic Design prepared through hydraulic analyses. Fig.12 shows the design improved based on the hydraulic model tests.

31. The structural design of the spillway and energy dissipator considers to secure its stability with anchor bars and dead loads, aiming at reducing the excavation volume. Further, the design considers to reduce the acting loads by providing sufficient drainage systems in back sides of the structures.

32. All conceivable loading conditions such as the normal loading condition, seismic loading condition and flooding condition are taken into consideration in the stability analyses against overturning, sliding and bearing, etc., and then necessary safety factors are given in the structures.

33. The structural analyses are made for each of the overflow weir, side channel portion, transition portion, chuteway, energy dissipator and slabs, of which details are presented in the Design Report.

Design of Spillway Bridge

34. An open channel of the spillway exists in the left abutment. The dam crest road has to cross this open channel, requiring a bridge to cross the open channel.

35. This bridge is provided with a span of 29.0 m and a width of 7.2 m (6.0 m in effective width) and is designed as the second grade bridge.

36. The reason why the bridge is designed as the second grade bridge is as follows:-

There is a consideration that the dam crest road may be used as a public traffic road. Even in the above case, the road will not be a trunk road but a branch road for which its bridge is designed as the second grade bridge.

37. The bridge is designed as a composite girder in which both the girder and concrete slab bear the acting loads in view that the composite girder will make its construction works easier and will be more advantageous economically in such a case as the long span (29 m) and the large height below the girder (about 20 m).

Design of Intake Facilities

38. The project plans to utilize the diversion tunnel for the water supply facilities. Hence, the water intake facilities are located on the left bank near the inlet of the diversion tunnel.

39. From an economical point of view, the intake structures will be constructed on the slope of the left bank as an inclined structure.

40. The project aims at supplying the water to the Port Louis City, and is expected to always supply water with a high quality. Therefore, the multi levels water intake is planned to be constructed so that water with a high quality can always be taken: that is, the intakes will be provided at three (3) levels of EL.169.0 m, EL.154.0 m and EL.140.0 m.

41. The project also considers to install a power generation facility in future by utilizing the water to be supplied to the Port Louis City. Thus, the design of the intake should take this future power installation into consideration.

The capacity of 1.0 m^3/sec will meet the requirement for the water supply. However, the peak discharge for power generation is assessed at 9.0 m^3/sec, and therefore, 9.0 m^3/sec is taken as the design discharge for the intake structure.

42. Based on the above design discharge, dimensions of 3 m x 3 m are provided for the screen at the entrance of intake in accordance with a standard which mentions the flow velocity at the screen should be limited within 1.0 m/sec to reduce the head loss.

Dimensions of 2.1 m x 2.4 m are provided for the inclined intake shaft with the following considerations:- that is, the water taken from the intake will be led to the tunnel through the inclined intake shaft and then, flow in a different direction through the steel pipe. Since the flow condition is rather complicated as mentioned, the flow velocity is desirable to be limited to not more than 2.0 m/sec.

43. Diameter of the steel pipe with which the water will be led to a valve located at the downstream of the tunnel main plug is provided with 1.5m which is determined as the economical diameter which minimises the sum of headloss and cost.

44. The capacity given to the valve at the downstream end of the tunnel main plug is 1.0 m^3/sec which meets the requirement of water supply. In the future power installation, this valve will be removed and the water is to be led to the power generating equipment through a penstock.

45. Various mechanical equipment for the water supply such as the trash racks (screen), intake gates, gate hoists, steel pipe and valves, etc. will be installed.

The design for these equipment takes the future dam expansion in mind into consideration.

46. The design of the water supply facilities is as seen in Fig.15 to 18.

Design of River Outlet Facilities

47. The river outlet facilities having the purpose to lower the reservoir water level at emergency will also be installed in the diversion tunnel.

48. The facilities are composed of the inlet tower, steel pipe with 1.5 m in diameter to be embedded in the tunnel main plug, and the valve to release the water located at the downstream end of the tunnel main plug.

49. Capacity of the river outlet facilities is determined with consideration that the reservoir should be made empty within one week.

This capacity given to the river outlet facilities is considered reasonable, referring to

examples of capacities provided in other similar projects.

50. The design of the river outlet facilities also take the future dam expansion in mind into consideration.

Design of the Tunnel Plug

51. The diversion tunnel is required to be closed after completion of the dam. This closure of diversion tunnel is made by placing the plug concrete in the tunnel.

In the case of the project, the tunnel concrete plug also has the purpose to install the said water supply and river outlet facilities.

52. The tunnel concrete plug is provided with a length of 30m in accordance with the minimum requirement for a complete water stoppage (0.3 to 0.5 of maximum water pressure), although the length of 10 to 15 m is possible to withstand the acting pressure.

53. The plug concreting work should be done as early as possible after the diversion gate closure. Thus, the concrete placing will be made for a short period, and therefore, the concrete temperature tends to become high when hardening. In the case that a large difference between the maximum temperature rise and the final stable temperature is caused, the plug concrete may be subject to shrinkage cracks, requiring the pipe cooling to restrict the concrete temperature rise when hardening. The pipe cooling is required to be made so that the difference between the maximum temperature rise and the final stable temperature be limited to 20°C. Hence, cooling pipes are arranged on each lift. The pipe cooling will be continued during the plug concreting.

54. Contact between the lining concrete and the plug concrete may be insufficient in the upper portion of the tunnel, requiring the pressure grout to fill the voids.

The design also makes arrangement of the above filling grout pipes.

Preparation of Tender Documents

55. The Tender Documents require (i) Instructions to tenderers, (ii) Condition of contract, (ii) General and technical specifications, (iv) Various forms for tendering, bill of

quantities, breakdowns, bonds and agreements, etc.,and (v) Tender drawings, etc.

56. Then, the Tender Documents are composed of followings:-

Vol. I :	Instruction to Tenderers Forms for Bonds and Agreement Condition of Contract
Vol.II:	General Specification Technical Specification
Vol.III :	Form of Tender Bill of Quantities Schedules of Particulars
Vol. IV :	Tender Drawings

57. The Tender Drawings are prepared with following considerations:

(1) The Condition of Contract is based on the FIDIC (International Condition of Contract for Civil Engineering Works).

There are two methods to prepare the Condition of Contract: that is, (i) In part-I, FIDIC is presented as it is, and in Part-II, necessary revisions, additions or elimination, etc. are made, and (ii) necessary revisions, additions or eliminations are all incorporated into the FIDIC without any division into Part-I and Part-II.

In the case of the Project, the latter is adopted in consideration that the latter will be much more convenient for the readers.

(2) It is considered important that the specification corresponding to each work item in the Bill of Quantities can easily be verified. Therefore, the clause of specification corresponding to each work item is shown in the Bill of Quantities.

(3) A special attention is paid for letting the tenderers submit schedules of particulars as much as possible, which will greatly serve for evaluating the tenders or for dealing with claims from the contractor in future.

The schedules of particulars to be submitted by the tenderers include the following:-

- Cash Flow Tabulation
- Labour Flow Tabulation, including Estimate of Mauritian Labour Employment
- Material Flow Tabulation
- List of Construction Plant and Materials, including Their Sources
- Field Personnel
- List of Sub-Contractor(s)
- Daywork
- Breakdown of Prices
- Construction Time Schedule
- Drawings and Documents submitted with Tender
- Information and Qualification for Works of Hydro-mechanical Equipment
- List of Spare Parts of Construction Plant

(4) The names of the consultant and financing agency, etc. which are not determined yet, are left blank.
Those should be filled before tendering.

(5) Constructions of the diversion tunnel and various preparatory works will be executed under Lot-I and furnished to Lot-II contractor.
The diversion gate will be manufactured under Lot-I. However, its closing work has to be done under Lot-II.

The cofferdams, access road from the outlet of diversion tunnel to the spillway and access road to be connected to the dam crest in the right bank, which are forced to be done under Lot-II, are included in Lot-II.

The tender and contract documents are prepared under the above conditions.

Construction Cost Estimate

58. The construction cost at price level in January 1991 is estimated on the basis of results of the detailed design and analyses on unit prices.

59. Division between the foreign currency and local currency portions is made as follows:-

- In principle, the local labour cost, inland transportation cost, etc. belong to the local currency portion

- The indirect foreign cost, which is defined as the cost of imported raw materials and depreciation cost of imported equipment for the production of local construction materials, etc., is included in the foreign currency portion.

- All the construction equipment and plants are considered to be imported by the contractor. Thus, most of the equipment cost is composed of the foreign currency portion. A part of the repair and management cost,which is mainly the local labour cost, is considered to be the local currency portion included in the equipment cost.

Construction Schedule

60. The construction schedule is shown in Fig.19 and 20.

61. The total construction period including Lot-I, II and III will be 47 months after completion of financing and contract procedures.

    The construction period of Lot-II will be 34 months after completion of the contract procedures.

TABLES

Table 1 (1) : PRINCIPAL FEATURES OF THE PROJECT

(1)	Reservoir	
	Catchment area	54.9 km^2
	Annual baisn rainfall	2,400 mm
	Gross storage capacity	6.7 x 10^6 m^3
	Effective storage capacity	6.3 x 10^6 m^3
	Flood water level	El.193.5 m
	High water level	El.189 m
	Low water level	El.139 m
	Surface area	30 ha
	Mean runoff	1.8 m^3/s
	Design flood	1,890 m^3/s
	Return period	(PMF)
(2)	Dam	
	Type	Rockfill
	Crest elevation	El.196 m
	Height	84 m
	Crest length	250 m
	Embankment volume	1,548 x $10^3 m^3$
(3)	Spillway	
	Type	Side channel
	Crest elevation of weir	El.189 m
	Width of weir	92 m
	Discharge	1,890 m^3/s
(4)	River Diversion	
	Type	Tunnel diversion
	Design flood	520 m^3/s
	Return period	(20 years)
	Discharge in tunnel	520 m^3/s
	Number of tunnel	1
	Diameter	6.8 m
	Tunnel length	499 m
	Gate type	Sluice gate

Table 1 (2) : PRINCIPAL FEATURES OF THE PROJECT

(5) Intake

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

(6) New Transmission Pipeline

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

(7) New Treatment Plant

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

Table 2 : DESIGN VALUES OF EMBANKMENT MATERIALS

Item		Earth	Filter		Rock
			Fine	Coarse	
Specific gravity		2.88			
(Oven dry condition)			2.87	2.87	2.87
Natural moisture content	(%)	40.0			
Water absorption	(%)		2.0	2.0	2.0
Dry density	(tf/m^3)	1.23	1.90	2.00	2.10
Wet density	(tf/m^3)	1.72	1.94	2.04	2.14
Saturated density	(tf/m^3)	1.80			
Submerged density	(tf/m^3)	0.80	1.23	1.30	1.37
Coefficient of permeability	(cm/sec)	$1x10^{-5}$	$1x10^{-3}$	$1x10^{-2}$	$1x10^{-1}$
Strength parameter (effective stress analysis)					
Cohesion c'	(tf/m^2)	0.0	0.0	0.0	0.0
Internal friction angle	(degree)	30.0	36.0	38.0	40.0

Table 3 : MIX PROPORTION OF CONCRETE SPECIFIED

Class of cement	Maximum grain size of aggregate (mm)	Slump (cm)	Air content (%)	Water cement ratio W/C (%)	Fine aggregate percentage s/a (%)	Unit Quantity (cubic meter)							
						Water W (kg)	Cement C (kg)	Fine aggregate S (kg)	Coarse aggregate 80~40 (kg)	Coarse aggregate 40~20 (kg)	Coarse aggregate 20~10 (kg)	Coarse aggregate 10~5 (kg)	Admixture (ml)
A	20	14 ~ 18	6.0	60	55	207	345	1028	0	0	496	291	863
B	20	12 ~ 14	6.0	65	56	200	308	1078	0	0	500	294	770
C	40	12 ~ 14	4.5	55	44	177	322	891	0	503	335	209	805
D	40	8 ~ 10	4.5	65	46	170	262	968	0	504	336	210	655
E	80	8 ~ 10	3.5	65	38	143	220	857	434	434	289	158	550
F	40	8 ~ 10	4.5	75	45	170	227	962	0	521	239	217	568

Specific Gravity of Fine Aggregate = 3.00

Specific Gravity of Coarse Aggregate (Max. 20mm) = 2.81

Specific Gravity of Coarse Aggregate (Max. 40mm) = 2.77

Specific Gravity of Coarse Aggregate (Max. 80mm) = 2.82

Specific Gravity of Cement = 3.13