CHAPTER 6 PLAN FORMULATION

6.1 Introduction

The Tana river has the largest flow among all the rivers in Kenya. The river flow and available water head have been harnessed for hydropower generation to date. Hydropower plants have been constructed at five locations along the river, the first being in service since 1968. The river profile of the Tana river are respectively shown in Figure 6.1.1. The Mutonga and Grand Falls dam sites have been identified as promising sites because of the affluent river flow and suitable topographical and geological conditions. The Project will meet the growing power demand as discussed in Section 5.2, and supply clean, stable and economical power.

The Project aims not only at attaining conventional benefits of hydropower generation and water supply of irrigation and municipal water, but also contributing to the environmental improvement in the downstream river corridor by regulating the river flow.

The field investigations as well as the preliminary study on Mutonga and Grand Falls Hydropower were carried out in the Kiambere Feasibility Study (1980), the Kenyan National Power Development Plan (KNPDP) in 1986 and the updated KNPDP in 1990. Four options are envisaged as follows:

1) Mutonga Dam (MT)
2) Low Grand Falls Dam (Low GF)
3) Low Grand Falls and Mutonga Dams (Low GF + MT)
4) High Grand Falls Development (High GF)

The difference of 98 m in river bed elevation between the tail water level of TWL 542 m at the Kiambere dam and EL 444 m at the proposed GF dam site can be utilised for hydropower generation. The High GF option intends to utilise this 98 m water head by constructing one large dam, while the Low GF and MT options envisage the creation of one or two middle scale dam(s). Depending on the development scale, the reservoir area and volume vary as shown in Figure 6.1.2.

The optimum development scheme of the Project is defined to maximize the net benefit and the internal rate of return obtained from the four schemes of the MT, Low GF, Low GF and MT, and High GF.

The studies of the Plan Formulation search for the optimum development scheme and alternatives of the Project and the optimal installation timing of the selected scheme with four steps as follows:

- The first step is to carry out the reservoir operation by establishing a simulation model consisting of five existing reservoirs and proposed reservoirs. The reservoir operation is simulated by the established reservoir operation policy to find out optimal operation rule of the existing and proposed reservoirs which produces the maximum power benefit.

- The second step is to find out the optimal development scheme of the Project among four schemes of the MT, Low GF, Low GF and MT, and High GF in consideration of power benefit, additional benefit obtained from irrigation and municipal water supply and artificial flood release. The artificial flood release which is one of the major function of the Project decreases the power and irrigation benefit, however, improve environmental assets. The negative benefit derived from the artificial flood release is gauged by the economic cash flow.
- The third step is to find out the optimum layout of the selected scheme at the second step by comparing alternatives of dam site and type taking into account the topographic and geological conditions at the site.
- The fourth step seeks the optimum installed capacity of the selected scheme.

6.2 Reservoir Operation

6.2.1 Simulation Model

A simulation model was established to find out the optimum reservoir operation rule of the MT, Low GF and High GF dams, taking into account the existing reservoir of the Masinga, Kamburu, Gitaru, Kindaruma and Kiambere dams. The model was developed with "EXTEND", a software for simulation, product by IMAGINE THAT Inc.

The upstream river basin as shown Figure 3.2.2 is modelled into Figure 6.2.1. The modelled basin is coded in the computer software. The structural view of the model is shown in Figure 6.2.2. River flow series at every sub-basin and intervening basin are estimated in Section 3.2, and water abstractions for irrigation supply and for municipal water supply are forecast in Section 5.3 and 5.4. The flow series and water abstractions at every sub-basin are incorporated in the hydrological model. The hydrological simulation is carried out by using the remaining flow series after abstracting the water supplies in the upstream area.

The major hydropower facilities on the Tana river are listed in Table 6.2.1. Five major hydropower stations have been in service: Masinga in 1981, Kamburu in 1974, Gitaru in 1978, Kindaruma in 1968 and Kiambere in 1988. One of the important factors to determine the project scale and reservoir scale is the reservoir regulation ratio, which is the ratio of the reservoir live storage volume to annual inflow volume. The ratio of the reservoirs on the Tana river is listed as follows:

Items	Masinga	Kamburu	Gitaru	Kindaruma	Kiambere	Mutonga	Low G/F	High G/F
Inflow (m^3/s)	87.37	107.52	107.76	107.96	111.18	145.45	160.67	160.67
Annual Inf.Vol.(10^6m^3)	2,755	3,391	3,398	3,405	3,506	4,587	5,067	5,067
Live Resv.Vol.(10^6m^3)	1,410	135	12.5	7.5	485	82.5	955	3,616
Regulation Ratio	0.512	0.040	0.004	0.002	0.138	0.018	0.185	0.714

The above table indicates that Masinga dam has a large river flow regulation capability. Because the dam is located in the upper most reaches of the Tana River, it governs power generation and reservoir operation at the downstream.

The model includes hydroelectric power stations on the Tana river. The power stations are divided into two types in the simulation model, run-of-river type and dam reservoir type. The stations with a reservoir regulation ratio of less than 5 % are regarded as run-of-river stations, since they can not regulate the river flow seasonally.

The stations are classified as follows:

- Run-of-river type stations : Kamburu, Gitaru, Kindaruma, Mutonga
- Dam reservoir type stations : Masinga, Kiambere, (Low/High) Grand Falls

Each power station has three inputs and six outputs. Inputs are composed of 1) inflow water discharge, 2) operation rule curve number, and 3) minimum plant discharge. Output consist of 1) reservoir water Level, 2) plant discharge, 3) spill out discharge, 4) power output, 5) energy output and 6) evaporation loss. For run-of-river type power stations, operation rule curve, minimum plant discharge and evaporation loss are omitted.

Each inflow receives the amount of the upstream plant discharge, spill-out discharge and water from the remaining catchment area. The water discharge from the remaining catchment area is deducted in advance according to the municipal and irrigation water requirement.

6.2.2 Conditions and Assumptions for Simulation Model

Reservoir operation is simulated from the upper reservoir to the lower one by applying the following policy on a daily basis:

(1) Water Balance in Reservoir

Water balance in a reservoir is computed applying the following equation:

$$S_i = S_{i-1} + I_i - O_i - EV_i$$

where, S_i : Reservoir water volume in the current day
S_{i-1} : Reservoir water volume in the previous day
I_i : Inflow into the reservoir in the current day
O_i : Outflow from the reservoir in the current day
Ev_i : Evaporation from the reservoir in the current day

The simulation flow is shown in Figures 6.2.3 and 6.2.4. Evaporation loss is deducted from a reservoir in proportion to the reservoir area. The inflow is assumed as same as that in the previous day since inflow into the reservoir in the current step is not known in the actual operation. Outflow from the reservoir is plant discharge plus spill-out discharge. Spill out discharge is calculated when the reservoir water level exceeds the full supply level.

(2) Plant Discharge

Five alternatives of reservoir operation rule curves are introduced to each reservoir-type power station as shown in Figures 6.2.5 and 6.2.6 and defined as follows:

- Rule curve 1 aims at retaining the reservoir water level at full supply level (FSL) as far as inflow exceeds the defined firm discharge.
- Rule curves 2 through 5 aim at lowering the reservoir water level to certain levels below FSL at the end of the dry season (beginning of March and October), and retaining the reservoir water level at FSL in the wet seasons.
- Firm discharge, which is a minimum discharge through the turbine, is introduced. This is to prevent continuation of very low power output and to guarantee the minimum power generation, even when inflow discharge is small in drought times. Firm power can be generated with this discharge until the reservoir water level finally reaches the minimum operation level (MOL).

The above rule curves define a target reservoir water level at the beginning of every month. Reservoir operation shall follow the monthly rule curves cyclically every year as far as the reservoir inflow discharge is smaller than the plant dischargeable flow and is larger than the defined firm discharge.

Reservoir inflow water shall be stored in the reservoir when the inflow is more than the plant dischargeable flow. It raises the reservoir water level over the rule curve. Excessive water shall spill out when the reservoir water level exceeds the FSL. Stored water in a reservoir shall be released when the inflow is less than the defined firm discharge. It lowers the reservoir water level below the rule curve. The plant discharge shall be as same as the inflow when reservoir water level reaches MOL. The plant dischargeable flow depends on the reservoir water level and can be expressed as follows:

$$Q = Q_{max} \times (h_e/h_r)^{0.5}$$

where, Q : Plant discharge (m^3/s)
Q_{max} : Plant maximum discharge at rated water head (m^3/s)
h_e : Effective water head (m) = Static water head - Head loss
h_r : Rated water head (m)

(3) Power Output

Power output (kW) and energy output (kWh) can be computed by the following equation:

$$P = g \times Q \times h_e \times E_t \times E_h$$

where, P : Power output (kW)
g : Gravity acceleration (9.8 m/s^2)
Q : Plant discharge (m^3/s)
h_e : Effective water head (m) = Static water head - Head loss
E_t : Turbine and generator combined efficiency
E_h : Turbine efficiency due to water head variation

$$E = P \times t$$

where, E : Energy output (kWh)
t : Generating time (hr.)

(4) Rule for Run-of-river Type Power Station

The existing Kamburu, Gitaru, Kindaruma and Mutonga power plants are simulated as run-of-river power station since the regulation ratio of the reservoirs is less than 5 % and they can not regulate the river flow seasonally. The plant discharge shall be as same as the inflow discharge up to the plant dischargeable flow. The reservoir water level is retained at FSL all the time in the simulation. Evaporation loss is omitted because of the small reservoir surface area.

(5) Evaluation on Operation Rules

The power benefit of hydropower generation can be evaluated by using the firm power output and the average energy output. The annualized power benefit is estimated by using unit prices of 121.4 $/kW for power output and 0.0705 $/kWh for energy output. The optimum operation rule is the one which maximises the power benefit.

(6) Outline of Project Options

The reservoir surface area and storage volume of the existing dams and proposed dams are shown in Figures 6.2.7 to 6.2.9. The Mutonga and Grand Falls dams are will be located at the downstream point of the Mutonga river and the Kathita river.

The following principal features of the proposed dams are adopted for the simulation:

Items	Mutonga	Low GF	High GF
MOL (m)	538.5	491.4	518.2
FSL (m)	550.0	512.0	550.0
Installed capacity (MW)	60.0	140.0	200.0
Unit nos.	2	2	2

It is noted that the above MOL was referred to the result of study on the reservoir water level in sub-section of 6.4.1. The optimization study was carried out by setting the certain water level as a first step to seek the optimum development scheme. Following determination of the MOL in sub-section 6.4.1, the optimization study was re-examined by using the above MOL.

6.2.3 Optimal Reservoir Operation

Reservoir operation is simulated on a daily basis by using 34-year river flow from 1957 to 1990. The estimated daily river flow of 12,410 days is generated in Chapter 3.

The optimal rule is examined step by step, firstly with an analysis of the existing dams located upstream of the planned dams, secondly with an analysis of the planned dams, and finally with an analysis for the whole reservoirs in the Tana river.

(1) Examination for Existing Dams

Masinga + Run-of-river Stations

Three run-of-river power stations, Kamburu, Gitaru and Kindaruma, are entirely controlled by the Masinga reservoir operation rule because those run-of-river power stations are located downstream of the Masinga dam and have too small reservoir capacity to regulate the river flow. The Masinga dam operation rule is evaluated from the simulation results of 4 power stations.

Firm discharges ranging from 35 to 60 m^3/s are applied for Rule Curve 1 through 5. The simulation is conducted for 30 cases (= 6 cases in firm discharge x 5 rule curves). Two evaluation indexes of annual average energy and firm power are shown in Figure 6.2.10. This figure indicates that the annual average energy increases following the increase of the firm discharge, while the firm output decreases when the firm discharge exceeds 40 to 55 m^3/s. In addition, Figure 6.2.10 also shows that the firm discharge of 50 m^3/sec gives the larger annual energy and firm output under the rule curve 3. Accordingly, the power benefit produced by the the Masinga is maximized under the operation rule 3 with a firm discharge of 50 m^3/s.

Kiambere

Firm discharges ranging from 55 to 80 m^3/s are applied for the rule curves 1 to 3. The same rule curves are assumed at Masinga and Kiambere. The simulation results are shown in Figure 6.2.11. This figure indicates that the annual average energy gradually increases in a range of 55 to 70 m^3/s under rule curves 2 and 3, while the firm output decreases when the firm discharge exceeds 60 m^3/s. Accordingly, it can be said that the power benefit produced by the Kiambere station could give higher value with a firm discharge of 60 m^3/s.

All existing dams

Simulations are carried out for the following combinations of rule curves, using the selected firms discharge of 50 m^3/s for the Masinga dam and 60m^3/s for the Kiambere dam.

Dams	Rule Curve Number											
Masinga	1	1	1	2	2	2	3	3	3	4	4	4
Kiambere	1	2	3	1	2	3	1	2	3	1	2	3

The results are shown in Figure 6.2.12 with annual average energy and firm power of the all existing power stations on the Tana river from Masinga to Kiambere. As seen in Figure 6.2.12, combinations of rule curves 3 the Masinga and rule curves 2 for the Kiambere introduce larger annual average energy and firm output than those of other rule curves. The power benefit produced by the existing stations as a group will be maximum under the operation rule applying the rule curve No. 3 for the Masinga dam and the rule curve No. 2 for the Kiambere dam.

(2) Examination for Planned Dams

The same simulations are carried out for the proposed downstream dams, Mutonga, Low Grand Falls and High Grand Falls dams.

Mutonga

The Mutonga dam is regarded as a run-of-river type dam. Firm discharge varying from 60 to 85 m^3/s and the rule curve 1 only are applied. The simulation results are shown in Figure 6.2.13. This figure indicates that the firm discharge of 65 m^3/s will yield the maximum power benefit since though the annual average energy does not largely vary in a range of 60 to 70 m^3/s, the firm discharge of 65 m^3/s introduces the large firm output.

Low Grand Falls

Firm discharge ranging from 65 to 95 m^3/s are applied for Rule Curves 1 through 3. The simulation results are shown in Figure 6.2.14. This indicated that the firm discharge of 75 m^3/s gives large firm power and annual average energy. Accordingly, the firm discharge is set at 75 m^3/s as the optimum one.

High Grand Falls

Firm discharges varying from 60 to 100 m^3/s are applied for rule curves 1 to 3. The simulation results are shown in Figure 6.2.15. As seen in Figure 6.2.15, the optimum firm discharge is selected as 70 m^3/s since though annual average energy is not changed in a range of 70 to 85 m^3/s, the large firm output is introduced by 70 m^3/s.

(3) Optimum Rule of Reservoir Operation

Simulations are carried out for the following cases in order to determine the final optimal rule curve for the combined operation of the existing and planned dams altogether. It is found that the power output of the MT, Low GF + MT and High GF are so similar that Low GF operation is studied as a representative.

Dams	Rule Curve Nos.	Firm Discharge
Masinga	3 and 4	Qf = 50 m^3/s
Kiambere	1, 2, 3	Qf = 60 m^3/s
Low Grand Falls	1, 2, 3	Qf = 75 m^3/s
Combined cases	18 cases	

The total average energy output and firm output of all stations from the Masinga to Grand Falls dams are shown in Figure 6.2.16. The figure indicates that the rule curve 3 for the Masinga station and the rule curve 2 for the downstream Kiambere and Low Grand Falls can generate the most beneficial power output. The Masinga is the heading reservoir with a large regulation capability, therefore it plays a principal role in regulating flows into the downstream dams and enables them to operate with higher reservoir water levels.

6.3 Optimum Development Scheme

6.3.1 Optimum Development Scheme with Power Output

To find out the optimum development scheme of the Project, the economic comparison of the alternative schemes among the Mutonga (MT), Low Grand Falls (Low GF), Low Grand Falls + Mutonga (Low GF + MT) and High Grand Falls (High GF) is carried out by gauging benefits and costs in terms of conventional economic effects including power output, irrigation, municipal water supply and flood control. This section deals with the economic comparion of power output.

(1) Preliminary Design

(A) Dams

The center core rockfill dam type is adopted for the comparative study of the optimum development scheme. The Kiambere Feasibility Report (1980, EPDC) and KNPDP (1986, Acres) identified the dam axes at the proposed dam sites. Alternative dam axes which are located near downstream of the identified axes in the above reports are considered in this study in consideration of dam abutment and dam crest length. As a the result of comparison of reservoir volume and dam volume including embankment and excavation, the previous axes of MT, Low GF and High GF are selected because of less construction cost and more superiority of specific reservoir volume produced by the unit embankment volume. The layout plans of the selected dam axes are shown in Figures 6.3.1 to 6.3.3. Figure 6.3.4 shows the typical section of rockfill dam.

The geological investigation including boring and seismic test confirmed that the assumed excavation depth is appropriate in general. The dam excavation volume and embankment volume are computed through data processing. The embankment volume of the MT, Low GF and High GF is estimated at 810,000 m^3, 5,560,000 m^3 and 22,320,000 m^3, respectively. In addition, the excavation volume of the MT, Low GF and High GF is estimated at 194,000 m^3, 860,000 m^3 and 2,058,000 m^3.

Safety of the proposed dams is examined for 10,900 m^3/s and 12,800 m^3/s of 10,000-year flood at the Mutonga dam and the Low GF and High GF dams. In addition, design floods of diversion tunnels are estimated at 2,400 m^3/s for the Mutonga and 2,800 m^3/s for the Low GF and High GF as 50-year flood. Main dam, spillway, cofferdam and diversion tunnel are designed to be safe against the above floods.

(B) Hydraulic Structure

Mutonga Dam

The spillway is designed to be located on the left bank about 200 m from dam body where the excavation volume will be minimised. The chuteway length and width will be 300 m and 81 m respectively. The diversion tunnel is designed to be located on the dam left bank where the tunnel length can be minimised. Two tunnels with a length of 460 m and a diameter of 10.5 m are required.

Two waterway tunnels with a length of 370 m and a diameter of 6.3 m will be aligned on the right bank. The original ground surface along at the end portion of tunnels becomes so thin that open-air steel conduits will be aligned. Two penstocks with a diameter of 4.8 m will be connected to the power house. An open-air power house is designed to be located beside the river.

Low Grand Falls Dam

The spillway is designed to be located on the left bank to minimise the cost since no suitable abutment for spillway is found on the dam right bank. The chuteway length and width are 200 m and 90 m. Two diversion tunnels will be located on the right bank. The tunnel length and diameter are respectively 650 m and 10.5 m.

Two waterway tunnels with a length of 460 m and a diameter of 5.0 m are aligned on the left bank. Surge chamber will not be required because of the short distance of the tunnel. The power house is designed to be of open-air type beside the Tana river.

High Grand Falls Dam

The dam is located at the same axis of the Low Grand Falls dam, except the river portion. A spillway will be provided on the left bank in consideration of the topographic condition. The chuteway length and width are respectively 520 m and 90 m. Two diversion tunnels with a length of 620 m and a diameter of 10.5 m are aligned on the left bank.

Two waterway tunnels with a length of 370 m and a diameter of 4.0 m are aligned on the left bank. Surge chambers will not be required. An underground power house is designed to be constructed below the spillway structure and a tailrace tunnel will drain power water to the river.

(2) Construction Cost

Construction work quantities are estimated on the basis of the above preliminary design, layout plan, structure profile and additional cross-sections.

Unit prices for civil works and electrical/mechanical works in this study are estimated according to the following criteria:

- Updating the unit price in KNPDP.
- Referring to similar Projects in Kenya such as Kiambere Project, Sondu/Miriu Project and Magwagwa Project.
- Referring to similar international hydropower projects.

The construction cost of MT, Low GF and High GF is summarized in Table 6.3.1.

(3) Conversion to Economic Cost

The construction costs are converted to economic costs by the following procedure:

- The costs in foreign currency are used as economic costs without any adjustment, because international prices are used.
- For the local currency portion:
    - Firstly, the internal transfer costs are excluded from material and equipment costs,
    - Secondly, the economic price of cement is estimated taking into account a shadow price equivalent to 38% of the financial price,

- Thirdly, the non-tradable part is converted to international prices using a standard conversion factor of 92%,
- Forthly, economic labour costs are estimated by applying a shadow wage rate of 85% and 60% to the financial cost of skilled and unskilled labour respectively.

The construction cost is assumed to be composed of the following components:

Items	Foreign currency	Local currency	Total
Material	0.031	0.262	0.293
Equipment	0.339	0.034	0.373
Engineering	0.050	-	0.050
Labour			
Skilled	-	0.142	0.142
Unskilled	-	0.142	0.142
Total	0.420	0.580	1.000

(Note) The share of cement is around 40 % of material cost in the local currency portion, which means that the share of cement cost is 0.105.

Equipment and materials such as turbine-generator, heavy construction machines and steel products are assumed to be procured in principle from abroad, sharing over 34 % of the total cost. Materials such as embankment rocks, concrete aggregates, cement and so on as well as labour are presumed to be available at the local market, accounting for 26 % of material cost and 28 % of labour cost. Engineering services for detailed design and construction supervision are assumed to be undertaken by a foreign consultant. As a result, a factor for the conversion from the construction cost to the economic cost is calculated at 0.89.

The construction cost and converted economic cost are summarized in Table 6.3.1. The economic construction cost of the MT, Low GF, Low GF + MT and High GF are US$ 187.7 million, 362.5 million, 550.2 million and 673.4 million, respectively.

(4) Construction Period and Disbursement Schedule

The critical path of construction work will be on the diversion tunnel construction, river diversion, dam embankment and impounding. Construction schedule of each scheme is shown in Figures 6.3.5 to 6.3.8. Construction periods of MT, Low GF, Low GF and MT and High GF are 4 years, 5 years, 9 years and 7.5 years, respectively. Based on the above construction schedule, construction cost of each scheme is disbursed.

(5) Power and Energy Benefit

Alternative thermal plant is applied for evaluation of the economic feasibility of hydropower plants. It is assumed that hydropower is replaced with an alternative thermal power source to meet power demand. The proposed hydropower stations will share firstly the peak power portion and secondly the intermediate power portion in rich water years. Gas turbine and low-speed diesel thermal power is selected as alternative source. Power value, energy value and O/M cost are quoted from KNPDP in 1992 as follows:

Items	Gas Turbine	LS Diesel	Average	O/M Cost
Power Value ($/kW)	593	1,034	814	16.0
Energy Value ($/kWh)	0.0879	0.0492	0.0686	

The energy value is estimated on the basis of the crude oil price of US$ 17/bbl which is referred to "Commodity Price Forecasts, First Quarter 1996" issued by the World Bank and a plant factor of 70 %. The following losses are assumed in the comparison between hydropower and thermal power:

(Unit : %)

Items	Hydropower	Thermal power
(for power value and energy vale)		
Power loss through transmission line	5.0	2.0
Power consumption at power station	0.3	6.0
(for power value only)		
Forced operation outage	0.5	5.0
Maintenance outage	2.0	8.0

The above losses show that hydropower generation is more advantageous. The advantage is converted into a "compensation factor" of 1.147 for power value and of 1.028 for energy value. Assuming a discount rate of 12 % and the thermal power plant life of 25 years, the capital recovery factor is computed as follows:

$$\text{Capital recovery factor} = (1 + i)^n \times i / ((1 + i)^n - 1) \times 100 = 12.75\ \%$$

Annualised power and equivalent energy values are thus estimated as follows:

Items	Soured Value	Compensation Factor	Capital Recovery F.	Equivalent Annualised Value
Power Value ($/kW)	830	1.147	0.1275	121.4
Energy Value ($/kWh)	0.0686	1.028	-	0.0705

(6) Economic Comparison with Power Output

The replacement cost of electrical equipment is scheduled to be incurred 30 years after commissioning. The operation and maintenance cost is assumed to be 0.5 % of the civil works, the metal works and the transmission line and substation plus 4 to 8 $/kW of installed capacity. The economic cash flow is prepared assuming a 12 % discount rate. Economic indices are shown in Table 6.3.1, and summarised below:

Items	Unit	Mutonga	Low GF	Low GF + Mutonga	High GF
Installed Capacity	MW	60	140	200	200
Firm Output	MW	58	134	192	197
Annual Average Energy	Gwh/year	337	715	1,052	1,108
Economic Project Cost	x 10^6 US$	187.7	362.5	550.2	673.4
Annual Benefit	x 10^6 US$/year	30.8	66.7	97.5	98.5
Present Worth of Cost	x 10^6 US$	161.9	297.8	387.9	514.1
Present Worth of Benefit	x 10^6 US$	182.1	351.8	455.6	414.4
Benefit/Cost		1.12	1.18	1.17	0.81
Net Beneit	x 10^6 US$	20.2	54.0	67.7	-99.7
Internal Rate of Return	%	13.4	13.8	13.8	10.1
Unit Generation Cost	¢/kWh	8.1	7.9	7.9	11.0

Remarks: Unit generation cost is obtained by dividing present worth of cost by present worth of average energy with discount rate of 12 % for 50 years in the year of commissioning power generation.

The economic indices of net benefit and internal rate of return (IRR) indicate that the Low GF + MT scheme is the most economical development scheme. The study on the development alternative scheme reveals that the Project be developed with the Low GF and Mutonga scheme, that is, the Low GF dam is firstly constructed, followed by the MT dam.

In addition to the Low GF + Mutonga sequence, the Mutonga + Low GF sequence is examined to clarify economical construction sequence of the Project. The economic comparison of the Mutonga + Low GF reveals that the net benefit and IRR are obtained at US$ 48.8 million and 13.4 %. The unit construction cost of the Mutonga + Low GF sequence is gauged at 8.1 ¢/kWh. The examination of the economical construction sequence also results that the Low GF + Mutonga sequence is more economical that the Mutonga + Low GF one since the Low GF + Mutonga one maximizes the net benefit and IRR as seen in the above table.

On the other hand, there is a possibilty that the raising scheme of the Low Grand Falls dam introduces economical construction cost. The raising scheme of the Low GF dam is examined to confirm the economical development scheme of the Project. According to the above table, the raising cost of the Low GF dam is estimated at US$ 310.9 million (US$ 673.4 million - US$ 362.5 million). On the condition that the dam raising is started in 10 years after constructing the Low GF dam and the raising period is four years, the economic study of the raising Low GF is carried out to obtain the net benefit and IRR. As a result, the net benefit and IRR of the raising scheme are calculated at US$ 36.6 million and 13.2 %, which are lower than those of Low GF + Mutonga scheme. Accordingly, the Low GF and Mutonga scheme is further clarified to be the optimum development scheme of the Project.

(7) Further Economic Comparison in Consideration of Runoff from 1947 to 1995

The hydrological study in Section 3.2 presents the long-term daily discharge which is generated by using the Tank Model in the period of 34 years from 1957 to 1990. On the basis of these daily runoff, the comparative study of the alternative development scheme is carried out by the reservoir operation study and the economic study to find out the optimum development scheme in the above sub-section.

The long-term daily runoff from 1957 to 1990 in Section 3.2 is further extended by adding the runoff from 1947 to 1956 and from 1991 to 1995 at MT dam site and Low GF and High GF dam site. The extension of the long-term daily runoff is made on the basis of the monthly runoff in the gauged subbasins which are given by KPC in July 1997. Runoff in the ungauged subbasins are estimated as follows:

- Runoff from 1947 to 1956 and from 1991 to 1995 in ungauged subbasin of MASI-RL is estimated by multiplying runoff in the gauged subbasin of 4CB4 + 4CA2 by ratio of catchment area between MASI-RL and 4CB4 + 4CA2 and ratio of specific runoff of MASL-RL and 4CB4 + 4CA2.
- Runoff in six downstream ungauged subbasins of KAMB-RL, GITA-RL, KIND-RL, MUTO-R, KIAM-RL and GRAF-RL is estimated by multiplying runoff of each neighboring upstream ungauged subbasin by ratio of catchment area and specific runoff of each on the basis of calculated runoff in MASI-RL.
- Runoff of MUTO-L is calculated by the above method with using runoff in the gauged subbasin of 4EA7.
- Runoff at the Mutonga and Low GF/High GF dam site is obtained by summing up runoff in the gauged and ungauged subbasins.

The estimated monthly runoff at each dam site from 1947 to 1995 is shown in Table 6.3.2 and Table 6.3.3.

The econmic comparison of alternative schemes is further executed by using the extended runoff from 1947 to 1995. The result of further comparative study is summaried below:

Items	Runoff of 34 years from 1957 to 1990				Runoff of 49 years from 1947 to 1995			
	Mutonga	Low GF	Low GF + Mutonga	High GF	Mutonga	Low GF	Low GF + Mutonga	High GF
Installed Capacity (MW)	60	140	200	200	60	140	200	200
Firm Output (MW)	58	134	192	197	58	135	193	197
Annual average energy (GWh/yr)	337	715	1,052	1,108	340	710	1,050	1,103
Net Benefit (Mill. US$)	20.2	54.0	67.7	-99.7	21.4	52.7	67.2	-103.0
IRR (%)	13.4	13.8	13.8	10.1	13.5	13.8	13.8	10.1

The further comparative study reveals that:

- Even though the runoff duration is further extended to 49 years from 1947 to 1995, the firm power outputs of alternative schemes are not changed from those obtained from daily runoff of 34 years from 1957 to 1990 since the large reservoirs of the Masinga, Kiambere and Low GF contribute to regulate the long-term runoff.
- Annual average energy of each scheme estimated by runoff of 49 years from 1947 to 1995 is not also changed from that calculated by runoff of 34 year from 1957 to 1990 by contribution of regulated outflow from the Masinga, Kiambere and Low GF.
- Net benefit and IRR of the alternative schemes estimated by using runoff of 49 years are almost similar to those estimated by runoff of 34 years. Accordingly, it is confirmed that the optimum development scheme of the Project is the construction sequence of the Low GF and Mutonga as selected in the above sub-section.

6.3.2 Optimum Develpment scheme with Additional Irrigation Effect

(1) General

Irrigation can be developed to some extent without the Project by using the discharged flow from the existing dams. The increment irrigable area constitutes the irrigation benefit of the Project, which will be created by improvement of river flow by the proposed dams. The increment irrigation benefit is studied in this section for comparison of options.

The reservoir operation of the proposed dams was simulated to maximize the power benefit in the previous sections, by making the outflow from the dams as stable as possible so as to ensure stable electric power supply. However, irrigation water demand differs from power demand month by month according to the cropping pattern. In this section, the reservoir operation is simulated to meet the irrigation water demand to maximize irrigation benefit, which decrease the power benefit on the contrary. The Mutonga dam does not have a seasonal river flow function because of its small reservoir capacity. The options examined in this section are, therefore, the Low GF + MT and the High GF.

The Project will be completed in 2008 for the Low Grand Falls dam and in 2010 for the High Grand Falls dam at the earliest. Irrigation schemes will be developed by use of the regulated flow by the existing dams for the time being. The irrigation benefits by the proposed dams will be created in far future. Therefore, the reservoir operation of the proposed dams is simulated on the basis of the water abstraction level in 2020. It is assumed that the construction of irrigation facilities for the incremental irrigable area will start immediately after completion of the proposed dams, and irrigation benefit will be produced 3 years after the construction commencement. The incremental irrigation benefit would reach the maximum value 8 years after the construction commencement and the benefit would decrease thereafter at an annual rate of 0.45 % because of water abstraction at the upstream area.

(2) Area Irrigated by the Project

The maximum incremental irrigation area is calculated for each alternative option in order to evaluate the irrigation benefit along the lower Tana river. In general, irrigation potential is assessed by evaluating suitability of three major factors, i.e. Land, Water and Climate. Of the three factors, Land and Climate were evaluated in the National Water Master Plan (MOWD/JICA) using various data such as soil, topography, climate and land control. In this Study, those evaluation results are applied for assessment of the irrigation potential along the lower Tana river.

<u>Land resources</u>

The National Water Master Plan selected potential lands for agricultural production considering 1) land gradient, 2) soil conditions and 3) temperature. After selecting potential lands, some areas such as major urban area, forest, park, national reserve, major road and remote area from the river, were deducted. Figures 6.3.9 and 6.3.10 show the spatial distribution of lands suitable for upland crops and lowland crops respectively, cited from the Master Plan results.

Water release from the dam

Agricultural lands in the downstream area will be irrigated by using the discharged water from the dams. The following assumptions are made to establish monthly water releasing pattern from the dam which maximizes the downstream irrigable area with the least water release amount from the proposed dam.

i) No storage facilities are considered in the downstream area.
ii) Surplus water is allocated to downstream area.
iii) Return flows are disregarded.
iv) Domestic & livestock water requirement is not taken into account.
v) For the area downstream of Mnazini town, the Type I cropping pattern is applied. (The typical cropping pattern used in MOWD's study was applied for estimation of water requirements, after discussion with TARDA's Agronomist.)
vi) For the area upstream of Mnazini town, the Type II cropping pattern is applied.

The river flow at downstream points is estimated based on the discharge at 4F13. The points are just upstream and downstream of each sub-basin as illustrated in Figure 6.3.11. River flow discharges at 4F13 gauging station are converted to each point assuming that the downstream flow regime may change at the same ratio as the naturalised flow estimated in the Master Plan study. The following releasing patterns for the downstream irrigation area are assumed:

Unit : m^3/s

Pattern	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Total
A	34	19	132	115	165	34	11	109	48	155	98	68	988
B	35	20	133	116	166	35	12	110	49	156	99	69	1,000
C	36	21	134	117	167	36	13	111	50	157	100	70	1,012
D	37	22	135	118	168	37	14	112	51	158	101	71	1,024
E	**38**	**23**	**136**	**119**	**169**	**38**	**15**	**113**	**52**	**159**	**102**	**72**	**1,036**
F	39	24	137	120	170	39	16	114	53	160	103	73	1,048
G	40	25	138	121	171	40	17	115	54	161	104	74	1,060
H	41	26	139	122	172	41	18	116	55	162	105	75	1,072
I	42	27	140	123	173	42	19	117	56	163	106	76	1,084
J	43	28	141	124	174	43	20	118	57	164	107	77	1,096

The water release from the proposed dam more than pattern E will saturate the downstream potential irrigable area. The pattern E is adopted for the simulation of irrigation operation.

Dam operation simulation

The river flow at the Grand Falls dam site (gauging station No. 4F13) is simulated in the cases of "without Project" and reservoir operation under irrigation purpose. Reservoir operation under irrigation purpose to meet the water releasing pattern E in the above Table is simulated in this section. The water demand in the upstream area in 2020 is abstracted in all cases. The simulation cases are defined as follows:

- Irrigation purpose simulation for the Low GF
- Irrigation purpose simulation for the High GF

The simulation results of water outflow from the proposed dam(s) are listed in Tables 6.3.4 to 6.3.6.

Irrigable area

The irrigation area is determined by the released water (m^3) divided by water requirements (m^3/ha). Irrigable area is computed at every sub-basin of 4GA to 4GG on monthly basis from 1957 to 1990. The results of calculation for irrigable area are summarised in the Table below:

(Unit : 1,000 ha)

"Without Project"	Low GF	High GF
83	102	124

(3) Benefits and Costs Estimate

The irrigation benefit of the each is estimated by comparing the "without-Project" and "with-Project" conditions, assuming that all the irrigable area is developed as small pumping irrigation schemes. The project benefit and cost are estimated by multiplying the unit cost and benefit by the incremental irrigation area.

The unit cost and benefit of small pumping irrigation scheme are estimated based on the district development plan and information provided by MOALD. According to this information, there are 2,433 ha of pumping irrigation area along the Tana river and administratively belonging to Garissa and Tana River Districts as shown in Table 6.3.7. The unit cost and benefit are calculated by dividing the total cost and benefit by total area (2,433 ha).

Cost

The construction cost for pumping irrigation schemes was estimated by summation of two kinds of cost: cost of pumps and cost of infrastructure. The cost of pumps varies according to the pump size. The required size and capacity (HP) for each scheme are calculated as shown in Table 6.3.8 applying the equations described in Table 6.3.9. The following assumptions are made for the calculation:

1) The size and number of pumps are limited by the recommended maximum area for the groups which can work together at 20 - 25 ha/group.
2) Areas larger than 20-25 ha should have at least two pump units, including a stand-by unit. Four kinds of pumping systems are considered for cost estimation, taking account the proposed scheme size and market availability:

Type A	5 HP	Ksh 75,000	(Suppliers quotations + 25% allowance)
Type B	8 HP	Ksh 100,000	
Type C	16 HP	Ksh 187,500	
Type D	20 HP	Ksh 225,000	

A total of 144 pump units would be required, costing Ksh. 21,012,500 (Garissa) and Ksh. 7,462,500 (Tana River). The total cost, including infrastructure development cost of Ksh. 20,000 per ha, is Ksh. 56,580,500 (Garissa) and Ksh. 20,562,500 (Tana river) totaling Ksh. 77,143,000. The unit cost per ha is about Ksh. 32,000/ha in 1995 price level. This unit cost is calculated at Ksh 39,000/ha by a escalation rate of 5% for two years from 1995 to 1997.

Benefit

Most of the pumping irrigation areas, particularly those in Garissa, are mainly cultivated with vegetables and bananas. The Garissa Irrigation Project Report (funded by Danida) indicates that farmers use half of the land for cultivating bananas (staple crop) and the other half for horticulture, normally a mixture of tomatoes, melon, chillies, green maize is also becoming common.

For calculation of benefit of each scheme, it was assumed that half of the area is cultivated with bananas and the other half with tomatoes representing horticultural crops. According to the farm budget survey report[1], the average gross benefit for tomatoes is Ksh. 88,960/ha and Ksh. 23,480/ha for bananas. The average gross benefit is assumed to be Ksh. 56,000/ha in 1995 price level. The average gross benefit is calculated at Ksh 68,000/ha by a escalation rate of 10% for two years from 1995 to 1997.

Summary of Cost and Benefit

Conclusively, the incremental irrigation cost and benefit are calculated and summarised bellow:

Alternative	Increment Area (1,000 ha)	Benefits expected (million Ksh/US$)	Capital Investment (million Ksh/US$)
Low GF	19.0	1,292/25.8	741/14.8
High GF	41.0	2,788/55.8	1,599/32.0

Note : Exchange rate, 1US$=50 Ksh.

(4) Economic Evaluation

Economic evaluation is carried out for both the cases of reservoir operation aimed at power benefit and irrigation benefit. The results are summarised below:

Items	Low GF + Mutonga	High GF
Firm Power (MW)	48	32
Average Energy (Gwh/y)	912	968
Annual Power Benefit (10^6 US$)	70.1	72.1
Annual Irrigation Benefit (10^6 US$)	25.8	55.8
Net Benefit (10^6 US$)	4.5	-88.7
Internal Rate of Return (%)	12.1	10.6

The result of comparative analysis demonstrate that the Low Grand Falls + Mutonga option is economically superior to the High Grand Falls option.

[1] Ministry of Agriculture Irrigation and Drainage Branch : Profitability of Small Holder Irrigation in Kenya, 1990.

6.3.3 Optimum Develpment Scheme with Additional Water Supply Effect

As discussed in the section of 5.4, the upstream water use for municipal water up to the point of the proposed dams is abstracted in the simulation model. The water demand at the downstream districts are estimated in the National Water Master Plan, which are quoted as follows:

(Unit : m^3/s)	Rural	Urban	Livestock	Industrial	Total
River Course					
Tana River	0.05	0.08	0.09	0.01	0.23
Garissa	0.01	0.25	0.15	0.01	0.42
Lamu	0.02	0.08	0.04	0.01	0.15
					Sub Total 0.80
Water Transfer					
Mombassa	0.00	1.57	0.00	0.77	2.34
Kilifi	0.27	0.47	0.05	0.09	0.88
					Sub Total 3.22
Total	0.35	2.45	0.33	0.89	4.02

Water demand in the districts along the downstream river course is negligible (0.8 m^3/s). Water demand even including Mombassa and Kilifi, to which water transfer facilities are required, will be 4.02 m^3/s only. The National Water Master Plan recommends that the municipal water to Mombassa and Coastal areas will be supplied by developing 1) Second Mzima scheme and 2) Sabaki Extension scheme. Water supply to the downstream districts may be abstracted from the Tana river without appreciable impact on the selection of the alternative options because of the small abstraction amount in comparison with available water.

The available water to the downstream area will be decreased due to water abstraction in the upstream area. The firm discharge was obtained at 75 m^3/s for the Low Grand Falls dam and 70 m^3/s for the High Grand Falls dam as explained in sub-section 6.2.3. Meanwhile, the minimum daily river flow discharge without the proposed dam will be 5.6 m^3/s under 2020-year water abstraction. Therefore, the discharge of 69.4 m^3/s (75 - 5.6) and 64.4 m^3/s (70 - 5.6) is water supply benefit by Low and High Grand Falls dams, respectively. The project cost without the cost for power generation facility is as follows:

Unit: Million US$

Structures	LGF	HGF
Preparatory Work	19.2	21.3
Diversion Tunnel	39.6	30.0
Main Dam and Coffer Dam	82.2	289.0
Spillway	36.7	101.7
Hydro-mechanical Works	40.4	37.9
Sub Total	218.1	479.9
Engineering Services and Administration	21.8	48.00
Land Acquisition and Compensation	15.2	31.0
Physical Contingency	25.0	62.3
Total Cost	280.1	621.2

The above construction cost is converted to economic cost by multiplying the ratio of 0.89. The economic cost is disbursed during the construction period of 5 years for Low Grand Falls and 7 years for High Grand Falls and discounted at the annual rate of 12 %. The cost is annualized by multiplying capital recovery factor of 0.1204 on an assumption of 50 year economic life. The seepage and evaporation loss ratio of 20 % is assumed. The calculation results are summarized below:

Items	Unit	LGF	HGF
Project construction cost	10^6 US$	280.1	621.2
Project economic cost	10^6 US$	249.3	552.9
Discounted cost	10^6 US$	197.2	410.7
Annualized cost	10^6 US$	23.7	49.4
Utilized water volume	10^6 m^3/yr	1,751	1,625
Water unit cost	US¢/m^3	1.35	3.04

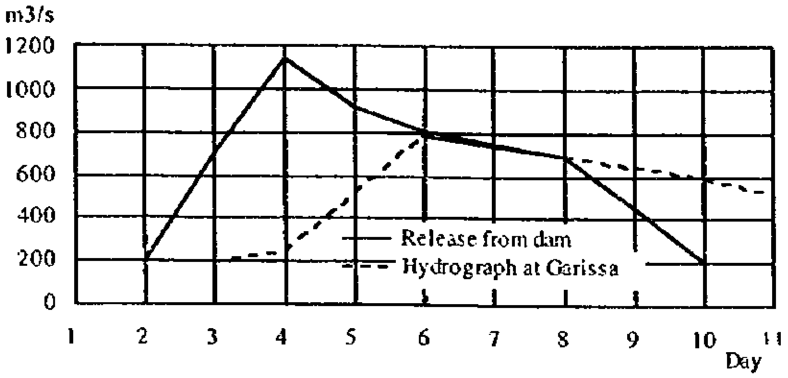
As seen in the above table, the unit water cost of HGF is higher than that of LGF.

6.3.4 Artificial Flood Release

The environmental assessment study of the downstream river morphology in Section 4.4 reveals that the existing environmental situation would be improved by artificial flood release from the proposed Low Grand Falls dam. Garissa is selected as the target point to evaluate the normal flood which will contribute to normal inundation at the downstream. As a result of hydrological study at Garissa, the normal flood is identified as two floods every year of more than 500 m^3/sec. Hydrological analysis of the correlation between the proposed Low Grand Falls dam site and Garissa introduces the required flood releasing pattern from the proposed dam. The hydrograph of the normal flood is set as the median flood having a period of 6.5 days and a peak discharge of 785 m^3/sec and 1,100 m^3/sec at Garrisa and the proposed Low Grand Falls dam site, and a total volume of 394 million m^3 and 490 million m^3 at the Low Grand Falls site and Garissa. The required hydrograph of the normal flood is show below:

(Unit : m^3/s)

Day	Release from Low G/F	Q at Garissa
0	(Power discharge)	(Power discharge)
1	200	(Power discharge)
2	707	200
3	1,146	250
4	921	529
5	807	785
6	750	739
7	689	689
8	450	646
9	200	600
10	200	520



It is simulated that the proposed dam are operated to release artificial floods. The reservoir operation rule is established so that an artificial flood is released at the beginning of the flood seasons every year, namely, the artificial flood starts on 1 April and 1 November. For power generation, the same reservoir operation policy and operation rule are applied as discussed in Section 6.2.

The simulation results of the Low Grand Falls dam with artificial flood function is shown in Figures 6.3.16 and 6.3.17. The simulation proves that the Low Grand Falls dam can provide two artificial floods every year. The simulation results are summarized below.

(Unit : m^3/s)

Items	Low Grand Falls	High Grand Falls
Average	157.2	152.9
Monthly Maximum	1,608.1	1,340.7
Monthly Minimum	75.0	70.0
Daily Maximum	2,237.2	2,091.1
Daily Minimum	69.2	70.0

Artificial flood operation shall decrease hydropower output. The economic cash flow in consideration of the artificial flood release is computed with the discount factor of 12 %. The result of the economic cash flow is summarized below:

Items	Unit	Low GF + Mutonga	High Grand Falls
Firm Power	MW	136.4	178.4
Annual Energy	GWh/yr	870.9	962.3
Annual Power Benefit	x 10^6 \$	78.0	89.5
Present Worth of Cost	x 10^6 \$	387.9	514.1
Present Worth of Benefit	x 10^6 \$	352.2	376.7
Benefit/Cost		0.91	0.73
Benefit - Cost	x 10^6 \$	-35.7	-137.4
Internal Rate of Return	%	11.0	9.4

The artificial flood release function decreases the economic viability of the Low GF + Mutonga and the High GF schemes. The negative net benefits are led by the artificial flood release to the Low GF + MT and High GF schemes with the discount rate of 12 % as seen in the above table. However, the artificial flood release mitigates the environmental degradation on the downstream environmental condition of the Tana river, and leads to the positive impact on the downstream environment. Though this positive impact is not measured by value because of its intangible benefits, the artificial flood release could contribute to an increase of the benefit of the Project.

In addition, the environmental assessment study in Section 4.4 recommended that the optimum reservoir operation was required to be established to aim at maximizing power output while maintaining enough excess storage capacity to allow for normal flood release. The study recommended that the actual power cost would be minimized and could be compensated by the integrated management of the combined power generating capacity of the seven dam system. The firm power output and annual average energy of the Project will be increased by the optimum reservoir operation. As a result, the benefits produced by the Project will be increased. Accordingly, EIRR which was obtained at 11.0 % at the condition of the peak discharge of 1,100 m^3/sec with total volume of 490 million m^3 is conceived as the minimum value as well as conservative value. When the proper management of reservoir operation is established, EIRR will be increased.

Taking into account the environmental improvement of the downstream morphology of the Tana river with the artificial flood release from the Low GF and MT dams, it could be said that the Low GF + MT scheme still has economic viability. In addition, the above result shows that the Low GF and MT scheme is superior to the High GF scheme.

6.3.5 Reservoir Water Level and Spill Outflow

The simulations for the economic comaprsion are carried out in this study, of which cases are 12 as shown below:

Items	Low GF + Mutonga			High Grand Falls		
Flow Regime in 1995	Power Operation	Irrigation Operation	Flood Operation	Power Operation	Irrigation Operation	Flood Operation
Flow Regime in 2020	Power Operation	Irrigation Operation	Flood Operation	Power Operation	Irrigation Operation	Flood Operation

The results of reservoir water level and total outflow in case of flow regime in 1995 are shown in Figure 6.3.12 to 6.3.23.

Reservoir Water Level

It is concluded most beneficial for power benefit to regulate the river flow regulation by the head reservoir of the Masinga dam in the power system on the Tana river. The proposed power station is located at the immediate downstream of the dam. Accordingly, power generation operation is made to retain reservoir water level as high as possible. The optimum reservoir scale is designed so that the reservoir water level recovers to the FSL in rainy seasons. If the reservoir water level does not reach to the FSL in rainy season, the reservoir scale may be over scaled. The following Table shows number of years out of 34 years when the reservoir water level does not recover to FSL under 1995 hydrological regime.

Items	Low Grand Falls	High Grand Falls
Power Operation	0	7
Irrigation Operation	4	11
Flood Operation	11	12

Once the reservoir water level of High GF dam goes down, the reservoir will not be filled with water quickly, which indicates that the dam does not function effectively and economically. In the case of flood operation, the frequencies of unfilled condition of the reservoir of LGF and HGF are similar. Flood requirement at the downstream is so large that even medium size reservoir of LGF will not be filled with water every year.

Spill Outflow

The ratio of average spill outflow to average inflow is shown as follows:

Items	Low Grand Falls	High Grand Falls
Power Operation	14.3	11.7
Irrigation Operation	14.0	11.4
Flood Operation	24.4	22.7

There is not an appreciable difference of the ratio between the LGF and HGF even though the reservoir volume is significantly different. The medium scaled reservoir of the LGF dam can utilized the river flow effectively since the river flow is regulated by the existing upper reservoirs of Masinga and Kiambere.

6.3.6 Optimum Development Scheme

Economic comparisons of the alternative development schemes in terms of hydropower development are summarized below:

Items	Alternatives			
	Mutonga	Low GF	Low GF + Mutonga	High GF
Economic Cost (10^6 US$)	187.7	362.5	550.2	673.4
Installed Capacity (MW)	60	140	200	200
Firm Power (MW)	58	134	192	197
Average Energy (GWh/yr)	337	715	1,052	1,108
Net Benefit (10^6 US$)	20.2	54.0	67.7	-99.7
Internal Rate of Return (%)	13.4	13.8	13.8	10.1
Unit Generation Cost (US¢/kWh)	8.1	7.9	7.9	11.0

From the above table, the Low Grand Falls + Mutonga scheme can be selected as the optimum development scheme among four alternative schemes from the technical and economical viewpoints. The Low Grand Falls + Mutonga scheme with implementation of the Low Grand Falls dam followed by the Mutonga dam also appears to preferred scheme in terms of conventional effects attributable to the Project, comparing with the Mutonga and Low Grand Falls sequence which has the net benefit of US$ 48.8 million and EIRR of 13.4 %.

In addition, the economic comparison in terms of the additional effect derived from irrigation water supply in sub-section 6.3.2 confirmed that the Low Grand Falls + Mutonga scheme was superior to the High Grand Falls scheme.

Though the artificial flood release decrease the economic viability of the Low Grand Falls + Mutonga and the High Grand Falls schemes, the Low Grand Falls + Mutonga scheme could have economic viability in consideration of the improvement effect for the downstream environmental condition.

Accordingly, it can be concluded the Low Grand Falls + Mutonga scheme is the optimum scheme from technical and economical viewpoints.

6.4 Development Alternatives

6.4.1 Reservoir Water Level

(1) Full Supply Level

KNPDP in 1987 concluded that the static water head between the Kiambere tail water level and the Grand Falls dam site would be fully utilized for hydropower development. This policy is adopted in this study. The riverbed elevation of the outlet structure of the Kiambere power station is 538.5 m. The Kiambere power station is designed to have the tail water level at the elevation of 550 m. The full supply level of the Mutonga reservoir is set at the elevation of 550 m.

At the downstream stretch about 2.8 km from the confluence of the Mutonga river, valley width becomes larger where a dam axis is not suitable. A dam axis for the Mutonga dam was proposed at the river stretch between 0.8 km and 2.1 km downstream river channel from the confluence. The dam axis located at the about 800 m downstream from the confluence of the Mutonga river was selected as the Mutonga dam site because the axis gives less construction cost. The tail water level of the Mutonga power station ranges between 507.3 m and 510.7 m, depending on power outflow discharge.

The Grand Falls dam sites were proposed at 4.1 km and 7.0 km downstream points from the confluence of the Kathita river. The full supply level of the Low Grand Falls reservoir is set on the elevation of 512.0 m so that the Mutonga power station may not be dammed up so much.

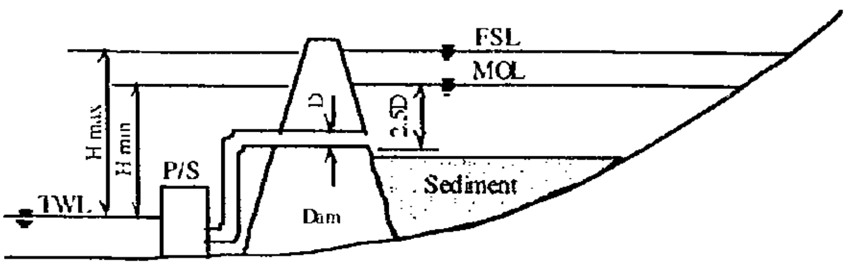
(2) Minimum Operation Level

The minimum operation level (MOL) is determined so as to satisfy the following conditions:

Condition 1: The sill of power-water intake structure is set on the designed sediment level. Sufficient clearance above the intake structure up to the MOL shall be provided to avoid entraining air into power water conduit. Therefore, MOL is elevated above the sediment level with the depth of about 2.5 times of conduit diameter.

Condition 2: MOL is lower than the simulated RWL (reservoir water level).

Condition 3: The ratio of the minimum water head to the maximum water head is preferably more than 70 % to avoid harmful effect to turbine. This ratio is applied for the Project.



To determine the sediment level, the criteria of 50-year sediment volume was applied for the Kiambere dam. The same criteria is applied for the Mutonga and the Low Grand Falls dam with sand flush facility. The designed sediment volume and the corresponding sediment level are computed by using annual sediment volume which is estimated in Section 3.2 as follows:

Dams	Volume ($\times 10^6$ m^3/year)	Designed sediment volume ($\times 10^6$ m^3)	Sediment level (m)
Mutonga dam	1.10	55	540.35
Low Grand Falls dam	1.40	70	477.79

The water level and dimensions of waterway and the above mentioned conditions of each dam are shown as follows:

Dam	FSL (EL m)	TWL (EL m)	Dia. (m)	Sediment Leve (m)	MOL Condition 1	Condition 2	Condition 3
Mutonga	550	510.6	4.7	540.4	552.1	538.9	538.2
Low Grand Falls	512	441.1	5.4	477.8	491.3	508.4	490.7

Through evaluating the above three conditions, the MOL of the Mutonga dam is set at EL. 538.5 m with an allowance to meet the condition 2. When sediment level reaches to EL. 526.8 m, sediment materials in the reservoir is flushed out through sand flushing facilities. The sediment volume below EL. 526.8 m is about 12.2 million m^3, which corresponds to about 10 year period. MOL of the Low GF is set up on EL. 491.4 m with the condition 1, taking into account the simulated minimum RWL in the condition 2 and the required volume of about 490 million m^3 for the artificial flood release.

6.4.2 Dam Alternatives

(1) Design of Dam

Type of Dam

The following dam types are applicable for the Mutonga dam site and the Low Grand Falls dam site in consideration of the geology, topography and available material quantity:

1) Rockfill dam type
2) Concrete dam type
3) Combined dam type
   (Concrete dam at the river section and rock fill dam at the right abutment)
4) Concrete facing dam type

Typical dam sections of alternative dam type are shown in Figure 6.4.1.

Available rock material at the Mutonga and the Low Grand Falls sites is gneiss or granite. The upstream and downstream slopes of the concrete facing dam are set on 1 : 1.4 in consideration of the properties of the rockfill materials. The downstream slope of the rockfill dam is designed to be 1 : 1.8 and the upstream slope at 1 : 2.0. Assuming the internal angle of rock material of 43° and the surface slope of 1 : 1.8, the safety factor of rock surface sliding is computed at 1.20 at the seismic state. For the concrete dam, the "basic triangular section" is applied for the concrete dam. The upstream surface is vertical and the downstream surface is inclined at 1 : 0.8. The fillet is provided at the upstream surface.

River Diversion

The peak discharge of the design flood for the diversion facility is set as follows:

(Unit : m^3/s)

Type of Dam	Return Priod	Mutonga	Grand Falls
Fill type dam	50-year flood	2,400	2,800
Concrete dam	5-year flood	1,600	1,800

Diversion tunnel and coffer dam are provided to divert the above flood during dam construction. The diameter of diversion tunnel diameter and height of cofferdam are determined on the basis of the above design flood.

Spillway

A flood of 200-year probability is adopted for the design of a concrete dam and spillway. A flood of 20 % increase of the 200-year flood is applied for a fill type dam. The safety of dams with a free-board of 1.5 m is further examined for 10,000-year probable flood. The peak discharge of the design flood for the spillway is as follows:

(Unit : m^3/s)

Type of Dam	Flood	Mutonga	Grand Falls
Fill type dam	200-year flood x 1.2	4,800	5,400
Concrete dam	200-year flood	4,000	4,500
Extreme flood	10,000-year flood	10,900	12,800

Waterway

The waterway consists of intake, headrace tunnel, power tunnel, surge tank and penstock. A surge tank is not required for concrete dam since power house is located just downstream from the dam and steel conduit is embedded in the concrete dam body. The tank is required for fill type dam since the headrace tunnel becomes relatively long and sudden water pressure variation is required to be absorbed by means of the surge tank. The diameter of conduit is preliminarily computed on assumption of the velocity of 3.0 m/s for headrace tunnel and the velocity of 5.0 m/s for penstock.

Powerhouse

A power house is located just downstream from the dam since the river longitudinal slope is so gentle that it is not economical to locate a powerhouse at far downstream stretch by power tunnel. An open-air type powerhouse is applied for the Mutonga and Low Grand Falls power stations. A power house is dimensioned so as to encase the turbine and generator equipment.

(2) Construction Cost

Mutonga Dam

The project costs of alternative dam types for the Mutonga dam are estimated in Table 6.4.1. The project cost of rockfill dam and concrete dams are estimated at 210.9 million and 208.8 million US$.

The Mutonga reservoir has so small capacity that the reservoir will be filled with sediment material. A sand flush facility is required to be provided at the dam structure to flush out the sediment above the sill level of the intake. A concrete dam can facilitate the sand flush function in the dam body. Rockfill dam or concrete surface dam would not effectively function sand flush operation. Concrete dam gives more economical cost than rockfill dam, as seen in the Table 6.4.1. The rock foundation is so sound that a small scale concrete dam can be constructed. Accordingly, the concrete dam is selected as the Mutonga dam. Figure 6.4.2 shows a plan of the concrete dam and the power station.

Low Grand Falls Dam

The dam axis of the Low GF was preliminarily selected at about 4.1 km downstream location from the confluence of the Kathita river, where three dam axis as shown in Figure 6.4.3 are suitable for dam construction from a topographical view point. The applicable dam types are concrete facing type, rockfill type and combined type. The combined dam has a roller compacted concrete dam (RCC) at the river section and rockfill dam at the remaining abutment section. The upstream dam axis has the longest crest length. Therefore, the concrete facing dam only is competitive on the upstream axis. The dam axis alignment is curved on the middle and downstream axis due to topographical condition. The rockfill dam and combined dam are conceivable on the middle and downstream axis.

The alternative dam axis and type of the Low GF dam are as follows:

Case-1 : Upstream dam axis, Concrete facing dam
Case-2 : Middle dam axis, Rockfill dam
Case-3 : Middle dam axis, Combined dam
Case-4 : Downstream dam axis, Rockfill dam
Case-5 : Downstream dam axis, Combined dam

Figure 6.4.4 to 6.4.8 show plan of alternative dam type and axis. The construction costs of the alternative cases are estimated in Table 6.4.2. The crest length of concrete facing dam is relatively so long as about 1,900 m. The concrete facing dam is fragile against water leakage through cracks on the surfaced concrete. The longer the dam crest is, the bigger the probability of the concrete cracks is. There is few examples of the concrete facing dam with so long dam crest.

Technical disadvantage on rockfill dam is on handling core material embankment. The rockfill dam requires core material of more than 850,000 m^3. The material is available in the reservoir area, however, it is deposited in thin layer of about 1 m depth on the bank slopes. The top layer shall be cleared and spoiled. Accordingly, the borrow area becomes wider and hauling distance becomes longer. Natural moisture content is so low as a few percent in dry season that strong sprinkle watering is required to attain the optimum moisture content of about 12 %. The soil layer has clayey soil and gravely soil, then blending work is required.

The combined dam on the downstream dam axis gives the lowest project cost as seen in Table 6.4.2. Accordingly, the combined dam at the downstream dam axis is selected as the Low GF dam axis and type. Mass concrete volume is estimated at 955,000 m^3. Roller compacted dam concrete is applied for dam body. The core embankment volume is far less than that of the above rockfill dam, or 333,000 m^3. The sand flushing facility is provided in the dam body as the measures against the downstream environmental effects.

6.5 Installed Capacity

6.5.1 Reservoir Operation

The optimum installed capacity of the Mutonga and the Low GF is studied in this section. The installed capacity is optimized in relation with power benefit and construction cost.

The Mutonga dam is regarded as a run-of-river type because of its small capacity of reservoir. The operation rule curve-1 and firm discharge of 60 m^3/s are applied for the Mutonga reservoir. On the other hand, the operation rule curve-2 and the daily firm discharge of 75 m^3/s are applied for the Low Grand Falls reservoir as beneficial ones. The alternative installed capacity and basic conditions of each dam are listed below:

Mutonga Power Station

Items	Unit	40 MW	60 MW	80 MW	100 MW
Qmax	m^3/s	114	174	240	316
TWL	m	510.1	510.6	511.3	512.0
Net Rated Head	m	39.5	39.0	37.5	35.8
Combined efficiency		0.90	0.90	0.90	0.90

Low Grand Falls Power Station

Items	Unit	100 MW	120 MW	140 MW	160 MW
Qmax	m^3/s	158	192	227	262
TWL	m	440.4	440.9	441.1	441.5
Net Rated Head	m	70.7	69.8	69.1	68.3
Combined efficiency		0.91	0.91	0.91	0.91

The tailwater levels in the above table at the Mutonga and the Low Grand Falls sites are shown in Figure 6.5.1, which is estimated by applying uniform flow formula on the surveyed river cross section.

Reservoir operation on the variation of the installed capacity is simulated on the same hydrological model in Section 6.2. Daily river discharge series from 1957 to 1990 are applied for the simulation. The result of the reservoir operation leads to the firm power and annual average energy of alternative installed capacity in the Mutonga and the Low GF as follows:

Mutonga

Items	Unit	40 MW	60 MW	80 MW	100 MW
Firm Power	MW	38.7	58.1	63.6	58.7
Annual Average Energy	GWh/yr	302.7	337.5	354.3	363.4

Low Grand Falls

	Unit	100 MW	120 MW	140 MW	160 MW
Firm Power	MW	94.4	114.3	135.0	133.6
Annual Average Energy	GWh/yr	664.9	688.2	714.9	727.3

6.5.2 Power Benefit and Construction Cost

Annual power benefit is obtained by multiplying firm power and annual average energy by annualized power benefit of 121.4 $/kW and energy benefits of 0.0705 $/kWh, respectively. Annual power benefits of alternative capacity for each scheme are summarized below:

Mutonga

Items	Unit	40 MW	60 MW	80 MW	100 MW
Powe Benefit	(Mill.US$)	4.7	7.1	7.7	7.1
Energy Benefit	(Mill.US$)	21.3	23.8	25.0	25.6
Annual Benefit	(Mill.US$)	26.0	30.9	32.7	32.7

Low Grand Falls

	Unit	100 MW	120 MW	140 MW	160 MW
Powe Benefit	(Mill.US$)	11.5	13.9	16.4	16.2
Energy Benefit	(Mill.US$)	46.9	48.5	50.4	51.3
Annual Benefit	(Mill.US$)	58.4	62.4	66.8	67.5

The construction costs of alternative installed capacity are shown in Table 6.5.1 to 6.5.2 for the Mutonga and Low Grand Falls dams. Project cost is summarized below:

Mutonga

Ulnit	Alternative Installed Capacity			
MW	40 MW	60 MW	80 MW	100 MW
Mill. US$	195.6	208.8	222.3	235.0

Low Grand Falls

MW	100 MW	120 MW	140 MW	160 MW
Mil. US$	375.0	383.9	392.3	400.8

6.5.3 Optimum Installed Capacity

To seek the optimum installed capacity of the Mutonga and Low GF, the economic comparison on the alternative installed capacity of each scheme is carried out. The result of economic comparison is summarized below:

Mutonga

Economic Index	Unit	40 MW	60 MW	80 MW	100 MW
Net Benefit	Mill. US$	5.2	20.4	20.1	9.9
IRR	%	12.39	13.37	13.27	12.60

Low Grand Falls

Economic Index	Unit	100 MW	120 MW	140 MW	160 MW
Net Benefit	Mill. US$	35.9	50.3	66.9	63.9
IRR	%	13.35	13.83	14.36	14.22

As a result of the econmic comparison, the optimum installed capacity is determined to be 60 MW for Mutonga and 140 MW for Low GF, which maximize net benefit and IRR.

In addition, the combination of the installed capacity for the Low Grand Falls + Mutonga are also studied, and summarized as follows:

Dams	Unit	Installed Capacity (MW)			
Mutonga	MW	40	60	60	80
Low Grand Falls	MW	100	120	140	140
Economic Index					
Net Benefit	Mill. US$	37.4	60.4	77.0	76.4
IRR	%	13.10	13.69	14.11	14.07

As seen in the above table, the combination of the Low GF with 140 MW + the MT with 60 MW gives the maximum net benefit and IRR. Accordingly, it is confirmed that the optimum installed capacities of the Low Grand Falls and Mutonga hydropower plants are 140 MW and 60 MW.

CHAPTER 7 PRELIMINARY DESIGN

7.1 Introduction

As the result of the plan formulation in Chapter 6, it is concluded that the preferred option of the Project consists of the Low Grand Falls and the Mutonga hydropower schemes. This Chapter describes the preliminary design of the preferred Project, by explaining the concept of the design and the principal features of the main structures, which results will be adopted for planning the time schedule and estimating cost of the Project.

Through the comparison studies to elect the preferred development of the Project, the following basic dimensions are worked out:

	Low Grand Falls Scheme	Mutonga Scheme
- Full Supply Level (FSL):	EL. 512.0 m	EL. 550.0 m
- Minimum Operation Level (MOL):	EL. 491.4 m	EL. 538.5 m
- Installed Capacity:	140 MW (70 MW x 2 units)	60 MW (30 MW x 2 units)
- Maximum Plant Discharge	227.6 m^3/sec	175.0 m^3/sec

7.2 Low Grand Falls Scheme

7.2.1 River Diversion

Diversion tunnel and cofferdams are provided to divert river flow during the construction works of the dam. Diversion tunnel is located at the left abutment of the dam, because it forms steep slope preferable in rock cover and geology for the construction of the tunnel. Layout of the diversion system including the tunnel and cofferdams is shown in DWG-03.

Diversion tunnel is designed to have two lanes to release the flood river flow of 50-year probable flood with a peak discharge of 2,800 m^3/sec, taking into considerations the type of the dam which is a combined dam of rockfill and concrete. The diameter of each tunnel is determined to be 10.5 m with the maximum velocity of 16.2 m/sec at the discharge of 2,800 m^3/sec, for preventing a damage due to hydraulic erosion.

An upstream cofferdam, about 26 m high rockfill type dam, is provided between the inlet of the diversion tunnel and the main dam. The crest elevation of the cofferdam is set at EL. 466 m with a freeboard of 1.0 m above the flood water level in the upstream of the dam at the time of releasing the peak flood through the diversion tunnel. A downstream cofferdam of rockfill type is also provided between the main dam and the outlet of the diversion tunnel.

7.2.2 Main Dam

(1) Dam Axis and Type

As the result of the comparison study of the location and type of the dam as described in Section 6.4, the main dam axis is located at the alternative downstream site and the type is combined dam consisting of a concrete dam at the riverbed section and a rockfill dam at the right abutment. The general plan of the main dam is seen in DWG-03.

(2) Dam Crest

The maximum level of the reservoir during the time of the regular (without flood) reservoir operation is the FSL at EL. 512.0 m. The crest elevation of the main dam is determined, taking into account of a freeboard for the FSL in the regular reservoir operation and a freeboard for water level at flood reservoir operation, whichever higher value, resulting in EL. 516.5 m as explained below.

Freeboard during the regular reservoir operation is calculated at 4.3 m, which is the sum of 2.3 m for wind wave runup (with a storm wind velocity of 30 m/sec and the fetch of 10 km), 0.5 m for earthquake wave runup and 1.5 m allowance for the rockfill type dam. Adding this freeboard to the FSL, EL. 512.0 m, dam crest level is assumed to be EL. 516.5 m

Freeboard during the flood reservoir operation was examined by adopting the result of the flood routing, which indicated that the maximum reservoir water level (FWL) at the 10,000-year probable flood with a peak inflow of 12,800 m^3/sec is EL. 513.0 m as shown in Figure 7.1. The flood is released by operating the spillway gates; when the flood discharge is less than the spillway capacity, the flood is released without any retention in the reservoir, namely inflow into the spillway equals outflow; and when the flood discharge is more than the spillway capacity, the flood is retained in the reservoir. A freeboard of 2.0 m, sum of 1.0 m for malfunction of gate and 1.0 m allowance for uncertainty at this study stage, was assumed, then the dam crest level is calculated at EL 515.0 m, which is lower than the dam crest level assumed for the regular reservoir operation.

(3) Concrete Dam

Section of Concrete Dam

The concrete dam at the river channel section is designed to have a vertical face for the upstream and an inclination of 1 to 0.8 for the downstream face. A fillet is provided at the low level section of the upstream face with an inclination of 1 to 0.2. The concrete dam has a crest length of 405 m and a height of 90 m. A gallery is constructed at the low level section in the dam body for providing drain and grouting holes at the upstream dam foundation as well as an access to a normal flood and sediment release facility.

A roller compacted concrete dam (RCC) type is adopted in consideration of its advantages in cost, construction sequence and construction period. The concrete volume of the dam is estimated at 1,175,000 m^3. Typical section is shown in DWG-04.

Spillway (overflow section)

A spillway is provided at the river channel section of the concrete dam to evacuate excess water including floods, which is able to discharge the 10,000-year probable flood with sufficient freeboard to the dam crest

The spillway has total width of 105 m including five 3 m wide piers and equipped with six numbers of radial gates, each having 15 m in width and 15.5 m in height, for regulating discharges. The spillway including an energy dissipater is shown in DWG-04.

Energy Dissipater

Taking into considerations relatively large height of the dam and economy, an energy dissipater of submerged bucket type is provided in the downstream end of the spillway to kill the excess energy of the outflow, for which a 100-year probable flood with a peak discharge of 4,600 m^3/sec is adopted as the design discharge. Downstream river water depth is sufficient during the discharge of the design flood to maintain the energy killing function with a formation of a hydraulic roller.

Normal Flood and Sediment Release Facility

As discussed in Section 4.4, downstream river reach requires a release of normal floods and sediments to maintain the existing conditions of environments and river morphology. A peak discharge of 1,100 m^3/s with total volume of 490 million m^3 is required to replicate the normal floods. A release facility is provided in the concrete dam section for this requirement, see DWG-05, which will be able to release the artificial flood from the dam by evacuating reservoir water between EL. 504.0 m and the minimum operation level (MOL) as the most severe condition of the reservoir water level. The facility is also able to release sediments which include sands and suspended loads associated with phosphate and organic matter. The artificial flood release operation will be carried out bi-annually at the beginning period of the rainy seasons (April and November), for the purpose to effectively release clay particles flowing into the reservoir at the beginning stage of the rainy season.

The release facility is located at the right side of the spillway and consist of two sets of each 5.0 m wide and 5.0 m high steel conduit, a high pressure roller gate for operation control and a stoplog gate in front of the roller gate. The entrance sill of the facility is set at EL. 469.0 m, which will contribute to catch a density current containing the suspended loads as anticipated by the analysis in Section 4.4.

(4) Rockfill Dam Section

A rockfill dam with center core is constructed at the right abutment, with the maximum height of 55 m and crest length of 1,025 m. The slope of the dam is 1 to 1.8 in the downstream face and 1 to 2.0 in the upstream face.

The center core has a section of slope 1 to 0.25 both up- and downstream sides. The results of the construction material survey, described in Section 3.4, showed good quality in permeability of less than 10^{-5} cm/sec and shearing strength with internal friction angle of 31 degree and the cohesion of 0.5 t/m^2. The volume of core materials will be 597,000 m^3.

The rock materials will be brought from quarry sites which are formed by granitic and semi-felsic gneiss at the right bank of the Tana river. The properties of rock materials showed good quality in surface-saturated specific gravity of 2.65, absorption of 1% less and shearing strength with internal friction angle of 41.0 degree. The volume of rockfill materials is estimated at 1.85 million m^3. The upstream and downstream surface of the zone will be protected by riprap. Both upstream and downstream face of the center core will be flanked by filter zones, the materials for which will be brought from the borrow areas in the riverbed along the tributaries at the right bank of the Tana river.

Typical section of the rockfill dam is shown in DWG-05.

(5) Foundation Treatment

In the concrete dam section, grouting will be carried out at the dam foundation as a foundation treatment, consisting of a curtain grouting for the ground seepage cutoff at the upstream toe of the dam with 3 m spacing and 30 m in depth and a consolidation grouting for ground strength enhancement at the full space of the dam foundation with 3 m spacing and 5 m, and further a drain relief along the gallery.

In the rockfill dam section, curtain and blanket groutings will be provided for ground seepage control at the bottom of the core section, consisting of one row of curtain grouting with 3 m spacing and 20 m in depth and a blanket with 3 m spacing grid and 5 m in depth.

7.2.3 Waterway

(1) Power Intake

A power intake with the dimensions of 9.5 m wide and 40.0 m high is provided at the upstream face of the concrete dam. The sill elevation of the intake invert is set at EL. 477.8 m in consideration of the formation of the 50-year sediment level.

Multistage gates are installed in front of the power intake as shown in DWG-06, aiming to effectively catch the suspended loads in a form of density current which will flow in the reservoir toward the dam.

A 3.9 m wide and 6.5 m high intake gate will be provided at the entrance of the penstock. A trashrack is installed in front of the gate to prevent floating materials from entraining into the penstock.

(2) Penstock

A penstock is embedded in the body of the concrete dam, which is 5.4 m in diameter and 90 m in length conveying water to the power station.

7.2.4 Power Station

The power station is located just downstream of the concrete dam. The power station is an open-air type, housing two units of vertical synchronous generators and Francis type hydraulic turbines, and accompanied with an outdoor switchyard located at the right abutment.

(1) Powerhouse

The powerhouse has dimensions of 32 m in width, 60 m in length and 50 m in height, with a basic arrangement shown in DWG-07.

The house consists of a machine room of 22 m wide and 36 m long, an erection bay of 22 m wide and 14 m long and a control room of 10 m wide and 60 m long. The main units have a space of 18.0 m between centers to provide an area for installation and arrangement of turbines and auxiliaries. The turbine center is set at EL. 436.5 m to secure the required draft head. Main transformers will be provided just behind the powerhouse from economic viewpoints.

(2) Outdoor switchyard

An outdoor switchyard is located at the right abutment neighboring the powerhouse, having an open area of 129 m in length and 50 m in width. The outdoor switchyard accommodates main switchgear and control equipment.

7.2.5 Permanent Access Road

A permanent access road to the Low GF dam site will be constructed from the existing Kiambere hydropower plant along the right bank of the Tana river, see DWG-02, with a distance of about 53 km and a width of 6.0 m. Along the road, nine concrete bridges will be constructed at crossing points of the tributaries which flow into the Tana river, consisting of seven submerged type concrete bridges and two concrete girder type bridges.

The permanent access road is branched off to access the Mutonga hydropower plant, with a branch distance of about 6.5 km.

7.2.6 Hydro-mechanical Works

Hydro-mechanical equipment are gates, trashracks and penstock, with main features listed below:

(1) Diversion Gate

Type	:	Roller gate
Number	:	Two sets
Dimensions	:	10.5 m wide x 10.5 m high

(2) Spillway Gate

Type	:	Radial gate
Number	:	Six sets
Dimensions	:	15.0 m wide x 15.5 m high

(3) Sand Flush Gate and Steel Conduit

Type	:	High pressure roller gate and stoplog gate
Number	:	Two sets
Dimensions of gate	:	5.0 m wide x 5.0 m high
Dimensions of steel conduit	:	5.0 m wide, 5.0 m high and 30 m long

(4) Power Intake Trashracks

Type	:	Fixed screen
Number	:	Two sets
Dimensions	:	9.5 m wide x 35.0 m high

(5) Power Intake Multistage Gate

Type	:	Four numbers of roller gate
Number	:	Two sets
Dimensions	:	9.5 m wide x 10.5 m high for one roller gate

(6) Power Intake Gate

Type	:	Roller gate
Number	:	Two sets
Dimensions	:	3.9 m wide x 6.5 m high

(7) Tailrace Gate

Type	:	Roller gate
Number	:	Two sets
Dimensions	:	5.6 m wide x 5.0 m high

(8) Penstock

The penstock will be embedded in the dam, with total length of 90 m between the entrance of the power intake and the inlet valve in the powerhouse.

7.2.7 Generating Equipment

Basic design condition of the generating equipment are summarized as follows:

- Full supply level (FSL):	EL. 512.0 m
- Minimum operation level (MOL):	EL. 491.4 m
- Maximum plant discharge:	227.6 m^3/sec
- Tailwater level at maximum plant discharge:	EL. 441.2 m
- Installed capacity:	140,000 kW

Number of the generating units are selected to be two, which is the same as in the case of the existing power stations on the Tana river, and will assure flexibility of operation and maintenance to be compared to one unit case and will be economical to be compared to three units case.

The Francis type turbine is selected taking into account of its performance in head and discharge. The vertical-shaft umbrella type synchronous generator is selected due to its rotational speed and capacity.

As seen in the single line diagram in DWG-12, unit system with low voltage synchronization system is applied. The generator voltage is stepped up with main transformer connected to one-and-half (1.5) CB busbar arrangement of 220 kV switchyard in outdoor switchyard which has two 220 kV outgoing feeders toward Mutonga P.S/Kiambere P.S. Space for two incoming feeders is reserved for future development of downstream power stations. In addition to two 1,500 kVA station transformers, one 1,000 kVA local service transformer is also provided for 33 kV local supply.

The principal features of the generating equipment are outlined as follows:

(1) Water Turbine

- Number of units	:	2
- Type	:	Vertical shaft Francis
- Rated head	:	69.0 m
- Maximum plant discharge	:	113.8 m^3/sec/unit
- Rated output	:	71,600 kW
- Rated speed	:	188 rpm

(2) Generator

- Number of units	:	2
- Type	:	3-phase synchronous alternator of vertical shaft with brushless exciter

- Unit rated capacity	:	82,400 kVA
- Rated voltage	:	11 kV
- Rated frequency	:	50 Hz
- Rated power factor	:	0.85
- Electrical output	:	70,000 kW

(3) Main Transformer

- Number of units	:	2
- Type	:	Outdoor use with oil immersed and forced air-cooled
- Unit capacity	:	82,400 kVA
- Voltage ratio	:	11/235 kV

(4) 220 kV Circuit Breaker

- Rated voltage	:	245 kV
- Raged continuous current	:	1,600 A
- Rated rupturing current	:	31.5 kA

7.2.8 Transmission Lines and Substations

The power plant has a total output of 140 MW, being located about 45 km north-east of the existing Kiambere hydropower plant. The Mutonga hydropower plant with total output of 60 MW is also planned to be provided approximately at a middle point of both power plants.

The generated energy of both power plants under the Project will be transmitted through new transmission lines to the existing Kiambere power station, and further transmitted to the existing Embakasi substation near the demand center Nairobi. A committed 220 kV line from Kiambere power plant to the Embakasi, in addition to the existing line, should have been completed in time before the commissioning of the Low GF power plant to send the energy to the Nairobi area, and further new line should be added for assuring the energy transmission from the Mutonga power plant.

(1) Transmission Line Facilities

Canary (ACSR 460 mm^2) is selected from one of the standard conductors for 220 kV line used in Kenya in consideration of the total transmitting power and conductor thermal rating under the anticipated maximum conductor temperature of 75°C. Double circuit line is to be constructed in the section of Grand Falls-Mutonga-Kiambere for uninterrupting energy supply from the power plants even in the emergency case of one-circuit fault and the future development in the downstream reach of the Tana river.

Galvanized steel towers will be used as supports of the line with symmetrical vertical arrangement of double circuit power conductors with two overhead earthwires (galvanized steel strand 90 mm^2) for protecting the power conductors from lightning strokes. The average span length of the tower will be 350 m and accordingly total numbers of the towers in the section of Kiambere-Grand Falls are estimated at about 130 over 45 km and 12 in the tapping-off section to Mutonga power plant.

Insulators will be of either blue/brown grazed glass or porcelain for minimizing visual impact in the most environment of typical insulator strings (254 ϕ mm x 146 mm x 15 pcs).

To meet the construction schedule of the comprehensive project, the transmission line for the Project will be constructed in two phases. Double circuit line facilities from the Low GF power plant to the existing Kiambere power plant will be first constructed during the construction stage of the Low GF power plant. The tapped off section from the line to the new Mutonga power plant will then be constructed during the Mutonga project with double circuit provisions. The planned route of 220 kV transmission lines between the proposed power plants and the Kiambere power plant is shown in DWG-02.

(2) Substation Facilities

Following substations facilities are required:

- A new outdoor substation at the Low GF hydropower plant,
- A new outdoor substation at the Mutonga hydropower plant, and
- Extension of the existing outdoor substations at the existing Kiambere hydropower plant.

Extension of the Kiambere outdoor substation is available in a space south-east of the existing yard, and its arrangement of substation equipment will be similar to the existing one.

7.3 Mutonga Hydropower Scheme

7.3.1 River Diversion

The Tana river meanders from northeast to southeast in the upstream section of the dam site, and again changes it direction to northeast in the downstream section. A diversion tunnel is constructed at the left abutment by taking advantage of this meandering of the river. The tunnel will have sufficient rock cover and appropriate geological conditions along the route. The layout of the diversion system including cofferdams is shown in DWG-08.

A 5-year probable flood with a peak discharge of 1,600 m^3/sec is adopted as the design flood for the diversion system, taking into account of the concrete type dam. The diameter of the diversion tunnels is determined to be 11.0 m with the maximum velocity of 18.9 m/sec at the design discharge.

The crest elevation of an upstream cofferdam is set at EL. 539.0 m by adding a freeboard of 1.0 m to the flood water level at the design flood. A cofferdam will be also constructed in the downstream section

7.3.2 Main Dam

(1) Dam Type

As the result of the comparison study of the dam type as described in Section 6.4, the main dam is selected as a concrete type. The general plan of the dam is shown in DWG-08.

(2) Dam Crest

The FSL and MOL are set at EL. 550.0 m and EL. 538.5 m. The crest elevation of the main dam is designed to be safe with a free board against the FSL and the maximum flood water level through the examination similar to that for the Low GF as explained in Sub-section 7.3.1. The dam crest level is calculated at EL. 552.0 m with a freeboard including 1.0 m wind runup, 0.5 earthquake wave runup and 0.5 m allowance for the concrete dam. The crest level for flood operation is obtained at EL. 555.0 m by adding a freeboard of 2.0 m to the maximum flood water level of 553.0 m at the peak discharge of 10,900 m^3/sec of the 10,000-year probable flood, as seen in Figure 7.2. The dam crest is accordingly determined at EL. 555.0 m.

(3) Section of Dam

The concrete dam is designed to have a vertical face for the upstream and an inclination of 1 to 0.8 for the downstream face. The fillet of 1 to 0.2 slope is provided at the upstream face. The dam has a crest length of 660 m and a height of 60 m. A gallery is provided in the dam body for providing drain and grouting holes at the upstream dam foundation and an access to a sand flushing facility. Conventional concrete dam type is assumed, since the dam body is so small that the RCC type will not contribute to any advantage. The concrete volume of the dam is estimated at 416,000 m^3. The typical section is shown in DWG-09.

(4) Spillway

A spillway is provided at the river channel section of the dam, which is able to manage the extreme flood of a 10,000-year probable flood with a peak discharge of 10,900 m^3/sec with sufficient freeboard to the dam crest.

Four radial gates, each type having 17.5 m in width and 16.0 m in height, are provided to release the floods. The spillway including an energy dissipater is shown in DWG-09.

A stilling basin type is adopted as the energy dissipater for relatively low height of the dam. A 100-year probable flood with a peak discharge of 3,400 m^3/sec is applied for the design of the energy dissipater. The stilling basin is designed to be 80 m in length for the 100-year probable flood.

(5) Sand Flushing Facility

The dam is equipped with two lanes of sand flushing facility to release the sediments filled in the reservoir, see DWG-10.

As discussed in Section 6.4, the reservoir of the Mutonga dam will be filled with sediments within relatively short period, therefore a sand flushing facility is provided for maintaining the reservoir life with the release of the sediments. The facility consists of a 5.0 m wide and 5.0 m high steel conduit, a high pressure roller gate, a radial gate and a stoplog gate. The high pressure gate will flush out the sediments including coarse particles in front of the dam. The radial gate is provided just downstream of the high pressure gate to reinforce the water tightness of the sand flushing facility. The steel conduit will protect the conduit from surface erosion caused by coarser particles in the sediments.

The facility is located at both sides of the spillway section. The entrance sill is set at EL. 512.0 m to effectively flush out the sediments filled in front of the facility. Through this facility, release of the same amount of the artificial flood as that of the Low GF will be allowed.

(6) Foundation Treatment

Grouting will be carried out at the dam foundation as foundation treatment, consisting of a curtain grouting with two rows of 3 m spacing and 30 m in depth at the riverbed and 20 m in depth at the abutments, and a consolidation grouting with 3 m spacing and 5 m in depth on the dam foundation. The foundation treatment is shown in DWG-09 and 10.

7.3.3 Waterway

(1) Power Intake

A power intake, two works each having dimensions of 8.0 m wide and 15.0 m high, is provided at the upstream face of the concrete dam. The intake sill level is set at EL. 526.5 m, taking into account of the minimum covering above the intake to the MOL to avoid entraining air into the penstock. The flow velocity at the entrance of the intake is 0.67 m/sec for the maximum plant discharge of 87.5 m^3/sec per unit. Trashracks are installed in front of the power intake to prevent floating materials entering into the penstock.

(2) Penstock

A penstock is embedded in the concrete dam, which is 4.7 m in diameter and 59 m in length conveying the water to the power station.

7.3.4 Power Station

An open-air type station is located at the downstream of the concrete dam, with an outdoor switchyard located at the left abutment, see DWG-08, housing two units of vertical synchronous generators and Francis type hydraulic turbines. Main transformers will be installed behind the powerhouse.

(1) Powerhouse

The powerhouse has a dimension of 30 m in width, 49 m in length and 44 m, with a basic arrangement shown in DWG-11.

The house consists of a machine room of 21 m wide and 36 m long, an erection bay of 30 m wide and 13 m long and a control room of 8.5 m wide and 49 m long. The main units have a space of 18.0 m between centers to provide an area for installation and arrangement of turbines and auxiliaries. The turbine center is set at 509.5 m to secure the required draft head.

(2) Outdoor switchyard

An outdoor switchyard is located at the left abutment near the powerhouse as shown in DWG-08. The yard has an open area of 129 m in length and 90 m in width for incoming line from the Low GF power station and outgoing line to the Kiambere power station, accommodating main switchgear and control equipment. Main transformers will be installed just behind the powerhouse.

7.3.5 Permanent Access Road

A permanent access road to the Mutonga dam will be branched off from the permanent access road of the Low GF, with a distance of about 6.5 km from the branching point, see DWG-01.

7.3.6 Hydro-mechanical Works

Hydro-mechanical are gates, trashracks and steel penstock pipes, with main features listed below:

(1) Diversion Gate

Type	:	Roller gate
Number	:	One set
Dimensions	:	11.0 m wide x 11.0 m high

(2) Spillway Gate

Type	:	Radial gate
Number	:	Four sets
Dimensions	:	17.5 m wide x 16.0 m high

(3) Sand Flush Gate and Steel Conduit

Type	:	High pressure roller gate, radial gate and stoplog gate
Number	:	Two sets
Dimensions of gate	:	5.0 m wide x 5.0 m high
Dimensions of steel conduit	:	5.0 m wide, 5.0 m high and 40 m long

(4) Power Intake Trashracks

Type	:	Fixed screen
Number	:	Two sets
Dimensions	:	8.0 m wide x 15.0 m high

(5) Power Intake Stoplog

Type	:	Roller gate
Number	:	Two sets
Dimensions	:	3.4 m wide x 5.9 m high

(6) Tailrace Gate

Type	:	Roller gate
Number	:	Two sets
Dimensions	:	4.2 m wide x 3.9 m high

(7) Penstock

The penstock will be embedded in the dam with total length of 59 m between the entrance of the power intake and the inlet valve in the powerhouse.

7.3.7 Generating Equipment

Basic dimensions of the generating equipment are summarized as follows:

- Full supply level (FSL):	EL. 550.0 m
- Minimum operation level (MOL):	EL. 538.5 m
- Maximum plant discharge:	175.0 m^3/sec
- Tailwater level at maximum plant discharge:	EL. 510.7 m
- Installed capacity:	60,000 kW

Two units of generating equipment are installed in the Mutonga power station with the same reason as explained in sub-section 7.2.7. The layout is shown in DWG-11. Since variation of head is very small due to almost no change in the reservoir water, a vertical type Francis turbine is selected adopting a lower specific speed, taking into account its performance and economy. The type of generator is selected to be the vertical-shaft umbrella type synchronous generator due to its rotational speed and capacity.

The single line diagram shows that unit system (Turbine-Generator-Transformer in tandem) with low voltage synchronization system. The generator voltage is stepped up with main transformer connected to one-and-half (1.5) CB busbar arrangement of 220 kV switchgear in outdoor switchyard which has two 220 kV incoming feeders from Low Grand Falls P.S. and two 220 kV outgoing feeders to the existing Kiambere P.S. Besides two 1,500 kVA station service transformers, one 1,000 kVA local service transformer is also provided for 33 kV local power supply.

The principal features of the generating equipment are outlined as follows:

(1) Hydraulic Turbines

- Number of units	: 2
- Type	: Vertical shaft Francis
- Rated head	: 38.9 m
- Maximum plant discharge	: 87.5 m^3/sec/unit
- Rated output	: 30,800 kW
- Rated speed	: 176 rpm

(2) Generators

- Number of units	: 2
- Type	: 3-phase synchronous alternator of vertical shaft with brushless exciter
- Unit rated capacity	: 35,300 kVA
- Rated voltage	: 11 kV
- Rated frequency	: 50 Hz
- Rated power factor	: 0.85
- Electrical output	: 30,000 kW

(3) Main Transformers

- Number of units	: 2
- Type	: Outdoor use with oil immersed and forced air-cooled
- Unit capacity	: 35,300 kVA
- Voltage ratio	: 11/235 kV

(4) 220 kV Circuit Breakers

- Rated voltage	: 245 kV
- Rated continuous current	: 1,600 A
- Rated rupturing current	: 31.5 kA

CHAPTER 8 CONSTRUCTION PLAN AND COST ESTIMATES

8.1 Construction Plan and Schedule

8.1.1 Introduction

A construction plan of the project is prepared on the basis of the preliminary design discussed in the preceding Chapter 7, giving an outline of possible procedures, construction sequences, methods and types of plant and equipment to implement the construction works. The project includes two construction schemes such as the Low Grand Falls scheme and the Mutonga scheme. The construction works will be divided into four packages shown below and will be executed by the contractors selected by international competitive tenders for respective packages including pre-qualification. As for the engineering services, a consultant will be required for the execution of the project on the detailed design stage and the construction supervision stage, respectively.

(1) Civil works including preparatory works

Civil works include the construction of diversion tunnel, cofferdam, main dam, power station, tailrace and outlet channel, access road and base camp as well as preparatory works.

(2) Metal works

Metal works are composed of the installation of diversion gate, sand flush gate, spillway radial gate, intake gate, steel penstock and tailrace gate.

(3) Generating equipment

The works for generating equipment are for the installation of turbines, generators, transformers, indoor switchgear, ancillary equipment, miscellaneous materials, transmission line protective relays and PLC communication.

(4) Transmission line and substation equipment

Low Grand Falls scheme comprises of transmission line from Kiambere to Grand Falls, Low Grand Falls outdoor switchgear and extension of Kiambere substation. While, Mutonga scheme includes tapping-off transmission section, and Mutonga outdoor switchgear.

Low Grand Falls scheme will be first implemented and then Mutonga scheme will follow as shown in Fig. 8.1 Construction Schedule. The target date for commissioning of power station is planned to be mid 2008 for Low Grand Falls scheme and mid 2012 for Mutonga scheme.

8.1.2 Low Grand Falls Scheme

(1) Basic Conditions

The construction method and sequence are planned on the basis of the mode of construction and the target schedule of construction. Availability of construction forces, weather condition, geological and topographic conditions at the site and the mechanizes construction method are as well taken into consideration besides the matters mentioned above.

The commencement of the construction works is scheduled in July 2003 after the contract award. The Low Grand Falls scheme is planned to be completed by the end of June 2008 using information and data available at this moment, giving a time of 5 years (60 months). Preparation of a more realistic construction schedule, i.e. shortening of the construction time period, will rely on the structural drawings prepared in the detailed design stage.

The workable days in a year are assumed as follows:

Earthworks	252 days (21 days/month)
Embankment, core, filter	215 days (18 days/month)
Embankment, rockfill	256 days (21 days/month)
Concrete, grout	260 days (22 days/month)

Especially, the core embankment may restrict in April and May, November and December by the rainfall from the previous rainfall record. The workable days are shown in Table 8.1.

(2) Preparatory Works and Construction Facilities

Access Road

The project site is located about 150 km north-east in air distance from Nairobi. The Mwingi route from Nairobi to Grand Falls site is approximately 275 km and another route of Embu route is about 230 km. The Mwingi route is used for the transportation road during the Grand Falls scheme construction considering with the road conditions. With regard to the permanent access road, the Kiambere - Law Grand Falls access road of about 52.2 km in length will be constructed newly along the right bank of the Tana river. The temporary access roads required for the construction services will be provided by the contractor taking into account the borrow area, quarry site, sand deposit area, construction facilities, dam concrete transportation roads, etc.

Temporary Buildings and Base Camp

The temporary buildings required for the construction works are planned to be provided mainly at the main dam site. The temporary buildings consist of contractor's offices, quarters, a repair shop, a steel workshop, warehouses, labor quarters and so forth.

Beside of the temporary contractor's buildings, the employer's base camp will be constructed near Low Grand Falls site and located along the new access road of Kiambere - Low Grand Falls route. This base camp will be used for the construction supervision and for the maintenance and operation purpose after the power station commissioning.

Water Supply

Water required for the construction and the base camp is planned to be taken from the Tana River and other nearby tributaries. Water supply facilities will be required at the main damsite, requirement of which is to be 15 m^3/min.

Power Supply

Electric power for the construction is at this moment planned to be supplied with a diesel generator of 3,500 - 4,000 kW to carry out concrete dam work, rockfill dam work, metal works and generating equipment installation works. Meanwhile, the electric power to the base camp is planned to be supplied from a diesel generator, since the transmission line between Kiambere and Low Grand Falls is scheduled to be erected in the last stage of the Low Grand Falls scheme. It is however noted that the studies to seek power supply sources for the base camp should be re-examined in the detailed design stage.

Telecommunication

Wired telephone facilities will be provided for the construction use, and an automatic telephone exchange of 50 circuits will be installed at the contractor's office.

(3) River Diversion and Cofferdam

River diversion for the construction of main dam is planned to be carried out by the diversion tunnel method. Construction works for the diversion tunnels and the cofferdam are on the critical path of the construction schedule. One concrete-lined diversion tunnels with the dimensions of 10.5 m in a diameter and 630 m in length will be driven in 8 months of 2004. A top heading and then lower half cut method is applied for the tunnel excavation and a driving rate is to be 70 m/month for upper half section and 90 m/month for lower half section. Rocks will be drilled by 6 - boom drill truck jumbo and the broken rocks will be loaded by 1.4 m^3 side dump muck loaders into 11 ton dump trucks.

A full circular concrete lining method is planned to be applied for concrete lining and its progress rate is to be 72 m/month with 9 m sliding form. The concrete lining period is to be five months in 2004. Concrete will be transported by 4.5 m^3 agitator trucks from the 3 m^3 x 3 concrete plant using for the dam concrete and will be placed by a 100 m^3/hr concrete pump car.

Succeeding the completion of diversion tunnel, the upstream and downstream primary cofferdams will be constructed, and the river water will be diverted into the diversion tunnel in January 2005.

Second river diversion is planned to be made by closing in December 2007 considering with the reservoir impounding of 5 months. The plug concrete will be placed before wet test of generating equipment.

The zoned rockfill type cofferdam is planned to be built after a dry space is secured for constructing the cofferdam by releasing river water through the diversion tunnel. The cofferdam will be constructed prior to the main dam riverbed excavation and RCC concrete placement. Impervious earthfill materials will be hauled by 11 ton dump trucks from the borrow area and compacted by 20 ton tamping rollers. Rockfill and riprap materials will be obtained from the quarry and hauled by 32 ton dump trucks.

(4) Main Dam

The construction works of the main dam, which is a combined dam of rockfill and RCC

(Roller compacted dam) types, are scheduled to be performed for a period of 3 years from 2005 to 2007. The following are the main works for the dam construction.

Rockfill Dam

The rockfill dam section is planned to be 1,025 m and located on the right abutment. The rockfill dam will be constructed considering with the concrete dam placement schedule.

(A) Foundation excavation

Excavation works will be carried out in parallel with the diversion tunnel works, since the excavation elevation is higher than riverbed elevation. The excavation of common and weathered rock will be carried out using 32 ton bulldozers with ripper, 5.0 m^3 wheel loaders and 32 ton dump trucks. Trench excavation of rock will be made using 10 m^3/min crawler drills and 30 kg jack hammers for drilling and blasting, 0.6 m^3 backhoes and 11 ton dump trucks. The foundation will be cleaned using hand tools and skips as the final treatment.

(B) Embankment

The embankment of rockfill dam is scheduled to be carried out 19 months after the concrete dam height reaches at the rockfill embankment elevation. The core material will be obtained from four borrow areas located within 4 km upstream main dam. The rockfill materials will be from the quarry site located within 10 km apart from the main dam. While, the sand deposit inthe tributaries are located at Ngoru river and Karange river of 3 km and 9 km from the dam site.

Developing the borrow area will be made two months before starting the embankment works. The removal of topsoil will be made using 21 ton bulldozer, 2.2 m^3 tractor shovel, 11 ton dump trucks and 11 ton bulldozer. Quarry site developing is scheduled in March 2004, taking into account the concrete aggregate production of diversion tunnel concrete lining.

The core material is to be residual soils and 1.0 - 1.5 m thick thin layers in the proposed borrow areas. The core material will be dozed and gathered by 21 ton bulldozers and then watered strongly using water sprinkler truck to secure the optimum moisture contents. The material will be loaded by 2.2 m^3 tractor shovel into 11 ton dump trucks and spread by 11 ton bulldozer. The core material will be compacted in 20 cm thick using 20 ton self propelled tamping roller.

The fine filter material will be obtained from the river sand deposit. However, the coarse sand will be produced by the crushing plant provided for main dam concrete aggregate.

Quarried rock excavation is planned to be done by 7.5 m to 10 m high bench-cut method. The rock will be drilled by 15 m^3/min crawler drills and loaded by 5.0 m^3 wheel loaders into 32 ton dump trucks. At the embankment site, the rock materials will be spread by 21 ton bulldozers and compacted by 15 ton vibrating rollers.

Riprap materials, supplied from the quarry site, will be handled by 0.6 m^3 backhoes on the slopes of rockfill and all voids will be filled with smaller rock fragments.

(C) Foundation treatment

Succeeding the foundation excavation and foundation cleaning, consolidation grout and curtain grouts are scheduled to carried out in 2005 and 2006. The curtain grout will be made in parallel with the core embankment progress. Holes for the injection of consolidation grout will be drilled with 7 m^3/min crawler drills and 5.5 kW rotary boring machines. Curtain grout will be drilled by 5.5 kW and 11 kW rotary boring machines. Cement grout will be mixed at the central plant, delivered to 200 lit x 2 grout mixer installed at work site and injected by 7.5 kW and 11 kW grout pumps. The grouting works of rockfill dam section and concrete dam section are separately planned and carried out.

RCC Dam (Roller compacted concrete dam)

Concrete dam section is planned to be 405 m long (including conjunction section of 140 m) and dam height is 91 m. The total concrete volume is about 1,175,000 m^3. The interior concrete is planned to be RCC concrete and the concrete placement will be made by a expanded layer concrete method. The concrete placement period is scheduled to be 30 months in 2005, 2006 and 2007, in parallel with the rockfill dam construction.

(A) Foundation excavation

Excavation works in riverbed will be carried out after the river diversion. The excavation of common and weathered rock will be carried out using 32 ton bulldozers with ripper, 5.0 m^3 wheel loaders and 32 ton dump trucks. Trench excavation of rock will be made using 10 m^3/min crawler drills and 30 kg jack hammers for drilling and blasting, 0.6 m^3 backhoes and 11 ton dump trucks. The foundation will be cleaned using hand tools and skips as the final treatment. Furthermore, the final rock surface will be cleaned using high pressure water jetting.

(B) Concrete dam

RCC concrete material sources are the same of the rockfill material sources. The concrete aggregate will be produced from the quarried rock and the quarry site is located within 2 km apart from the main dam at the right aboutment. While, the sand deposit in the tributaries are located at Ngoru river and Karange river of 3 km and 9 km from the dam site.

Quarry site developing is scheduled in March 2004, taking into account the concrete aggregate production of diversion tunnel concrete lining. The removal of topsoil will be made using 21 ton bulldozer, 2.2 m^3 tractor shovel, 11 ton dump trucks and 11 ton bulldozer. The quarried rock excavation is planned to be done by 7.5 m to 10 m high bench-cut method. The rock will be drilled by 15 m^3/min crawler drills and loaded by 5.0 m^3 wheel loaders into 32 ton dump trucks for transporting to the crushing plant.

The sand material will be obtained from the river sand deposit. However, the screening and water washing are required for the required grading. The sand deposit is in the seasonable rivers and the stockpiling is necessary.

The concrete dam requires a lot of cement and the proposed dam site is situated under hot temperature from the temperature record of Kindarma and Masinga dams. Considering with above conditions and the previous concrete dam project in Kenya, Pozzolana cement will be used for a dam concrete and this kind of cement is produced by Bamburi Cement Co., Ltd located at Mombasa.

The concrete dam is at this moment planned to be constructed by a dump truck hauling method. It is however noted that the concrete placement method should be re-examined in the detailed design stage. This dam is planned to be a combined dam with RCC and rockfill dam. Therefore, it is very difficult to install a cable crane, taking into account the topographical conditions and a long dam crest length including rockfill dam section. In this view point, the dump truck hauling method is a quite reasonable dam concrete placement.

In case of RCC concrete placement, another fixed type tower crane of 13.5 ton with self climbing is necessary for miscellaneous handling works of construction materials and equipment and for installation works of sand flush gates, spillway radial gates, intake gates and steel penstock provided at spillway portion and power intake portion.

The concrete will be produced by 2 sets of 3 m^3 x 3 concrete plant considering total concrete volume and concrete placing period. The concrete transportation will be made with 20 ton dump trucks (9 m^3 concrete volume). The hauling road is planned to be a asphalt pavement surface and a tire washing facility will be provided. Also, the replacement of road embankment or the lift-up type temporary bridge to connect the dam placement elevation.

The zero slump concrete will be directly transported by 20 ton dump truck to the RCC placing spot and spread using 15 ton low pressure bulldozer in 0.75 m thick. The RCC concrete will be compacted using 7 - 10 ton vibrating roller and then the transverse joints will be provided at 15 m interval using vibrating cutting machine with a wedge shaped blade. The RCC concrete surface will be cured by water and sprinkling water like a mist. Before starting next layer, the green cut will be carried out and the mortar will be spread by 1.2 m^3 wheel loader and manpower.

The exterior concrete and the concrete around structures and embedded metal works will be transported by 20 ton or 11 ton dump truck and dumped at the placing spot. The concrete will be again hauled in position using 1.2 m^3 wheel loader and compacted by concrete vibrating machine with 4 nos. of 150 mm concrete vibrator. Upstream and down stream steel form adjacent to the exterior concrete will be provided.

The concrete placement at the crest portion will be made by applying a layer concrete method instead of an ordinary concrete block method. The concrete will be transported by 11 ton dump trucks and dumped at the RCC concrete surface, and then 3 m^3 load haul dumper. The concrete will be compacted by the similar way applied for the exterior concrete.

(C) Foundation treatment

Succeeding dam foundation concrete placement, consolidation grout and curtain grouts are scheduled to carried out in 2005, 2006 and 2007, in parallel with the dam concrete placement. Holes for the injection of consolidation grout will be drilled with 7 m^3/min crawler drills and 5.5 kW rotary boring machines. Curtain grout will be drilled by 5.5 kW and 11 kW rotary boring machines. Cement grout will be mixed at the central plant, delivered to 200 lit x 2 grout mixer installed at work site and injected by 7.5 kW and 11 kW grout pumps. The grouting works of concrete dam section and rockfill dam section are separately planned and carried out.

Other Structures

The flip bucket at spillway portion will be placed for 10 months in parallel with the main dam concrete placement work. The concrete will be handled using a 13.5 ton tower crane and compacted by a similar way applied for the exterior concrete. The assembled steel metal form for curb portion of flip bucket will be provided and the concrete placement will be carefully made.

Mechanical equipment comprising of sand flush gate, spillway radial gates, intake gate and penstock are installed in the main dam concrete structure. These installation works will be made according to the dam concrete placement schedule and The sand flush gate and penstock will be provided at the lower to middle portions in the main dam and these installation will be made using 13.5 ton tower crane in parallel with the RCC concrete placement. The installation of spillway radial gates and intake gate is scheduled to be carried out at the last stage of 2008 after the completion of crest concrete placement and pier concrete.

Intake concrete works will also made using 13.5 ton tower crane. After the installation of steel penstock and sand flush gate portion, the embedded concrete will also made by 13.5 ton tower crane and compacted by hand type 120 mm concrete vibrators.

(4) Power Station

The powerhouse is planned to be located just downstream toe of the main dam. The powerhouse is designed as a surface type reinforced concrete structure with the dimension of 60 m long, 32 m wide and 50 m high. The powerhouse structure is scheduled to be completed by July 2006 before the commencement of installation of overhead traveling crane. The first stage construction consists of open excavation, placement of substructure concrete, installation of the draft tubes and placement of the second concrete of them. The second stage consists of the construction of superstructure, installation of overhead traveling crane and casing, placement of the second concrete and installation of turbines and generators.

Foundation excavation of powerhouse and tailrace will be carried out using 10 m^3/min crawler drills, 21 ton bulldozers, 32 ton bulldozers with ripper, 2.2 m^3 tractor shovels and 11 ton dump trucks. The substructure concrete works will be carried out using 4.5 m^3 agitator trucks, 100 m^3/hr concrete pump cars and 1 m^3 buckets with 30 ton truck crane. The concrete will be produced by 3 m^3 x 3 concrete plant provided for the main concrete dam. Following the substructure concrete and draft tube installation, the superstructure concrete will follow. And then the overhead crane is scheduled to be provided in August 2006 to install the casings, turbines and generators. The powerhouse building works will be performed in parallel with the installation of turbines and generators.

(5) Major Construction Equipment

As the result of the study of construction method, the major plant and equipment to be used for the construction works of Low Grand Falls scheme are shown in Table 8.2.

8.1.3 Mutonga Scheme

(1) Basic Conditions

The commencement of the construction works is scheduled in January 2008 after the contract award. The Mutonga scheme is planned to be completed by the end of June 2012 using information and data available at this moment, giving a time of 4.5 years (54 months). Preparation of a more realistic construction schedule, i.e. shortening of the construction time period, will rely on the structural drawings prepared in the detailed design stage.

The workable days are the similar of Low Grand Falls scheme.

(2) Preparatory Works and Construction Facilities

Access Road

The access road between Kiambere and Low Grand Falls is planned to be constructed in 2005. The branch road is required as a permanent access road to Mutonga dam site. This access road is to be 6.5 km.

The temporary access roads required for the construction services will be provided by the contractor taking into account the quarry site, sand deposit area and construction facilities, etc.

Temporary Buildings and Base Camp

The temporary buildings required for the construction works are planned to be provided mainly at the main dam site. The temporary buildings consist of contractor's offices, quarters, a repair shop, a steel workshop, warehouses, labor quarters and so forth.

Beside of the temporary contractor's buildings, the employer's base camp will be constructed near Mutonga site. This base camp will be used for the construction supervision and for the maintenance and operation purpose after the power station commissioning.

Water Supply

Water required for the construction and the base camp is planned to be taken from the Tana River and other nearby tributaries. Water supply facilities will be required at the main damsite, requirement of which is to be 10 m^3/min.

Power Supply

Electric power for the construction is at this moment planned to be supplied with a diesel generator of 2,000 - 2,500 kW to carry out concrete dam work, metal works and generating equipment installation works. Meanwhile, the electric power to the base camp will be connected from the transmission line provided in the Low Grand Falls scheme. It is however noted that the studies to seek power supply sources for the base camp should be re-examined in the detailed design stage.

Telecommunication

Wired telephone facilities will be provided for the construction use, and an automatic telephone exchange of 50 circuits will be installed at the contractor's office.

(3) River Diversion and Cofferdam

River diversion for the construction of main dam is planned to be carried out by the diversion tunnel method. Construction works for the diversion tunnels and the cofferdam are on the critical path of the construction schedule. One concrete-lined diversion tunnels with the dimensions of 11 m in a diameter and 660 m in length will be driven in 8 months of 2008 and 2009. A top heading and then lower half cut method is applied for the tunnel excavation and a full circular concrete lining method is applied. The concrete lining period is to be 5 months in 2009. Concrete will be transported by 4.5 m^3 agitator trucks from the 1.5 m^3 x 2 concrete plant using for the dam concrete and will be placed by a 100 m^3/hr concrete pump car.

Succeeding the completion of diversion tunnel, the upstream and downstream primary cofferdams will be constructed, and the river water will be diverted into the diversion tunnel in January 2009.

Second river diversion is planned to be made by closing in February 2012 considering with the reservoir impounding of 1 month. The plug concrete will be placed before wet test of generating equipment.

The zoned rockfill type cofferdam is planned to be built after a dry space is secured for constructing the cofferdam by releasing river water through the diversion tunnel. The cofferdam will be constructed prior to the main dam riverbed excavation.

(4) Main Dam

The construction works of the main dam, which is a concrete dam type, are scheduled to be performed for a period of 24 months from 2009 to 2011. The concrete dam is planned to be 60 m high and 620 m long. The total concrete volume is to be 415,660 m^3. The following are the main works for the dam construction.

Foundation Excavation

Excavation works will be carried out after the river diversion, in parallel with the construction of cofferdam. The excavation of common and weathered rock will be carried out using 32 ton bulldozers with ripper, 5.0 m^3 wheel loaders and 32 ton dump trucks. Trench excavation of rock will be made using 10 m^3/min crawler drills and 30 kg jack hammers for drilling and blasting, 0.6 m^3 backhoes and 11 ton dump trucks. The foundation will be cleaned using hand tools and skips as the final treatment. Furthermore, the final rock surface will be cleaned using high pressure water jetting.

Concrete Dam

The concrete aggregate will be produced from the quarried rock and the quarry site is located within 2 km apart from the main dam. While, the sand deposit in the tributaries are located at Mukindu river, Kanyaka river and Konyu river of 2 km , 4 km and 5 km from the dam site.

Quarry site developing is scheduled in September 2008, taking into account the concrete aggregate production of diversion tunnel concrete lining. The removal of topsoil will be made using 21 ton bulldozer, 2.2 m^3 tractor shovel, 11 ton dump trucks and 11 ton bulldozer. The quarried rock excavation is planned to be done by 7.5 m to 10 m high bench-cut method. The rock will be drilled by 15 m^3/min crawler drills and loaded by 5.0 m^3 wheel loaders into 32 ton dump trucks for transporting to the crushing plant.

The sand material will be obtained from the river sand deposit. However, the screening and water washing are required for the required grading. The sand deposit is in the seasonable rivers and the stockpiling is necessary.

Type of cement is to be Pozzolana cement applied for Low Grand Falls scheme. The cement will be transported from Bamburi Cement Co., Ltd. located at Mombasa.

The concrete dam is at this moment planned to be constructed by a trestle-crane method. It is however noted that the concrete placement method should be re-examined in the detailed design stage. It seems to be difficult to install a cable crane, taking into account the limited construction period, an environment impact due to excess abutment excavation and a long dam crest length of 620 m. In this view point, the tower crane concrete placement method is a quite reasonable dam concrete placement.

At the riverbed portion, the steel girder trestle will be provided for placing concrete and transporting concrete and miscellaneous materials. As for the both abutment, the rail tracks with concrete base will be provided on the excavated ground , taking into account the limit of trestle-crane concrete placement of 75 m. Two numbers of 9.5 ton self propelled tower cranes will be provided for concrete placement. The concrete will be transported by transfer cars from the concrete plant located at both abutments. These tower cranes will be used for the installation works of sand flush gates, spillway radial gates, intake gates and steel penstock provided at spillway portion and power intake portion. The concrete will be produced by 2 sets of 1.5 m^3 x 2 concrete plant located ar both abutments, considering total concrete volume and concrete placing period.

The concrete block width is to be 15 m and a concrete lift is 1.5 m. The concrete will be charged into the spot by 9.5 ton tower crane with 3 m^3 concrete bucket and compacted by a concrete vibrating machine with 3 nos. of 150 mm vibrator.

The block concrete method will require a cooling pipe installation and grouting pipe installation, and then the cooling a block and joint grouting will be carried out. Before starting next block, the green cut will be carried out and the mortar will be spread by manpower.

Foundation Treatment

Succeeding dam foundation concrete placement, consolidation grout and curtain grouts are scheduled to carried out in 2009, 2010 and 2011, in parallel with the dam concrete placement. Holes for the injection of consolidation grout will be drilled with 5.5 kW rotary boring machines. Curtain grout will be drilled by 5.5 kW and 11 kW rotary boring machines. Cement grout will be mixed at the central plant, delivered to 200 lit x 2 grout mixer installed at work site and injected by 7.5 kW and 11 kW grout pumps.

Other Structures

The stilling basin concrete will be placed for 12 months in parallel with the main dam concrete placement work. The concrete will be handled using a 9.5 ton tower crane and compacted by a similar way applied for dam concrete.

Mechanical equipment comprising of sand flush gate, spillway radial gates, intake gate and penstock are installed in the main dam concrete structure. These installation works will be made according to the dam concrete placement schedule and The sand flush gate and penstock will be provided at the lower to middle portions in the main dam and these installation will be made using 9.5 ton tower crane in parallel with the dam concrete placement. The installation of spillway radial gates and intake gate is scheduled to be carried out at the last stage of 2011 after the completion of crest concrete placement and pier concrete.

Intake concrete works will also made using 9.5 ton tower crane. After the installation of steel penstock and sand flush gate portion, the embedded concrete will also made by 9.5 ton tower crane and compacted by hand type 120 mm concrete vibrators.

(4) Power Station

The powerhouse is planned to be located just downstream toe of the main dam. The powerhouse is designed as a surface type reinforced concrete structure with the dimension of 49 m long, 30 m wide and 44 m high. The powerhouse structure is scheduled to be completed by July 2010 before the commencement of installation of overhead traveling crane. The first stage construction consists of open excavation, placement of substructure concrete, installation of the draft tubes and placement of the second concrete of them. The second stage consists of the construction of superstructure, installation of overhead traveling crane and casing, placement of the second concrete and installation of turbines and generators.

Foundation excavation of powerhouse and tailrace will be carried out using 10 m^3/min crawler drills, 21 ton bulldozers, 32 ton bulldozers with ripper, 2.2 m^3 tractor shovels and 11 ton dump trucks. The substructure concrete works will be carried out using 4.5 m^3 agitator trucks, 100 m^3/hr concrete pump cars and 1 m^3 buckets with 30 ton truck crane. The concrete will be produced by 1.5 m^3 x 2 concrete plant provided for the main concrete dam.

Following the substructure concrete and draft tube installation, the superstructure concrete will follow. And then the overhead crane is scheduled to be provided in August 2010 to install the casings, turbines and generators. The powerhouse building works will be performed in parallel with the installation of turbines and generators.

(5) Major Construction Equipment

As the result of the study of construction method, the major plant and equipment to be used for the construction works of Mutonga scheme are shown in Table 8.2.

8.1.4 Construction Schedule

After the completion of the feasibility study, two years of additional environment study will be required. The detailed design and preparation of tender document are scheduled to be made during 1.5 years. This engineering service includes those for Low Grand Falls and Mutonga schemes.

The target date for the commissioning of generating equipment is planned to be mid 2008 for Low Grand Falls scheme and mid 2012 for Mutonga scheme. The main construction works of Low Grand Falls scheme are planned to be 5 years from July 2003 to June 2008, while 11 years will be required after the completion of the feasibility study as shown in Figure 8.1.

The implementation of Mutonga scheme is planned to be commenced 6 months before the completion of Low Grand Falls scheme. The main construction works of Mutonga scheme are planned to be 4.5 years from January 2008 to June 2012, while 15 years will be required after the completion of the feasibility study as shown in Fig. 8.1. The financial arrangement required for the additional environment study, the detailed design stage and the construction work stage shall be made by the Kenya Power Co., Ltd.

The following basic schedule shall be kept in order to secure the commissioning target of the Project:

(1) Low Grand Falls Scheme

a) Financial arrangement for the additional environment study:
6 months from January to June 1998
b) Additional environment study:
24 months (2 years) from July 1998 to June 2000
c) Financial arrangement for the detailed design:
6 months from January to June 2000
d) Selection of a consultant for the detailed design:
3 months from July to September 2000
e) Detailed design and preparation of tender document:
18 months (1.5 years) from October 2000 to March 2002
f) Financial arrangement for construction works:
12 months (1 year) from April 2001 to march 2002
g) Selection of consultant for construction works
3 months from April to June 2002

h) Tender and contract including prequalification:
12 months (1 year) from July 2002 to June 2003

i) Main construction works:
60 months (5 years) from July 2003 to June 2008

j) Commissioning of the commercial operation
: Beginning of July 2008

(2) Mutonga scheme

a) Financial arrangement for construction works:
12 months (1 year) from October 2005 to September 2006

b) Selection of consultant for construction works:
3 months from October to December 2006

c) Tender and contract including prequalification:
12 months (1 year) from January to December 2007

d) Main construction works:
12 months (1 year) from January 2008 to June 2012

e) Commissioning of the commercial operation:
Beginning of July 2012

8.2 Cost Estimate

8.2.1 Introduction

Construction cost for the Project comprising of the Low Grand Falls scheme and Mutonga scheme are estimated on the basis of the preliminary design. Unit prices for each work item are established considering local conditions, available construction equipment and materials and suitability of the construction method and referring to similar international projects.

The foreign and local currency of the project costs are estimated on basis of US Dollar and Kenya Shilling respectively, and the foreign currency portion expressed in US Dollar is converted to Kenya Shilling for assessing the total cost.

Assumptions and conditions applied for the cost estimate are as follows:

a) Price level : Price as of June 1997

b) Exchange rate : US$ 1.00 = K.Shs.54 = J.Yen 120

c) Work quantity : Quantities estimated from the preliminary design for the work items given in Table 8.13 and 8.14.

d) Construction works will be carried out by the contractor selected through an international competitive bidding.

e) Construction costs are divided into direct construction cost and indirect construction cost.

The direct construction cost (contract cost) are the cost for preparatory works, civil works, metal works, generating equipment, transmission line and substation equipment. While, the indirect construction costs are the ones for land acquisition and compensation, administration expenses, engineering services and contingencies.

8.2.2 Preparatory Works

Costs for the preparatory works comprise of the cost for insurance of works, temporary buildings, water supply system, electric power supply system, telecommunication system, provision of medical facilities, operation of medical services, inland transportation, testing laboratory, temporary access roads, installation works for the construction plant, temporary dam facilities and so on. The cost for preparatory works is estimated at 5 percent and 10 percent of the sum of civil works, respectively for Low Grand Falls scheme and Mutonga scheme, taking into account the project magnitude and dam construction method.

8.2.3 Civil Works

The costs of civil works are estimated by adopting unit rates including labor cost, material cost, equipment cost and contractor's overhead expenses and profits.

(1) Labor Cost

Wages obtained in Nairobi are based on the direct daily wages in 8-hour shift of labor. The labor cost is shown in Table 8.3.

(2) Material Cost

Most of construction materials are supplied from local markets, and therefore the local material prices were canvassed in Nairobi and Mombasa as shown in Table 8.4. These local material prices include local net price, inland transportation, duties and value added tax, i.e. the purchase price at the project site. The imported materials which are not available in local markets are estimated on the exemption of import duties and taxes.

(3) Equipment Cost

Equipment and plant for the construction will be provided by the contractor. Referring to prevailing prices in Japan as of June 1997, the prices of equipment itself are estimated based on the CIF price in Mombasa. The duties and taxes charged for the import of equipment are excluded in this cost estimate.

The equipment cost is divided into foreign and local portions. The foreign currency portion mainly includes the costs of depreciation, spare parts and consumable, while the local currency portion includes the cost of mechanic labor, the cost for the repair and maintenance and administration expenses. The costs of equipment requirement for the project are listed in Table 8.5.

(4) Contractor's Indirect cost

Overhead expenses and profits are contributed to the unit rates of each work item. These expenses are estimated at 25 percent of the direct cost including labor cost, material cost and equipment cost.

(5) Unit Price

The unit prices for each work item are estimated in accordance with the above conditions as shown in Table 8.13 and Table 8.14.

(6) Bill of Quantities

Cost estimate of civil works are based on the work quantity obtained from the preliminary design of the following structural components:

- Diversion tunnel
- Cofferdam
- Main dam
- Flip bucket or stilling basin
- Intake
- Power station
- Tailrace and outlet channel
- Access road and bridge
- Base camp

8.2.4 Metal Works

The prices for metal works are based on the recent international contract prices of similar works. The cost of imported equipment and materials is estimated at the CIF price in Mombasa excluding import duties and taxes. Costs for supply and delivery of imported items, ocean freight and insurance are included in the foreign currency portion. Costs for unloading and other charges at the port and for inland transportation are estimated in the local currency portion. Installation costs are portionally shared by the foreign and local portions. Structural components to estimate the cost of metal works are as follows:

- Diversion tunnel closing gate
- Sand flush gates
- Spillway radial gate, hoist, stoplogs
- Intake gate, intake trashracks
- Penstock
- Powerhouse tailrace gate, hoist

8.2.5 Generating Equipment

Price estimates of generating equipment are the same as those of metal works. Costs components of generating equipment are as follows:

- Turbines
- Generators
- Transformers
- Indoor switchgear
- Ancillary equipment
- Miscellaneous materials

- Transmission line protective relays
- PLC communication

8.2.6 Transmission Line and Substation Equipment

The prices for tower materials, conductors and substation equipment are estimated at the CIF price in Mombasa excluding import duties and taxes. Civil works such as site clearance, earthwork and foundation treatment are included in the transmission line cost. Other conditions to estimate the foreign and local portions are the same as those of metal works. Following are the transmission line routes and and substations to estimate the construction cost:

<u>Low Grand Falls scheme</u>
Transmission line, Kiambere - Grand falls section
Low Grand Falls outdoor switchgear
Extension of Kiambere substation

<u>Mutonga scheme</u>
Transmission lie, Tapping-off section
Mutonga outdoor switchgear

8.2.7 Land Acquisition and Compensation

All required land acquisition and compensation shall be carried out before the commencement of construction for Low Grand Falls scheme and Mutonga scheme. Those costs include the costs required for the acquisition of the reservoir and construction areas, and the compensation of resettlemenmt.

8.2.8 Administration Expenses

An allowance of 0.5 percent of the direct construction cost is provided for the executing agency of Kenya Power Co., Ltd., which is a part of the local currency portion.

8.2.9 Engineering Services

The cost of engineering services for the detailed design including the preparation of tender documents is estimated based on assumed man-months. The detailed design will be performed against both schemes for Low Grand Falls and Mutonga. As for construction supervision, the engineering services cost is estimated at 7 percent of the sum of direct construction cost, sharing the foreign currency portion of 85 percent and the local portion of 15 percent.

8.2.10 Contingencies

Contingencies are provided to cope with unforeseen physical conditions (physical contingency) and inflation (price contingency). The rate of physical contingency is assumed at 10 percent of an amount required for preparatory works, civil works, administration expenses and engineering services, while 5 percent for metal works, generating equipment, transmission line and substation equipment.

The cost of price escalation for Low Grand Falls scheme is estimated over of 12 years from 1997 to 2008 applying the inflation rate of 2 percent per annum for the foreign currency portion. As for the local currency portion, the price escalation is converted to the foreign currency portion applying the same annual rate, since the implementation schedule is on a long term basis and the escalation ratio is assumed to be related with the devaluation rate of US$. As for Mutonga scheme, the cost of price escalation is estimated over 16 years from 1997 to 2012 applying the same escalation rate. The amount of price escalation is estimated at US$ and counted in the foreign currency portion.

8.2.11 Construction Cost

Total construction cost for both schemes is shown in Table 8.6. The construction cost of the Project including the Low Grand Falls scheme and the Mutonga scheme excluding price escalation is estimated at KShs. 30,561 million (US$ 566 million) equivalent in total. While, the total construction cost of the Project including price escalation is estimated at KShs. 36,686 million (US$ 679 million) equivalent in total.

The construction cost of the Low Grand Falls scheme excluding price escalation is estimated as KShs. 20,609 million (US$ 382 million) equivalent in total, consisting of KShs. 15,124 million in the foreign currency portion (US$ 280 million, 73.4%) and KShs. 5,485 million in the local currency portion (26.6%). While, the total construction cost of the Project including price escalation is estimated at KShs. 24,002 million (US$ 444 million) equivalent in total, consisting of KShs. 18,518 million in the foreign currency portion (US$ 343 million, 77.2%) and KShs. 5,485 million in the local currency portion (22.8%). The construction cost of Low Grand Falls scheme is shown in Table 8.7.

The construction cost of the Mutonga scheme excluding price escalation is estimated at KShs. 9,953 million (US$ 184 million) equivalent in total, consisting of KShs. 7,553 million in the foreign currency portion (US$ 140 million, 75.9%) and KShs. 2,400 million in the local currency portion (24.1%). Whilst, the total construction cost of the Project including price escalation is estimated at KShs. 12,684 million (US$ 235 million) equivalent in total, consisting of KShs. 10,284 million in the foreign currency portion (US$ 190 million, 81.1%) and KShs. 2,400 million in the local currency portion (18.9%). The construction cost of Mutonga scheme is shown in Table 8.8.

The breakdown of construction cost for both schemes is shown in Table 8.9 and Table 8.10.

The detailed construction cost for both schemes is shown in Table 8.13 and Table 8.14.

8.2.12 Annual Disbursement Schedule

Annual disbursement of the construction cost for the foreign and local currencies is estimated on the basis of the construction schedule as summarized below and the details are shown as shown in Table 8.11 and Table 8.12.

Year	Foreign Currenc (1,000 US$.)	Local Currency (1,000 KShs.)	Total Equivalent (1,000 KShs.)
2000	2,642.27	23,760	166,443
2001	8,260.50	183,266	629,333
2002	3,153.91	247,731	418,042
2003	45,850.72	1,012,065	3,488,004
2004	44,580.91	476,518	2,883,887
2005	54,125.13	1,010,731	3,933,488
2006	83,911.05	1,388,796	5,919,993
2007	88,771.85	1,125,163	5,918,843
2008	59,867.29	705,609	3,938,443
2009	22,003.60	312,082	1,500,276
2010	48,197.38	731,024	3,333,683
2011	61,481.16	574,563	3,894,546
2012	10,514.07	93,383	661,143
Total	533,359.84	7,884,691	36,686,122

CHAPTER 9 PROJECT EVALUATION

9.1 Introduction

This Chapter deals with the economic and financial viability of the Mutonga/Grand Falls hydropower project. The following Section 9.2 discusses the economic viability of the Project as a hydropower development project itself. It also deals with the economic viability of the additional effects which would be derived from the artificial flood release. Financial viability of the Project is further discussed in Section 9.3, including an examination of loan repayability.

9.2 Economic Evaluation

9.2.1 Economic Benefit and Cost

The economic evaluation is to ascertain the contribution of the Project towards the national economic development. The viability of the Project is examined with the economic internal rate of return (EIRR) on the basis of economic benefit and cost.

The economic benefit of hydropower project is gauged at the cost of the alternative thermal plant which will produce equivalent power and energy. As explained in the sub-section 6.3.1, the alternative thermal plant of the Project was selected as gas turbine and low-speed diesel thermal plants, taking into account the allocation of power generated by the Low Grand Falls and Mutonga hydropower plants into the power system.

The power and energy values were estimated at US\$814/kW and US\$0.0686/kWh, referring to basic data in KNPDP in 1992. Annual O & M cost was set at US\$ 16/kW. Adjustment factors for power and energy values were set at 1.147 and 1.028.

As the result of the optimization study in Section 6.3, the firm power output and annual average energy were estimated at 134 MW and 715 GWh for the Low Grand Falls hydropower plant, and estimated at 58 MW and 337 GWh for the Mutonga hydropower plant. The benefits and O/M cost of the hydropower productions are therefore estimated as follows:

		Lower Grand Falls	Mutonga
Firm Power Output	(MW)	134	58
Annual Average Energy	(GWh)	715	337
Power Benefit	(US\$1,000)	125,110	54,152
Energy Benefit	(US\$1,000/year)	50,408	23,759
O/M cost	(US\$1,000/year)	2,459	1,064

Taking into account the period for the construction of the alternative thermal power plant to be 2 years, the power benefit is distributed in 2 years so that the completion of alternative thermal plant coincides with respective plant of the Low Grand Falls and Mutonga.

The construction cost excluding price escalation was estimated at US\$ 382 million for the Low Grand Falls and US\$ 184 million for the Mutonga as seen in the sub-section 8.2.11. Economic cost is obtained by adopting the standard conversion factor of 0.89 for the part local currency of the project cost, resulting in US\$ 339.7 million for the Low Grand Falls and US$ 164.0 million for the Mutonga with annual distribution as shown in Table 9.2.1.

Replacement cost of generating equipment is scheduled to be incurred 30 years after commissioning. The operation and maintenance cost (O/M cost) is assumed to be 0.5 % of the construction cost for the civil works, metal works and transmission line and US$ 5.5/kW for generating equipment of the Low Grand Falls and US$ 8/kW for the Mutonga.

Replacement of the alternative thermal plant is assumed to be every 25 years in consideration of their service life, with cost at 90 %.

9.2.2 Economic Evaluation

(1) Evaluation for Power Generation

The economic viability of the Project is examined by estimating the internal rate of return (EIRR), taking a 50-years project life. The economic cash flow is shown in Table 9.2.2, and economic indexes are summaiized below :

Items	Unit	Economic Index
Present worth of cost	US$1,000	229,837
Present worth of benefit	US$1,000	286,710
Internal rate of return (EIRR)	%	14.98
Net benefit (B - C)	US$1,000	56,873
Benefit/cost (B/C ratio)		1.25

The EIRR of the Project is calculated at 14.98 %, which is higher than the opportunity cost of capital, 12 % adopted in Kenya. Net benefit and benefit/cost ratio are computed at US$ 56.87 million and 1.25 at a discount rate of 12%, respectively.

(2) Evaluation for Additional Effect from Artificial Flood Release

The environmental assessment (see Section 4.4) recommends to release normal biannually flood with a peak discharge of 1,100 m^3/sec with total volume of 490 million m^3 from the Low Grand Falls dam to maintain the existing downriver conditions in environments. This will contribute to a decrease of the firm power and energy. The result of the reservoir simulation showed that the firm power and annual average energy were 77.25 MW and 533.36 GWh/year in the case of single operation of the Low Grand Falls, and 136.38 MW and 870.89 GWh/year in the case of combined operation of the Low Grand Falls and Mutonga.

The benefits of power and energy are gauged at US$ 72.125 million in total and US$37.606 million per annum in the case of single operation of the Low Grand Falls, and at US$ 127.332 million in total and US$ 61.416 million in the combined operation case of both dams.

The economic cash flow is shown in Table 9.2.3, and economic indexes are summarized below:

Items	Unit	Economic Index
Present value of cost	US$1,000	235,535
Present value of benefit	US$1,000	220,282
Internal rate of return (EIRR)	%	11.23
Benefit - cost (B - C)	US$1,000	-15,253
Benefit/cost (B/C ratio)		0.94

The EIRR indicates 11.2 %, lower than the opportunity cost (12%). The Project would however have economic viability with the following reasons:

- The downstream system of environment and production has already been affected as the result of the existing five dams, and has been deteriorated due to the decrease of biannual normal floods. Construction of additional dams without flood release will hasten this destruction. However, additional dam with flood release measure would be able to reverse negative situation and improve the downstream values.

  Though the value is not quantified because of its intangible nature, the artificial flood release could contribute to an increase of the benefit, the planned artificial flood release of the Project should have positive value in this context.

  Volume of artificial flood release is estimated at 490 million m^3, which occupies about 51 % of the effective storage volume of 955 million m^3 of the Low Grand Falls dam. Assuming that the above percentage cost of the dam including cofferdam and diversion tunnel is allocated to the environmental part, the cost born by the power generation will be decreased, with a result of EIRR at 14.4 % and a net benefit of US$ 37.5 million.

- The 490 million m^3 is the maximum volume of artificial flood release, with the adoption of "Predefined Fixed Flood Release" as explained in the sub-section 4.5.3. By adopting the "Variable Supplementary Flood Release" which takes into considerations contribution of additional flows from the downstream catchment, volume of artificial flood release will be decreased by about 12% as an initial estimate, which will increase the EIRR to be 12.53%. Clearly if the entire upstream system of 7 reservoirs were managed according to upstream rainfall events to maximise use of seasonal flows, the potential to increase power output from the whole system would also be increased.

9.2.3 Sensitivity Analysis

(1) Sensitivity Analysis for Cost and Benefit

Inflation rate in Kenya varied from 1.6 % to 53.91 % per annum since 1981 as shown in Table 2.1.1, an annual average at 17.56 %. Sensitivity analysis is adopted for 2 cases: 1) Case 1 for cost increase by 5%, 10% and 15%, and 2) Case 2 for benefit decrease by 5%, 10% and 15%, with results summarized as follow:

Case 1			
Benefit	Base case	Base case	Base case
Cost	+5%	+10%	+15%
EIRR	14.27%	13.63%	13.03%
Case 2			
Benefit	-5%	-10%	-15%
Cost	Base case	Base case	Base case
EIRR	14.24%	13.49%	12.74%

(Base case: the values are the same as these in Table in 9.2.2 (1))

In all cases, EIRR is larger than the opportunity cost of capital (12%).

(2) Sensitivity Analysis for Construction Sequence

Parallel Construction of Low Grand Falls and Mutonga Schemes

A parallel construction of Low Grand Falls and Mutonga Schemes is examined. In this case, general items of the civil works including temporary camp and access roads could be decreased as common use for both schemes, to be compared to the series construction. In addition, head office overheads will be reduced. The construction costs of the civil works for the Low Grand Falls and Mutonga schemes are assumed to be about 97% of the original sequence (the Low Grand Falls followed by the Mutonga), and the economic costs at US$ 333.1 million for the Low Grand Falls and US$ 161.3 million for the Mutonga.

Construction of the Low Grand Falls scheme is assumed to be commenced in July 2003 after the additional environmental study, detailed design and tender procedures including financial arrangement. On the other hand, the construction of Mutonga scheme will be commenced in January 2004, half year later than the Low Grand Falls construction. Both schemes will commence power generation in July 2008, producing total average energy of 1,052 GWh/year.

As the result of economic evaluation, EIRR is calculated at 15.33% which is slightly higher than that (14.95%) of the original sequence. However, it is noted that: 1) a possibility exits that the integrated power system is not be able to absorb total energy generated by both power station in 2008, and 2) a difficulty would remain in financial arrangement for two schemes at once because of huge amount of construction cost.

Reverse Construction of Low Grand Falls and Mutonga Schemes

A reverse construction sequence of the Low Grand Falls and Mutonga schemes (Implementation of Mutonga followed by Low Grand Falls) is examined. The construction cost of the reverse sequence will be the same as the original sequence. The construction of the Mutonga scheme will be started July 2001 after detailed design and tendering, and completed at the end of 2005. The additional environmental assessment will be executed in parallel with the construction of the Mutonga. The Low Grand Falls will be commenced in July 2005, and completed in June 2010. The Mutonga scheme will start production of annual 337 GWh in January 2006, and the Low Grand Falls scheme, annual 715 GWh in July 2010.

The EIRR is calculated 14.65%, slightly inferior to the original sequence.

(3) Sensitivity Analysis for Increased Cost of Additional Transmission Line

As a result of the power flow analysis discussed in the Section 5.3, an additional single-circuit 220 kV line between the Kiambere and Embakasi substations in Nairobi will be required to the power system in 2012 to secure the reliable power supply in the power system, especially for the transmitted power toward Nairobi. Taking into account this enhancement nature, the construction cost of the additional transmission line is assumed to be borne by other financial source as a scheme.

However, a sensitivity analysis is made be adding construction cost of the additional transmission line to cost of the Project.

The unit construction cost of the additional transmission is assumed to be US$140,000/km, and the distance between the Kiambere and Embakasi is 162.5 km, therefore the cost is estimated at US$ 22.75 million, and economic cost at US$ 20.25 million. The construction period line is assumed to be 3 years, and annual economic cost at US$6.75 million. The EIRR is obtained at 13.43%.

9.2.4 Installation Timing

The optimum installation timing of the Mutonga and Low Grand Falls power plant is defined to be the year of the proposed schemes put in the least cost sequence on the long-term installation program of power plants to the national power grid so as to meet growing power and energy requirements.

A software package "EGEAS" developed by Massachusetts Institute of Technology (MIT) and Electric Power Research Institute (EPRI) in the United States was adopted to find out the least cost sequence. The reliabilty of power supply is examined on loss of load probability, LOLP.

(1) Conditions and Assumptions

Examination to search for the optimum installation timing of the proposed scheme was carried out on the following assumptions:

- Thermal power plant is installed to avoid power outage in drought year when hydro power generation decreases.
- Installation program is executed on quarterly seasonal period to reflect annual dry and wet seasons in Kenya. The seasonal generation ratio is listed in Table 9.2.4.
- Power outage rate or loss of load probability (LOLP) is given as 10 days in a year, which was adopted in KNPDP. In addition, cost of unserved energy is assumed to be 0.79 US$/kWh.

- Power plants have to meet with growing demand of power (MW) and energy (GWh) which is estimated in the Section 5.4.

- Table 9.2.5 shows a list of existing, committed and proposed hydropower plants. The third unit 72.5 MW of the Gitaru hydropower plant is committed to start operation in 1999, and the Sondu / Miriu hydropower in 2002 with an installed capacity of 60 MW.

- The existing thermal power plants are composed of geothermal at Olkaria, steam plants at Kipevu and combustion turbines at Nairobi and Kipevu as listed in Table 9.2.6. First and second units of 75 MW LS diesel plant at Kipevu are committed to start operation in 1999. Olkaria North East geothermal plants (Olkaria II and III) of 64 MW each are committed to start operation in 1999 and 2000.

- Geothermal power plant (64 MW), MS diesel plant (60 MW) and LS diesel plant (50 MW) are employed as alternatives, of which principal features are as the same as those in KNPDP such as investment cost, plant life, oil price, heat rate, and generated energy, outage rate in Table 9.2.7.

- The firm output and annual average energy of the Mutonga and Low Grand Falls are shown in Table 9.2.5.

- The "Commodity Price Forecasts, First Quarter 1996" issued by the World Bank forecast that the current fuel price was US$ 17/bbl for crude in 1997. On the other hand, the construction schedule in the Section 8.1 shows that the earliest commissioning year is the year of 2008 for the Low Grand Falls and the year of 2012 for the Mutonga. However, it is assumed that the earliest installation timing of hydropower plant would not be appeared by the fuel price of US$ 17/bbl because of large initial investment of hydropower plant. Accordingly, the first step of the study on the installation timing is carried out by changing fuel prices from US$ 17/bbl to US$ 20/bbl with unfixed installation year.

  The second step of the study examines the installation timing of the Low Grand Falls and Mutonga at the fixed fuel price of US$ 17/bbl and fixed installation year, taking into account the fuel price, the above construction schedule, and the construction sequence. Power plant capacities of the Low Grand Falls and Mutonga are referred to the result of study on the optimum installed capacity in Section 6.5. The study of the second step seeks total system costs for three installation sequences of the Low Grand Falls and Mutonga, that is, the original sequence (the Low Grand Falls in 2008 followed by the Mutonga in 2012), the parallel sequence of the Low Grand Falls and Mutonga in 2008 and the reverse sequence of the Mutonga and Low Grand Falls (the Mutonga in 2006 followed by the Low Grand Falls in 2010).

- "EGEAS" computes installation timings to avoid power outage in dry year and to attain the least system cost by means of "dynamic programming" (DP).

(2) Installation Timing

The installation timings of the Low Grand Falls and Mutonga schemes are sought by running "EGEAS" by changing fuel prices. As the result, "EGEAS" finds out that the Low Grand Falls is best installed in the year of 2009, and the Mutonga in the year of 2014 at the fuel price of US$ 19/bbl. Total expansion costs for commissioning years of the Low Grand Falls in 2009 and Mutonga in 2014 are calculated at US$ 981 million for 25 years and US$ 1,287 million for 50 years.

The study of the second step to seek the installation sequence of the Low Grand Falls and Mutonga is carried out at the fuel price of US$ 17/bbl. The result is summarized below:

Unit: Million US$

Installation Sequence	Commissiong Year	Total Cost for Expansion Plan (Low GF: 140 MW and Mutonga: 60 MW)	
		For 25 years	For 55 years
Original (Low GF followed by Mutonga)	2008/2012	988	1,275
Parallel (Low GF and Mutonga)	2008/2008	993	1,281
Reverse (Mutonga followed by Low GF)	2010/2006	997	1,285

The result reveals that the input year of 2008 for the Low Grand Falls power plant and 2012 for the Mutonga power plant will lead to the least cost case at the fuel price of US$ 17/bbl among three sequences. Figures 9.2.1, 9.2.2 and Table 9.2.8 show the result of the study on the installation timing of the power plants. Tables 9.2.9 and 9.2.10 also give the power and energy balance of the system with the Low Grand Falls in 2008 and the Mutonga in 2012.

9.3 Financial Evaluation

9.3.1 Financial Evaluation

(1) Financial Evaluation for Power Generation

The financial viability of the Project is evaluated in terms of the project entity. The analysis of the Project focuses on the financial internal rate of return (FIRR) based on the present worth of expenditures and revenues.

Financial benefit is gauged at increased revenue which will be led by the commercial operation of the Project. The construction schedule in the Section 8.1 shows that the Low Grand Falls project will commence the power generation from July in 2008. Full power generation of the Project will be started after the completion of the Mutonga hydropower plant from July in 2012. According to KPC, the overall average tariff in 1997 is KShs. 5.04/kWh at the current price level. In addition, KPC predicts that the overall average tariff in 1999 will be KShs. 6.72/kWh. Actual and projected tariffs are tabulated as follows:

(Unit:KShs./kWh)

Calendar year	Domestic + Street Lightening	Commercial + Industrial Use	Off-peak	Overall average	Increasing ratio against previous year
1993	2.398	2.493	2.518	2.462	46.90%
1994	3.536	3.818	3.461	3.709	50.65%
1995	4.355	4.571	4.219	4.466	20.41%
1996	4.588	4.641	4.321	4.610	3.22%
1997	5.266	4.850	4.628	5.040	9.33%
1998	6.166	5.659	5.399	5.880	16.67%
1999	7.049	6.467	6.171	6.720	14.29%
Increasing since 1993	19.69%	17.22%	16.11%	18.22%	
Increasing since 1995	12.79%	9.06%	9.97%	10.75%	

Source : Average Tariff (KSh./KWh) by Consumption Classification (Current Prices) (Actual Revenue/Actual Sales), projected by KPC, March 1997 Load Forecast.

In the above table, tariff increasing ratio between 1993 and 1999 is calculated at 18.22% as an overall average. The increasing ratio against previous year in 1993 and 1994 seems to be irregular since the tariff was unusually increased, linking to the inflation rate as seen in Table 2.1.1. The increasing ratio from 1995 to 1999 is calculated at 10.75% varying between 3.22 % and 20.41% as that against previous year. In addition, the increasing ratio from 1995 to 1997 is calculated at 6.24 %, showing low value.

Taking into account the above tendency of increasing ratio, the average electricity tariff in 2008 is estimated at KShs.10.42/kWh by projecting KShs. 6.72/kWh in 1999 by using an annual increasing ratio, assumed at 5%.

The annual available energy to the customers is estimated at 601 GWh from the Low Grand Falls and 283 GWh from the Mutonga by considering the transmission line loss of 16%. Annual revenue in 2009 from the Low Grand Falls is calculated at US$ 115.9 million, and that from the Mutonga in 2013 is calculated at US$ 54.6 million.

As the result of construction cost estimate in the Section 8.2, the project cost is assumed as shown in Tables 9.3.1. The replacement cost of electrical equipment is assumed to be incurred 30 years after commissioning. The operation and maintenance cost (O/M cost) is assumed to be 0.5 % of the construction cost for the civil works, metal works and transmission line and US$ 5.5/kW for the generating equipment for the Low Grand Falls and US$ 8/kW for the Mutonga. Sub-transmission and distribution cost is assumed to be US$ 0.03/kWh. Table 9.3.2 shows the parameters for calculation of financial benefit and other parameters.

FIRR is calculated by using a cash flow shown in Table 9.3.3, to be 15.10%.

(2) Financial Evaluation for Additional Effect from Artificial Flood Release

Annual average energies of the Low Grand Falls and Mutonga were estimated at 533.4 GWh in the Low Grand Falls only and 870.9 GWh/year in combined operation of the Low Grand Falls and Mutonga. In consideration of 16 % transmission loss, annual revenue in 2009 from the Low Grand Falls is calculated at US$ 86.5 million, and that from the Mutonga in 2013 is calculated at US$ 54.7 million. The project cost including price escalation and O/M cost is distributed with the same condition as the above (1). As the result, FIRR is calculated at 12.22% .

9.3.2 Sensitivity Analysis of FIRR

Sensitivity tests for FIRR are carried out, taking into account future cost and electricity tariff in consideration of the increasing tendency of power tariff and price escalation as follows:

Case 1: Price escalation is not counted for both costs and revenue: (power tariff: KShs.5.04/kWh)

Case 2: Price escalation is counted for both costs and revenue: 1) foreign currency portion of 2% per annum and local currency portion of 10% per annum by the commissioning year of the Low Grand Falls, and 2) electricity tariff of 10 % per annum (power tariff: KShs.14.38/kWh)

The results of sensitivity tests for FIRR are summarized as follows:

Case	FIRR
Case 1	9.06%
Case 3	15.75%

Even Case 1 which is most unfavourable condition for the Project, the Project will have the return of 9.1% in FIRR.

9.3.3 Loan Repayability

The loan repayability of the Project is examined with the following conditions:

- Amount of loan : 85% of total cost
- Interest rate : 2.3 %
- Repayment period : 30 years
- Grace period : 10 years

Land acquisition cost, tax and duties are non-eligible for the loan. All the repayments will be made in form of annual equal installments with the interest during construction period.

Table 9.3.4 presents the loan repayability. The Project will start producing surplus from Year 9, when the revenue of the Low Grand Falls scheme is expected for the first time. From that year, the revenue would counterbalance the cumulative deficit, and the revenue gets it surplus in Year 10.

CHAPTER 10 ADDITIONAL ENVIRONMENTAL ASSESSMENT STUDY

The environmental assessment study in Chapter 4 recommended the mitigation measures for the reservoir resettlement and the degradation of the downstream riverine corridor. However, it is necessary to carry out an additional assessment of the environments in the downstream reach of the Tana river without delay, taking into considerations the presently taking place adverse effect on the downstream environment due to the existing dams and further increase of the impacts caused by development of the Project.

The additional environmental assessment study will aim at the implementation of long-term and overall environmental management plan in the Tana river flood plain and delta, which is assumed to be realized through a procedure as:

(1) Implementation of an environmental assessment
(2) Development of a management plan based on the assessment, and
(3) Implementation of the monitoring management plan.

The object of the study covers the first step, above (1), of the procedure, which is to implement an environmental assessment on the Tana river flood plain and delta, paying due considerations on the linkage among the resources, human activities and river flow regime. The study will include as explained in Section 4.9 in detail:

1) Review and compilation of data and information
2) Survey and investigation of physical conditions:
3) Identification of values, functions and sensitivities of ecosystem:
4) Identification of uses and dependencies on natural resources:
5) Investigation of impacts of socio-economic activities:
6) Identification of development opportunity and constraints
7) Identification of management issues
8) Development of management policy
9) Training and technology transfer

The study area will cover: 1) The river downstream of the Grand Falls, the Tana river flood plain downstream of Garissa and the Tana delta which will include all permanent and seasonal wetlands and the coastal resources, including marine resources off-shore of the delta to a depth of 15 meters as well as "hinterland" resources seasonally dependant on the Tana river, and 2) the Tana river upstream of Garissa in relation to flow regime including sediment.

The study period is assumed to be total 24.0 months from the time of commencement and to the submittal of the draft final report, where two wet seasons in the early study period are allowed for the measurement of runoff and sediment because the flow regime in this nature will have key role on the environment in the flood plain. The time schedule of the respective study item is assumed to be as shown in Figure 10.1 as follows:

- Review and compilation of data and information will be carried out for three months between the first month to the 3rd month.
- Survey and investigation of physical conditions including review work is planned to be done for 13 months between the 4th month to the 16th month. Two wet seasons are be included during this period, taking into considerations the measurements of floods and sediments in wet seasons.

- Identifications of ecosystem functions and natural resource uses will be carried out for 13.0 months between the 4th month to 16th month.
- Investigation of impacts of socio-economic activities including investigation of guarantee river flow will be executed for 10.5 months between the 9th month to 19th month.
- Identification of development opportunity and constrains will be carried out for 2 months from 14th month to 15th month.
- Identification of management issues will be carried out for 2 months from 14th month to 15th month.
- Development of management policy will be carried out for 3.5 months from 20th month to 23th month.
- Technology transfer will be done for 2 months from the 23th month to 24 month.

Through a whole period of the study, the training will be carried out to obtain the required long-term studies and to continue working on the above and related monitoring topics.