(c) Commercial and industrial development

Development of commercial and industrial activities is expected to boost economy in the district, through forward and backward linkages with agricultural sector : utilization of agricultural inputs and provision of inputs and implements to agriculture development.

DDC places an emphasis on improvement in urban infrastructure, especially housing, sewage, water systems and electrification so that private sector will invest more in industrial commercial activities in Nakuru municipality and other larger towns.

(d) Infrastructure provision

DDC recognizes that the growth of small urban centers in the district and their contribution to agricultural and industrial development will depend on the availability of the requisite infrastructural facilities, especially roads, water and sewerage systems, electric power and the telephones.

(e) Provision of social services

Improvement in the peoples' ability to participate in development activities will be encouraged by increasing the provision of health facilities and schools and literacy campaigns. Other proposed actions are as follows :

- rehabilitation and expansion of urban water supplies,
- expansion of primary health care,
- establishment of more effective family planning, and,
- more involvement of "Harambee" activities and NGOs.

3.3 Present Public Water Supply

3.3.1 The Stage 1 Project

The Stage 1 Project has been planned to urgently remedy the acute water shortage in both the Nakuru municipality and Gilgil town, including four bulk water consumers as KMB, GMB, NYSTC and ASTU in the vicinity of Gilgil. A general layout of the Stage 1 Project is as shown in Fig. 3.2.

The construction of the Stage 1 Project was actually commenced in January, 1989 for a scheduled completion in December, 1991. It actually produce a fully treated water amounting to 18,000 m^3/day, which has been determined to be allocated to the Nakuru municipality, Gilgil town, KMB, GMB, and NYSTC at the rates of 13,300, 1,300, 1,200, 870, and 1,300 m^3/day respectively.

The source of the raw water is the unregulated runoff of the Turasha river and a run-of-river intake has been sited at approximately 10 km upstream from the confluence of the Malewa river. An intake dam is of concrete gravity type with 11.2 m at the maximum height and 67.4 m in the total crest length.

The raw water is gravitated to the treatment works through a steel raw water main, composed of a combination of D600 mm with a length of 1,690 m and D500 with a length of 7,870 m. The treatment works with a treatment capacity of 19,000 m^3/day is located at about 6 km east of Gilgil town and is of the rapid sand filtration type, consisting of a receiving well, a rapid mixing chamber, two flocculation chambers, two sedimentation basins, 8 filter beds, a clear water reservoir, a wash water pond and two sludge lagoons.

The treated water is initially conveyed into R6 reservoir in Nakuru municipality through a steel treated water main, which runs via the Central Reservoir with a storage capacity of 3,000 m^3 in Gilgil town. The treated water main is 39,350 m in length and is composed of four different pipe sizes : D600 mm, D500 mm, D450, and D400 mm. It is characterized by a gravity flow for its entire length. The Central Reservoir has been contrived in order to automatically distribute the treated water to the Nakuru municipality and Gilgil town including bulk consumers without artificial discharge regulation.

The Stage 1 Project will strengthen and expand the water distribution facilities within both Nakuru municipality and Gilgil town. Upon completion of the project, the reticulation networks are capable of conveying the 2005 hourly maximum water demand. The Gilgil town, NYSTC and GMB are directly fed from the Central Reservoir, while KMB is directly catered by the treated water main. The Nakuru municipal area has been divided into 7 reticulation zones, each of which has been provided with the service reservoir.

The water demand in both Nakuru municipality and Gilgil town has been forecast to grow at an average annual growth rate of more than 5 % during a 15-year period from 1990 to 2005. As a consequence both the areas have been predicted to fall into failure of sufficient water supply even after completion of the Stage 1 Project. The potable water supply in Naivasha town and rural areas would be more serious, since no specific elaboration for improving water supply has been programmed so far. It is, therefore, urgently required to materialize the Project as soon as possible.

3.3.2 Nakuru Municipality

For potable water supply, the municipal area may be divided into Nakuru town area and Lanet area. The Lanet area is a military station.

The Lanet area is exclusively served by the Lanet treatment works and two wells located on the premises of the treatment works. The treatment works have a nominal production capacity of 1,050 m^3/day and the boreholes yield at the rate of 200 m^3/day. The treatment works are relying on the unregulated runoff of the Meroroni river. The average daily supply in the past was 1,150 m3/day. All the source works are operated and maintained by MOWD, while the distribution facilities are under the jurisdiction of the armed forces.

The public water supply in the town area is fully managed by NMC and is at present served by four sources, namely the Meroroni treatment works and Kabatini, Playing Field and Baharini borehole fields. The location of those source works are shown in Fig. 3.2. In 1988 there were 10,096 individual connections and 1,456 standpipes in the town area.

The Meroroni treatment works are of the conventional rapid sand filtration type and have a nominal production capacities of 5,740 m^3/day. The number of wells are 3 in the Playing field and Baharini fields respectively and four in the Kabatini field. The water supply in the recent years has been unstable as summarized below.

Source of Supply	Quantity of Supply(m^3/day)		
	Maximum	Minimum	Average
Meroroni treatment works	5,740	1,030	5,200
Kabatini borehole field	10,730	3,510	6,840
Playing Field borehole field	3,860	1,330	3,330
Baharini borehole field	8,130	4,360	5,540
Total	45,730	10,230	20,910

Such a large fluctuation had been attributed to the large variation in the runoff of the river and frequent break down of the pumping facilities in the well fields.

The existing reticulation pipes are very dense in the central part of the municipal area but are very scarce in both the Nakuru East and West areas, ever though these represent more than 50 % of the municipality's whole population.

3.3.3 Gilgil Town

There are two sources of water supply at present in Gilgil town : one is the Gilgil Malewa treatment works, which has been serving mainly KMB, and the other is the Gilgil Murindati treatment works, bearing the public demand in the town and bulk supplies to GMB, ASTU and NYTSC.

The Malewa treatment works with a nominal production capacity of 1,000 m^3/day is located at about 6 km west of the Gilgil town and has been pumping the raw water up from the Malewa river at about one km upstream from the confluence of the Turasha river. It has however, been deteriorated seriously and has already been programmed to be abandoned upon completion of the Stage 1 Project.

The Malewa treatment works were recently innovated by MOWD to augment its production capacity to 1,680 m^3/day and is located at about three km north of the town. Its source of raw water supply is depending on the unregulated runoff of the Murindati river and it major components are three sedimentation tanks, two composite filtration units, three filters , two clear water reservoirs and an elevated washwater tank. It is currently delivering treated water to the Gilgil town, ASTU, NYSTC and GMB at the rates of 640, 45, 420, and 575 m^3/day respectively. It has also planned that, upon completion of the Stage 1 Project, the Murindati treatment works specifically serves ASTU.

Water distribution within the town of Gilgil is currently made from an elevated tank, which is connected with the Murindati treatment works through a treated water main with a diameter of 150 mm and a length of 3,100 m, and has a storage capacity of 110 m^3. There were 292 connections in the town in 1988.

3.3.4 Naivasha Town

The public water supply is currently available within a limited area of the township, of which the boundary is the same as the proposed water service area. NTC is managing the water distribution, while MOWD is making a bulk water supply to NTC. All the source works are, therefore, operated and managed by MOWD. There were 1,657 connections in the existing water serving area.

There are two sources of supply : one is the Kinangop Ring Main treatment works and the other is the borehole field on the left bank of the Karati river. In the recent years the treatment works have, however, been obliged to operate intermittently during the wet season, because of augmented abstraction from the Kinangop Ring Main. The borehole field is, therefore, the substantial source of the water supply. In 1988 the annual amount of the supply was approximately 540,000 m^3, composing of 525,000 m^3 by the borehole filed and 15,000 m^3 by the treatment works.

In addition to the public supply, there two institutions being furnished with their own water supply facilities : one is WFTI equipped with one borehole having a production capacity of 100 m^3/day and the other is the Naivasha Prison provided also with one borehole with a production capacity of 720 m^3/day.

3.3.5 Gilgil and Eburu Rural Areas

There is no specific public water supply facility in the Eburu rural area, while portion of the Gilgil rural area is served by the Gilgil Nakuru treatment works. Most of inhabitants are putting untreated water available from boreholes and/or streams to a good use but are obliged to travel a long distance to reach water sources from place to place.

The Gilgil Malewa treatment works with a net production capacity of 950 m^3/day is located in the vicinity of the Gilgil Malewa treatment works and pumps the raw water up from the Malewa river at about 10 km upstream from the confluence of the Turasha river. Originally it was constructed to transmit the treated water to Nakuru municipality and treated water main was laid down along the railroad. The water has, however, been consumed by the consumers enroute to the municipality through 278 connections

3.4 Existing Sewerage System

In general sewage disposal in the proposed service area is being made by means of aqua privies, cess pits and/or septic tanks. In particular in the rural area the former two methods are much more common than the third one. The water-borne sanitary service is at present available within a limited area of Nakuru municipality and rendered by NMC.

The existing sewers in Nakuru municipal area covers only the central part of the municipal area, approximately 10.7 km^2 out of the entire municipal area of 91.7 km^2. The sewer coverage is divided into two areas, namely, western and eastern areas, by an intercepter. The sewage from the eastern and western areas has been treated by the Town and Njoro treatment works respectively.

The Town treatment works are composed of the conventional treatment works and experimental lagoons. The conventional works was commissioned in 1956 and the lagoons in 1961. The conventional treatment works mainly comprises the primary treatment works, clarifiers and biofilters and sludge digestion plant, but the sludge digestion plant has been out of order. The primary treatment works have a nominal capacity of 3,400 m^3/day of medium strength sludge. The lagoons with a nominal capacity of 450 m^3/day receives inflow from the primary treatment works through a distribution chamber. Inflow into the works was 3,100 m^3/day on an average during the period from 1988 to 1989 and effluent is led into the Lake Nakuru through an open channel.

The Njoro treatment works were commissioned in 1974 and are composed of two anaerobic ponds for pre-treatment and three series of faculative pond for waste stabilization. The anaerobic ponds are 4 m in the depth and were designed for a retention time of 1.25 days. Each series of faculative pond consists of two primary ponds in series followed by a secondary pond and a tertiary pond and has a treatment capacity of 1,200 m3/day. The length of the primary, secondary and tertiary ponds are 142, 161, and 48 m respectively and the width is 73 m for all the ponds. The faculative ponds were designed at a retention time of 39 days. This works have been operating under over-loading conditions : inflow was recorded at 5,720 m^3/day on an average during the period from 1988 to 1989 and effluent has been released into Lake Nakuru without maintaining the water quality standard stipulated by WAB in 1973. The effluent standards by WAB are as given in Table 3.10.

The increased water supply in the municipal area will naturally result in augmenting sewage effluent, which will flow down into Lake Nakuru which will cause adverse effects on the ecology of the lake. Accordingly suitable countermeasures will have to be taken up.

IV. FORMULATION OF DEVELOPMENT PLAN

4.1 Planning Scenario

The primary purpose of the Project is to secure and augment safe water supply to the proposed service area by means of creating a reservoir in the Malewa river basin by construction of dam. Such water resources development is generally planned to deal with the long term perspective and so as not to cause conflict with the existing water uses and cause substantial adversed effects on the environment . At the outset of the Study the basic scenario was defined on some key issues significant to formulation of the Project through discussions between the MOWD/NWCPC, JICA's Advisory Committee and the Study Team.

(1) Planning horizon

The ultimate target year of the water supply has been extended to the year 2015.

(2) Water supply method

The water supply by the Project will actually be divided into two categories : one is the bulk water supply to both NMC and NTC and other four consumers such as NYSTC, ASTU, KMB and GMB, and the other is the direct distribution to consumers in the Gilgil town and Eburu and Gilgil rural areas. The water distributions within the bulk water supply areas will be directly managed by the authorities concerned.

(3) Safe and reliable supply

It is most desirous that the water resources development of the Malewa river basin be planned to ensure the whole of the water demand at the ultimate target year in a joint operation with the existing facilities even at a certain drought year, but not at the sacrifice of the precious natural environment. It may be allowed to reduce the water supply in severe drought year to minimize the adversed effects. Further it is proposed to suspend the use of the groundwater to make the inhabitants free from a possible fluoride-oriented disease.

(4) Effective and rational use of water resources

The runoff of the Malewa river basin is the precious water resources in and around the Study Area, and NWCPC will, therefore, strain its efforts to conserve such water resources as long as possible. It is the most significant to formulate the Project placing emphasis on the efficient and effective use of the water resources in order to attain this purpose.

(5) Environmental considerations

The natural environment is the important resources not only for Kenya but also for the world and must be preserved at the present status and nature as much and long as possible. In particular there are three natural lakes in the Study Area, all of which are the famous for their natural environments over the world. It will, therefore, be a key issue to continuously maintain such atmosphere that the natural environments and human activity coexist with each other.

(6) Consistency with the Stage 1 Project

The Project will be implemented following and operated in integration with Stage 1 Project. In order to reduce the operation and maintenance cost as a whole, it is required to design the water supply scheme to be consistent with the Stage 1 Project.

4.2 Water Demand Forecast

4.2.1 Forecast Population

The population in the proposed water service area is estimated to be approximately 340,000 in 1987, of which nearly 70 % concentrated in Nakuru municipality. The future population forecast has been made available from the previous studies for the entire service area excluding Naivasha town, which have already been authorized by MOWD/NWCPC as fundamentals for the Study. For Naivasha town the forecast has been updated as a result of a joint study of the Study Team and NTC's task force as described in Annex E. Table 4.1. presents the forecasted population and growth rate for the respective service area.

It is expected that the population in the service area will reach approximately 1.56 million in 2015, of which about 1.23 million, corresponding to 78 % of the whole population concentrates in the Nakuru municipality. The population growth rate exceeds 4 per cent per annum on an average, and it is rather higher in the urban areas than the rural area, because of the rural - urban migration.

4.2.2 Water Demand Forecast

The future water demand forecast has been made available from the same studies as the population forecast for all the service areas excluding Naivasha town. For Naivasha town a specific forecast has been elaborated by the Study based on the latest socio-economic parameter. A full description on the water demand forecast is reported in Annex E

In principle the water demand forecast had been prepared in accordance with the criteria set forth in the Design Manual. According to the Design Manual, the water demand is classified into 6 categories, i.e., residential, institutional, commercial, livestock, industrial and others, and unit water consumption by category has been set out as given in Table 4.2. The residential water consumption rate varies largely with income level. Therefore population distribution by income level has been projected as given in Table 4.3. The unit water consumption includes the un-accounted for water of 20 %.

As given in Table 4.4, the average daily water demand amounts to 50,200 m^3 in 1990, 65,000 m^3 in 1995, 85,940 m^3 in 2000, 110,740 m^3 in 2005, 143,090 in 2010, and 185,230 m^3 in 2015. The main water consumption area among the proposed service areas is the Nakuru municipality, sharing approximately 70 % of the entire water demand, followed by Naivasha town with a share of approximately 12 %. In terms of the water demand category, the residential category is predominant, generating nearly 65 % of the whole water demand.

4.2.3 Need for Water Resources Development

The proposed water service area has been suffering from an acute shortage of water supply even at present and the situation eventually becomes worse than ever as the water demand increases rapidly and continuously. The future water deficit has been calculated for the respective water service area by deducting the available water from the forecasted water demand as given in Table 4.5.

The balance calculation was made based on the following assumptions.

(1) In the light of development scenario all the existing groundwater sources suspend its operation upon completion of the Project to provide a good quality of water, but will be kept in reserve for unexpected drought year and other emergency cases. In due consideration of a project implementation schedule, it is realistic that the water supply by the Project could be initiated only after a year 1995 at the earliest.

(2) The existing treatment works such as Meroroni, Murindati and Nakuru Gilgil continue to serve during the planning horizon at the same production rate as the present.

(3) There will be no effort to augment water supply capacity other than by the Project during the planning horizon.

(4) The Naivasha town and the other two rural areas will be integrated into the service area only upon completion of the Project.

The water deficit was calculated not only for the average daily demand but also for the maximum daily demand. The maximum daily demand corresponds to 1.2 times of the average daily demand. The forecasted average and maximum daily deficits are 158,250 m^3 and 195,300 m^3 respectively at the ultimate target year 2015. The water resources development scheme must be planned to remedy the whole of the average daily deficit, while the water supply scheme should cover the daily maximum deficit. It is not possible to take such great amount of water from the Malewa river on a run-of river basis. It is, therefore, essential to regulate the runoff throughout the year by creating a reservoir with construction of dam.

4.3 Formulation of Water Resources Development Scheme

4.3.1 Alternative Development Schemes

Through map study and field reconnaissance, there have been identified two dam sites : one in the Malewa river is called as Malewa Dam Scheme and the other in the Turasha river is named as the Turasha Dam Scheme, as shown in Fig. 4.1. Both dam sites are located at approximately 10 km upstream from the confluence of the Malewa and Turasha rivers. The catchment areas of the Malewa and Turasha rivers are 635 km^2 and 711 km^2 respectively at the proposed dam site. The topographic mapping and geological investigation were carried out to facilitate the assessment of technical feasibility of the both schemes. The area - storage curves of the Malewa and Turasha reservoirs are as presented in Figs. 4.2 and 4.3 respectively. The geological and hydrological conditions in the areas of both the schemes are presented in Annexes B and D respectively.

In due consideration of the location and topographic condition of two damsites, there have been recognized conceptually three different approaches for the water resources development as briefly described below.

(1) Case 1 : Single dam construction

This is to realize either the Malewa Dam Scheme or the Turasha Dam Scheme. The dam should be high enough to create a reservoir sufficient to meet the required water supply.

(2) Case 2 : Double dam construction

Both the Malewa and Turasha Dam Schemes would be implemented in stages in line with the growth of the water demand. The storage capacity of the respective reservoir is accordingly smaller than Case 1.

(3) Case 3 : Single dam with a trans-basin diversion tunnel

This contemplates to use the runoffs of both the Malewa and Turasha rivers in integration. Either the Malewa or Turasha Dam Schemes will be brought into effect and a trans-basin diversion tunnel will be driven between the Turasha river and the Malewa river in order to convey water released from the reservoir into the another river, where an intake will be located. The reservoir releases the water only when the runoff of the other river is short of the required water demand. The required reservoir capacity will be smaller than Case 1 but larger than Case 2.

In case of the single dam construction, the height of the dam would exceed 80 m at both damsites in order to secure the required reservoir storage, whereas it has been ascertained that the topographic and geological conditions are not favorable to construct such high dams. Case 2 is deemed not attractive in view of the investment cost and management of water resources and operation and maintenance of the project facilities. Eventually Case 3 is judged to be technically and economically the most promising and sound, and has been subsequently subjected to further detailed investigation and study.

4.3.2 Selection of Proposed Dam Scheme

The selection of the proposed dam scheme has resulted from technical and economical comparative study of the two alternative schemes. The comparative study has been elaborated in three steps : firstly water balance study, secondary preliminary design and thirdly preliminary cost estimate. Detailed procedures for the selection of the proposed dam scheme are wholly described in Annex F.

(1) Water balance study

In order to facilitate the selection of the most promising scheme among the two alternatives, a water balance study is a very essential tool. The study defines the storage capacity required to ensure the water demand at a specified dependability. The major elements of the study are :

- setting up of control points at appropriate locations of the rivers,
- estimation of the amount of the conservation flow to be maintained throughout the year,
- water balance calculation, and
- statistical treatment to define the active storage capacity of the reservoir.

The process of the study and schematic model are presented in Figs. 4.4 and 4.5 respectively. The water balance was simulated based on 5-day runoffs of the rivers during a 26-year period from 1952 to 1987 and the forecast 2015 raw water deficit. Three control points were arbitrary selected : one is at the Malewa dam site, the second at the Turasha dam site and the third at the stream gauge station 2GB1.

The conservation flow is required to preserve the existing water rights, aqua-ecology, river course, etc. and has been conservatively estimated in accordance with the prevailing criterion in consultation with MOWD. Further the expected water abstraction in the upstream reach of the control point is taken into consideration for the sake of safety. The conservation flow and upstream abstraction are as follows.

Control Point	Conservation Flow (m^3/sec)		Upstream
	Low Flow Period	Flood flow period	Abstraction(m^3/sec)
1	0.22	0.22	0.28
2	0.24	0.24	0.23
3	0.35	0.83	-

The required storage capacity is determined from the water balance calculation for every year during the simulation period. The statistical treatment was then deliberated to define the reservoir storage capacity to ensure the 2015 water demand for 24 out of 25 years. As a result, the net active storage requirement of the Malewa and Turasha reservoirs have been assessed to be 51.5 million m^3 and 50.0 million m^3 respectively as shown in Fig. 4.6.

(2) Layout design

In case of the Malewa Dam Scheme, the raw water transmission system will be aligned along that of the Stage 1 Project, while it must be laid down independently along the Malewa river in case of the Turasha Dam Scheme. For the purpose of economic comparison of the two alternatives, it is, therefore, indispensable to include the construction cost of the raw water transmission system in addition to the construction cost of the dam and trans-basin tunnel. The layout design was accordingly prepared with the aid of CAD for all those facilities based on the interim results of the geological investigation and construction material survey made available from the Phase 1 Study.

The layout designs of the dams and routes of the raw water transmission systems are shown in Figs. 4.7, 4.8 and 4.1 respectively. The principal features of the respective components resulted from the layout design are as presented in Table 4.6. It should, however, be noted that the layout designs of the dam and its appurtenant structures as well as the raw water transmission system presented herein are only for the purpose of the economic comparative study and those of the selected scheme have been explicitly upgraded incorporating more field investigation and survey results as explained in Chapters V and VI of this report.

(3) Preliminary cost estimate

The construction quantities have been calculated for the major work items for the purpose of the preliminary cost estimate. The construction costs of the Malewa and Turasha Dam Schemes have been estimated as presented in Table 4.7 and amount to Kshs. 1,578 million and 3,064 million respectively.

The Malewa Dam Scheme possesses great technical and economic advantages over the Turasha Dam Scheme as summarized below.

(1) Its construction cost has been estimated as less than the Turasha Scheme by 50 %.

(2) There so far has been observed no geological constraint in the foundation of the Malewa dam for construction of a rockfill dam with a height of 60 m, while it is unavoidable to construct a special and costly seepage cut-off beneath the Turasha dam to safely seal a highly permeable foundation.

(3) The water resources management could be rationalized as the raw water intake is integrated with the intake of the Stage 1 Project in the Turasha river and accordingly the operation and maintenance of the raw water transmission system could be saved to great extent in terms of cost, labour force and equipment.

(4) With conjunctive use of the runoffs of the Malewa and Turasha rivers, the height of dam is largely reduced : in terms of the required storage capacity, it is more than 90 million m^3 in the single dam development case and only 55.8 million m^3 in the Malewa Dam Scheme with the trans basin diversion tunnel.

It is, therefore, recommended to realize the water resources development by the Malewa Dam Scheme.

4.3.3 Optimum Development Scale of Malewa Dam

It is common practice that a dam is normally constructed at its optimum height to take advantage of the topographic, geological and water resources conditions prevailing over the site. It is obvious that heightening of a dam in the later stage is very costly and is sometimes technically difficult. Emphasis is, therefore, placed on identification of the optimal development height of the Malewa dam through the similar method as the selection of the damsite. Initially a storage - draft analysis was deployed, and followed by the preliminary design and cost estimate for the varied dam heights. A full description of these procedures is furnished in Annex F.

(1) Storage - draft analysis

The withdrawal from the reservoir is called the net draft rate, in other wards the water supply capacity, and varies proportionally with the active storage capacity of the reservoir. It is obtainable by means of either a mass curve analysis or a mathematical solution of the reservoir water balance. In the Study the later method has been adopted and a relationship between the reservoir active storage capacity and the draft was finally constructed as shown in Fig. 4.9, based on the results of the reservoir operation simulated by using 5-day runoffs during a 36-year period from 1952 to 1987. The net draft given has a dependability of 96 %.

(2) Preliminary design

The following six alternative dam heights were arbitrarily selected for the economic comparative study, and layout design of dam and its appurtenant structures was evolved also with the aid of CAD for the respective dam height by using the same interim results of the geological investigation and construction material survey as adopted in the selection of the dam scheme. It is technically not feasible to raise the height of the dam more than El. 2,160 m, owing to the topographic constraints on both abutments of the proposed damsite.

Description	Alternative Dam Heights					
	1	2	3	4	5	6
Reservoir storage ($10^6 m^3$)						
Active storage	9.16	16.32	26.62	43.88	55.92	78.52
Dead storage	15.88	15.88	15.88	15.88	15.88	15.88
Gross	25.04	32.20	42.50	59.76	71.80	94.40
MSL (El.m)	2,123.5	2,123.5	2,123.5	2,123.5	2,123.5	2,123.5
FSL (El.m)	2,130.0	2,134.0	2,139.0	2,145.0	2,149.0	2,155.5
Dam						
Crest (El.m)	2,135.0	2,139.0	2,144.0	2,150.0	2,154.0	2,160.0
Height (m)	50.0	54.0	50.0	65.0	69.0	75.0
Water supply capacity (m^3/day)	71,000	103,500	131,000	160,500	176,300	200.000

The economic comparison is based on the construction costs of the dam and its appurtenant structures, and the the trans-basin diversion system. The construction cost of the dam is largely dominated by the volumes of the dam excavation and embankment and spillway excavation. Characterized by the topographic condition, the former two increase proportionally with increasing dam height, whereas the later decreases drastically with increasing dam height as demonstrated in Fig. 4.10.

(3) Preliminary cost estimate

The construction cost of the respective alternatives was initially estimated at a preliminary level as shown in Table 4.8, and then construction costs per reservoir active storage and per water supply were calculated to assess the economic behavior of the respective alternative.

Description	Alternative Dam Heights 1	2	3	4	5	6
Reservoir active storage ($10^6 m^3$)	9.16	16.32	26.62	43.88	55.92	78.52
Dam						
Crest (El.m)	2,135.0	2,139.0	2,144.0	2,150.0	2,154.0	2,160.0
Embankment ($10^6 m^3$)	455.9	528.4	637.7	831.6	988.2	1,151.9
Water supply capacity (m^3/day)	71,000	103,500	131,000	160,500	176,300	200,000
Construction cost						
Total (10^6Kshs)	1,129	1,121	1,117	1,146	1,180	1,402
Per active storage (KShs/m^3)	123.3	68.7	42.0	26.1	21.1	17.9
Per water supply(KShs/m^3/day)	15,901	10,831	8,527	7,140	6,693	7,010

The initial cost and the construction cost per reservoir active storage/water supply have been related to the dam crest elevation as shown in Figs. 4.11 and 4.12 respectively, in order to easily find out the economically most optimum dam height.

The alternative 5, dam crest at El. 2,154.0 m, indicates the lowest construction costs per water supply among all the alternatives. It also makes it possible to cater for the required water demand at the specified dependability, unless there is no environmental constraint. It is, therefore, concluded that the Malewa Dam with the crest at El. 2,154.0 m is technically and economically the most optimum plan.

4.4 Formulation of Water Supply Scheme

4.4.1 Component of Water Supply Scheme

In line with development scenario stated in Section 4.1, the water supply scheme will be planned and designed at preliminary level. It includes the following components :

(a) Raw water transmission system, including intake in the Turasha river;
(b) Water treatment works ;
(c) Nakuru treated water transmission system from the treatment works to Nakuru;
(d) Naivasha treated water transmission system from treatment works to Naivasha;
(e) Gilgil rural supply system;
(f) Eburu rural system;

(g) Bulk supply systems in Gilgil such as KMB, ASTU, and GMB-NYSTC.

The planning and design of the reticulation systems are not included in the Study. All the above water supply facilities are to be designed based on the daily maximum discharge. The required transmission and /or treatment capacity of those facilities have been calculated as given in Table 4.9.

4.4.2 Optimum Development Phasing

The development concept of the water supply scheme may be qualitatively different from the that of the dam scheme. The water supply facilities could in general be completed within a relatively short period so that they can be implemented in a stage-wise way in consistent to a growing water demand. This will relieve the executing agency from a heavy financial overburden and will finally bring about high economic rate of return.

A comparative study was also evolved to indicate the optimum development phasing of the water supply scheme. The facilities involved in the Study are the raw water transmission system, the treatment works, and both the Nakuru and Naivasha treated water transmission systems. The other facilities were disregarded, since they are deemed to be minor structures and not influential in choosing the optimum plan.

Two development sequences have been examined : one in two phases and the other in three phases. The development sequences of the respective component are as presented in Figs. 4.13 through 4.16 and are summarized below.

Description	Case A Stage 2-1	Case A Stage 2-2	Case B Stage 2-1	Case B Stage 2-2	Case B Stage 2-3
Year of Commissioning	1997	2004	1997	2004	2011
Raw water transmission system(m^3/day)	102,500	102,500	102,500	51,250	51,250
Treatment works(m^3/day)	100,000	100,000	100,000	50,000	50,000
Nakuru treated water transmission(m^3/day)					
Treatment works - Gilgil	82,520	89,550	82,520	44,775	44,775
Gilgil - R6 reservoir	71,740	76,450	71,740	38,225	38,225
Naivasha treated water transmission (m^3/day)	16,700	9,900	16,700	4,950	4,950

The layout designs were considered for all the above components under consideration as summarized in Table 4.10. Of the raw water transmission system, intake works, composed of an inlet, a short tunnel and a desilting work, were planned to be constructed with a full discharge capacity at the initial stage for the all the cases. The construction costs were estimated based on the layout designs at the preliminary level as given in Table 4.11 and the annual operation and maintenance cost was also estimated.

In a case study as the present, the least cost solution is normally selected as the optimal plan. All the investment costs and the annual O&M costs were, therefore, converted into present worth at three different discount rates, i.e., 8, 10, and 12 per cent per annum, for the comparison. The sum of the present worth of the respective component was calculated as given below.

Components	Case A (Kshs. million)			Case B (Kshs.million)		
	8 %	10 %	12 %	8 %	10 %	12 %
Raw water transmission	134.1	115.9	100.9	138.7	117.9	101.3
Treatment works	503.9	416.3	351.4	502.6	411.6	345.2
Nakuru treated water transmission	631.6	548.6	481.0	652.4	558.0	483.7
Naivasha treated water transmission	150.2	130.1	113.4	155.0	132.3	114.2

It is clarified that the least cost sequence is a combination of the three-stage development in the treatment works and the two-stage development in the three water transmission systems. It is, therefore, recommended that the water supply scheme is to be implemented along with this least cost sequence.

4.4.3 Route and Material for Water Transmission System

The Project will include the construction of a number of the lengthy water transmission systems, which will consume not only a large portion of the initial investment but also of the annual O&M cost of the Project. It is, therefore, a key element to design the water transmission systems with gravity flow as much as possible along a route which is as short as possible, and to define the most economical pipe materials.

Both the raw water intake and treatment works have been proposed to be sited at the same locations as those of the Stage 1 Project. Therefore both the raw and Nakuru treated water transmission systems as well as three bulk supply systems in Gilgil are naturally proposed to be installed along those of the Stage 1 Project. The routes of the Naivasha treated water system and the three rural supply systems are the same as those selected by the 1985 Preliminary Design Report. DWG.2 shows the alignment of the all the water transmission systems. All the water transmission systems, excluding the Gilgil East and West rural, Eburru rural and ASTU bulk systems, have been possible to design for gravity flow. Those four systems will need pumping facilities owing to the topographic conditions.

The purchase cost of the pipe shell will be the largest item of construction cost of the water transmission systems. The raw water and both the Nakuru and Naivasha treated systems receive a relatively high hydrostatic pressure, nearly 20 bars at the maximum in case of the Nakuru system. Accordingly the pipe material shall be chosen for its technical and economic feasibility. DCI, GFR and steel pipes technically meet the design requirements. Although PCP is widely used for the water transmission system, it is judged to be not adaptable in view of allowable stress. The construction costs of the three compatible pipe materials were compared as indicated in Fig. 4.17. The cost includes the pipe shell, jointing and installation. As is clearly seen from the figure, the steel pipe is the least cost one of the three and is virtually defined as the most technically and economically sound one. UPVC pipes are, however, proposed to be adopted for distribution pipes with smaller diameters.

4.4.4 Service Reservoirs

The service reservoirs will be strengthened to meet the hourly peak demand and to store adequate quantity of water for fire extinguishment. Their locations have been selected in due consideration of the existing water supply systems and/or in consistent to the design principles of the Stage 1 Project. The Nakuru municipality will be served through 7 reservoirs and the town of Gilgil will be directly fed by the Central Reservoir, which will also cater for the four bulk supplies in Gilgil. In Naivasha town, there are two services reservoirs at different locations and their storage capacity is ascertained in need of expansion for the proper water distribution. For both the Gilail and Eburu rural areas, a number of the service reservoirs will be newly constructed in mid course of and at the terminal of the pipelines.. The locations of the proposed service reservoirs are shown in DWG.2.

The service reservoir will be expanded along with the proposed phased development of the treated water transmission systems. The expansion plan has been worked out taking into account the forecast water demand, the existing storage capacity, the expansion by the Stage 1 Project, and the proposed treated water transmission expansion schedule as shown in Table 4.12. Fig. 4.18 shows the capacities of the water transmission systems and the required water demand by the proposed phasing development.

4.5 Micro Hydroelectric Power Generation

There is the possibility to generate hydroelectric power by harnessing the hydraulic head between the Malewa reservoir and the Turasha river and the water to be diverted into the Turasha river Fig. 4.19 shows a flow duration curve of the diversion discharges.

The facilities for the hydroelectric power is composed of a head tank, a penstock and a power station. The transmission line will be also constructed to connect to the existing Gilgil substation from the power station. The principle feature of the development plan are described below.

- Head Pond
  - Operation level : El. 2,118.8 m
- Penstock
  - No : 1
  - Length : 14.5 m
  - Diameter : 1.2 m

- Power station

Turbine	: Cross flow type, 1 no.
Max. plant discharge	: 2.3 m^3/sec.
Net head	: 11.1 m
Tailwater level	: El. 2,105.0 m
Installed capacity	: 200 KW
Annual energy output	: 427 x 10^3 KWh

- Transmission line : 20 km

Based on the preliminary layout and design, incremental cost is estimated at Kshs. 32,390 x 10^3, corresponding to Kshs. 162,000/KW and Kshs. 7.9/KWh.

A preliminary evaluation was made by comparison with the energy value of the alternative diesel power plant. It is judged that this micro hydropower scheme is not economically attractive. Further, the micro power plant can be operated only 220 days annually, the reliability is very low. Therefore the micro hydro-electric power generation is not taken into consideration in the proposed development.

4.6 Overall Development Sequence

The Project has been designated as Stage 2 of the Greater Nakuru Water Supply Project, Eastern Division and comprises two main components, namely, Malewa Dam Scheme and Water Supply Scheme. Their optimum development scale and development sequence have been revealed through the elaboration of the various technical and economic comparative studies as aforementioned. Characterized by the optimum phasing development, the Project has been recommended to be implemented in three stages as summarized below.

(1) Stage 2-1 (Expected commissioning year : 1997)

- Malewa dam with the trans-basin diversion tunnel
- Raw water transmission system, Phase 1, conveyance capacity of 102,500 m^3/day
- Water treatment works, Phase 1, treatment capacity of 100,000 m^3/day
- Nakuru treated water transmission system, Phase 1, conveyance capacity of 82,520 m^3/day, including the Central Reservoir and 7 reservoirs in Nakuru.
- Naivasha treated water main, Phase 1, conveyance capacity of 16,700 m^3/day, including two reservoirs in Naivasha

- Bulk supply system to KMB, conveyance capacity of 2,030 m^3/day, including a reservoir

- Bulk supply system to GMB-NYSTC, conveyance capacity of 7,060 m^3/day

- Gilgil West rural supply system, Phase 1, conveyance capacity of 780 m^3/day, including two reservoirs

- Gilgil East rural supply system, Phase 1, conveyance capacity of, 780 m^3/day including three reservoirs

- Eburu rural supply, Phase 1,conveyance capacity of 3,700 m^3/day including 9 reservoirs

(2) Stage 2-2 (Expected commissioning year : 2004)

- Raw water transmission system, Phase 2, conveyance capacity of 102,500 m^3/day

- Treatment works, Phase 2, treatment capacity of 50,000 m^3/day

- Nakuru treated water transmission system, Phase 2, conveyance capacity of 89,550 m3/day, including the Central Reservoir and 7 reservoirs in Nakuru :

- Naivasha treated water main, Phase 2, conveyance capacity of 9,900 m^3/day, including two reservoirs in Naivasha

- Bulk supply system to ASTU, conveyance capacity of 1,140 m^3/day, including a reservoir

- Gilgil East rural supply, Phase 2, conveyance capacity of 550 m^3/day, including 3 reservoirs

- Gilgil West rural supply, Phase 2, conveyance capacity of 550 m^3/day, including 2 reservoirs

- Eburu rural supply, Phase 2, conveyance capacity of 2,220 m^3/day, including 9 reservoirs

(3) Stage 2-3 (Expected commissioning year : 2011)

- Water treatment works, Phase 3, treatment capacity of 50,000 m^3/day

- Nakuru treated water transmission system, Phase 3, construction of the Central Reservoirs and 7 reservoirs in Nakuru

- Naivasha treated water transmission system, Phase 3, construction of 2 reservoirs in Naivasha

V. PRELIMINARY DESIGN OF MALEWA DAM

5.1 Site Conditions

5.1.1 Topographic Condition

The Malewa dam site is characterized by very steep topography ; both banks of the river form cliffs which an almost vertical. Both the banks become flat above the top of cliffs. Riverbed elevation and top of the cliffs are approximately El.2,083.3 m and El. 2,140 m respectively. The width of the gorge is only 190 m at the tops of the cliffs. Especially the left bank above El. 2,140m is characterized by a narrow ridge. This topographic condition determines the maximum height of the dam.

5.1.2 Geological Condition

The geological investigation including the test drilling and bore mode permeability testing was conducted during the period of the Study. The results of the investigation are reported in detail in Annex B. The geological conditions of the reservoir area and damsite are briefly summarized below.

The geological maps of the reservoir and damsite are presented in Figs. 5.1 and 5.2 respectively. The terrain of the Malewa reservoir and dam site area is composed of a series of pyroclastic rocks which are stratigraphically classified as members of a Pliocene Kinangop Tuffs, comprising welded tuffs and massive tuffs interbedded with the lake sediments, being covered with colluvial deposits, the Quaternary overburden on the flat slopes. Of all the young volcanic and pyroclastic bedrocks in this area, the Pliocene lake sediments are the weakest from foundation engineering view point. This sediments are deemed empirically, however, still stable enough for foundation of fill-type dams of some 60 m in height, if it is not weathered to more than moderate grade.

The geological profile along the proposed dam axis is shown in Fig. 5.3. Both the overburden and highly weathered rock zone are thin, excepting at the foot of the slopes on the both banks. Competent bedrock for the dam foundation can be reached at the depth of several metres from the present ground surface in the dam abutments, while approximately excavation will be required to approximately 8 m below the ground surface for foundation of impervious earth core of dam in the river bed section in order to remove colluvial overburden and loose zone of the lake sediments. The depth of excavation in dam shell zone foundation would be about 5 m in the river bed and 2 m in the abutments.

The field permeability tests in the boreholes show that the permeability in fresh rocks and slightly to moderately weathered rocks is usually 5 Lugeon or less in this site, with a few exceptions. Higher Lugeon values over 10 are only occasionally observed near boundaries of the lake sediments. Ordinary treatment with curtain grouting will be sufficient for underseepage cut-off. Considering the unusually low groundwater table on both banks, it is recommended to extend the grout curtain deep into both abutments with rim grouting.

No sign of significant land sliding, existing or potential, has been found in the reservoir area. The slopes are, however, subject to considerable erosion, particularly below cultivated lands and barren lands. Minor collapses of the slopes are seen being incurred by this sort of surface erosion. It is difficult to eliminate the possibility of minor local landsliding to occur in the colluvial deposits on the slopes around the level of the reservoir water level, but such sliding, if occurs, will be of too small scale to result in any practical damage. Any potential passage for substantial water leakage from the reservoir has not been found.

5.1.3 Construction Materials

As reported in detail, various field investigation and reconnaissances were executed under the Study over a vast area to find out adequate embankment materials: As the results, one borrow area and two quarry sites were finally selected. Their locations are shown in Fig. 5.4.

General characteristics of the borrow area and quarry sites are as follows:

Borrow Area	Work Items	Classification	Available Quantities
East Road borrow area	Core zone	Lateritic soils	300,000 m^3
Kipipiri Road quarry site	Outer shell-1	Welded tuffs	1,000,000 m^3
South Gilgil quarry site	Outer shell-2 Filter zone Concrete aggregates	Trachyte	Sufficient quantity

(1) Earthfill material

The "East Road" borrow area is situated close to the Gilgil golf course, about 3.5 km east from the Gilgil town. The distance to the Malewa damsite is approximately 17 km through the existing Kipipiri road and rural roads. Both residual and lateritic soils, originated from tuffaceous bedrocks, are the predominant type of soil. This area covers about 200 ha (200 m x 1,000 m) and the available core materials lie with 1.5 m in thickness below the 0.5-m-thick top soil. The available quantity is estimated at approximately 300,000 cu. m. There is a little variation especially in gradation (soil classification) from place to place and seasonal moisture content. The gradation and moisture adjustment are essential for construction of the impervious core.

(2) Rock embankment material

The "Kipipiri Road" quarry site is selected for rock embankment materials for outer shell-1 zone. The site covers conspicuous hills formed by welded tuff and high degree welded tuff. A distance to the Malewa damsite is approximately 5 km through the Kipipiri Road and rural road.

The "South Gilgil" quarry site is selected for rock embankment materials for outer shell-2 zone. The site is located in the southwest rim of the Gilgil town and presently rock quarrying is operated by the Ministry of Public Works. The site is formed with steep cliff of trachytes. A distance to the Malewa damsite is approximately 23 km through the Kipipiri road and rural roads.

(3) Concrete aggregate materials

As a result of field survey, it was found that no sand and gravel deposits are available within reasonable hauling distances of the Malewa damsite. Therefore, it is proposed to produce coarse and fine concrete aggregates by crushing and mixing trachyte quarried from the "South Gilgil" quarry site.

5.2 Permanent Access Road

Permanent access roads to the Malewa damsite are comprised of the National Highway Route A104, the Kipipiri road and rural road to be improved. The existing Kipipiri road including bridges requires minor improvement works for some portions prior to commencement of the construction works. After branching from the Kipipiri road, much improvement works such as widening and repairing will be required for the rural road. The distance between the branch point and the damsite is approximately 2.5 km. A feeder road with a distance of aproximately 800 m is constructed to reach to the intake of trans-basin tunnel. The location of the permanent access road is shown in DWG. 4

5.3 River Diversion

5.3.1 Design Condition

Diversion tunnels and cofferdams are provided for the purpose of diverting riverflow during the period of the construction works of the main dam. The diversion tunnels are provided on the left abutment as shown in DWG. 5, considering the flow characteristic of the river and topographic conditions.

Two lines of diversion tunnels would be provided, because a river outlet facility will be installed in the No.2 tunnel for discharging river maintenance flow.

A 20-year probable flood with a peak discharge 240 m3/sec was adopted as the design flood discharge for the diversion scheme without the effect of the flood routing in the reservoir. At that time, the downstream outlet water level is estimated at El. 2,089.00 m from the Fig. 5.5.

In designing the diversion tunnel, the maximum velocity should be restricted to less than 15 m/sec for the tunnels lined with reinforced concrete.

5.3.2 River Diversion Scheme

The optimum diversion scheme is elaborated in terms of the sum of the construction cost of upstream cofferdam embankment and diversion tunnels. In the case of the Malewa dam, however, since a substantial portion of the cofferdam embankment is included in the main dam, it is more economical to construct the cofferdam as high as possible resulting in reducing a diameter of diversion tunnel. Therefore the diversion tunnels could be designed as small as possible within the limitations of the allowable flow velocity in the tunnel and constructed within one dry season.

In order to find the least cost solution, a relationship between the cofferdam crest elevation and the total embankment volume was constructed as shown in Fig. 5.6. Finally, the upstream cofferdam with the crest at El. 2,115.00 m is selected, considering the maximum possible embankment volume to be constructed within a given period and in due consideration of relative location of the cofferdam slope toe and the main dam core trench.

The diameter of the diversion tunnels was determined with consideration to the upstream reservoir water level and flow velocity in tunnel. The reservoir water level is calculated by the following equations :

R.W.L. = O.W.L + he

he = $(1.0 + f_e + f_{sr})$ V2/2.0/g

where,

R.W.L	: Reservoir water level (El. m)
O.W.L	: Outlet water level (El. m)
he	: Loss head in tunnel (m)
fe	: Coefficient of entrance loss (= 0.2)
fsr	: Coefficient of friction loss
	$(= f\,L/D = 124.5\,n^2\,L\,/\,D^{4/3})$

where,

n	: Manning's coefficient (= 0.018)
L	: Tunnel length (m)
D	: Tunnel diameter (m)

The outlet water level is estimated at El. 2,089.00 m for the design discharge of 240 m^3/s from the rating curve shown in Fig. 5.5.

Finally, the optimum river diversion scheme is determined as follows :

- Diameter of diversion tunnel : D = 3.65 m
- Elevation of cofferdam : El. 2,115.00 m

In this diameter, the maximum velocity in the diversion tunnel is estimated at 11.5 m/sec.

5.4 Main Dam

5.4.1 Dam Crest Elevation

As demonstrated in Chapter IV, the proposed reservoir has FSL at El. 2,149.00 m and MSL at El. 2,123.50 m.

In accordance with the ICOLD criterion the crest elevation of the dam was designed to be safe against the flood discharge, water waves by wind or earthquake and other allowances. This design was made so that the maximum water level thus estimated should not overtop the top of the impervious core zone, being set 0.5m below the dam crest elevation for the road pavement. Hence, the dam crest elevation is determined at El.2,154.00 m by adding a freeboard of 5.00 m to FSL 2,149.00 m:

The required freeboard is calculated as follows:

- Surcharge by design flood above FSL	:	2.83 m	
- Wind wave runup	:	0.64 m	
- Allowance for fill dam	:	1.00 m	
- Road pavement	:	0.50 m	
Total (Freeboard)		4.97 m	≑ 5.00 m

5.4.2 Foundation Treatment

The foundation for the core and filter zones are trench-excavated upto the firm rock with about 5 m in depth below the existing riverbed. Furthermore, a grout gallery is provided in the foundation for expediting of the embankment works and for the sake of countermeasure against future leakage problem whenever necessary in future.

At this feasibility stage, the following grouting works are taken into account.

(1) Blanket grout : on 3 m grid and 5 m in a depth, at 3 m intervals in 4 rows and 10 m in a depth under the core zone

(2) Curtain grout : at 2 m intervals in 3 rows and 30 to 70 m in a depth

(3) Rim grout : at 2 m intervals in 3 rows and 30 to 70 m in a depth

5.4.3 Zoning

The final output of the design of the main dam is shown in DWGs. 5 and 6. The height of the proposed dam would be 80 m at the maximum section above the bottom of the foundation and the crest length is 360 m. The width of the crest is 10 m. The upstream and downstream slopes are 1 : 2.70 and 1 : 2.20, respectively. The dam cross section is zoned by 5 different embankment materials. The features of each zone are detailed below :

(1) Impervious core zone

The impervious core materials would be both residual and lateritic soils, being obtained from "East Road" borrow area. The core zone has a width of 4.0 m at the top and is sloping by 1 : 0.25 both for the upstream and downstream. The width of the core becomes about 44 m at the bottom, having a creep length of 55 % of the maximum water pressure.

(2) Shell zone

The shell zone is divided into three zones, i.e., inner shell, outer shell-1 and outer shell-2, from the viewpoint of economy and availability of construction rock materials. The inner shell zone is of selected excavated rock materials from the main dam foundation and spillway. This zone is arranged only in the downstream slope in view of the slope stability.

The outer shell-1 zone is formed by welded tuff and high degree welded tuff, quarried from the Kipipiri Road quarry site.

The outer shell-2 zone is trachyte which has sufficient strength as rock embankment materials. This zone is arranged in the upstream slope with 10 m in thickness to ensure the surface slope stability and in the downstream slope with 2 m in thickness for the prevention of weathering of the outer shell-1 zone.

(3) Filter zone

Between the core and the shell zones, a filter zone is provided to prevent erosion of the core materials. The materials will be produced by crushing and processing trachyte rocks in the South Gilgil quarry site. The filter zone is designed at 2 m in thickness at the top and with a slope of 1: 0.30 in both the upstream and downstream sides.

Below the downstream shells, there a 5-m-thick drain is to be provided in due consideration of the low permeability of the embankment materials and the seepage line in the downstream zone for the slope stability. Drain materials are the same as the filter zones.

The dam embankment volumes based on the proposed design are as summarized below:

Zone	Volume(m^3)	Materials
(1) Main dam		
- Core	166,900	Lateritic soils
- Filter	110,900	Crushed and processed trachyte rocks
- Inner shell	59,100	Selected excavated rock materials
- Outer shell-1	516,200	Quarried welded tuff
- Outer shell-2	148,100	Quarried trachyte
Subtotal	951,200	
(2) Cofferdam		
- Core	16,700	Lateritic soils
- Filter	8,900	Crushed and processed trachyte rocks
- Outer shell-1	50,200	Quarried welded tuff
- Outer shell-2	48,700	Quarried trachyte
Subtotal	124,500	
Total	1,075,700	

5.4.4 Stability Analysis

Safety of the dam was examined in terms of slope stability using the slip circle method and plane failure surface method. Design values used for the analysis are as tabulated below:

Items	Unit	Riprap	Outer Shell	Inner Shell	Filter	Core	Foundation Dam	Foundation Spillway
Unit weight, dry	ton/cu.m	1.85	1.22	1.22	1.85	1.29	–	–
Unit weight, saturated	ton/cu.m	1.90	1.25	1.25	1.90	1.60	–	–
Internal friction angle	degree	40.00	38.00	35.00	35.00	29.50	–	–
Cohesion	kg/sq.cm	0.00	0.00	0.00	0.0	0.95	–	–
Permeability	cm/sec	Free	5.0×10^{-3}	1.0×10^{-3}	5.0×10^{-4}	1.0×10^{-5}	1.25×10^{-4}	1.59×10^{-4}

(1) Plane failure surface method

The factor of safety of an infinity long cohesionless soil slope against surface sliding is calculated by the following equation :

$$SF = (m - k.g) / (1 + k.g.m) \tan(a)$$

where,

SF	:	Factor of safety
m	:	Slope (height/unit horizontal length)
k	:	Horizontal seismic coefficient (= 0.10)
g	:	Saturated density / submerged density for submerged slope and 1.0 for slope which is not submerged

For the upstream slope,

$$SF = (2.7 - 0.1 \times 1.9 / 0.9) / (1 + 0.1 \times 1.9 / 0.9 \times 2.7) \tan(40) = 1.330 > 1.2$$

For the downstream slope,

$$SF = (2.2 - 0.1 \times 1.0) / (1 + 0.1 \times 1.0 \times 2.2) \tan(38) = 1.345 > 1.2$$

(2) Slip circle method

Safety factors against slip circle were analyzed by the aid of electronic computer. The results of study are summarized below, and shown in Fig. 5.7.

Cases	Seismic Coeff.	Static Condition U/S slope	Static Condition D/S slope	Seismic Condition U/S slope	Seismic Condition D/S slope
FSL	0.10	2.194	1.520	1.219	1.218
Reservoir Empty	0.05	2.222	1.781	1.896	1.781
Rapid Drawdown	0.05	1.678	1.809	1.338	1.586

As seen in the above table, the safety requirements are satisfied in each case as the resultant safety factors are more than the minimum required safety factor 1.20. Therefore, it is concluded that the Malewa dam is safe against the slope failure.

5.4.5 Seepage Analysis

Seepage flow which percolates through the dam and foundation was analyzed using the finite element method. The design values adopted are reported in the sub-section 5.4.4 of this report. The analysis was made for two sections: one is the main dam typical cross section and the other is the left abutment mountain ridge section near the spillway. The results are as shown in Fig. 5.8 through Fig. 5.13.

As a result of the seepage analysis, the maximum velocity in the foundation rock are obtained a follows:

- Dam foundation $V_{max} = 1.87 \times 10^{-4}$ cm/sec
- Spillway foundation $V_{max} = 7.52 \times 10^{-5}$ cm/sec

Possibility of piping through the foundation are checked by the Justin's equation against the above maximum velocity. Considering the foundation component as silt, the critical velocity (Vcr) is calculated as $V_{cr} = 7.25 \times 10^{-1}$ cm/sec for a diameter of a silt particle at 0.000.5 cm and specific gravity at 2.6 g/cm^3. It is, therefore, concluded that piping would not occur through dam and spillway foundation because of $V_{max} < V_{cr}$.

5.5 Spillway

5.5.1 Design Condition

The spillway is basically designed to discharge a 1,000-year probable flood with a peak discharge of 960 m^3/sec. For the sake of free operation and maintenance, it is predetermined to be ungated and is located on the left abutment to suit the topography as shown in DWG 5. It consists of ungated weir, open chuteway and stilling basin with horizontal apron.

5.5.2 Optimum Layout

(1) Type of Spillway

In view of the topographic condition on the left bank of the damsite, it would be possible to adopt either the side spillway or fan-shaped spillway. A comparative study was therefore carried out for two alternatives : one is a side spillway and the other is a fan-shaped spillway. Layouts for the two alternatives are shown in Fig. 5.14 and Fig. 5.15. The cost comparison was made for the ungated weir length of 101 m as summarized below :

(Unit : K shs.10^6)

Item	Side-spillway	Fan-shaped spillway
Excavation	110.7	97.1
Concrete	125.7	125.6
Others	0.6	0.6
Total	236.4	223.3

The fan-shaped spillway is virtually selected as is superior to the side spillway in terms of construction and hydraulic stability.

(2) Length of weir

In case of the ungated spillway, the height of the dam varies with the length of the ungated weir. A further comparative study is, therefore, made to define the length of the ungated weir. The total cost of the dam and spillway is estimated for four cases as follows :

	Length of Ungated Weir (m)			
	64.0	80.0	101.0	135.0
Dam crest elevation (El. m)	2,155.00	2,154.50	2,154.00	2,153.50
Construction cost (10^6 Kshs)				
Dam	411.2	379.5	363.8	351.1
Spillway	191.3	209.6	223.3	257.0
Total	602.5	589.1	587.1	608.1

The least cost plan is the case of dam crest at El.2,154.00m and ungated weir length 101.0m. The resultant flood water level is at El. 2,151.83 m.

5.5.3 Hydraulic and Structural Designs

The 1,000-year probable flood with a peak inflow of 960 m3/sec was adopted as the design flood of the spillway, while the safety of the dam was further checked by the PMF (probable maximum flood) with the retardation effect of the reservoir.

The flood routing studies were carried out to check the safety of main dam at the times of the PMF discharges. The results of the computation are shown in Fig. 5.16. It shows that the PMF can be discharged through the spillway safely within the freeboard.

The chuteway was designed to have two slopes to save the excavation cost taking into account the topographic conditions. The slopes are determined at 1 : 20.0 for the upstream part and 1 : 2.0 for the downstream part. The width of the chuteway is 20 m obtained from the nonuniform flow hydraulic analysis. The results of the calculations are as shown in Fig. 5.17. The maximum velocity is estimated as 32.3 m/sec for the 1,000-year probable flood and 34.4 m/sec for the regulated PMF.

A 100-year probable flood with a peak discharge of 460 m3/sec was applied, for design of an energy dissipator. A stilling basin type was adopted from the hydraulic viewpoint. It consists of a 80 m long horizontal apron with 15-m-high walls.

5.6 Trans-basin Diversion Tunnel

5.6.1 Design Condition

The trans-basin diversion tunnel consists of an intake in the Malewa reservoir, an trans-basin diversion tunnel and an outlet in the Turasha river. The waterway has a discharge capacity of 2.30 m^3/sec, which is the maximum rate of flow to be diverted into the Turasha river according to the result of the water balance calculation. A general layout is shown in DWGs 4 and 9.

5.6.2 Designs of Intake, Tunnel and Outlet

(1) Intake

The intake will be located on the left bank and comprises an inlet, a vertical shaft and a valve chamber. In order to regulate the diversion flow, a hollow jet valve with 0.5 m in a diameter as a flow regulating device is installed at immediately downstream from the inlet. The inlet will be of shaft type, taking into accont the topographic characteristics.

(2) Tunnel

The trans-basin tunnel is driven through the mountain range forming the water division between the Malewa river and the Turasha river and have a total length of approximately 2,420 m. Its diameter is 1.80 m, minimum diameter in view of tunnel construction works. The minimum operation level in the Malewa reservoir is El. 2,123.50 m, while the normal water level in the Turasha intake is El. 2,103.20 m. The water released from the reservoir is diverted into the Turasha river by harnessing a hydraulic head of 20.30 m between the reservoir and the Turasha intake. Slope of the tunnel is 1 to 1,000. The free flow type tunnel was selected from the economical viewpoint. Hydraulic characteristics in the tunnel are shown in Fig. 5.18. The water depth and velocity are calculated at 1.16 m and 1.11 m/sec respectively, for the design discharge of 2.3 m^3/sec.

(3) Outlet

At the end of the tunnel a step-wise energy dissipator is constructed to smoothly evacuate the flow into the Turasha river and to protect the slope of the river bank against slope erosion. The details are shown in DWG.9.

5.7 Building Works

For the purpose of operation and maintenance of the dam and trans-basin diversion tunnel, a control house, a warehouse, an emergency generator house, a valve control house and two Type D staff quarters are constructed.

The locations of those buildings are as shown in DWGs 4. and 5.

5.8 Metal Works

The major items of the metal works gates, stoplogs, trashracks, and valves as per listed below :

(a) Diversion gate

Type	:	Roller gate
Number	:	1 set
Dimensions	:	3.65 m wide x 3.65 m high

(b) River outlet facilities

Type	:	Hollow jet valve and ring roller gate
Number	:	1 set
Dimensions	:	0.5 m diameter

(c) Intake trashrack

Type	:	Fixed screen
Number	:	4 sets
Dimensions	:	2.0 m wide x 2.5 m high

(d) Intake valve

Type	:	Hollow jet valve and ring follower gate
Number	:	1 set
Dimensions	:	0.75 m diameter

VI. PRELIMINARY DESIGN OF WATER SUPPLY SCHEME

6.1 Raw Water Transmission System

6.1.1 Design Condition

The raw water transmission system will be installed between the intake in the Turasha river and the water treatment works in Gilgil. It has been recommended to be implemented in two stages, namely, Stages 2-1 and 2-2. The design discharge is 102,500 m^3/day for all stages, including a conveyance and treatment loss of 5% of the daily maximum treated water requirement.

The hydraulic design of the raw water transmission system is in accordance with the Design Manual and topographic maps at a scale of 1 to 5,000, which were made available from NWCPC. In particular the head loss calculation due to the pipeline has been based the Colebrook - White formula.

6.1.2 Intake

The intake is composed of an inlet, a tunnel and a desilting works as shown in DWG.10, and is sited at approximately 30 m upstream from the intake of the Stage 1 Project. It has been proposed to be constructed with a full intake capacity of 205, 000 m^3/day at the initial stage, namely, Stage 2-1.

The Malewa reservoir diverts the water into the Turasha river through the trans-basin diversion tunnel, when the Turasha river runoff runs short of the raw water demand. It is, therefore, required to hold a certain amount of storage capacity in the Turasha river in order to offset a time lag of water supply from the reservoir. The travelling time from the reservoir is estimated at about half an hour, while there is a storage capacity of about 35,000 m^3 between the spillway crest level of the intake dam (El. 2,103.2 m) and the sill of the inlet (El. 2,102.0 m) as shown in Fig. 6.1. This available storage, corresponding to 3.8 times of the maximum daily demand, is sufficient enough to cover the time lag.

The normal intake level is set at El. 2,103.2 m, the same level as the Stage 1 Project. The inlet is divided into two compartments, each 1.2 m in height and 2.5 m in width. A trash rack will be installed at the front of the inlet.

6.1.3 Tunnel

The tunnel with a length of 190 m is driven through a narrow ridge and is capable of carrying 2.30 m^3/sec. As shown in DWG.11 it runs in parallel with that of the Stage 1 Project and is of the semi-circle shape with a height of 1.8 m and a width of 1.8 m. The invert of the tunnel is sloped at a gradient of 1 to 1,500. The tunnel will be wholly lined with concrete for its entire length.

6.1.4 Desilting Work

In order to prevent the sediment inflow into the pipeline, the desilting work is proposed at the outlet of the tunnel. It is designed based on the conditions that all the sediment particles larger than 0.2 mm be settled in and disposed out of the desilting works. As presented in DWG.12 the sand settling basin is divided into two chambers by a partition wall. Each chamber is 14.0 m in a length and 4.0 m in a width and is furnished with a bulkhead scouring sluice to flush out the sediment deposit hydraulically.

In order to remove rubbish, trash and other foreign floating matters, screen is installed at the end of the basin. The regulating gate is installed on the respective pipeline for the convenience of the operation and maintenance.

6.1.5 Raw Water Mains

Two raw water mains, each having transmission capacity of 102,500 m^3/day (1.19 m^3/sec), are aligned in parallel with that of the Stage 1 Project as shown in DWG.13. The total length of the raw water main is 9,400 m. The optimum configuration of each raw water main has been identified through an economic comparative study as summarized below.

(a) Given design condition

Normal level in desilting works	:	El. 2,102.5 m
Water level in receiving well	:	El. 2,077.5 m
Pipe material	:	Steel pipe

(b) Pipeline configuration

D1,000 mm	:	6,800 m
D900 mm	:	2,600 m

As shown in the hydraulic profile in DWG.14 , the hydraulic gradient is 2.20 per mill along the D1,000 mm pipeline and 3.85 per mill along the D900 mm pipeline. On each pipeline air valves and washouts are installed at 13 locations respectively. The air valve is 100 mm in a diameter and the washout 300 mm in diameter. In addition there are river - crossing at 8 locations and a road - crossing at one location along the pipeline.

6.2 Water Treatment Works

6.2.1 Basic Configuration and Design Values

In accordance with the optimum development phasing the water treatment works will be constructed in three stages and the production capacity has already been defined as 100,000 m^3/day in the Stage 2-1 and 50,000 m^3/day in the Stages 2-2 and 2-3 respectively. This capacity is sufficient enough to meet the maximum daily demand throughout the planning horizon. For the convenience of construction, operation and maintenance, it is deemed beneficial to adopt a treatment unit with a treatment capacity of 50,000 m^3/day. Therefore two units will be completed in the Stage 2-1.

The same treatment method as the Stage 1 Project, a rapid sand filtration is selected in view of the water qualities of both the Malewa and Turasha rivers. The major components of each treatment unit consist of receiving well, rapid mixing and flocculation chamber, sedimentation basin, filter, clear water reservoir, sludge lagoon, wash water pond and high level tank.

The basic configuration and design values of the treatment unit are as given below, which are the same as those of the Stage 1 Project.

(a) Receiving well

Retention time	:	Approximately 1.5 min.

(b) Rapid mixing chamber

Type	:	Hydraulic
Mixing head	:	About 0.8 m

(c) Flocculation chamber

Type	:	Hydraulic, vertical flow
G value	:	45sec^{-1}

GT value	:	65,000
Velocity in channel	:	0.1 - 0.3 m/sec
Velocity in orifice	:	0.6 m/sec at maximum
Retention time	:	20 -25 min.

(d) Sedimentation basin

Type	:	Conventional, horizontal flow basin
Surface loading rate	:	1 $m^3/m^2/hr$
Desludging method	:	Manual

(e) Filters

Type	:	Conventional, constant flow rate, with backwashing and surfacing washing
Filtration rate	:	5 $m^3/m^2/hr$
Backwash rate	:	50 $m^3/m^2/hr$
Surface wash rate	:	9 $m^3/m^2/hr$, backwash and surface wash
time	:	8 min

(f) Clear water reservoir

Retention time	:	1 hr

(g) Chemical dosing facility

(g.i) Aluminium sulphate

Form		:	Lump
Stock volume		:	60 day's quantity
Dosing method		:	Gravity
Dosing rate,	max.	:	90 mg/l
	ave.	:	50 mg/l
	min.	:	20 mg/l

(g.ii) Soda ash

Form		:	Lump
Stock volume		:	60 day's quantity
Dosing method		:	Gravity
Dosing rate,	max.	:	100mg/l
	ave.	:	50 mg/l
	min.	:	10 mg/l

(g.iii) Chlorinated lime

Form		:	Lump
Stock volume		:	60 day's quantity
Dosing method		:	Gravity
Dosing rate,	max.	:	2.0 mg/l
	ave.	:	1.5 mg/l
	min.	:	1.0 mg/l

6.2.2 General Layout

The treatment works are located adjacent to those of the Stage 1 Project for integrated operation. For construction of the new treatment works, approximately 16 ha of the land is needed. A general layout is given in DWG.14.

Each component of the treatment unit will be lined up along the slope of the hill so that the entire treatment processing can be accomplished by a gravity flow. All the hydraulic calculations have been based on the Design Manual and the hydraulic profile along the major treatment facilities are the result of these calculations as given below.

Treatment Unit		Design Water Level (El.m)	
		HWL	LWL
(1)	Receiving well	2,077.5	-
(2)	Rapid mixing chamber	2,075.1	-
(3)	Flocculation basin	2,075.1	2,074.8
(4)	Sedimentation basin	2,074.8	-
(5)	Filter chamber	2,074.4	2,072.9
(6)	Clear water reservoir	2,072.0	2,069.0

The hydraulic profile of the treatment works is given in Fig. 6.2. The pumping facilities will be used only for lifting clear water for the backwashing purposes from the clear water reservoir to the high level tank.

6.2.3 Receiving Well and Rapid Mixing Chamber

The preliminary design of the receiving well down to the filters are shown in DWG.15.

The receiving well has been designed as 4.0 m square with a depth of 3.8 m, giving a storage capacity of 60.8 m^3 and is equipped with a rectangular weir to maintain the constant water level. A flow control valve is installed at immediately upstream from the receiving well to regulate the raw water flow into the treatment works. Both the aluminium sulphate and soda ash are to be dosed in the mixing chamber, which is 4.0 m in width and 2.0 m in length.

6.2.4 Flocculation Chamber and Sedimentation Basins

There are four flocculation chambers, each of which is 12.5 m in width, 10.2 m in length and 2.5 m in effective depth. A tapered G-value distribution has been adopted for the hydraulic design.

The sedimentation basins adjoin to the flocculation chambers are of the horizontal flow type. Based on the design surface load and discharge, area required for the respective basin is calculated at 625 m^2. The dimension of the basin is, therefore, fixed at 12.5 m in width, 50.0 m in length and 2.5 m in effective depth.

Sludge is to be drained to the sludge lagoons through drainage pits, and flushing water facilities are arranged to facilitate sludge removal.

6.2.5 Filters

The filters are of the same type as those of the Stage 1 Project, constant rate filtration, and is divided into 8 chambers, each 8.7 m in width, 8.0 m in length and 4.3 m in depth. The filter media is 600 mm in thickness and is composed of the quartz sand, while the supporting bed consists of three gravel layers : the first and second layers are 100 mm in thickness respectively and the third layer 150 mm.

Both the backwashing and surface washing are to be done with the clear water gravitated from the high level tank. The backwashing rate is assumed to be in 0.83 $m^3/min/m^2$ at the maximum. Each chamber is, therefore, provided with a number of the backwashing troughs and surface washing nozzles.

Chlorination is designed to use chlorinated lime, which is to be piped from the operation building into the filter.

6.2.6 Clear Water Reservoir

The clear water reservoir is planned to be increased gradually in accordance with the expansion of the treatment works. There will be four reservoirs at the final stages, each reservoir with the same storage capacity of 2,084 m^3. In the Stage 2-1 two reservoirs are to be constructed. As shown in DWG.16, the reservoir has been designed of the underground type and is divided into two chambers, each with 25.0 m in width, 30.0 m in length and 3.0 m in effective depth.

As the same as the raw water main a flow meter is installed on the teated water main at the outlet from the clear water reservoir.

6.2.7 Sludge Lagoons and Wash Water Ponds

Each treatment unit will have two sludge lagoons and a wash water pond as shown in DWG.14. Each lagoon will have a storage capacity of 3,400 m3 and two lagoons are to be used alternately for the convenience of the desludging works. The side slopes of the lagoon will be lined with concrete. The wash water pond will regulate the flow from both the sedimentation basin and filter and the rain water with a storage capacity of 7,400 m^3. It will be provided with an emergency spillway for un-expected inflow.

6.2.8 High Level Tanks

Two high level tanks, each with two chambers, are to be constructed : one will be constructed in the Stage 2-1 and the other in Stage 2-2, on the top of the hill as shown in DWG.15.

Each tank has a storage capacity of 1,100 m^3 with a dimension of 20.0 m in width, 25.0 m in length and 2.2 m in effective depth as shown in DWG.16. It is of reinforced concrete structure.

6.2.9 Operation Building and O&M Staff Housing

For the operation and maintenance of the water treatment works and water transmission systems, operation building and O&M staff housings are indispensable, and they will be strengthened in accordance with the expansion of the treatment works..

The operation building is composed of the chemical storages and solution tanks, laboratory for water quality test, administration office and operators' room. The building is of the masonry structure as is common in Kenya. In addition workshop will also be built up adequately.

The O&M staff housings are of the MOWD's standard. In the Stage 2-1, 10 houses are proposed to be newly constructed, composing of 8 in Grade 9 double and 2 Grade 9 single. In both the Stages 2-2 and 2-3, 6 houses will be additionally be furnished, consisting of 5 houses in Grade 9 double and one house in Grade 9 single.

6.3 Nakuru and Naivasha Treated Water Transmission Systems

6.3.1 Design Conditions

The Nakuru and Naivasha treated water transmission systems are to convey the treated water from the treatment works to the Nakuru municipality via the Gilgil town and to the Naivasha town respectively. Both the systems are basically to be brought about in the Stages 2-1 and 2-2, excluding the service reservoirs. Each transmission system is principally composed of a transmission pipeline, bulk water meter, distribution main and service reservoir.

The transmission capacity of the respective pipeline has been already defined through the economic comparative study of the phasing development and the water levels of the concerned reservoirs have been fixed up as the same as those of the Stage 1 Project as shown in Table 6.1. Further the design discharges of the distribution mains are also given in Table 6.2.

6.3.2 Treated Water Mains

The Nakuru treated water system supplies not only the Nakuru municipality and Gilgil town but also feeds four bulk water consumers such as KMB, GMB, ASTU and NYSTC in Gilgil town. In the outskirts of the Gilgil town, the Central Reservoir is to be constructed as the same as the Stage 1 Project for the purpose of the proper and even water distribution to the town of Gilgil and bulk water consumers. Another reservoir is to be provided for exclusive use of KMB in between the treatment works and the Central reservoir. The downstream end of the pipeline is R6 reservoir in Nakuru. Thus the system is, therefore, divided into adequate numbers of reach.

The Naivasha system is originated from the clear water reservoir in the treatment works and connected to R1 reservoir in Naivasha.

The Nakuru transmission system is aligned along that of the Stage 1 Project as shown in DWGS. 17 and 18, while that of the Naivasha transmission system follows as that recommended by 1985 Design Report as drawn in DWGS. 19 and 20. The optimum pipeline configurations have been worked out under a given hydraulic condition as summarized in Table 6.1.

The designed hydraulic profiles of both the Nakuru and Naivasha transmission systems are presented in DWG.17 through DWG.20 respectively. Both the systems associate with the construction of the following appurtenant structures on the pipeline of the respective stage.

Appurtenant Structures	Nakuru System	Naivasha System
Air valve	23 (D100 mm)	3 (D80 mm)
Washout	13 (D25 mm)	5 (D150 mm)
Railway - crossing	3	0
Road - crossing	10	10
River - crossing	13	2

At the end of each pipeline a bulk water meter will be installed to meter the quantity of the treated water supply to NMC and NTC.

6.3.3 Distribution Mains

The distribution mains will be installed to connect among the service reservoirs. Their configurations are presented in Table 6.2. The route of the Nakuru distribution system is the same as that of the Stage 1 Project throughout the stages.

The water distribution systems include the installation of air valves and washouts and the construction of appurtenant structures as listed below:

Appurtenant Structures	Nakuru System	Naivasha System
Air valve, D100 mm	3	0
Air valve, D75 mm	1	0
Railway - crossing	1	0
Road - crossing	9	1

The above quantities are only for a single stage.

6.3.4 Booster Pumping Facilities

The booster pumping facilities will be installed on the distribution mains in Nakuru municipality. One is at R_3 to deliver to R_1 and the other is at R_4 to convey to R_2. The principal features of the booster pump stations are as follows:

Location of Pump	Design Discharge (m^3/min)	Gross Lift Height (m)	Diameter of Pump (mm)	No. of Pump	Power Supply	
					Required Power (kW)	Source of Supply
R_1 reservoir	1.86	88.0	150	2	55	Electricity
R_2 reservoir	0.31	54.0	65	2	10	Electricity

The multi-stage volume pump is selected and will be installed in two stages.

6.3.5 Service Reservoir

The Nakuru municipal and Naivasha town areas have been divided into 7 and two reticulation zones respectively, while the Gilgil town area is to be fed directly from the Central Reservoir. The service reservoirs have been programmed to be expanded in three stages as presented in Table 4.12 and the water levels of the respective reservoir have been set at the same levels as those of the Stage 1 Project throughout the stages to keep consistency in water distribution. The location and dimension of the respective reservoir are shown in DWGs.2 and 24 respectively.

The service reservoir will be a reinforced concrete structure and of a semi-underground type.

6.4 Rural Water Supply System

6.4.1 Design Condition

The Project will include three rural water supply systems : Gilgil East and West systems and Eburru system. Although the water consumers scatter over a wide area, the pipeline routes have been selected to embrace as many as the rural trading centers and communities. All the systems have been programmed to come into effect in the Stages 2-1 and 2-2.

The main components of the transmission system will be the transmission pipeline, boosting facility, bulk water meter and service reservoir. The design transmission capacity and dominant water levels of the service reservoirs will be as summarized in Table 6.3.

6.4.2 Treated Water Mains

The Eburu system will originate from the Central Reservoir and the Gilgil East system from the Clear Water Reservoir in the treatment works. All the transmission systems require the boosting pump facilities. The Gilgil West system bifurcates directly from the Nakuru treated water transmission system at the 25.2 km point. The configuration of the respective treated water main is presented in Table 6.3

The water transmission systems will include the installation of air valves and washouts and the construction of appurtenant structures as listed below.

Appurtenant Structures	Gilgil East System	Gilgil West System	Eburu System
Air valve, D25 mm	0	5	4
Washout, D75 mm	0	5	4
Railway - crossing	1	0	1
Road - crossing	2	8	6
River - crossing	2	2	2

The water transmission facilities for Stage 2-2 will be the same as those of Stage 2-1 because of the similar design discharges.

Before discharging into the service reservoir a bulk water meter will be installed in each pipeline to measure the quantity of the treated water supply. The routes of all the proposed transmission systems will be the same as those proposed in the 1985 Preliminary Design Report and as drafted in DWGS. 21 and 22.

6.4.3 Booster Pumping Facilities

The principal features of the booster pump station are as follows :

Location	Design Discharge (m^3/min)	Gross Lift Height (m)	Diameter of Pump (mm)	Nos. of Pump	Power Supply	
					Required Power (kW)	Source of Supply
Gilgil East System						
- Clear water reservoir	0.596	171	80 x 65	2	40	Electricity
- Reservoir No. 1	0.374	136	80 x 65	2	30	Generator
- Reservoir No. 2	0.225	149	65 x 50	2	15	Generator
Gilgil West System						
- Reservoir No. 1	0.114	104	40	2	10	Generator
Eburru System						
- Reservoir No. 3	1.450	297	125 x 100	2	125	Generator
- Reservoir No. 4	1.450	134	125 x 100	2	110	Generator
- Reservoir No. 5	1.450	199	125 x 100	2	80	Generator
- Reservoir No. 6	0.726	294	100 x 80	2	75	Generator

The types of the pumps will be multistage volute.

6.4.4. Service Reservoir

The service reservoirs with adequate storage capacity will be constructed and the required storage capacity and expansion schedule have been established through the procedures as described in the sub-section 4.4.4. The locations of the reservoirs are shown in DWG.2. The dimensions of the reservoirs are given in DWG.24.

6.5 Bulk Supply Systems in Gilgil

6.5.1 Design Condition

There are four large scale institutions such as KMB, GMB, ASTU, and NYSTC in the vicinities of the Gilgil town and all of them have been dealt as bulk water consumers. It has been planned that every consumer shall be furnished with an independent treated water feeder system as is done by the Stage 1 Project. ASTU is to be served by both the existing Murindati treatment works and Project. KMB system installs its own service reservoir in the course of the Nakuru treated water main in between the treatment and the Central Reservoir, while GMB, NYSTC and ASTU receives the water directly from the Central Reservoir.

All the bulk water consumers, excluding ASTU, has been programmed to be partly facilitated by the Stage 1 Project. Therefore the additional system is to be implemented by the Project. Such additional systems have been plannned to be implemented with a full capacity in the Stage 2-1, since they are smaller in transmission capacity and shorter in pipeline length. ASTU system is, however, to be realized in the Stage 2-2.

The water conveyance pipeline has been designated as the distribution main and designed under the following conditions.

Transmission System	Reservoir W.L (El.m)		Design Discharge (m^3/day)	
	HWL	LWL	Stage 2-1	Stage 2-2
KMB System				
Nakuru treated main		2,056.2	2,030	-
KMB reservoir	2,029.1	2,025.5		
ASTU System				
Central Reservoir	2,051.7	2,047.0	-	1,140
ASTU reservoir	2,063.6	2,060.0		
GMB - NYSTC Systm				
Central Reservoir	2,051.7	2,047.0	7,060	-
Flow meter room		2,046.6		

6.5.2 Distribution Mains

The routes of the distribution mains and locations of the reservoirs are as presented in DWG. 24. GMB and NYSTC will be served by a common system. Based on the design conditions the optimum pipeline configuration has been identified as shown below.

Transmission System	Dia. of Pipe (mm)	Length of Pipeline (m)	Hydraulic Gradient (per mill)
KMB System			
Nakuru treated main - KMB reservoir	100	129	119.0
ASTU System			
Central reservoir - ASTU reservoir	150	2,530	2.8
GMB - NYSTC System			
Central Reservoir - Flow metre point	500	259	0.467

Out of the three systems, ASTU system requires a booster pumping facility.

6.5.3 Booster Pumping Facilities

The principal features of the booster pump station for ASTU as follows:

Location	:	Central reservoir
Design discharge	:	0.825 m^3/min.
Gross lift height	:	25 m
Diameter of pump	:	100 m
Required Power	:	10 kW
Source of power supply	:	Electricity

Two pumps will be installed, one of which will be a stand-by.

6.5.4 Service Reservoir

The service reservoir site has been selected at the same place as the existing and/or the Stage 1 Project in order to maintain the consistency in water supply system and their capacity has been determined as noted in the sub-section 4.4.4. The dimensions of the reservoirs are presented in DWG. 24.

VII. CONSTRUCTION PLANNING AND COST ESTIMATE

7.1 Mode of Construction

It is assumed that the construction of the Project will be performed on the basis of contract and the contractors will be procured through both the local competitive bid (LCB) and the international competitive bid (ICB). It is, therefore, proposed to divide the entire project works into the adequate numbers of contract works in due consideration of the natures of the works involved, the project implementation schedule and the amount of the construction cost. The proposed packaging and mode of procurement are as follows.

Stages	Package Number	Contract works	Mode of Procurement
2-1	D-1	Access Road	LCB
	D-2	Malewa dam	ICB
	D-3	Trans-basin diversion tunnel	ICB
	W-1	Raw water transmission system, Phase 1	ICB
	W-2	Water treatment works, Phase 1	ICB
	W-3	Nakuru transmission system, Phase 1	ICB
	W-4	Naivasha transmission system, Phase 1	ICB
	W-5	Gilgil East and West and Eburru rural systems, Phase 1	ICB
	W-6	KMB and GMB - NYSTC bulk supplies	LCB
2-2	W-7	Raw water transmission system, Phase 2	ICB
	W-8	Water treatment works, Phase 2	ICB
	W-9	Nakuru transmission system, Phase 2	ICB
	W-10	Naivasha transmission system, Phase 2	ICB
	W-11	ASTU bulk supply	LCB
	W-12	Gilgil East and West and Eburru rural systems, Phase 2	LCB
2-3	W-13	Water treatment works, Phase 3	ICB
	W-14	Nakuru transmission system, Phase 3	LCB
	W-15	Naivasha transmission system, Phase 3	LCB

7.2 Construction Plan and Method

7.2.1 Basic Conditions

The construction plan has been worked out in due consideration of the climatological conditions prevailing over the construction sites, transportation of the construction materials, equipment, machineries and plants, availabilities of the dam embankment materials, concrete aggregates and pipe material and quantities of the construction works. The following basic conditions have been adopted for the construction planning.

(1) Number of workable days

It is usual practice that the dam embankment and concrete works be suspended when the daily rainfall exceeds 5mm and 1.0 mm respectively. Based on this criterion the number of working days are determined at 298 days and 268 days in a year for concrete works and embankment works respectively, based on the daily rainfall records at Gilgil during a 10-year period from 1977 to 1986. Of course the public holidays in Kenya, including the national holidays are not included.

(2) Transportation

The constructional plant, equipment, machineries to be imported will be unloaded at the port of Mombasa and transported to the site by the National Highway A104.

(3) Working hour

The gross working hours is assumed to be 10 hr/day, while the actual working hour of the constructional plants, equipment and machineries is set at 8 hours per day.

(4) Filter material and concrete aggregates

Both the filter materials of the dam and coarse concrete aggregates have been proposed to be produced by exploiting the trachytes available from the South Gilgil quarry site, since there are no suitable natural deposits in the vicinity of the damsite.

(5) Construction quantity

The construction quantities have been calculated for the major construction works based on the basic design as given in Tables 7.1 and 7.2.

7.2.2 Construction Plan for Malewa Dam Scheme

(1) Preparatory works

The preparatory works include the construction of the project offices, quarters, clinic, field laboratory, permanent access road and the installation of water and power supply and sanitary facilities. The general layout of the preparatory works is shown in Fig. 7.1.

The permanent access road to the damsite is extended from the existing Kipipiri Road and its length is 4.0 km. It is 5.5 m in a width and its surface is to be macadamized.

The electric power requirement for the construction use in the damsite is estimated at 2,000 KVA. The power distribution line is extended from the Malewa pumping station to the temporally sub-station at the damsite in a distance of approximately 10 km. The water supply is made through two water tanks : the primary tank is used for the water supply to the constructions of both the diversion and trans-basin diversion tunnels and the secondary tank feeds the other water uses. The water treatment works are located on the left bank of the dam and the raw water is pumped up from the Malewa river upstream from the cofferdam. The required water is estimated to be 2.8 m^3/min.

(2) Diversion tunnel

The construction works have been planned to be executed in two shifts. The excavation is progressed by a full face and heading-bench method using leg drills, load haul dump trucks, in view of the tunnel section, accessibility to the portal and the location of the spoil bank, and steel support is placed at intervals of one meter. Lining with concrete is executed by using a sliding form, excepting the invert portion, with a combination of agitator trucks and concrete pump cars. Backfill grouting is performed after lining works.

(3) Coffer damming

After completion of the diversion tunnel, a temporary dyke will be built across the Malewa river with the materials derived from the diversion tunnel excavation works. The coffer dam, forming an integral part of the main dam, is then embanked with the specified material. The embankment of the coffer dam is planned by the same method as that of the main dam.

(4) Main dam

The excavation of the dam foundation will be commenced soon after completion of the cofferdaming and done by mobilizing bulldozers with ripper, backhoes, wheel loaders and dump trucks. In the abutment area it will be proceeded progressively from the upper portion towards the valley bottom. The rock surface in the core trench will be finished by using pick hammers and manpower and be furnished with suitable drainage facilities.

The dam embankment will have an impervious core of 166,900 m^3, filter materials of 110,900 m^3, outer shell-2 of 48,700 m^3 and outer shell -1 of 516,200 m^3. The impervious earth material and the outer shell -1 are tarnsported from the East Road borrow area and Kipipiri quarry site respectively, while both the filter material and the outer shell -2 are quarried from the South Gilgil quarry site.

The quarry and borrow operation will be done using crawler drills, air compressors and bulldozers with rippers, while dozer shovels, wheel loaders and dump trucks are operated for the loading and hauling of the embankment materials. The core material will be transported from the East Road borrow area and will be spread by 21 ton bulldozers on the embankment area in a layer of 200 mm thick. Compaction will be achieved by eight passes of tamping roller.

The filter materials will be produced by a crushing plant to be installed at the South Gilgil quarry site, and will be transported by 11 ton dump trucks. They will then be dumped into a spreader box and then spread by 21 ton bulldozers in a layer of 300 mm thick. Compaction will be accomplished by five passes of vibratory roller. The inner shell will use excavated fresh rocks piled at the dam site. The material will be transported by 11 ton dump trucks after loading by 1.9 m^3 dozer shovels, and they will be dumped and spread by bulldozers in up to 1,000 mm thick layers. Then they will be compacted by five passes of vibratory roller.

The materials for the outer shell-1 will be loaded at Kipipiri Road quarry site by 1.9 m^3 dozer shovels and transported to the damsite by 11 ton dump trucks. They will be dumped and spread by bulldozers in up to 1,000 mm thick layers, then compacted by five passes of vibratory roller. The outer shell-2 materials will be loaded by 11 ton dump trucks at the South Gilgil quarry site and will be placed at embankment area by dumping. Spreading will be executed by 21 ton bulldozers in a layer of 1,000 mm thick and compaction will be accomplished by five passes of vibratory roller.

The trench excavation will be executed along the gallery, which will be finally lined with concrete. The curtain grouting will be done from the gallery and blanket grouting will be completed prior to commencement of the embankment works.

(5) Spillway

Spillway is to be excavated from the upper section to the lower with crawler drills and bulldozers with ripper. Dozer shovels and dump trucks will be used for loading and hauling respectively. The amount of soils and rocks to be excavated is about 446,300 cu.m and hauled to the spoil banks. Fresh rocks out of excavated rocks will be used for dam embankment.

Concrete works of 41,000 cu.m in volume will be carried out in the order of overflow weir, stilling basin and chuteway. Two tower cranes with the working radious of 45 m are to be used for concrete works. A crawler crane with 40 ton is planned to assist the work outside of the tower crane.

(6) Trans-basin tunnel

Tunnel will be excavated by full-face blasting method using leg drills from both portals considering its length (2,400 m). Rocker shovels, train loaders and battery locomotives with grambee toro are to be used for mucking. Concrete lining except invert is done using sliding forms with presscretes and locomotives.

Table 7.3 shows the major constructional equipment, plants and machineries required for the construction of the dam, appurtenant structures and trans-basin diversion tunnel.

7.2.3 Construction Plan for Water Supply Scheme

(1) Intake

The intake works are scheduled to be constructed during the dry season the left bank of the Turasha river. The work is to be executed so as not to disturb the operation of the Stage 1 Project. Sand bags and piles will be used for temporary coffer damming.

(2) Tunnel

Tunnel is planned to be constructed by full-face blasting method using leg drills from the upstream portal. Rocker shovels, train loaders and battery locomotives with grambee toros are to be used for mucking. Concrete lining excepting the invert will be done using sliding forms with a presscrete and a locomotive.

(3) Water treatment works

Excavation works is to be executed during the dry seasons with bulldozers and backhoes, loaded with wheel loaders and hauled with dump trucks.

Concrete plant will be installed at the site and agitator cars and concrete pumps will be used for casting concrete.

(4) Pipelines

Excavation of raw water mains and treated water mains will be mainly executed by backhoes. Pipes are to be installed with crawler cranes, truck cranes or trucks with cranes. Backhoes, compactors and tampers will be used for backfilling.

7.3 Construction Time Schedule

7.3.1 Malewa Dam Scheme

The Malewa Dam Scheme will be realized in the Stage 2-1 and its construction time schedule is presented in Fig. 7.2. The construction of access roads is scheduled to commence advancing the main works. Major part of diversion tunnels will extend over a period from

April, 1994 to June, 1995. Pipe and valve installations, and plug concrete are to be executed during a three-months period from November, 1996 to January, 1997.

The construction of the upstream coffer dam including a temporary closing dyke is planned to be commenced from July, 1995, immediately after completion of the diversion tunnel, and completed by October, 1995. The foundation excavation of the main dam is commenced from February, 1995 at the upper part of the abutment. The embankment works will extend over a 17-month period from September, 1995 to January, 1997 and concentrated in the dry season.

The foundation excavation of the spillway will be started from June, 1995 and completed by May, 1996. The concrete works are to be started after the installation of temporary facilities, such as tower cranes and other relevant facilities.

Construction of the trans-basin diversion tunnel including intake structure will be done during the period from March, 1994 to July, 1996.

7.3.2 Water Supply Scheme

The water supply scheme has been programmed to be implemented in three stages, each of which requires a three-year period for the construction, namely, the Stage 2-1 during the period from February, 1994 to January, 1997, the stage 2-2 during the period from January, 2002 to December, 2004, and the Stage 2-3 during the period from January, 2009 to December, 2011, as shown in Fig. 7.3

7.4 Cost Estimate

7.4.1 Construction Cost

The construction costs were estimated at the 1990 price level for the Malewa Dam and Water Supply Schemes respectively. They comprise foreign and local currency components and are divided into the direct and indirect construction costs and contingencies. The indirect construction costs includes the government administration cost, engineering services, and land compensation and acquisition. The contingency consists of the physical contingency and the price escalation. The direct construction cost is based on the work quantity and unit price of the corresponding work item.

The basis and condition of the cost estimate are as follows:

(1) Unit price by work item

This covers the direct costs of labours, materials and equipment and the indirect costs such as contractor's overhead, profit and site expenses. The quantities of the major work items is presented in Tables 7.1 and 7.2, while the labour wages, unit prices of the major construction materials adopted are as shown in Table 7.4 and 7.5 respectively.

(2) Exchange rate

The foreign exchange rate was assumed at US$1.00 = Kshs. 22.5 = Japanese Yen 150.75.

(3) Land acquisition and compensation

It has been based on the area to be submerged by reservoir and land acquisition cost given by MOLH.

(4) Government administration and engineering service

These are assumed to be 3% and 8% of the direct construction cost respectively.

(5) Contingency

The physical contingency is assumed to be 10% of the sum of the direct and in direct construction costs.

The price escalation is also assumed at 3 per cent per annum for the foreign currency component and 8 per cent per annum for the local currency component.

The estimated construction cost is presented in Tables 7.6 and 7.7, and is summarized below.

	Project component	Foreign Currency Portion (US$$10^6$)	Local Currency Portion (Kshs. 10^6)	Total (Kshs. 10^6)
(1)	Malewa Dam Scheme	41.5	349.1	1,282.6
(2)	Water Supply Scheme			
	Stage 2-1	59.7	511.5	1,854.6
	Stage 2-2	58.3	544.6	1,856.8
	Stage 2-3	13.1	201.6	497.4
	sub-total for (2)	131.1	1,257.7	4,208.8
	Total	172.6	1,606.8	5,491.4

The annual disbursement schedule of the construction cost has been worked out for the respective stage on the basis of the proposed construction time schedule as shown in Table 7.8.

7.4.2 Operation and Maintenance Cost

The annual operation and maintenance costs including the salaries of project operation and maintenance staff, materials, labour, costs for repair and maintenance of equipment, and the running costs for the project facilities were estimated for the respective project component as shown in Table 7.9.

7.4.3 Replacement

Some of the facilities, especially mechanical and electrical equipment, have shorter useful lives than the civil works, and require replacement within the project service life. The replacement costs of these facilities are also listed in Table 7.9.

IIX PRELIMINARY ENVIRONMENTAL STUDY

8.1 General Description

8.1.1 Background

(1) Needs of Environmental Investigation

The Project is expected to directly benefit more than one million people by augmenting and securing safe water supply. On the other hand, it is foreseeable that the Project would implicate the various effects on the natural and social environment, especially in Lake Naivasha and Lake Nakuru. It is indispensable to create an appropriate atmosphere for coexistence of the contemplated water supply and the natural and social environment.

(2) Background

Originally it had been planned that MOWD would perform a full scale environmental study concerning the Project but unfortunately it was not completed by the time of the Study. Accordingly JICA provided a technical assistance to MOWD for the environmental study in accordance with an additional request made by the Government of Kenya. The environmental investigation has been conducted during the Phase 2 and 3 Studies.

	Field Investigation Period	Home Office Work Period
Phase 2 Study	October to November, 1989	November,1989 to January, 1990
Phase 3 Study	May, 1990 to July, 1990	July to September, 1990

(3) Scope of work

The study was actually composed of two different levels of studies as follows :

(a) Preliminary environmental study for the proposed Malewa reservoir area and Lake Naivasha
   - Field reconnaissance

- Ecological investigation
- Agriculture and irrigation surveys
- Groundwater survey
- Water quality survey
- Lake Naivasha water balance study
- Data collection on environmental fundamentals

(b) Fundamental environmental study for Lake Nakuru
- Field reconnaissance
- Topographic mapping of Lake Nakuru
- Lake Nakuru water balance study
- Data collection of environmental fundamentals

Since the environmental investigation involves various fields such as wildlife, irrigation, hydrology, agriculture, horticulture, groundwater, vegetation, the Study Team has got in touch with the various governmental and non-governmental organizations concerned and received valuable information.

This Chapter present a summary of the preliminary environmental study. Detailed elaboration and all the data collected are given in Annex G.

8.1.2 Objective Area

(1) Lake Naivasha Drainage Basin

The lake drainage basin covers an area of approximately 3,400 sq.km, comprising three sub-basins as follows:

Sub-basins	Drainage Area (sq.km)
Malewa river	1,653
Gilgil river	511
Lake Naivasha and minor rivers	1,237
Total	3,401

The Malewa river is the largest river sustaining nearly 95% of inflow into the lake. The second largest river is the Gilgil river. There are also a large number of another small tributaries flowing into Lake Naivasha. Among them, both the Karati and Marmonet rivers have relatively large catchment area but they supply with the lake only seasonal flow. There is no river outgoing from the lake.

The proposed Malewa damsite is approximately eight km upstream from the confluence point. The elevation at the confluence is approximately El. 1,960 m. The Malewa river finally debouches Lake Naivasha at about 7 km to the west from the Naivasha town.

As shown in Fig.8.1, the lake has three distinct parts, The first one is the main lake having a surface area of some 130 sq.km. The second one is connected to the main lake on the east side. An ancient volcanic crater rim has formed a crescent-shaped peninsula which shelters the Crescent Island Bay. The third one, on the south, is called " Lake Oloidien " or " Small Lake ", which is disconnected at present and saline lake. There are the irrigated lands of approximately 7,500 ha acres extending the lake shore and they are depending on the lake, river and boreholes nearby for irrigation supply. The general characteristics of the lake are summarized in Table 8.1.
Vegetation map over the Drainage area of Lake Naivasha is shown in Fig. 8.2.

(2) Lake Nakuru drainage basin

Lake Nakuru is located at the floor of Rift Valley and bounds with the southern boundary of Nakuru Municipality. The lake is a part of the Lake Nakuru National Park with an international reputation for "the Lake of a million flamingos." It is a very important tourism resource in Kenya, which is nowadays one of the most important foreign exchange earnings. It is also a wetland of international importance.

Its drainage area covers an area of approximately 1,536 sq.km comprising the following sub-basins.

Sub-basins	Drainage Area (sq.km)
Enjoro river basin	273
Makalia river basin	331
Enderit river basin	523
Lamudiac river basis	131
Ngosor river basin	80
Lake Nakuru and minor river basins	198
Total	1,536

There is no river outgoing from the lake. Geographically the lake acts as a recipient of sewage effluent from the Nakuru municipality, as shown in Fig. 8.3. The lake's hydrologic regime is quite the similar to that of Lake Naivasha but the lake water is the saline as shown in Fig. 8.4.

The lake surface area is approximately 43 sq.km at El.1,760 m and varies largely with the lake level, since the lake is characterized by a shallow water depth. During the Phase III Study period, a topographic map for the lake and its surrounding has been prepared in a scale of 1 to 10,000, by using aerial photo covering 140 sq.km and sounding survey of the lake floor. The general characteristics of the lake are summarized in Table 8.1. Vegetation map over the drainage area of Lake Nakuru is shown in Fig. 8.2.

8.2 Present Natural and Social Environment

8.2.1 Malewa River

(1) River water quality

The water quality analysis has been carried out during both the Phase 2 and Phase 3 Studies. During the Phase 2 Study, the water quality test was entrusted to the local laboratories, while that during the Phase 3 Study was more comprehensive than during the Phase 2 and was executed directly by the Study Team. The water quality of the Malewa River is, therefore, reported based on the results obtained during the Phase 3 Study.

Water has been sampled during a three-month period since July, 1990 by the Study Team and NWCPC's counterpart at six monitoring points on the Malewa river. The summary of the water quality analysis are presented in Table 8.2 and are briefly explained below.

- The average values of SS and DO was 94 mg/l and 7.2 mg/l respectively. This fact means that the river water is not polluted.
- Suspended solid value accordingly increases with increase of stream flow, and COD, N, and inorganized P also increases due to inflow of organic matter.

(2) Ecology

According to the literatures made so far available, lists of aquatic animals and terrestrial animals is as given in Table 8.3 and 8.4 respectively. There are two species of vertebrates and 25 species of invertebrates. Terrestrial animals include seven species of mammals, ten species of birds, three species of reptiles, and two species of amphibians. Cheetah and Leopard, the trade in which is prohibited by the Washington Convention, are reported to live within the Malewa river basin.

In the proposed reservoir area, Rainbow Trout, Barbus, and Crayfish inhabit. There has been no commercial fishery in the river.

(3) Land and river uses

The proposed reservoir extends over 3.85 sq.km below the dam crest level of El. 2,154 m. Majority of the area is classified into bushland. Most of the area is grazing land for cattle and 13.1 ha of the land are used for cultivation of maize as shown in Fig.8.5. Five households or approximately 30 peoples are living below El. 2,154 m.

Along the Malewa River between the proposed damsite and Lake Naivasha and the Turasha River between the intake and the confluence of the Malewa river, there are a number of water users as under-listed.

Categories of Water Rights	Malewa River				Turasha River			
	Low Flow Season		Flood Season		Low Flow Season		Flood Season	
	Nos.	Discharge (cu.m/s)	Nos.	Discharge (cu.m/s)	Nos.	Discharge (cu.m/s)	Nos.	Discharge (cu.m/s)
Domestic	16	0.007	-	-	-	-	-	-
Public	2	0.030	-	-	4	0.003	-	-
Minor irrigation	-	-	3	0.001	-	-	-	-
Industrial	-	-	1	0.028	-	-	-	
Power	-	-	1	0.008	-	-	-	-
General irrigation	-	-	8	0.068	-	-	2	0.010
Others	-	-	1	0.007	-	-	-	-
Total	18	0.037	14	0.112	4	0.003	2	0.010

8.2.2 Lake Naivasha

(1) Lake level fluctuation

The lake level has been recorded by MOWD at the gauge station 2GD1 for the period from 1933 to 1985. The recording at this station has also been abandoned since 1986. During the Phase III Study, the Survey of Kenya checked the altitude of the zero point of the staff gauge and found that it had been set 3.82 m higher than the national datum level. Therefore the water level recorded so far has been corrected accordingly. The Study Team also obtained the water level records monitored by the Elsamere Conservation Center since 1950s up to date. The recorded lake level is given in Fig. 8.6. In 1988, the most severe drought year in the last 25 years, the lake level descended to El. 1,882.0 m.

(2) Lake water quality

As with the Malewa River, the water quality analysis was conducted in both Phase 2 and Phase 3 Studies. The water quality is, however, reported herein based on the results of the Phase 3 Study. During the Phase 3 Study, the water has been sampled over a three-month period from July to September, 1990 at 15 points, evenly distributed over the lake surface area as shown in Fig. 8.1, and further two different layers in a vertical direction ; upper and lower layers. The summary of the water quality test is given in Table 8.2 and is report as follows:.

- High pH value of 9.04 is measured at the northeast part of the lake, where a lot of submerged plants are growing. It is presumed that photosyntheisis by these plants increases the pH value.
- The lake water shows a higher pH value than that of the Malewa River.
- The lake water of Lake Oserien is characterized by a high salinity. Its water quality and comparison to the main lake are are also given in Table 8.2.

The historical records of pH in relation to the lake level is shown in Fig. 8.7.

(3) Ecology

(a) Aquatic plants (Macrophyte)

The aquatic plants in the lake are categorized into such three kinds as emerged, floating leaved, and submerged. Nine species of the aquatic plants has been confirmed during the field investigation.

Among the aquatic plants, Salvinia molesta, and Papyrus are at present notable species from the environmental view point.

(i) Salvinia molesta

Salvinia molesta is a tropical fern and floating leaved plant. Although one piece of Salvinia molesta is more or less 10 cm long, it increases explosively. It can occupy the surface of calm nutrient-rich water quickly. The Salvinia mat is easibly movable by a wind.

At present, Salvinia molesta are mainly spreading over approximately 15 sq. km over the northern side of the lake. The distribution of Macrophytes is shown in Fig. 8.8. By forming the surface mat, it brings the following impacts on lake environment.

- Impeding navigation,
- Invading into and clogging intake facilities,
- Obstructing submerged plants growth owing to shading effect
- Increase in sedimentation of organic matter, and
- Reduction of oxygen concentration beneath mat.

Although much research work has been done so far, a causal relation between Salvinia and water quality has not yet been determined. Some studies suggest that the optimum conditions for Salvinia mats may be found where slightly moving water is transporting nutrient to the submerged leaves. The river outlets into the lake provide such an environment.

In order to prevent increase of the Salvinia mat, the following countermeasures are, however, being considered by the Human Settlements Secretariat.

- Chemical control method by using herbicide
- Mechanical removal by machine
- Biological control by releasing Cyrtobagous singularis (a weevil), an enemy of Salvinia

(ii) Papyrus

Papyrus is a dominant emergent species and grows in shallows less than 1.5 m in water depth. Before 1983, Papyrus fringed the lake shore, extended to some 12 sq km. In 1983 when the lake level declined, the large area of Papyrus was reclaimed for agriculture proposes. At present the Papyrus area still grows at the mouths of the Gilgil and Malewa rivers in the northern part of the lake, where it traps floating matter carried by the river.

(b) Plankton

Total numbers of Phytoplankton cells observed range from 6,300 to 15,000 / ml. Dominant species are Microcystics aeruginosa and Synedra sp.A. Aphanocapusa sp.A and Aphanocapusa sp.B, and Cylindrospermopsis raciborskii are also founded. As for Zooplankton, the total number of cells are in the range of 270 - 285 / liter. The dominant species are Brachiflorus, Filinia sp., and Diaphanosoma sp. Such species in general is found in the eutrophic water.

(c) Aquatic animals

The list of the aquatic animals is shown in Table 8.3. Dominant species are Tilapia, Blackbass, and Crayfish, which have been introduced within the last 25 years for the fishery. They can tolerate against relatively high pH and salinity.

(d) Terrestrial animals

There are enormous kinds and numbers of birds as shown in Table 8.4. White Pelican, Heron, Coot, and Duck are the dominant. It has been reported that number of the birds had fluctuated depending on mode of life of aquatic plants. Various kinds of mammals inhabit bushland around the lake. Only Hippopotamus lives in the lake.

(4) Groundwater survey

The groundwater survey has been carried out aiming to grasp a relation between the lake level and the groundwater table during the period from May to July, 1990. The survey area is limited to the surroundings of Lake Naivasha, based on the Borehole Record and Location Map of the existing boreholes furnished by MOWD.

(a) Hydrogeology

Alluvial plain extends on the north of the lake and very gentle piedmont slope of the Longonot mountains is on the south. On the east of the lake, comparatively steep slope of the edge of the Kinangop plateau is found and those of the Eburu mountains are on the west.

The terrain of the survey area is mainly composed of a series of tertiary volcanic and pyroclastic rocks interbedded with lake sediments, quaternary lake sediments, and a series of quaternary volcanic and pyroclastic rocks as shown in Fig. 8.9.

The series of tertiary volcanic and pyroclastic rocks is distributed from the northeast to the east to form the Kinangop plateau. The quaternary lake sediments, which are benthonic sediments of old Lake Naivasha, are distributed below El. 2,000 m with a thickness less than 30 m around the lake. The series of quaternary volcanic and pyroclastic rocks are distributed in the remaining area to form the Eburu and Longonot mountains. Alluvium overlays only the quaternary lake sediments being distributed over the north of the lake.

Since the alluvium in the area is distributed only in the north of the area and composed of mainly silt and clay less than 10 m in thickness, there is no very thick and extensive aquifer that can be found generally in alluvium being composed of sand or gravel. Groundwater in this area shows both of "Stratum water" and "Fissure water". The former is in the porous zone like coarse-grained sand of lake sediments or scoria of pyroclastic rocks. The latter is in fissured zone of volcanic rocks. Faces of the lake sediments or the volcanic/pyroclastic rocks are generally variable both horizontally and vertically, therefore many aquifers of small magnitude seem to exist in the area.

(b) Existing boreholes

There are 137 boreholes around Lake Naivasha as shown in Fig. 8.10. 31 boreholes, out of 137 existing boreholes are working, 37 boreholes are not working including 17 abandoned boreholes. 69 boreholes are unknown whether they are working or not. Most of all the working boreholes are used for irrigation. Although 10 farms out of 47 farms depend on borehole only for irrigation supply, other boreholes are used as supplement to the other main water source such as direct lake water pumping.

The existing borehole area is divided into six sub-areas, in due consideration of regional topography, i.e. NP (North Plain), SP (South Plain), EL (East Plateau), WM (West Piedomont), EM (East Piedomont) and SM (South Piedomont).

There are some boreholes encountering saline water problems in the "NP" area, especially to the south of the railway and in the "SM" area.

(c) Preliminary examination on lake level - groundwater table

Since no geological log with sufficient accuracy has not available, distribution and structure of each rock faces are not made clear. Judging from various surrounding geologic conditions, the groundwater seems to exist in many small magnitude aquifers distributed over different horizons as "Stratum water" or "Fissure water". Therefore it is impossible to prepare a groundwater contour map in the survey area.

There are many boreholes in which water levels are lower than or the same as the lake level. This, however, does not always suggest that the lake recharges the groundwater. If the lake always recharges groundwater, both of recharge and its reverse could be observed in two boreholes located closely each other. However it has been found out actually in the field that the water level of one borehole is higher than the lake level and that of the other is lower than the lake level.

Provided that aquifer exists on the bottom of the lake, rest level and struck level of all boreholes would be almost the same elevation. This, however, also contradicts with the actual record. The rest levels appear higher than the struck levels in many boreholes located in the "NP" or "SP" areas. It indicates that the groundwater around the "NP" or "SP" area is under the conditions of confined water with high pressure. Mechanism of such high pressure is deemed to attribute to connection of aquifer and the lake bottom with high angle or by aquifer with steep dip. Both of the mechanism mentioned above are therefore not applicable in the surveyed area. Since the rest levels are nearly equal to the lake level, there may exist seepage flow of the lake water to the boreholes through permeable weathered zone because of imperfect seal. This concept is considered to be more realistic than the other concept of existence of the confined water with high pressure.

Furthermore, water levels in most boreholes, except boreholes in the "NP" or "SP" areas, are almost the same lake level with a difference within 10 m. As for these boreholes, the lake water also seems to seep into the boreholes considering the location of the boreholes in a limited area. If the lake water seeped into the boreholes through the weathered zone, water level in the boreholes would lower following the decline of the lake level. If borehole pumps are set low enough, no problems will occur.

(5) Land and lake water use

The area around the lake is not only a large-scale ranching area but also a large-scale exportable crop production center in the Nakuru District. In particular, the horticulture crops export are being highlighted as one of the major foreign currency earning commodities. Owners of farms have formed an association named " Riparian Owners Association.'

Two luxury hotels are operated on the eastern lake shore as a leisure resort area on commercial basis. A yacht club has maintained a club house on the Crescent Island since 1931. There are two self camp sites and a hostel. The Elsamere Conservation Center contains a small museum and offers residential facilities for conservation researchers.

The lake water is being used for various purposes and 80 water rights have so far been registered by WAB as summarized below.

Categories of Water Right	Low Flow Season		Flood Flow Season	
	Nos.	Discharge (cu.m/sec)	Nos.	Discharge (cu.m/sec)
General	40	0.039	-	-
Public	1	0.003	-	-
Small irrigation	-	-	16	0.002
Commercial	1	0.001	-	-
Power generation	-	-	1	0.008
Large irrigation	-	-	62	0.819
Others	-	-	1	0.030
Total	42	0.043	80	0.858

(6) Agricultural

Although there is a little discrepancy with the WAB's data, there are a vast irrigation land of 7,895 ha around Lake Naivasha, according to the Provincial Irrigation Unit. The irrigated areas by the water sources are as follows.

Categories of Water Abstraction	Nos. of Farms	Total Area (Acres)
Lake water only (direct or/via canal)	53	2,426
Open channel only	13	630
Bore hole only	8	1,888
River only	6	1,381
Lake & borehole	5	815
River Borehole	2	725
Open well & boreholes	1	30
Total	88	7,895

A door to door survey has been carried out to grasp actually the existing irrigation and agricultural situation in June, 1990. The survey was completed for 47 farms, covering as large as 6,908 acres, out of 7,895 acres.

(a) Physical condition

In general climate is very suitable for exportable crop cultivations such as cut flowers, strawberry, and french beans.

The soil around the lake are classified into six categories such as Phaeozems, Cambisols, Xerosols, Solonetz, Regosols, and Lithosols. The Phaeozems and Cambisols extend over the middle-west part of the lake, while Xerosols, Solonetz, Regosols and Lithosols are developed over the lake except the middle-west of the lake.

(b) Agricultural land use type

Out of the total surveyed land area of 45,219 acres, 7,431 acres are cultivated, of which 6,908 acres are irrigated. As shown in Table 8.5, the irrigated lands are classified into fodder crops land of 3,543 acres and horticulture crops land of 3,365 acres.

(c) Crops

The horticulture crops includes vegetables, ornamental flowers, and fruits. The number of farms by type of crops are summarized below and irrigated area by crop is shown in Table 8.5.

Type of Crops	Number of Farms	Main Crops
Vegetable	12	Cabbages, Onions, French beans, Carrots, Tomatoes, Potatoes, Asparagus
Pasture	15	Lucern, Maize, Sorghum Rhodes grass and Nappir
Vegetable & Pasture	10	
Cut flowers	7	Astrcemerias, Carnation, Chrysanthemums, Ornithegdum, Tuberesen
Flower bulb	1	
Fruit	2	Strawberry, Apple, Orange
Total	47	

Most of vegetables are locally marketed. French beans and asparagus are mainly exported. Strawberries are grown mainly for European markets.

The cut flowers production is highlighted in this area. Most of flower growers gather in the south side of the lake. Cut flowers are planted throughout a year with a drip irrigation systems. The crop intensity is 200 percent on average. Some 30 million blooms a year are exported to European countries.

(d) Source of irrigation supply

As a result of the door-to-door survey, the following were noted.

- The farms around Lake Oloidien, a saline lake, use fresh water from boreholes.
- Some farmers have reported that floating plants such as Salvinia molesta and Water Hyacinth have stagnated for some period in front of their intake pumps or in the canals, resulting in clogging of the pumps.
- Some large farms have provided a small pond for water storage from where the water is boosted to the terminal point. Therefore their water abstraction are not affected by fluctuation of the lake level.
- Some flower growers are using the lake water after neutralizing pH value even at present.
- 32 farms out of 47 have reported that they have shifted pumps or extended canals by some tens meters to cope with set back of the shoreline in the 1988 drought period. A few farms deepened their boreholes by approximately 2 meters.

(e) Irrigation method

The sprinkler irrigation is the most common. However a large number of farmers have come to recognize that much water is wasted by using sprinklers, and are using long hoselines to supply water to the cut flowers and other crops. The cultivations of the flower and strawberries are mostly resorted to a drip irrigation.

(7) Fishery

A commercial fishery was initially commenced in the year of 1959. Major fishing target are Black bass, Tilapia, and crayfish. The Black bass was firstly introduced in 1927, Tilapia in 1956 and Crayfish in 1975. According to the 1987 report, 83 boats were given a license, of which 51 were actually under operation.

The annual fish catch has fluctuated greatly from 692 tons at maximum in 1983 to 245 tons at minimum in 1985. In 1983, 78 % of the total catch weight was Tilapia, 17 % crayfish fish, and the rest Black bass. The market value was Kshs. 9.6 million in 1983, 64 % of the total value was Tilapia, 30 % Crayfish fish, and the rest Black bass.

8.2.3 Lake Nakuru

(1) Lake Nakuru National Park

The present landscape of the park is characterized by hills, ridges, cliffs, rocky outcrops and lake basin. On the southern rim of the lake, the river mouth of the Makalia and Enderit rivers forms a flat grass land, some 60 sq.km, which is the major feeding ground of the wildlifes. The bird and wild animals life forms an unique spectacular display that attracts thousands of both local and foreign tourists into the park. There are two lodges in the park. The visitors have increased recently from 98,000 in 1983 to 129,000 in 1987. The revenue collected in 1987 amounted to approximately Kshs. 7.3 million. Kenya signed the Ramsar Convention on Conservation of Wetlands of International Importance on June 6, 1990, and nominated Lake Nakuru as the first Kenyan Wetland on the list.

(2) Ecology

The list of the terrestrial animalsis shown in Table 8.4 and the aquatic animals in Table 8.3.

The land is composed of bushlands in the east and west, grasslands and woodlands in the south and north, and other such types as forest, sedge marshes, swamp vegetation, and cliff vegetation. It exhibits a wide ecological diversity with characteristic of habitats that stretch from the lake through the shoreline and up to the escarpments and ridges.

(a) Aquatic flora

There are no tall aquatic flora growing in the lake. High saline and alkaline water is unsuitable for their growth. There is, however, an enormous concentration of blue-green algae, Spirulina platensis, that forms the basis of all the food chains in the lake. Spirulina platensis is a small filamentous alga living in alkaline and saline water and is sometimes so prolific that the water is colored bright green.

It has been noted by many literatures that there is a close relationship between chemistry, Spirulina, and Flamingoes, although it has not been clearly identified yet so far. Some literatures have reported some findings as below:

- Spirulina cannot survive in fresh water nor too extremely saline one.
- Spirulina is found in water with total dissolved solids of 8.5 to 270 g per litter.
- As Spirulina densities dropped, mean number of flamingos were correspondingly and drastically reduced.
- At such times as Spirulina densities dropped, diatoms and other species of blue-green algae (e.g. Anabaenopsis arnoldii) became the main primary producers. Tilapia almost certainly dominated primary consumers.

Fig. 8.11 shows the historical records of conductivity of the lake water, algae concentration and flamingo population.

(b) Plankton

The total numbers of Phytoplankton cell are observed in the range of 147,000 to 208,000 / ml. Dominant species is Cryptomonadales, which reveals that the lake water is under eutropic state.

(c) Aquatic fauna

There is a single species of small fish, Tilapia grahami, which was introduced from Lake Magadi into the lake in 1961 as a mosquito control measure. Tilapia can tolerate a wide range of temperature and salinity ; it is found in water with salinity up to about 40 mil percent, pH 10.5 and temperature 40°C.

(d) Birds

There are more than 30 species of birds in the park, out of which Lesser Flamingo and White Pelican are the dominant.

Lesser Flamingo is a deep rose-pink small flamingo with a height of about one meter. This year very few The Lesser Flamingoes are observed in Lake Nakuru but many in Lake Elementeita. Lesser Flamingo has the following characteristics:

- Inhabits typically large alkaline or saline lakes.
- Sudden movement in enormous numbers to other lakes due to such causes as breeding and reduction of Spirulina density, etc.
- Feeding by a special tongue to extract Spirulina in size ranging 40-200 µm. Food requirement of individuals some 60 g in dry weight per day. Therefore a high density of Spirulina is indispensable for their habitat.
- Breeding in Lake Natron in North Tanzania. No breeding in Lake Nakuru has been recorded.

With introduction of Tilapia in the lake, the obvious effect on the ecology of the lake has been the appearance of more than fifty species of fish eating birds. Among the birds, the Great White Pelican is the main predator. The Pelicans consume large quantities of fish.

(e) Mammals

There are various and numbers of aquatic animals in Lake Nakuru, including Hippopotamus (Hippopotamus amphibus) and Clawless Otters (Aonyx Capensis). More than 70% of Hippopotamus live in the pools of the northern shore rushy swamps, where water is fresher due to the springs. The rushes provide cover and shade of the Kikuyu grass (Pennisetum clandestinum) under Acacia trees which is preferred food for the Hippopotamus.

(3) Lake level fluctuation

The lake level have been recorded by MOWD at the gauging station 2FC4 for a 34-year period from 1951 to 1984. The recording has unfortunately been interrupted since 1984. During the Phase III Study period, the altitude of the zero point of the staff gauge has been rectified; it was actually 2.7 m higher than the national datum level. The lake level records are given in Fig. 8.6. The lake level was surveyed at El. 1,758.5 m in July, 1990.

The lake level fluctuation shows almost the same tendency as Lake Naivasha. During the recorded period, the highest level was recorded at El. 1,763.3 m in August, 1979, while it has been reported that Lake Nakuru dried up in 1939, 1947, 1955, 1956 and 1961. In dry seasons before 1953, it has been reported that the floor had been subjected to wind erosion resulting in clouds of alkaline dust being blown over the surrounding area. Most recently in 1987, the lake was almost dried up, leaving water on a very limited area.

(4) Lake water quality

The water sampling was carried out by the same manner as Lake Naivasha. 11 sampling points were distributed evenly over the lake area as shown in Fig. 8.3 and the water were sampled from both the upper and lower layers. The summary of the water quality analysis are summarized in Table 8.2 and explained below.

- The lake water is extremely high alkaline, having pH value more than 10.3 and the conductivity more than 17,000 μs/cm.
- The lake water is excessively under eutrophication, judging from high concentration of N and P values.
- Low Do value zone was recorded in the aphotic. Especially, on the northern part of the lake (sampling point. 1), very low DO value is detected (shown in Fig. 8.12) probably due to a large amount of organic sediment. Organic sediment consumes DO, and "sampling point. 1" is closely located to outlets of sewage treatment plants.
- High DO value in (9.7 on average) the upper layer means that it is saturated by photosynthesis by plankton.

(5) Sewage Inflow

There are two sewage treatment works, the Town and Njoro treatment works, with a total treatment capacity of 7,000 cu.m/day in Nakuru municipality, The actual quantity of sewage inflow has, however, amounted to 8,840 cu.m/day. All the outflow from the treatment works is directly being discharged into Lake Nakuru.

Sewage water in raw and after treatment has been sampled from the Town and Njoro treatment works. The summary of the water quality analysis is summary is given in Table 8.2 and is briefly reported below.

- Such items as pH, conductivity, DO, and COD have showed of increased value after treatment in the Njoro treatment works. It is considered that the treated water is stored in lagoon where are plenty of nutrient salts such as N and P and Phytoplankton, resulting in increasing photosynthesis. The photosynthesis also results in increase in pH and DO.
- The values of COD and SS have been exceeding the effluent standards set forth by NMC and MOWC. The standards are shown in Table 3.10.

In addition to the above, it is to be noted that residues of different heavy metals (Arsenic, Tin, Copper, Zinc, Mercury and Cadmium) have been detected in different tissues of birds (liver and kidneys of Pelicans and Pink Flamingos) as well as in the fish (Tilapia grahami) according to Dejoux in 1981

8.3 Forecast Environmental Impact

8.3.1 Proposed Malewa Reservoir Area

(1) Inundation of land and resettlement of inhabitants

The reservoir impounds an area of 3.9 sq.km, of which 13.1 ha is cultivated. Five households needs resettlement.

(2) Eutrophication of reservoir

COD value is forecasted to reach 16 mg/liter, which is twice the present value of the Malewa river water. The water quality analysis predicts that the reservoir will be eutrophic. It is also predicted that I-P and T-N values will not increase so much from the present level, which means that the magnitude of eutrophication will remain at low level.

The total storage volume of the reservoir (72 million cu.m) is relatively small against the annual mean inflow (100 million cu.m) and the annual mean discharge to be diverted to the Turasha river (32 million cu.m). Therefore water in the is expected not to stagnate for a long period in the reservoir. It is also expected that water released from the reservoir will not cause acceleration of eutrophication of Lake Naivasha.

8.3.2 Lower Malewa River

It the water supply is made as planned, the Malewa dam will divert the water amounting to 166,000 cu.m/day at a full development stage. The reduction in stream flow occurs accordingly in the reach downstream from the dam, by some 27 % on average at the confluence of the Malewa and Turasha rivers. It is necessary to conserve the existing water right and aqua-eco system, function of river, etc.

8.3.3 Lake Naivasha

(1) Change in lake waer balance

It is explicit that the implementation of the Project primarily reduces the inflow into Lake Naivasha, which subsequently causes to decline its lake level.

The water balance of Lake Naivasha is subjected to the direct precipitation on and evaporation from the lake, inflows into the lake by rivers, abstractions by existing intakes and change in subterranean flow. The lake level virtually fluctuates as a result in change in water volume in the lake. A water balance simulation model was constructed to simulate the future change in the lake water balance. The lake water balance has been calculated for the varied water supply quantities in order to grasp a sensitivity.

The water supply quantity by the Stage 1 Project (19,000 cu.m/day), the Kipipiri Project (6,100 cu.m/day), and the Ol Kalou Project (16,400 cu.m/day) are also taken into consideration. The Kipipiri and Ol Kalou Projects are located upstream of the proposed Malewa dam site.

The water balance calculation is iterated until a new equilibrium balance is gained, assuming that the same hydrologic and climatological conditions during the period from 1961 to 1984.

The results of the simulated water balances are shown in Fig. 8.13 and summarized below.

Items	Without the Project	Case No.					
		1	2	3	4	5	6
Water supply (m^3/day)	0	56,000	105,000	121,000	138,000	151,000	166,000
Lake level (El.m)							
Max	1,886.9	1,886.4	1,886.2	1,886.0	1,885.8	1,885.7	1,885.5
Average	1,885.2	1,884.8	1,884.3	1,883.9	1,883.6	1,883.4	1,883.1
Minimum	1,883.2	1,883.0	1,882.5	1,882.0	1,881.5	1,881.0	1,880.0
Fall of lake level (m)							
Highest	0	0.5	0.7	0.9	1.1	1.2	1.4
Average	0	0.4	0.9	1.3	1.6	1.8	2.1
Lowest	0	0.0	0.5	1.0	1.5	2.0	3.0
Lake area (sq.km)							
Highest	297	257	235	226	213	207	198
Average	185	170	156	150	145	143	139
Lowest	138	138	133	129	123	117	101

If the water supply quantity of 166.000 cu.m/day is fully abstracted from the Malewa river basin even in drought period (Case 6), the lake level will fall extremely to El. 1,880 m.

(2) Water quality change

The change in the lake water quality has been forecast by using the numerical simulation method for varied lake levels as summarized below.

Lake level (El.m)	DO (mg/l)		COD (mg/l)		T-N (mg/l)	T-P (mg/l)
	Upper	Lower	Upper	Lower		
1,883.1	9.6	8.3	38.9	38.3	3.33	0.3
1,883.4	9.6	8.1	38.8	38.1	3.34	0.3
1,883.6	9.6	8.0	38.6	37.9	3.34	0.3
1,883.9	9.6	7.8	38.1	37.4	3.35	0.3
1,884.3	9.6	7.7	37.8	37.0	3.35	0.3
1,884.8	9.6	7.2	39.3	38.3	3.4	0.3

The water quality show is forecast to remain almost the same as the present, little change, probably due to the fact that the decreased water quantity is quite smaller compared to the lake volume. The conductivity and pH may be likely to remain within the same fluctuation range as recorded in the past.

(3) Ecology

The lake shore will set back some 3 km at maximum in the northeast part of the lake and surface area will reduce to 73 % as shown in Fig.8.14. The share of the aquatic plant in the surface area becomes larger.

(4) Isolation of crescent island Bay

At the lake level below El.1,881 m, the lake bed between the island and the eastern shore of the lake would be exposed as shown in Fig. 8.14. There would appear an another small lake in a dry period on the east of the Crescent Island. Otherwise a dredging work will be required in order to avid salifying lake water and to secure water traffic for fishery and tourism.

(5) Groundwater

It is hardly possible to state quantitatively the impact of the Project on the groundwater resources for a moment, since the investigation period was short and available data were limited. However, it is hardly likely that the groundwater resources will be affected seriously as far as the lake level maintains higher than the lowest level reached in 1988. It has been endorsed by such facts that there had been any serious draw down in the groundwater table and change in water quality in the 1988 drought year.

(6) Agriculture

If the lake level falls extremely to El. 1,880 m, most of the existing pumping facilities eventually will become impossible to be operated properly for a long period, probably resulting in severe damages in horticulture crops and fodder production.

(7) Fishery

The causal relationship between the water quality, aquatic plant, plankton, and the number of fish in the lake have not been cleared yet. It is explicit, however, that the fish catch area would decrease to a certain extent. Fig. 8.15 shows the relationship between the lake level and the fish catch records since 1962, which shows such general tendency that the fish catch decreases as the lake level decline.

8.3.4 Lake Nakuru

(1) Change in lake water balance

The water balance of Lake Nakuru is principally the same as that of Lake Naivasha. A simulation model was also developed based on the climatological and hydrological records during a 24-year period from 1959 to 1982.

An attempt was made to measure a sensitivity of the lake water balance against the sewage inflow rate. The augmented sewage inflow will continuously induce rising the lake level until a new water balance between the inflow and the evaporation is created. Thus the water balance calculation was iterated until the new water balance is gained, assuming that the same hydrological and climatological conditions during the period from 1959 to 1982 are repeated. The results of the water balance calculations are given for the respective sewage inflow rate in Fig. 8.16 and below.

Items	Without Stage 2 project	Case No. 1	2	3	4	5
Sewage inflow (m^3/day)	8,840	17,400	34,700	52,000	69,200	95,000
Lake level (El.m)						
Max	1,760.6	1,761.2	1,762.9	1,765.1	1,767.3	1,771.7
Average	1,758.6	1,759.3	1,761.0	1,763.3	1,765.6	1,770.1
Minimum	1,756.6	1,757.5	1,759.5	1,762.0	1,764.3	1,768.9
Rise of lake level (m)						
Highest	0	0.6	2.9	4.5	6.7	11.1
Average	0	0.7	2.4	4.7	7.0	11.5
Lowest	0	0.9	2.9	5.4	7.7	12.3
Lake area (sq.km)						
Highest	52.0	53.9	58.7	65.4	72.4	90.0
Average	43.7	46.7	53.2	59.9	67.2	81.2
Lowest	26.0	36.9	47.0	56.1	63.0	76.9

The impact on Lake Nakuru would be more serious than on Lake Naivasha. The water supply to the municipality area will increase year after year, and reach as much as 135,800 cu.m/day in 2015. Along five cases studied, the Case 5 shows the whole of the potential sewage effluent of 95,000 cu.m/day, corresponding to 70 % of the water supply to the municipality area in 2015, discharging into the lake after treatment by such treatment works as the Njoro treatment works.

(2) Submergence of land

With increasing the lake level, the lake area naturally expands, in other words, a large extent of feeding ground (60 km^2) and ring road along the lake rims will be submerged. The effects of the lake level rises on the land and road are shown in Fig. 8.17 and as summarized below.

Lake level	Submergence of Ring Road		Submergence of Land
El. 1,758.5 m	Existing lake level in July, 1990		0
El. 1,760.0 m	4 %,	Southern part	1 %
El. 1,761.0 m	12 %,	ditto	4 %
El. 1,762.0 m	26%,	Southern and northern parts	7 %
El. 1,764.0 m	65 %,	ditto	13 %
El. 1,766.0 m	70 %,	ditto	20 %
El. 1,768.0 m	74 %,	ditto	25 %
El. 1,770.0 m	90 %,	Eastern part	33 %
El. 1,772.0 m	100 %,	All of road is submerged	40 %

It is to be noted that at the lake level El. 1,771.7 m, the lake area envelops part of the Nakuru Municipal area of about 12 sq.km, the whole of the ring road, and the grass land of about 25 sq.km. The submergence of the land will create a severe problem for human activities and habitual condition of wildlife, while large expenditure will be incurred for relocation of the ring road.

(3) Change in water quality

The change in the lake water quality has been forecasted by using the numerical simulation method as adopted for Lake Naivasha. It was also made for varied lake levels as given below.

Lake Level (El.m)	DO (mg/l)		COD (mg/l)		T-N (mg/l)	T-P (mg/l)
	Upper Layer	Lower Layer	Upper Layer	Lower Layer		
1,758.6	9.9	7.6	191.5	192.4	31.85	5.28
1,761.0	9.7	0.1	183.5	177.1	63.98	18.57
1,765.6	9.7	0.0	180.9	187.5	64.37	20.86
1,770.1	9.4	0.0	188.8	193.3	73.58	23.82

In Lake Nakuru, it is deduced that T-N and T-P will continue to increase over a considerable period of time. COD value is too high in present situation, so it is predicted that COD value may not change much. It is presumed that, with increasing sewage inflow, the lake area and volume expand proportionally, resulting in increasing the sedimentation and decomposition of organic matter (COD). The increased decomposition of organic matter will probably augment the consumption of oxygen, and the rise of water level will impede the supply of oxygen from the surface to the bottom, resulting in a great reduction of DO value to the level of almost nil in the aphotic zone.

Salinity in the lake water is foreseen to decrease with increasing the sewage inflow.

(4) Ecology

Particular attention should be paid to the the possible changes in the lake ecology due to the augmented sewage inflow.

As is recognized widely, Lesser Flamingoes are the precious and representative bird of the lake, and depend on Spirulina for their food. The growth of Spirulina is sensitively influenced by salinity of the lake water. It is inferred that the increased sewage and resultant rise in the lake level would have a great impact on a linkage between the water quality, Spirulina and Flamingo.

It is hardly possibly to forecast quantitatively and qualitatively the impact of the sewage inflow on a complicated lake ecology at this stage. Comprehensive and systematic research is requisite to assess properly the present and anticipated future ecology.

8.4 Conservation of Environment

8.4.1 General Description

It has been pointed out in Section 8.3 of this report that the Project implicates various effects on the present natural and social environment, unless adequate counter measures should be taken up. On the other hand the Project will ensure a safe water supply for an increasing population and a sustained economic growth in the region.

It is, therefore, indispensable to create an appropriate atmosphere for coexistence of the contemplated water supply and the natural and social environments. A conceptual countermeasure is introduced herein, which is, however, subject to further verifications and study in later stage.

8.4.2 Malewa River

As noted in the sub-section 8.1.2 of this report, there exists a large number of water users in the downstream reaches from the proposed Malewa Dam and Turasha intake, which should be preserved even after completion of the Project to keep their benefits. Further some additional water needs to be maintained in a river channel throughout the year for conservation of aqua-eco system, function of river, etc.

In consultation with MOWD/NWCPC, the rates of the conservation flows have been estimated as set forth below, which are larger than the existing water rights and the ones derived from the prevailing practice.

River	Low Flow Season (cu.m/sec)	Flood Season (cu.m/sec)
Malewa river, downstream from dam	0.22	0.22
Malewa river, downstream from 2GB1	0.35	0.83
Turasha river, downstream from intake	0.24	0.24

It is to be noted that the above conservation flows have been taken into consideration in formulating the proposed water development scheme.

8.4.3 Lake Naivasha

The conservation of the lake should be planned not only from the view point of natural environment but also from the national economic point of view. The lake is an important tourism resources and its water has been widely used for the cultivation of exportable horticulture crops. The excessive decline of the lake level will directly lead to loss of the national benefit.

In order to alleviate the impact of the Project, it is a prime concern to reduce the water supply to a certain extent. By such measure the fall of the lake level could be reduced and naturally the magnitude of the impact is decreased to a certain extent.

It is very difficult to set forth the irreducible lowest lake level, but it is deemed that the lake level El. 1,882.0 m, recorded in the 1988 drought period would be indicative. It is presumed that as far as the lake level is maintained at the lowest level even the drought year like 1988, no serious problem is foreseeable based on the following facts.

(a) Most of the existing intake facilities have already adjusted their locations to cope with the lake level El. 1,882 m.

(b) Segregation of the Crescent Island Bay from the main lake is virtually eliminated.

(c) Such level occurs very rarely and for a limited time in a year. The runoff of the Malewa river in 1988 correspond to a drought frequency once in 25 years.

(d) The water quality changes little and accordingly there will be no water quality problem in view of source of irrigation supply.

It has been verified through a water balance study the Project output will be reduced to 73 per cent of the initially envisaged quantity on a long term average, if the lowest lake level is set at El. 1,882 m.

8.4.4 Lake Nakuru

The impact on Lake Nakuru will be more serious than those on Lake Naivasha. It would be unavoidable to introduce a dynamic physical structural measure to settle the augmented sewage inflow to the lake. There would be several opinions such as diversion of the sewage into the other basin, re-use of sewage, and combination of them in order to minimize/eliminate sewage inflow into the lake.

One of the options is to use the treated sewage for irrigation. A very indicative attempt is presented below.

In view of soil, climate and consumptive use of water, it is deemed that cultivation of sugarcane seems appropriate. Its consumptive use is roughly estimated at 1,300 mm/year, ranging from 90 mm/month in April to 120 mm/month is September. The average annual rainfall is 900 m in Nakuru, of which 70% is roughly accounted for effective rainfall. Accordingly the required irrigation supply is 570 mm/year, corresponding to 5,700 cu.m/ha/year. Very roughly speaking, a potential sewage quantity of 95,000 cu.m/day may be wholly consumed by cultivation of sugarcane over 6,100 ha.

IX. ORGANIZATION AND MANAGEMENT

9.1 Organization for Project Implementation

As noted elsewhere of this report, the public water supply within the proposed service area is being managed by the various government agencies concerned. Further the implementation of the Project will have very sensitive environmental aspects. It is, therefore, deemed essential to organize an inter-ministerial committee within the Government of Kenya to provided overall management and to coordinate all the issues relative to implementation of the Project. The committiee should be organized by representatives from the Ministries of Environment and Natural Resources, Water Development, Tourism and Wildlife, Regional Development, Agriculture, Local Government, and Finance, and should be chaired by the representative from the Ministry of Water Development.

NWCPC will be assigned as an executing agency of the Project. For the pupose of the construction of the Project, it is proposed to establish a project office outside the headquarters. The basic functions of the office are :

- to arrange all the legal procedures for the implementation of the Project;

- to carry out the survey and investigation necessary for design and quality control of the works;

- to make tenders for the procurement of the contractors;

- to execute the construction supervisory works;

- to undertake accounting and auditing of the contract works.

The proposed organizational structure of the main office is indicated in Fig. 9.1. The project office is proposed to be built in Gilgil, but it may be required to place branch offices at such locations as the Malewa damsite, treatment works site, Nakuru municipality and Naivasha town to facilitate the day-to-day supervisory works.

9.2 Operation and Maintenance of the Project

NWCPC will have direct and overall responsibility for the operation and maintenance of the project facilities. Under the Stage 1 Project two offices are being built up : one in Gilgil town and the other in Nakuru municipality, for the purpose of the operation and maintenance of the project facilities and collection of water tariff. These offices will be expanded and strengthened in terms of manpower and facility. In addition it is advisable to construct offices at the terminal points of the Gilgil West and East, Ebrru and Naivasha transmission systems for the same functions as those of the Nakuru and Gilgil offices. For the Malewa dam and treatment works a control house and operation building have been proposed respectively for the pupose of the water resources management and proper water production.

Both NMC and NTC will be resposible for the water distribution within their administrative areas. It is also advisable to expand and strengthen the existing O&M system in both agencies in compliance with the increased water supply by the Project.

9.3 Proposed Organization for Environmental Monitoring

In accordance with the recommendation set forth in Chapter VIII of this report, an environmental monitoring will be executed not only during the pre-construction stage but also during and after the construction of the Project to assess the impact of the Project on the various fields. It could be affirmed that such objective can be achieved through effective and rational coordination and elaboration among the existing government and government organizations. Fig. 9.2 present a proposed organization, consisting of four basic elements, i.e., an inter-ministerial committee, a data bank, and environmental management agency, and coordinating organizations.

	Elements	Related Organizations	Major Roles
(1)	Inter-ministerial Committee	various government ministries	- Evaluation of environmental impact assessment - Approval of countermeasures
(2)	Data Bank	NWCPC	- Execution of the project including countermeasures - Establishment of data base for systematic storage of data and information collected by various organizations
(3)	Environmental Management Agency	NES	- Environmental impact assessment - Proposal for countermeasures

(4)	Coordinating Organizations	Ministry of Water Development	- Water quality observation - Hydrological observation
		Meteorological Department	- Climatological observation
		Kenya Wildlife Service	- Wildlife observation and management - Game control
		Fisheries Department	- Aquatic animal observation - Fishing control
		Land and Farm Management Service	- Land use monitoring
		Livestock Production and Management Service	- Livestock management
		Horticulture Crop Development Corporation	- Production reporting
		Nakuru Municipality and Naivasha and Gilgil Townships	- Population, water supply and sewage

X. ECONOMIC AND FINANCIAL EVALUATION

10.1 Introduction

This chapter presents the economic and financial evaluations, the water cost and assessment of the socio-economic impacts of the Project. Main issues for each of these aspects are summarized below.

(1) Economic evaluation

In economic evaluation, it is a key issue to identify the economic benefits accrued from the Project. Major primary benefit will be the sum of consumer's surplus and water revenue. It will be valued as much as possible, based on the estimation of consumer's willingness-to-pay, taking into account the differences among the beneficiaries' characteristics.

The economic cost of the Project will be obtained by converting the financial cost with some adjustments. With the economic costs valued, then, Economic Internal Rate of Return (EIRR) will be computed to assess the economic viability of the Project. Allowing for the inherent features of the Project, it is necessary to test the sensitivity of the EIRR under the following conditions:

(a) Varied lowest level of Lake Naivasha

As reported in Chapter VIII, the quantity of the water supply varies largely with the level of Lake Naivasha to be maintained, and finally from the environmental point of view, it is proposed to limit the lake level at El. 1,882 m at the lowest. The sensitivity will, therefore, be tested for the varied lake level.

(b) Upstream development effect

Also as reported in Chapter VIII, a certain amount of water abstraction has been allowed in the upper Malewa and Turasha river basins. The sensitivity will also be tested for the increased abstraction rates.

(2) Financial evaluation

Financial evaluation usually consists of the assessments of both the financial viability of the Project and of the impacts of the Project on the financial soundness of the executing body. It was found out, however, that the later is difficult to be assessed due to the insufficiency of financial data on NWCPC, which was just established in 1988. Financial Internal Rate of Return (FIRR) will be computed.

(3) Socio-economic impacts

Socio-economic impacts of the Project will be examined focusing on the impacts on the regional development and low income households.

10.2 Economic Evaluation

10.2.1 Method and Assumptions

The economic aspect of the Project is evaluated using the EIRR as an evaluation index. The evaluation procedure consists measurement of economic costs, identification of benefits and estimation of unit benefit, calculation of benefits, and calculation of the EIRR.

The economic evaluation was made under the conditions that the evaluation period is taken at 35 years from the completion of the initial stage of the Project. The Opportunity Cost of Capital (OCC) in Kenya is at 10% as adopted by IBRD.

10.2.2 Economic Cost

The cost of the Project basically consists of the capital cost, replacement cost, O&M cost and associated cost.

The associated cost is for the expansion of he reticulation networks in Nakuru municipality and both Gilgil and Naivasha towns, without which the economic benefit of the Project could not be gained. It has been estimated at Kshs. 291.5 million.

All the costs have been estimated at the 1990 market price as stated in Chapter VII, and therefore have to be converted into the economic cost, i.e., the real resources costs or "opportunity costs" incurred from the view point of the national economy. The following simplified procedures have been adopted for the adjustment.

(1) Elimination of internal transfer payments

Internal transfer payments such as taxes and interest charges must be excluded from the local cost portion, because they do not reflect real resource costs. The indirect tax concerning the Project is roughly estimated at about 8%, based on the recent macro data on indirect tax and GDP at the current price.

(2) Shadow pricing

(a) Land

There are the cultivation lands of 13.1 ha in the proposed Malewa reservoir area. The economic price of these lands can be represented by the production foregone, which is expected to be accrued from the most productive land use, cultivation of cabbage in this case. The economic price and yield of the cabbage were assumed to be Kshs. 500/ton, being the same as the market price, and 20 tons/ha in 1989. The annual gross revenue, therefore, amounts to Kshs. 131,000.

The present value of the gross benefit is estimated at Kshs.1,310,000 in total, resulting in Kshs.969,400 of the net benefit by applying the value-added ratio of 0.74.

(b) Labour

The shadow wage rate (SWR) of skilled labour is assumed to be the same as that at market price. For unskilled labour, 70% of the market wage rate is taken as its SWR, adopting the estimate by IBRD in the "Kericho-Kisii-Isebania Road Study in 1987-88".

(c) Cement

The cement is representative input of the Project, and can be procured domestically. Its price is, however, administered by the Government. Therefore the valuation of its economic price depends on how it will be procured. The latest statistic indicates that its production has recently stagnated and its volume for export had been decreased at 13.7% per annum from 1983 through 1988, while its domestic consumption had increased at 4.5% per annum during the same period. This means that the increasing demand of the