RIVER FLOW REGULATION: A CASE STUDY OF THE UPPER EWASO NGIRO NORTH RIVER

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DECLARATION

I declare that this thesis is my original work and it has not been presented to any University. All sources of information have been cited.

Signed..**Date**....15/10/99..

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This thesis has been submitted for examination with my approval as the University Supervisor.

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DEDICATION

To My Beloved wife Susan and Loving Children Eusebio and Grace

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ABSTRACT

Water shortage during dry season and very high flows during wet seasons has been identified as the major challenges in water resources management in Ewaso Ngiro North River Basin, Kenya (ENNRB). The increasing water demand of the growing population, unequitable allocation of water resources and the uneven distribution of dry weather river discharges lead to tension and conflicts over the water resources in the basin. This study has investigated the feasibility of developing proposed Archers Post and Kihoto reservoirs to mitigate this problem of water shortage. The regulation of outflow from these reservoirs should be such that the downstream water users are assured of certain quantity of flow in the year round while the upstream water users are able to use the dry season river flow to meet their irrigation water requirements.

The study objectives were to determine suitable reservoir sizes for the two sites and assess the performance of the reservoir in augmenting low flows. This was done through simulation of reservoir operations using Interactive River-Aquifer Simulation (IRAS) model for reservoirs with active storage capacity ranging from 50-million m^3 to one with 350-million m^3 at an interval of 50-million m^3 for each site. The criterion of selecting the suitable reservoir size was based on the quantity of water that was made available by the reservoir during low flow periods over the simulation period which was 27 years (1960-1986). If the resultant increase in volume of water made available by an increase in reservoir capacity was less than 15% of the volume of water made available by the preceding reservoir active storage capacity the increase was considered unsuitable. The next small reservoir size was considered the suitable reservoir capacity of the site. The assessment of reservoir performance of the selected reservoir sizes was done using low flow frequency analysis and the ability of the reservoir outflow to meet target releases.

The reservoir size that was selected for the proposed Archers Post site has a gross capacity of 400-million m^3 while the one for Kihoto site has a gross capacity of 161-million m^3. The dead storage for the Archers Post reservoir is 200-million m^3 with useful life span of 80 years and for Kihoto reservoir is 11-million m^3 with a life span of 100 years if the current sediment load conditions at the two sites prevail in future.

The reservoir operating policy used to simulate reservoir releases allowed the releases such that the available water in the reservoir was adequate for 14 weeks. With this operating policy the two reservoirs did not run dry at any time of simulation. It was found that with

this operating policy the Archers Post reservoir made available 3,001.4-million m^3 of water in the low flow periods during the 27 years of simulation while 897.25-million m^3 of water was made available by Kihoto reservoir over the same period. This water, that was made available by these reservoirs, has improved flow reliability together with the return period of low flow events, which are shown to have a marked improvement at the two sites.

This study is expected to be an important contribution to the management of water resources of ENNR, which Ewaso Ngiro North Development Authority (ENNDA) has identified as strategic resource for the development of the area under its jurisdiction. One way ENNDA can use results of this study will be to build on it to identify the water demand that is required to boost the economy of the region and can be satisfied by the outflows of these reservoir at a known reliability. It is also expected that based on the findings of this study ENNDA will now carry-out detailed economic analysis of these reservoirs to justify their development on the economic point of view.

ACRONYMS

ENNDA – Ewaso Ngiro North Development Authority

ENNR – Ewaso Ngiro North River

ENNRB – Ewaso Ngiro North River Basin

IRAS - Interactive River-Aquifer Simulation Model

LRP – Laikipia Research Programme

MLRRWD – Ministry of Land Reclamation Regional and Water Development

MOWD – Ministry of Water Development

NRM3 – Natural Resources Monitoring, Modeling and Management Project

NWMP – National Water Master Plan

RGS – River Gauging Station

SWMP – Soil and Water Management Program

UENNR – Upper Ewaso Ngiro North River Basin

WMO – World Meteorological Organization

WRAP – Water Resources Assessment Project

1. INTRODUCTION

1.1 Background and Problem Statement

Ewaso Ngiro North River Basin (ENNRB) is among the 5 drainage areas of Kenya and is code named Drainage Area 5. This area covers most of the northern Kenya and comprises of part or whole districts of Nyeri, Nyandarua, Laikipia, Samburu, Meru, Marsabit, Nyambene, Moyale, Garissa, Wajir, Isiolo and Mandera. The total area of this drainage area is 210,000 km^2 (MOWD, 1992) and is approximately 36% of the total land area of Kenya.

The main river in this drainage area is Ewaso Ngiro North River (ENNR) and its catchment is just a part of the whole drainage area. It extends from the latitude 0.5^{0}S to 1^{0}N and from longitudes 36.5^{0}E to 41^{0}E. ENNR drains the northern slopes of Aberdare Ranges, the northwestern slopes of Mt. Kenya, the northern slopes of Nyambene Hills and Slopes of Matthew's Ranges in Samburu District. The various study on the water resources of the ENNRB have sub-divided it differently depending on the objective of the respective study. The MOW [1963] sub-divided it into the upper, middle and lower catchment. The upper catchment is the catchment area in the upstream of the confluence of Ewaso Narok River and ENNR. This area is endowed with a network of perennial rivers and it is also in this part of the basin where Ranches and Agricultural activities are concentrated. While middle catchment is the area between this confluence and Melka Bulfayo which is short distance upstream of Merti. This area is characterized with dry river beds which provide incremental flow to ENNR only during wet seasons otherwise during dry seasons the ENNR does not get incremental flow in this section of the catchment except for the few springs on the southern banks plus discharges from Isiolo and Ngara Mara Rivers which cater for water loss through evaporation in this section. Another notable issue of Melka Bulfayo is that it is the point where ENNR leaves the rock bed to start flowing on the alluvial bed. The area to the downstream of this point is the lower catchment and ENNR in this area keep on shifting its course with seasons depending on the deposition of sediments. Lorian Swamps marks the physical end of this river after flowing for more than 700km. In this study the catchment has been sub-divided into upper and lower catchment where upper catchment is comprised of all that area that is to the upstream of River Gauging Station (RGS) 5E3. The RGS 5E3 is the last RGS from the upstream with continuous flow data over a long period of time and at the same time most of the water abstraction is done to the upstream of this point and very little water abstraction is currently being done to the downstream of this point. Therefore change in water resources management (particularly river water) in the area considered here as the upper catchment is bound to have a big impact to the flows in the downstream of RGS 5E3.

The upper Ewaso Ngiro North River Basin (UENNRB) is just a small part (7.3%) of the whole ENNRB (see Figure 1) and covers part of Nyeri, Meru, Samburu, Laikipia, Nyambene, Isiolo and Nyandarua districts. The Laikipia district covers most of this area (see **Table 1**) and the increased population in the district due to new settlement, which are mainly located in the marginal area of the district, has a big bearing in water resources management in the basin.

Table 1. District covering the UENNRB and the area they cover.

Districts	Area of Coverage (km^2)	Percent of the Total Area (%)
Laikipia	7453.3	48.5
Nyandarua	1223.1	8
Isiolo	2088.1	13.6
Nyeri	943.8	6.1
Meru and Nyambene	2104.2	13.7
Samburu	1544.5	10.1
Total	**15357**	**100**

Source SWMP GIS, U.o.N

Extremely low flows in the ENNR during dry periods to the extent that some of its section have turned ephemeral in recent times [Decurtin, 1990; MOWD, 1990; LRP, 1994] is a major problem that is encountered in water resources management in the basin. The drying of ENNR downstream of Bulesa has become a common phenomena [MOWD, 1990] and it is feared the same might happen to the section upstream of Buffalo Springs which is reported to have dried severally in the recent times [Decurtin, 1990; MOWD, 1990; MOWD, 1987; LRP, 1994]. The main cause of this problem is the high abstraction of river water during dry seasons to meet irrigation water requirements which is at the peak this time and these activities are concentrated at the upper catchment. This problem is bound to even get worse with the increased irrigation activities particularly in the newly opened marginal lands of Laikipia District.

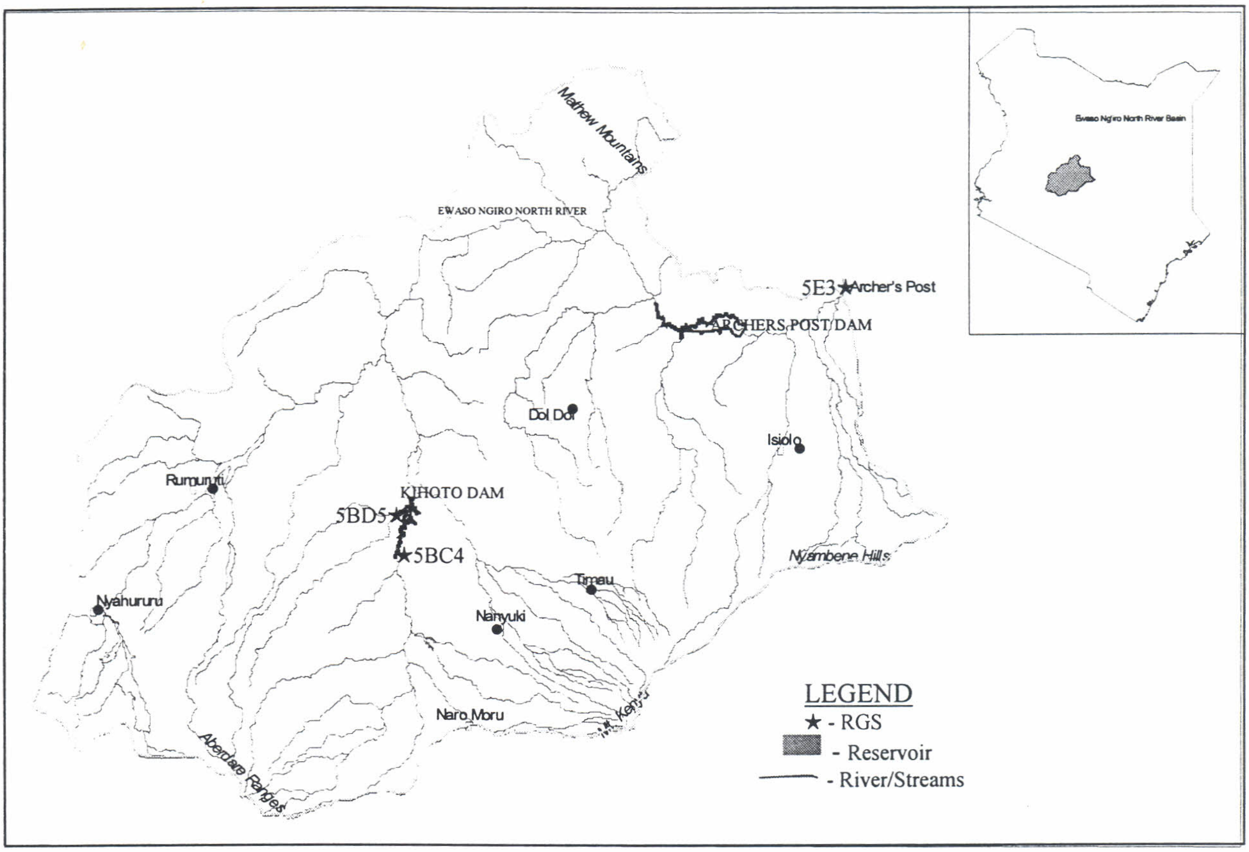


Figure 1 Location of the Study Area

The ground cover in the basin is being degraded through deforestation, opening up of marginal land for agricultural activities and overgrazing and in turn it is reducing the ground water recharge, which is subsequently reducing the base flow of the rivers in the basin. At the same time except at the slopes of the mountains and in the section between Buffalo Springs and Channelers Falls there is limited ground water contribution to the flow in the ENNR during the dry seasons. This implies that the continued over drawing of river water for irrigation in the upper catchment will result to even a bigger section of the ENNR drying in the upstream of Archers Post.

This problem of extremely low flows to the extent of the river drying in the downstream has adversely affected the inhabitants of lower parts of the basin, which is an arid area and livestock keeping is the mainstay of economy. Pastoralists in the area have established grazing patterns that rely on availability of both pasture and water and drying of parts of

ENNR has contributed to disruption of this pattern. Other negative effects that are related to reduced low flows are highlighted in Riggs et al., [1980] and Liniger [1995] as;

a) Degradation of water quality by increase in concentration of dissolved chemicals and increase in travel-time of pollutant through a river reach.

b) Degradation of aquatic life of this river as a result of rise in water temperature and decline in re-aeration capability of water pools.

c) Degradation of important aesthetic features such as the Channelers falls of the ENNR.

d) Increased conflicts over the water resources for instance when the pastoralists mostly inhabiting the lower reaches of river basins move to the upstream in search of water it often result to human conflicts on the resources whereas movement of elephants and other wildlife to the upstream in search of water result to destruction of crops and irrigation fields which leads to human/wildlife conflict.

The variability in flows within a year closely corresponds to the distribution of rainfall within the year [Decurtin, 1990]. During wet season high flows are registered in this river and it tends to increase with distance from the highlands. Sometimes floods in the downstream displace communities who have settled close to the river to higher grounds. Also affected are the pasturelands [MOWD, 1980], which are denuded and are not available for grazing for sometimes. Road communication is the other area that is adversely affected by flooding of this river in the downstream.

It is then hypothesized here that with proper water regulation the problem of water resources management in the basin will be adequately solved. Such regulation can be achieved through the use of proposed Kihoto and Archers Post reservoirs which this study aims at investigating the feasibility of using them for this purpose.

Most of the flood originate from the middle and lower parts of the basin [MOWD, 1980; Decurtins, 1990] and storage of floods in the UENNRB may reduce the impact of these floods in the lower parts of the basin without necessarily affecting the swamps and other riverine ecology that benefit from the flood flow of this river. Controlled release of water to the downstream is yet another benefit that will be derived from flood storage and this will enable water resources development in the downstream area. The flood storage is also expected to facilitate the farmers in the upstream to continue irrigating their land while the communities in the downstream are not adversely affected by their activities.

However the proposed sites in Archers Post and Kihoto are currently habitat of a number of wild animals and with denudation of the land these animals will be displaced and their migratory routes will be disrupted which may result to isolation of some species. Increased cases of diseases like malaria and bilharzia are yet other negative impacts that are expected if these reservoirs will be developed due to the increased free water surface that will result to increased multiplication of mosquitoes and snails which transmit these diseases. The developed reservoir will trap sediment load from the catchment in their upstream and the discharge water will have a lower sediment concentration which will result to the river-bed and banks being eroded in the process of the river regime trying to regain back its sediment carrying capacity. The trapped sediment will also result to reduced nutrients to the flood plains which will affect the ecosystem that depend on these nutrients. The increased surface area of water will increase water loss through evaporation and seepage. The water lost through seepage may even result to water logging in another area with detrimental effects.

1.2 Rationale of the Study

According to MOWD [1990] the flow data of this river have not been analyzed in detail with an aim of determining whether it is possible to develop storage reservoir along this river. However good sites for reservoir development along the stretches of this river have been identified largely based on the available topographical data [MOW, 1963; MOWD, 1980; MOWD, 1992]. This study aims to build on this information by analyzing the river flow and topography data and evaluate the extent to which the outflow from the proposed reservoirs can be used to regulate the flow in the downstream.

The importance of storage facilities being developed along this river has been cited severally [MOWD, 1990; MOWD, 1991; MOWD, 1992; ENNDA, 1995] since there is no potential left for a permanent domestic and livestock water supply without storage in this river [Gichuki et al., 1995; MOWD, 1990]. The proposed reservoirs will then be very important in water resources management in the basin and their impact to the downstream hydrology should be studied in detail. This study will contribute toward this end by quantifying the impact of the proposed reservoirs in augmenting low flows.

Ewaso Ngiro North Development Authority (ENNDA) has identified ENNR as a strategic resource in its pursuit of developing the region under its jurisdiction [ENNDA, 1995] and the control of this river is of paramount importance if ENNDA will ever use this river for this purpose. It is in this line this study was initiated to investigate the feasibility of developing the proposed Archers Post and Kihoto reservoirs for flow regulation.

The location of the reservoir to be developed in the basin has been an issue given that the inhabitants in lower parts of the basin view a reservoir being developed in the upstream as a favour to the already endowed people in the upper catchment despite the suitability of such a site in terms of it having low siltation rate, low evaporation losses and good topography for dam construction. This study brings forward these issues and pin points the importance of each of these reservoirs.

Interactive River-Aquifer Simulation (IRAS) model which is used in this study was introduced just recently to be used in water resources management by the Water Resources Assessment Programme (WRAP) in the Ministry of Land Reclamation Regional and Water Development (MLRRWD). It has not been used in reservoir operation study in the ENNRB and this study will serve as demonstration of how it can be used.

Developing of storage reservoirs requires heavy investments. This calls for good planning which should result to efficient use of the resources. This study aims at recommending the reservoir size that is suitable for a particular site based on the topography and flows at that site.

1.3 Objectives of the Study

The overall objective of this study is to investigate the feasibility of developing reservoirs at the proposed sites in Kihoto and Archers Post areas.

The specific objectives are

i) To collect, collate and analyze the topography, flow, evaporation and sediment data for both Kihoto and Archers Post reservoir sites.

ii) To simulate reservoir operation study using IRAS model at the two reservoir sites.

iii) To quantify stream flow regulation due to the proposed reservoirs.

1.4 SCOPE OF THE STUDY

The scope of this research study was limited to hydrologic study of flow regulation by the proposed reservoirs. Other reservoir design considerations such as structural design of the dam embankment, environmental impact assessment, analysis of economic viability, analysis and collection of geological information and the impact of land use change to the proposed reservoirs were not addressed in this study.

2. LITERATURE REVIEW

2.1 PREVIOUS STUDIES ON THE WATER RESOURCES OF EWASO NGIRO NORTH RIVER BASIN

According to MOW [1963] in 1958 the Government of Kenya initiated a study to investigate into the water resources of the ENNRB with a broad terms of reference. The main objective of the study was to give factual account of water resources of the catchment and agricultural potential and then make specific recommendations as to the best means of water development bearing in mind the economic and humanitarian aspects of the problems.

In water resources management it was found that the dry season river flow is limited and as a consequence the availability of water rather than the availability of irrigable land limit the area that can be irrigated in the basin. Construction of dams in the upper catchment was viewed as a means of enabling greater flow regulation, which will result to flood peaks being reduced and low flow run of river being increased. This way the available water resources would be utilized in a better way and at this stage is when Archers Post and Kihoto Reservoirs among others were identified. Kihoto (Randall) Reservoir was highly recommended for development and its main function would have been to provide compensation flow to the downstream and some pumped irrigation could be undertaken. Archers Post reservoir was recommended for development only if the irrigation development in the lower catchment was envisaged.

In the National Master Water Plan Study [MOWD, 1980] sites that were identified in MOW [1963] were investigated further in the point of view wheather they can be used in promoting irrigation agriculture in the basin. This study identified irrigable land in the upper catchment and in the lower catchment to be 30,000 ha. (15,000 ha. apiece) and concluded that the regulated storage at Archer Post reservoir will be sufficient to irrigate 25,000 ha.

In National Water Master Plan Study (NWMP) these reservoirs (see table 2) identified in the previous studies were taken through a detailed screening process to rank them among other national water project [MOWD, 1992]. The Archers Post reservoir was discarded at the second screening stage on the view that the available active storage capacity was less than the required storage that was derived from reservoir yield-draft curve developed in the study for this purpose. Kihoto reservoir was discarded at third screening stage, which was the last before the project was identified as a prospective project. The reason of discarding it was that it was remote to the demand point. However these two reservoir projects were recommended as alternative future projects for subsequent detailed investigation at regional level.

Table 2 Proposed Reservoirs Investigated in the NWMP Study

Dams	Rivers	Estimated Active Storage (m^3)	Co-ordinates X (m)	Co-ordinates Y (m)
Rumuruti	Ewaso Narok	17.1×10^6	37 216,500	0016,650
Nyahururu	Nyahururu	9.8×10^6	37 201,900	0003,950
Archers Post	Ewaso Ngiro	214.3×10^6	37 335,500	0063,200
Crocodiles Jaw	Ewaso Ngiro	21.7×10^6	37 264,000	0067,000
Kirimuni	Ewaso Ngiro	*	37 269,750	0082,500
Kihoto	Ewaso Ngiro	672.3×10^6	37 267,600	0027,650
Ngadurumuto	Ewaso Ngiro	74.1×10^6	37 269,000	0039,800
Gage	Ewaso Narok.	*	37 261,000	0057,200
Barsalinga	Ewaso Ngiro	-	37 273,712	0082,950

Source MOWD [1992] - not established * negative value due to siltation

In all these studies the problems of high siltation and high water loss through evaporation at Archers Post reservoir site are highlighted together with its problem of lack of an obvious good site for dam construction. Kihoto reservoir site is viewed as a good site on the basis of having a site with a good topography for dam construction and it experiences low siltation problem. But this site has low flows and the flow regulation is expected to have an impact only in the upstream of Archers Post.

2.2 Impact of Reservoir Development to the Environment

Reservoir development like many other developments is an effort of man to manipulate the environment for the betterment of the human population [OAS, 1978]. The overall effect of reservoir development is complex, as there are both positive and negative impacts related to environmental, economic and social factors [Liniger, 1995]. Once a reservoir is developed a portion of land is submerged and associated problems to this include among others the displacement of people living on the land to be submerged as happened in the case of Ndakaini Dam in Thika District, denudation of natural forests and extinction of endangered plant species, displacement of animals from their natural habitats and disruption of their migration route, denudation of infrastructures such as roads and buildings and foreclosure of future development alternatives. The increased surface area of water due to reservoir development is associated with two main problems, that is, one it provides a suitable area for breeding of mosquitoes and other water-borne diseases vectors which in effect result to increased health risk for the people living within the vicinity of the reservoir [OAS, 1978] and two, the evaporating surface of water is increased and as a result more water is lost from the river basin water system through evaporation [Liniger, 1995].

The dam structure that is used to block the river when a reservoir is developed provide a barrier to the movement of riverine species [OAS, 1978]. This affects adversely those species that require different environments for bleeding and living in which the river provides the only migratory route.

A reservoir will trap most of the sediment from its catchment area which alter the quality of the flow to the downstream and in turn the river regime is altered [Liniger, 1995]. Bed degradation due to release of clear water from reservoir can be extremely dangerous not only to the existing structures along the river but also to the planned ones. According to Gasser and Gamel [1994] after the development of Aswan High Dam, Nile River started to degrade its bed and banks in the downstream causing a change in the regime of the river. This affected the barrages along this river and navigation. Development of reservoir in some cases results to exaggeration of both the degree and incidences of disasters occasioned by natural phenomena such as earthquakes.

2.3 RIVER FLOW REGULATION BY A STORAGE RESERVOIR

Streams and Rivers are important sources of surface water and they are the source of water for various uses such as water supply for municipal and domestic use, irrigation water supply, hydropower generation and navigation among others. Satisfactory functioning of water development schemes is largely dependent on the flow variation in the river. For instance a river with extremely low flows in parts of a year and devastating flood in other parts of the year will not provide suitable conditions for the above functions. A storage reservoir will then be an important facility in such a case to regulate flows mainly to assure that either certain quantities of water is available when and where needed, certain quantities of water is not present when and where it is not needed or a stated quantity of hydropower is generated when and where needed.

2.4 Reservoir Design Consideration

2.4.1 General

A reservoir site is a natural resource of a country and should be used to its optimum potential [Davies and Sorensem, 1969]. The optimal use of a reservoir site entail among others maximization of reservoir storage capacity that can be attained at a site and minimization of cost of developing a reservoir at a site. The factors for a given site that would influence its optimal use include topography, geology, availability of flow, sediment load discharge and proximity to point of water use from the site.

2.4.2 Topography of a Site.

Topography of a suitable site for developing a reservoir is ideally where the river valley is narrow and expands in the upstream. Another requirement for such a site is the river channel to have a gentle slope to obtain a long reservoir in proportion to the height of the dam [Linsely and Franzini, 1979; MOWD, 1991; Fair et al., 1966; Golzé, 1977]. This combination will ensure that the reservoir will have a large storage capacity.

The ratio of reservoirs active storage to the volume of embankment material, which is also a factor that is governed by the topography of an area, is used in assessing economic suitability of a reservoir [Fair et al., 1966]. MOWD [1991] recommends for development only those reservoirs whose ratio is above 8 and in MOWD [1992] it was found that the marginal value for the existing reservoirs in Kenya is around 15.

In addition to this the topography of the site should have favorable site for spillway, water diversion conduit and a suitable route for pipeline to convey water from the reservoir to the point of use [MOWD, 1991; Fair et al., 1966].

2.4.3 Geology of a Site

On all dam projects the water tightness of a reservoir, suitability of foundation for a dam embankment plus its appurtenant structures and availability of construction materials within the proximity of reservoir site are important geological and engineering considerations [USBR, 1977].

Seepage losses from a reservoir are important since dam failure may occur if seepage forces causes piping, interference of project functions could also be experienced if there is excess water loss from the reservoir through seepage or water logging may occur in the area where seepage water gets to the surface. Water tightness of a reservoir should then be such that the foregoing are prevented to the acceptable levels which will vary from one site to the other depending on the geological conditions prevailing in the area [USBR, 1977; Davies and Sorensem, 1969; Linsely and Franzini, 1979].

Compressibility of foundation material is the main factor that determine the extent of foundation settlement which has been found to be the common cause of foundations failure [Smith, 1981]. It is an important consideration especially where foundation material is firm soil or a solid rock with an underlying layer of clay. Degree of settlement is dependent of the void ratio of the foundation material, weight to be placed on the foundation and duration when this weight will be applied. The expected settlement should not exceed the allowable settlement, which depend on the type of dam embankment [Terzaghi and Peck, 1967; Smith, 1981; Davies and Sorensem, 1969].

The strength of the foundation material is the other important consideration in design of a reservoir. The two important dam embankment failures that are considered here are failure by sliding and sinking. The total frictional resistance to sliding of the foundation material plus its ultimate shearing strength must exceed the total horizontal force on the dam embankment (for all conditions of loading) by a safe margin (see equation (2.1)) [Terzaghi and Peck, 1967; Smith, 1981; Davis and Sorensem, 1969].

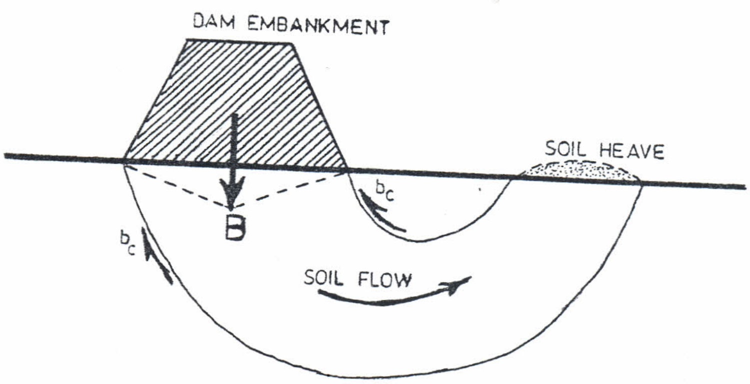
$$\frac{H_s + H_f}{H} \geq \text{factor of safety} \tag{2.1}$$

Where H_S – Ultimate shearing strength of foundation material, H_f – Frictional resistance to sliding and H – Total horizontal forces on the dam embankment.

When stress exerted by the weight of dam embankment exceed the bearing capacity of a soil forming the foundation failure may occur (see Figure 2) and to reduce the chances of such a failure stresses exerted by a dam embankment to the foundation should be less than the safe bearing capacity (equation (2.2)) of the foundation material [Smith 1981].

$$B_{safe} = \frac{b_c}{f_s} \quad (2.2)$$

Where B_{safe} –safe bearing capacity of foundation material, b_c – ultimate bearing capacity of foundation materials and f_s – Factor of safety.



Source: Smith, 1981

Figure 2. Foundation failure through rapture

2.4.4 Availability of Water at a Reservoir Site.

The available water for storage at a given point in the river is the flow that is in excess of the upstream water demand.

$$Q_{a(t)} = Q_{r(t)} - Q_{d(t)} \quad (2.3)$$

where $Q_{a(t)}$ – Available flow for storage, $Q_{r(t)}$ – River flow and $Q_{d(t)}$ – Upstream Water demand at t^{th} time period.

The cumulative quantity of available flow for storage will determine the rate at which a reservoir will fill after a drought sequence but it should be within a period of 2 or 3 years of average rainfall [Rofe, 1987].

2.4.5 Sediment Load Discharge at a Site

Dead storage in a reservoir is normally provided to accommodate the deposition of silt and will not be brought into use in a yearly cycle of inflow and outflow, rather it is an idle capacity of a reservoir. Its capacity is designed such that the operation of the reservoir will not be impaired within its useful life which ranges between 50 and 100 years [MOWD, 1992; USBR, 1977; Golzé, 1977; Singh et al., 1990; Linsely and Franzini, 1979]. The size of dead storage is a product of useful life span of a reservoir and the sedimentation rate. Its proportion to the gross storage capacity, is an important factor in determination of suitability of a reservoir but the acceptable value will vary from one project to the other depending on the importance of a reservoir.

2.4.6 Proximity of the Site to the Point of Use

Operation and capital cost of conveying water from a reservoir to the point of use is an important cost in the economic analysis of a reservoir development project. Certainly those sites that display low unit cost of conveying water will be preferred though this should be integrated into other economic considerations.

2.5 RESERVOIR OPERATION CONSIDERATION.

2.5.1 General

A conceptual model of inflows and outflows of a reservoir is presented in Figure 3.

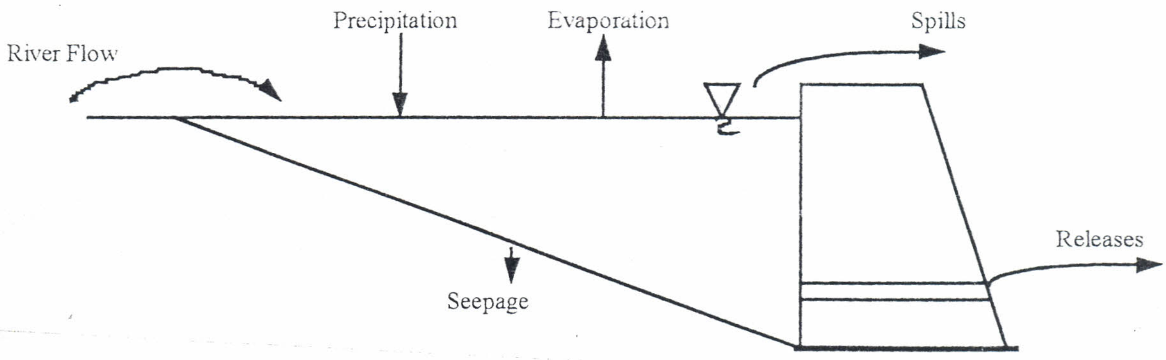


Figure 3. A Conceptual Model of Inflows and Outflows of a Reservoir

From this model the inflow into a reservoir will consist of river flow and precipitation at the reservoir site while outflow is made up of evaporation and seepage losses, spills and reservoir releases. Reservoir inflow and storage must be kept in proper balance if a certain reservoir yield is to be assured. This can either be done graphically by use of mass diagram where mostly data for a critical period is used or empirically through reservoir operation study using available historical data.

2.5.2 Reservoir Operation Study

The basic concept of a reservoir operation study is the law of conservation of matter, where in a reservoir the inflows equal the outflows plus the change in volume.

The change in volume in a reservoir at any time is expressed empirically in equation (2.4).

$$V_{(t+1)} - V_{(t)} = I_{(t)} + P_{(t)} - R_{(t)} - E_{(t)} - S_{(t)} - O_{(t)} \qquad (2.4)$$

Where $V_{(t+)}$, $V_{(t)}$ - Volume of reservoir contents at the end and beginning of t^{th} time period respectively, $I_{(t)}$ - Reservoir inflow, $P_{(t)}$ - Precipitation, $R_{(t)}$ - Reservoir releases, $E_{(t)}$ - Evaporation loss, $S_{(t)}$ - Seepage loss and $O_{(t)}$ - Spills during t^{th} time period expressed in volume units respectively.

End storage of a reservoir at any time can be evaluated by rearranging equation (2.4) to equation (2.5).

$$V_{(t+1)} = V_{(t)} + I_{(t)} + P_{(t)} - R_{(t)} - E_{(t)} - S(t) - O_{(t)} \qquad (2.5)$$

In which

$O_{(t)} = 0$ for $V_{max} > V_{(t+1)}$
$O_{(t)} = V_{(t+1)} - V_{max}$ for $V_{(t+1)} > V_{max}$
$R_{(t)} = 0$ for $V_{(t+1)} < V_{min}$

Where V_{max} - The maximum capacity of the reservoir and V_{min} – Dead Storage while other variables remain as defined in (2.4).

Equation (2.5) forms the basis of reservoir operation study [Linsely and Franzini, 1979; McMahon and Mein, 1978; Carthy et al., 1990]. This equation has been used severally for modeling and simulation of reservoir operations [Loucks et al., 1981; Loucks et al., 1995; Strycharczyk et al., 1987; Nazar et al., 1981; Goodman et al., 1981; Linsely and Franzini, 1979] but caution should be taken on the effect of starting condition and especially for short data [McMahon and Mein, 1978; Carthy and Cunane, 1990].

2.5.3 Quantifying Of Reservoir Inflow.

The inflow into a reservoir consist of river runoff at the reservoir site and precipitation over the surface of the reservoir. More often the runoff resulting from precipitation over the reservoir surface is included in the measured river runoff. Thus it is often ignored in the reservoir operations study.

The historical data of river runoff is used to simulate reservoir inflows and preferably the length of record should be greater than 20 years [Linsely and Franzini, 1979]. The use of historical data is based on the assumption that hydrological events are cyclic in nature and the flows will repeat themselves though this is rarely true [Beard, 1967].

An important consideration in the use historical data is the time period to be used in the analysis. It mostly depends on the requirement of the project, for instance MOWD [1986] recommend use of daily means for urban water supply projects and monthly means for rural water supply projects. For flood flow study a time period of a day or less is suitable as flood only last for few days or hours but for projects with large storage reservoir with annual carry-overs the mean annual flows will be adequate for analysis [WMO, 1983].

In using historical flow data to simulate reservoir inflows, problems of a site either having no flow data, flow data being too short or some missing data in the record are often encountered [Kuiper, 1965; Linsely and Franzini, 1979; MOWD, 1992; USBR, 1977].

In the situation where there is completely no record for river flow at a reservoir site, records for a nearby gauging station on the same river may be used to estimate flows at the site. In absence of a gauging station on the particular river, different methods are developed that enable use of data of a gauging station in another similar drainage basin where both precipitation and runoff data are available. In both cases comparative hydrologic study between the two drainage basin is done where after the correction for rainfall and other flow related factors the records are transferred on a unit-area basis [Hallas and Titford, 1971; Kuiper, 1965; Linsely and Franzini, 1979; MOWD 1992; USBR, 1977].

Where data length is too short to be used in a reservoir operation study, it can be extended either by

i) Correlating flows at the station with concurrent flows at nearby station(s), which has a longer data, to develop a regression relationship that is used to extend the shorter data by applying it on the data for the time periods where data is required to be extended. Both WMO [1983] and MOWD [1992] recommend the coefficient of determination of such regression equations to be greater than 0.6.

ii) Correlating flows at the station with meteorological data of the catchment area and use this relationship to generate flow data of the length equal to that of meteorological data [Kuiper, 1965; MOWD, 1992; USBR, 1977; WMO, 1983] or

iii) Generating synthetic data from the statistical parameters of the available flow data [WMO, 1983; Linsely and Franzini, 1979; Thomas et al., 1962; Fair et al., 1966]. Equation (2.6) is commonly used in generating synthetic flow data from the available data.

$$Q_{i+1} = \overline{Q}_{j+1} + \beta_j(Q_i - \overline{Q}_j) + t_i\, \sigma_{j+1}(1-\rho_j^{2})^{1/2} \tag{2.6}$$

where Q_i, Q_{i+1} - Estimated runoff during i^{th} and $(i+1)^{th}$ time period respectively both counted from start of the operational sequence, $\overline{Q}_j$, $\overline{Q}_{j+1}$ - Mean value of flow record during j^{th} and $(j+1)^{th}$ time periods of an annual cycle respectively, β_j - Regression coefficient for estimating flow in $(j+1)^{th}$ time period from flow in $(j)^{th}$ time period, t_j - Random normal deviate with zero mean and unit variance,

σ_{j+1} - Standard deviation of flows in $(j+1)^{th}$ time period of a year, ρ_j - Correlation coefficient between the runoff in j^{th} time period of year and $(j+1)^{th}$ time period of a year, i = 1, 2 .... Total number of time period in the operation sequence, j = 1, 2............ Total number of time periods with an annual cycle.

The same procedures are applicable in filling gaps in a data record where there are missing data but where gaps are only for few days interpolation can be done to fill the gaps for instance in the WRAP report of Laikipia District [MOWD, 1987] gaps of 7 days and less were filled by interpolation. The 7 day gap is reasonably small particularly when the length of the data under consideration is long and is not expected to affect the overall statistical parameters of the data.

2.5.4 Quantifying Water Loss from a Reservoir.

2.5.4.1 General

The mean reservoir outflow is always less than the mean reservoir inflow due to reservoir water losses through seepage and evaporation [Goodman et al., 1981; Linsely and Franzini, 1979; MOWD, 1991; Loucks et al., 1982]. Evaporation losses are evaluated either empirically or by direct measurement while seepage losses are evaluated using seepage functions.

2.5.4.2 Evaporation Loss

Evaporation is the net loss of water from a surface. The rate at which water will evaporate from water surface in a given area will depend on the wind velocity, temperature of water and air, humidity, solar radiation and area of water surface.

These are the factors used in empirical methods of evaluating evaporation from a surface. The two approaches normally used are the mass transfer and energy budget approaches.

The mass transfer equations are based on Dalton law (equation 2.7) on vapour transfer due to turbulence in the air [Bruce and Clark, 1966; Linsely and Franzini, 1979; Sharp and Sawden, 1984].

$$E = f(u)\ (e_s - e_a) \tag{2.7}$$

where E – Evaporation (depth/time), f(u) – A function of horizontal wind speed, e_s – Saturation vapour pressure at the temperature of the water surface and e_a – Atmospheric vapour pressure

This equation is empirical and the function of wind speed (f(u)), must be determined for each particular locality. The equation developed is normally not transferable to other situations [Sharp and Sawden, 1984]. Lake Hefner equation (equation 2.8) is one such equation and equation (2.9) was developed after tests on this particular Lake [Bruce and Clark 1966; Linsely and Franzini, 1979; Sharp and Sawden 1984].

$$E = 0.122(e_s - e_2)\ u_2 \tag{2.8}$$

$$E = 0.097(e_s - e_8)\ u_8 \tag{2.9}$$

Where E – Evaporation from the reservoir (mm/day), e_s – Saturation vapour pressure at the temperature of the water surface (mbar), e_2,e_8 – Atmospheric vapour pressure at 2m and 8m above the water surface respectively (mbar) and u_2,u_8 – Wind velocity at 2m and 8m above the water surface respectively (m/s)

Equation (2.9) will change if the atmosphere vapour pressure and wind velocity are measured at different heights from the ones given. The vapour pressure and wind velocity must be measured carefully otherwise large errors will result. Another limitation of the use of this method in estimating evaporation loss from a proposed reservoir is the alteration of micro-climate of an area once a reservoir is developed in an area [Linsely and Franzini, 1979].

The energy balance equations are based on the principle that heat energy that get into a water mass must balance with heat lost from a water mass [Bruce and Clark, 1966; Linsely and Franzini, 1979]. This approach entails measurement of net radiation, heat flux into and out of the ground, heat flux from and to the air and heat flux from and to water mass. These measurements are highly specialized and costly which limits the use of this method in estimating evaporation from a proposed reservoir. The method is also limited further by the requirement of measuring water temperature, which is only possible in an existing reservoir.

Penman combined the two approaches to develop equation (2.10), which permits evaporation to be estimated from climatological observations of frequently measured elements [Bruce and Clark, 1966; Sharp and Sawden, 1984].

$$E = \frac{\Delta Q_n + \gamma E_a}{\Delta + \gamma} \tag{2.10}$$

Where Δ - Vapour pressure gradient at air temperature, Q_n – Net radiation expressed in evaporation units, Ea – Mass transfer evaporation, (Ea = f(v) (e_s – e_a) (Water temperature is assumed to be equal to air temperature for determination of e_s)) and γ - psychomatic constant (0.61 if temperature is measured in ^{0}c).

Δ, Q_n, and E_a are evaluated empirically and require rigorous computations using a wide range of meteorological data [Sharp and Sawden, 1984] and this is the main limitation of the use of Penman equation in the estimation of evaporation losses from reservoir. The other limitation of this method is the assumption that air temperature just on the surface of water and that of the water surface is the same which have been found to be true only when the lake is extremely shallow or the surface being considered is a small water mass [Bruce and Clark, 1966; Sharp and Sawden, 1984]. Coefficients are then used to correlate reservoir evaporation to evaporation evaluated using this equation.

Direct measurements of evaporation in the field are done by use of evaporation pans. Unfortunately a pan cannot simulate a lake wholly and test have shown that pan over-estimate evaporation from the surface of a lake [Hounam, 1973]. Small amount of water in a pan is more exposed to energy input in form of heat than a large mass of water in a lake, which experiences stabilizing effects of convection currents and of the earth around it [Sharp and Sawden, 1984; Linsely and Franzini, 1979]. In effect the evaporation from a pan tends to be higher than that experienced from a lake.

Evaporation as measured by the pan is correlated to evaporation from the surface of a lake by applying pan-to-lake coefficient. The value of the coefficient depends on the type and exposure of evaporation pan. For class A pans, which are widely used in Kenya [Kaila, 1983; MOWD, 1992] the coefficient ranges from 0.6 to 0.8 with an average value of 0.7 [Hounam, 1973; WMO, 1983; Linsely and Franzini, 1979; Bruce and Clark 1966].

The higher value of 0.8 is used in humid climates and seasons while the lower of 0.6 is used for arid climates and dry seasons [WMO, 1983; Linsely and Franzini, 1979; Bruce and Clark, 1966]. Factors that can be attributed for this difference in pan-to-lake coefficient for different climates and seasons are

(i) Difference in temperature of pan water and the surrounding air in different climates.

(ii) Thermal inertia effects of a large water mass as season changes.

In humid climates the temperature of water in a pan tends to be higher than the air temperature and this result to heat being transferred from the pan to the surrounding air reducing the evaporation from the pan while in arid climates the temperature of water in the pan tends to be less than the air temperature and this result to increased evaporation from the pan due to heat energy transferring from air to pan [WMO, 1983; Kaila, 1983].

During humid seasons the heat stored in the water mass of a lake during summer contribute to the high evaporation from the lake relative to pan evaporation and hence the high pan coefficient while during dry season the pan warms up more rapidly than the lake resulting to the high evaporation from the pan relative to lake evaporation and hence the lower pan coefficient[Linsely and Franzini, 1979; Hounam, 1973].

2.5.4.3 Seepage Losses

When a reservoir bank and dam embankments are pervious a steady flow of water is set-up through them owing to the head difference between the upstream and downstream of a dam.

In evaluation of water loss through this steady flow, otherwise referred to as seepage, an assumption is made that the flow through the soil under consideration follows Darcy's law (Equation 2.11) [Smith, 1981; Terzaghi and Peck, 1967].

$$Q = KA\frac{H}{L} \tag{2.11}$$

Where Q – Flow rate through a soil, K – Coefficient of permeability for the soil, A – Area of cross – section through which the water flows, H – Hydraulic head across soil and L – Length of flow path through soil

The method used commonly in computing seepage loss from reservoir is flow-net which is based on the Darcy law. Flow-net (see Figure 4) is a pictorial representation drawn to scale, of the paths taken by water in passing through a soil material [Smith, 1981]. It consists of flow-lines and equipotential-lines. Flow-lines represent the paths of flows through a soil while equipotential-lines joins points along the flow lines which are of equal head [Smith 1981, Davis and Sorensem, 1969; Terzaghi and Peck, 1967].

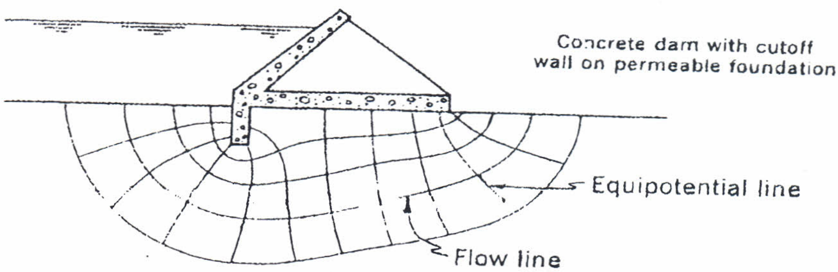


Figure 4 Flow net

The two important requirements while developing a flow net are

(i) Flow-lines should be drawn with each one approximately parallel to the last. The uppermost flow line is assumed to be the base of the dam and the sides of the cut-off wall while the lowest flow line is assumed to be the base of the pervious stratum. The rest will lie between the two and are drawn such that they do not close each other [Smith, 1981; Terzaghi and Peck, 1967]

(ii) Equipotential-lines are drawn such that they cross the flow lines at right angles and fields formed should near a square [Smith, 1981; Terzaghi 1967]

Flow-net is used to evaluate seepage from a unit length of a dam length under consideration using equation (2.12) [Smith, 1981, Davis and Sorensem, 1969 and Terzaghi and Peck, 1967].

$$Q = KA\frac{N_f}{N_e} \tag{2.12}$$

where Q – Seepage flow rate per unit length of the dam under consideration [Flow rate/length], K – Coefficient of permeability [Length/Time], N_f – Total number of flow lines and N_e – Total number of equipotential lines.

The main limitation of this method is the variability of soil permeability from one point to the other in contrast to the assumption in its derivation that a soil stratum under consideration is uniformly permeable. This certainly results to different flow pattern in real soil than that depicted in the flow net [Terzaghi and Peck, 1967].

2.5.5 Reservoir Sedimentation

2.5.5.1 General

Reservoir sedimentation is the process of deposition of eroded soil particles (sediments) into a reservoir. The deposited sediment will reduce the reservoir storing capacity equivalent to its volume. The operation of a reservoir is set such that the volume of deposited sediment does not hamper it.

The rate at which sediment is deposited in a reservoir will depend on both the rate of sediment discharge at a given reservoir site and the trap efficiency of a given reservoir.

2.5.5.2 Determination of Weight of Sediment Load Discharged at a Reservoir Site

The three widely used methods of estimating rate of sediment discharge into a reservoir are;

(i) Use of results of surveys of sediment accumulation in existing reservoirs

(ii) Use of results of catchment soil loss and

(iii) Sampling of suspended sediments.

The first technique entails measurements of accumulated sediments in the existing reservoirs by surveying the reservoir bed profile. This technique is laborious and expensive despite the fact that the sediment deposit determined are not very accurate [El-Moattasen, 1994].

The sediment carried in a river mostly originates from a catchment of a particular river. Thus, the estimates of soil loss from a catchment can be used to estimate sediment load to be deposited in a reservoir [MOWD, 1992]. There are several models that have been developed for the purpose of estimating soil loss from a catchment and uses different parameters. Thus this method will be best suited for reservoirs with small and homogenous catchments. Otherwise the determination of reservoir sedimentation from soil loss of a catchment will be laborious and may not be accurate at all.

In suspended sampling method of historical data of suspended sediment and the corresponding river discharge data are used to develop a sediment rating equation. This equation relate sediment concentration to water discharge at a particular point on a river, which is normally presented in the form expressed in equation (2.13) [Linsely and Franzini, 1979; Golzé, 1977]

$$C = a\,Q_r^{\,b} \tag{2.13}$$

Where C – Concentration of sediment [weight/volume], Q_r – River discharge [volume /Time], a and b – Constants for a particular gauging station.

Sediment rating equation is used together with the flow duration curves to compute the average annual sediment load. The water discharge values of the mid-ordinates of increments of the flow- duration curves are used to obtain the matching values of sediments load from the sediment rating equation curves. The values of sediment load are multiplied by the percentage of time of the corresponding increment in the flow duration curve and the cumulative sum is the average annual sediment load [MOWD, 1992; Linsely and Franzini, 1979; USBR, 1977; Golzé, 1977; Davies et al., 1969]. This method reduces considerably the amount of computation, however with the use of a computer it is possible to simulate daily sediment discharge and the average annual sediment discharge using equation (2.14) and (2.15).

$$Q_{d(j,y)} = f.Q_{r(j,y)}[aQ_{r(j,y)}{}^{b}] \tag{2.14}$$

where Qd(j,y) – Weight of sediment discharge through a gauging station on jth day of yth year, Qr(j,y) – Mean river discharge on jth day of yth year, f - Conversion factor to convert mean discharge to flow volume per day, a and b – Constants as defined in (2.13).

$$\overline{Q}_d = \frac{\sum_{j=1}^{N}\sum_{i=1}^{365} Q_{d(i,\,j)}}{N} \tag{2.15}$$

Where, $\overline{Q}_d$ - Average annual sediment discharge through a gauging station

$Q_{d(i,j)}$ – As defined in equation (2.14), N - Total number of years in record, j = 1, 2... N, and i = 1, 2, ...365.

The sediment discharged into a river is adjusted to include the bed-material, which are not included in the measured suspended material. Generally bed-material concentration is estimated as a percentage of suspended sediment and ranges between 3% and 25% [Joglekar, 1971]. The recommended ranges for estimating bed-material from suspended sediment concentration by the USBR, 1977 are presented in Table 3.

Table 3 Bed Load as Percentage of Suspended Sediment for Different Stream Bed Material

Concentration of Suspended Material (mg/l)	Type of Material forming Stream Channel	Characteristic of Suspended Material	Bed load as % of suspended material
< 1000	Sand	Similar to bed material	25 – 100
< 1000	Gravel, rock or Consolidated clay	Small amount of sand	5 – 12
1000 - 7500	Sand	Similar to bed material	10 – 35
1000 - 7500	Gravel, rock or Consolidated clay	Small amount of sand n.e 25%	5 – 12
>7500	Sand	Similar to bed material	5 – 15
>7500	Gravel, rock or Consolidated clay	Small amount of sand n.e. 25%	2 – 8

Source; USBR [1977]

2.5.5.3 Determination of Sediment Trap Efficiency of a Reservoir

Trap efficiency as pointed earlier is an important factor in determining the rate of reservoir sedimentation. It is the weight of particles that are retained in a reservoir expressed as a percentage of the total sediment load discharge at a reservoir site. Brunes developed curves (see Figure 5) that relate trap efficiency to the ratio of average inflow into a reservoir to its total capacity [Singh et al., 1990; MOWD, 1991; Linsely and Franzini, 1979; USBR, 1977; Golzé, 1977; and Joglekar 1971]. These are the curves normally used to estimate the trapped sediment in a reservoir.

2.5.5.4 Determination of Sediment Trap Efficiency of a Reservoir

Density of sediment trapped in a reservoir is used in computing volume it occupies from its weight. According to Singh et al., [1979] and USRB [1977] the sediment density will depend on age of sediment and composition of sediment (that is the % sand, % silt and % clay). Linsely and Franzini [1979] notes that sediment samples collected from a number of reservoirs had their densities ranging from 650kg/m3 to 1800km/m^3 with an average of 1000kg/m^3 for fresh sediment and 1300kg/m^3 for old sediments. Singh et al., [1990] gives equation (2.16) for computing average density of sediment in a reservoir over its useful life span.

$$\overline{\delta_N} = \frac{1}{100}\sum_{i=1}^{3} P_i\left[\delta_{(i,1)} + \frac{M_i}{N}\sum_{J=1}^{N} Log(j)\right] \qquad (2.16)$$

Where $\overline{\delta}_N$ - Average density of sediment in the reservoir after N years (weight/unit volume), $\delta_{(i,1)}$ – Density of sediment constituent i in the first year (weight/unit volume), P_i – Percentage of constituent i in the sediment deposit, M_i – Compaction factor for i^{th} constituent and N – Number of years (Length of useful life of a reservoir in years).

Compaction factor (M) and initial density ($\delta i,1$) depends both on the sediment constituents and mode of reservoir operation and these values are given in Table 4 for the various reservoir operation conditions.

Table 4 Compaction Factor and Initial Densities of Sand, Silt and Clay for different reservoir operations.

Reservoir Operation	Sand		Silt		Clay	
	$\delta_{(sand,1)}$	M	$\delta_{(Silt,1)}$	M	$\delta_{(Clay,1)}$	M
Submerged	1490	0	1040	91	481	256
Moderate Drawdown	1490	0	1190	43	737	171
Considerable Drawdown	1490	0	1270	16	961	96
Normally Empty	1490	0	1310	0	1250	0

After Singh et al., [1990]

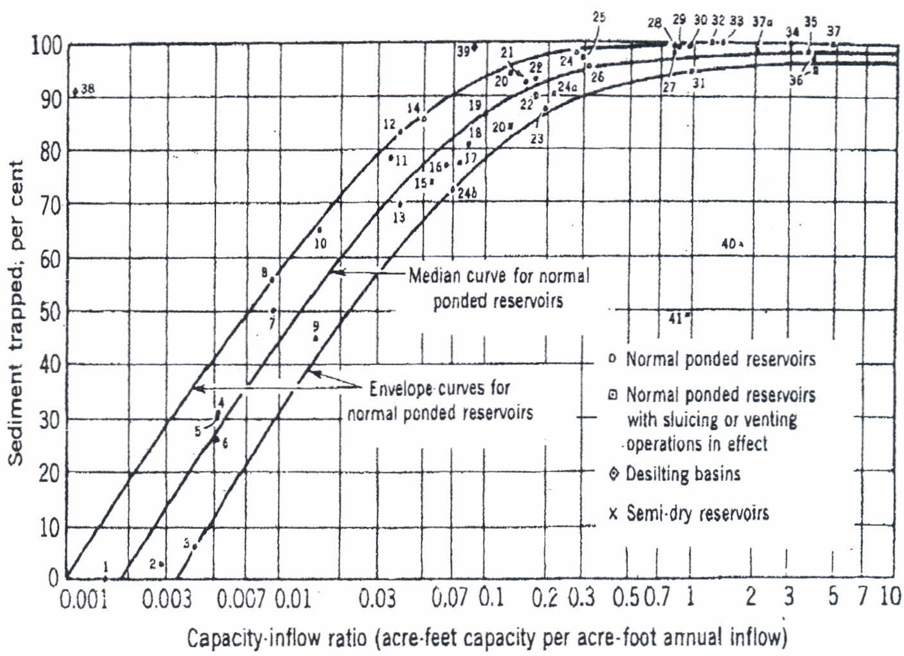


Figure 5 Reservoir Trap Efficiency as a Function of the Reservoir Capacity – Inflow Ratio [after Linseley and Franzini [1979]]

2.6 RESERVOIR OPERATING POLICY

2.6.1 General

Operating policy of a reservoir is a set of rules for storing and releasing water from a reservoir. These rules are either in tabular or graphical forms and will apportion storage and release among purposes and time periods. They have been used mainly to control reservoir releases so as

(i) To reduce the impact of a drought sequence in a water supply scheme in a case where reservoir is used for storage purpose

(ii) To reduce the risk of flooding in a case where a reservoir is used for flood control and

(iii) To optimize on benefits of a reservoir which has a limited quantity of water in its storage in case of a multi-purpose reservoir

The above are normally the objectives of an operating policy and the objective of operating a reservoir must be clear right from the inception of a reservoir operating policy [Bowler, et al., 1962].

2.6.2 Storage Reservoir.

Standard operating policy (equation 2.17) is one of the operating rules used to control storage reservoirs and it aims at supplying the demand all the time as long as water is available in the reservoir.

$$\begin{aligned} R_{(t)} &= D_{(t)} && \text{for } V_{min} < V_{(t)} < V_{max} \\ R_{(t)} &= I_{(t)} && \text{for } V_{(t)} < V_{min} \\ O_{(t)} + R_{(t)} &= I_{(t)} && \text{for } V_{(t)} > V_{max} \end{aligned} \qquad (2.17)$$

Where $R_{(t)}$ – Reservoir release during t^{th} time period, $D_{(t)}$ – Water demand during t^{th} time period, $I_{(t)}$ – Reservoir inflow during t^{th} time period, $O_{(t)}$ – Reservoir Spill during t^{th} time period, V_{min} – Minimum reservoir capacity [dead storage], V_{max} – Maximum reservoir capacity and $V_{(t)}$ - Reservoir storage contents during t^{th} time period

This is the simplest reservoir operating policy and good results have been claimed by many people who have used it to operate reservoirs [Simonovic, 1987 and Strycharczyk, et al., 1987]. But this policy cannot be used in controlling reservoir releases such that the available storage content is able to satisfy restricted demands over a specified time horizon. This limits its use in reducing impact of a drought flow sequence and hence the need of operating rule that can be used to satisfy these needs.

According to Edward and Johnson [1978] the release from a reservoir where the available storage content is limited and has to satisfy some demands over a known time horizon will depend on present storage contents, time period of a year, expected inflows, acceptable risks of failure [reservoir emptying] and time horizon being used for planning. Based on this principle Walsh and Walker [1988] have given a description of three operating policies that have been developed by improving one to the other and they have been used to control releases from a storage reservoir to meet water demand for different drought events. These rules are;

(i) Draw down based policy
(ii) Refill based policy and
(iii) Spill based policy.

A draw down based policy aims at drawing water from the reservoir upto a level where the remaining reservoir contents will be able to satisfy specified future demand. Walsh [1971] developed these rules in equation (2.18).

$$V_{r(t)} = V_{min} + D_{s(t)} - Y_{a(t)} - I_{(t)} + C_{(t)} + L_{(t)} \qquad (2.18)$$

Where $V_{r(t)}$ – Required storage contents at the beginning of the t^{th} time period of a year, V_{min} – Minimum storage contents (Dead storage), $D_{s(t)}$ – Total volume of water required to be supplied from the reservoir with a specified time horizon storing from t^{th} time period of a year, $C_{(t)}$ – Total volume of water required for compensation release within a specified time horizon starting from the t^{th} time period, $I_{(t)}$ – Design drought inflow (expected inflow) within a specified time horizon starting from t^{th} time period, $Y_{a(t)}$ – Total volume of water available from alternative source with a specified time horizon starting from t^{th} time period and $L_{(t)}$ – Total losses (in volume) from the reservoir within a specified time horizon starting from t^{th} time period.

Drought inflow is random [Edward and Johnson, 1978] and should be determined in a probabilistic term. Walsh [1971] determined total flows over specified time horizons starting from each time period in the record. Design drought inflow during each time period of a year for a specific time horizon was considered to be that one exceeded with a 98% probability. By applying equation (2.18) and fixing all variables other than demand a series of curves can be developed for different demands. These curves can then be used to control the reservoir releases so as to protect the reservoir against a repeat of a design drought inflow sequence on which the analysis was based on [Walsh and Walker, 1988].

Refill based policy aims at making reservoir to be as full as possible just after a wet season. The main purpose of this is to make available as much water as possible during the dry season other than releasing it during wet season. Releases are controlled so that the reservoir is likely to fill with a specified probability. Walsh and Walker [1988] indicates that this is an improvement of drawdown based policy to improve its performance during abnormal drought periods where equation (2.19) is used for this purpose.

$$V_{r(t)} = V_{max} - D_{(t)} + Y_{a(t)} + I_{(t)} - L_{(t)} \tag{2.19}$$

Where Vr(t) – Required flow during tth time period, Vmax – Maximum reservoir capacity, $D_{s(t)}$ – Total volume of water required to be supplied from the reservoir with a specified time horizon storing from t^{th} time period of a year, $Y_{a(t)}$ – Total volume of water available from alternative source within a specified time horizon starting from t^{th} time period, I(t) – Expected inflow over the time horizon under consideration starting from t^{th} time period and $L_{(t)}$ – Total losses (in volume) from the reservoir within a specified time horizon starting from t^{th} time period.

The expected inflow is determined by identifying the start and end of wet periods in a year. Then total flow over different consecutive time periods ending at the end of wet periods is determined for the data under consideration. These totals are used to compute the expected inflow over a specified time horizon at the desired probability and it is used in equation (2.19) to determine the required storage of a reservoir during the wet period. The rest of the curve follows drawdown curve.

According to Loucks et al., [1981] spills from a reservoir is wasted flow and Walsh and Walker [1988] describe spills as expensive in view of reservoir operations, which require to be minimized without adversely affecting reliability of the system. In this case the reservoir releases are made such that there is adequate free volume in a reservoir to accommodate expected inflow (see equation (2.20)) and are based on the probability of spill for different reservoir storage throughout a year [Walsh and Walker, 1988].

$$R_{(t)} = V_{(t)} + I_{(t)} - V_{max} \tag{2.20}$$

Where $R_{(t)}$ – Reservoir release at t^{th} time period, $V_{(t)}$ – Reservoir volume at t^{th} time period, V_{max} – Maximum reservoir capacity and $I_{(t)}$ – Expected flow over the time horizon under consideration starting from t^{th} time period.

Equation (2.20) is just used to modify the drawdown curve to minimize chances of reservoir spilling without necessarily affecting the reliability of reservoir releases. When releases from the drawdown curve is higher than the release in equation (2.20) the drawdown curve is used to control reservoir operations.

2.6.3 Flood Control Reservoir.

For flood control reservoir an operating policy will aim at reducing the risk of flooding. During normal flow periods the releases are made such that the reservoir will empty as much as possible such that at time of arrival of flood peak there will not be need to release a lot of water and hence reduce the peak flow. During flood flow period the release should be low to the level where they will not synchronize with flow from the downstream catchment to cause flooding.

2.6.4 Multipurpose Reservoir

Releases from a multipurpose reservoir are used to satisfy more than one water demand which are usually interrelated in a complex way. In case the release are less than total water demand then specific allocation must be reduced and allocation of the limited releases to various demands should aim at optimizing the reservoir benefits.

The two are achieved through the use of appropriate reservoir operating policy [Simonovic, 1987; Georgakakos et al., 1987] which are developed mainly based on two concepts. One of these concepts is where different uses of a reservoir are treated equally and the allocation of water will be such that the reduced allocation will induce equivalent losses for all uses [Simonivic, 1987]. That is the ratio of deficit occurring for a particular use to deficit that will induce unit loss for that particular use should be the same for all uses. Then an operating policy is derived such that it will minimize the total loss.

The other concept is where benefits for one reservoir use are maximized while the fulfillment of the remaining uses are required to be above certain minimum levels. This approach arises frequently in reservoir systems, which have operating priorities mandate by institutional agreement [Georgakakos et al., 1987].

2.7 Reservoir Operation Models

2.7.1 General

The mass balance equation (2.5) forms the basis of a reservoir operation study. It has been modeled severally to develop empirical models for reservoir operations, which has been used in various ways that includes designing of proposed reservoirs, management of existing reservoirs and studying the effect a reservoir has on the water management in a basin.

The two broad reservoir operation models classes are the simulation models and Optimization models (linear and dynamic programming). The reservoir operation study simulation models from the Hydrologic Engineering Center (HEC) are widely used. Ford [1990] reports the use of Texas Water Development Board Simulation models and Acres Simulation model beside one he developed for PC computers. Goodman et al. [1981] reports the use of a simulation program called HYPO for the study of a system of reservoirs and power plants in the upper Hudson River System in New York State. GIS based simulation model called Geostorm has been developed for simulating hydrologic events in a river basin [Arc News, 1997]. Tabular modeling of reservoir operation study is possible and it is demonstrated in Linsely and Franzini [1979] and this is made much easier by use of computer spreadsheet as demonstrated by Hancock and Heany [1987].

Interactive River-Aquifer Simulation (IRAS) model simulate over time operations in a river basin system [Loucks et al., 1995]. Reservoirs are modeled within a river basin and their effect to the water management in a basin is studied. By just modeling a reservoir and the river system that flows into it, it will be possible to use this model to simulate reservoir operations as is demonstrated in this study.

2.7.2 Use of IRAS Model in Reservoir Operation Study

IRAS model is a generic simulation program which is used to simulate over time operations in a river basin water system. A river-basin water system is represented in a node-link networks where nodes represent either aquifers, gauge sites, consumption sites, natural lakes, reservoirs wetlands, confluence(s) or diversion(s). Links represent water flow paths such as river channels or diversions.

Simulation of reservoir operations study using IRAS model will require at least 3 nodes and 2 links. These nodes and links will be arranged as shown in

Figure 5.

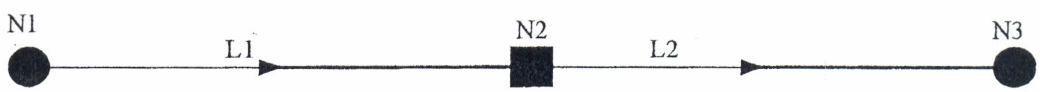


Figure 5. Nodes and Links model of a Reservoir System in IRAS model

The first node (N1) and link (L1) represent the river system in the upstream of the reservoir, where N1 simulate a gauge whose flow data has been used to simulate reservoir inflows and L1 simulate river channels in the upstream that drain into the reservoir. The specific data that is required for node (N1) are flows in a flow file, which will have the following

i) Flow conversion constants where one of them is for converting flow rate from one unit to the desired flow units and the other is for changing flow rate to flow volume per day.

ii) Beginning year, number of years of data and number of within year periods in each year.

iii) Average flow for each within year period.

The second node (N2) simulate reservoir and the specific data required for this node include

i) Annual mean daily evaporation for each within year time period

ii) Seepage loss function and

iii) Elevation-Surface Area-Storage relationship.

The second link (L2) and third node (N3) simulate the draw-off system that get water from the reservoir to the consumer point.

The general data required for simulation while using this model are

i) Number of within year time periods to be used in the simulation

ii) Length of each within year period

iii) Number of simulation time steps for each within year period

iv) Node names

The model's value limits of parameters used here is as follows

i) Number of within year time period should be equal or less than 60

ii) Number of incoming links per node should be equal or less than 5

iii) Number of outgoing links per node should be equal or less than 4

iv) Number of days in a year should be equal or less than 365

v) Number of simulation per within year time period should be equal or less than 12

The model simulation is based on the mass balance equation (2.5). This equation is applied on daily basis for each within year period where seepage and evaporation losses are computed based on the initial storage for each day, that is for evaporation losses computation equation (2.21) is used and seepage losses equation (2.22) is used.

$$De_{t\,(d)} = A_{t\,(d)}.\ EV_t \qquad (2.21)$$

Where $De_{t\,(d)}$- Evaporation loss on d^{th} day of t^{th} time period of a year, $A_{t\,(d)}$. - Surface area of reservoir on the beginning of d^{th} day of t^{th} time period of a year and EV_t – Annual mean daily evaporation rate for t^{th} time period of a year.

$$L_{t\,(d)} = f\,(L) \qquad (2.22)$$

Where $L_{t\,(d)}$ – Seepage loss on d^{th} day of t^{th} time period of a year and f (L) – Seepage loss function

The total loss over a within year time period is computed by summing the daily losses in that within year time period. The initial storage for each day is revised to account for these losses. That is

$$V_{(t,\,d)} = \text{Max.}\ (0,\ (V_{(t,\,d)} - De_{t\,(d)} - L_{t(d)})) \qquad (2.23)$$

Where the term $V_{(t,d)}$ on the left side of equation (2.23) represent the revised initial storage and it must be non-negative while $De_{t(d)}$ and $L_{t(d)}$ remain the same as in equation (2.21) and (2.23) respectively.

The end storage for each day is computed as detailed in equation (2.24).

$$V_{(t,d+1)} = V_{(t,d)} + Q_{(t,d)} - R_{(t,d)} \qquad (2.24)$$

Subject to; $0 \leq V_{(t,d+1)} \leq V_{max}$

Where $V_{(t,d)}$ – Revised initial storage on d^{th} day of t^{th} within year period, $V_{(t,d+1)}$ – Storage at the end of d^{th} day of t^{th} within year period, $Q_{(t,d)}$ – Inflow during d^{th} day of t^{th} within year period and $R_{(t,d)}$ – Reservoir release during d^{th} day of t^{th} within year period

If the right hand of equation (2.24) is greater than V_{max} the excess is released as spills. The left-hand side must be non-negative.

Reservoir release is user defined and can either be based on release rules, which depend on storage contents and time of the year , downstream demand deficits, minimum release or target storage volumes for a reservoir group.

2.8 Flow Analysis

2.8.1 Hydrographs

A hydrograph is defined as the plot of a river discharge against time. The hydrographs are normally used to study flow distribution within a specified time period which is normally a year. They are the one best suited for comparing inflow and outflow of a reservoir and they would easily be used to quantify the volume of high flows that is retained in a reservoir for redistribution as is demonstrated in this study. Still using them the flow that is made available by the existence of a reservoir would be easily evaluated.

2.8.2 Flow Duration Curves

A flow duration curve shows availability of water as a percentage of time. The procedure of developing this curve involves first ranking the data. Then evaluating the plotting position of each ranked data using any of the existing methods. Linsely and Franzini [1979] have shown that the existing methods differ very little especially where the data is of a long period and points out that Weibull method (equation (2.25)) is mostly used.

$$Pn = \frac{n}{m+1} \times 100 \qquad (2.25$$

Where P_n – Percentage of time duration when n^{th} flow is equaled or exceeded, n – Rank number of nth flow and m – Total number of samples.

According to WMO, 1983 a flow duration curve for daily discharge will show the percentage of time that the flow of a stream is greater than a given value whereas that for either the weekly or monthly discharge will represents the percentage of weeks or months rather than time. The importance of duration curves for yearly discharges in appraising the yearly variation in flow is highlighted here.

Use of flow duration curves in hydrologic studies is limited because it does not give sequence of flow and thus not useful where sequence of flow is needed [WMO, 1983; Linsely and Franzini, 1979; Davis and Sorensem, 1969].

2.8.3 Frequency Curves

Frequency curves are used in hydrologic studies to determine the frequency of occurrence of rare hydrologic events such as floods and droughts. The danger of flooding will be studied using maximum annual flood while risk of drought will be studied using low flow [Kuiper, 1965; Golzé, 1977].

Use of frequency curves for hydrologic analysis is based on findings by Fisher ad Tippet who have shown that distribution of rare events (extreme values) of many large samples of a particular population is independent of population distribution and conforms to a limiting function [Linsely and Franzini, 1979]

In low flow analysis the annual minimum of a selected length of time period such 7 or 30 consecutive days is used [WMO, 1983; Riggs et al., 1980; Linsely and Franzini, 1979; MOWD, 1986; MOWD 1992]. In Kenya, 1 day time length is used in design of urban water supply and 30 days (monthly) time length is used for design of rural water supply schemes [MOWD, 1986; MOWD, 1992]. Riggs et al., [1980] report wide use of 7 day (weekly) time length and it is used in this study.

In the 7-day period a year is divided into 52 time periods (weeks) and the total flows within each of these time period is determined. The minimum value for each year should be picked independently [Kuiper, 1965] such that the value picked for a particular year should be from an independent drought event. Where a drought event extend beyond a calendar year only one value, which is the lowest is peaked even if in the other year the drought event had yielded the lowest value for that year. The values peaked are then ranked in the ascending order and plotting position determined as detailed in equation (2.25). The resulting curve shows the percentage of time duration when a flow equal or less than a given value will occur. The return period of droughts of different magnitudes is determined from the inverse of their probability of occurrence.

This is suitable for evaluating the impact of a reservoir in improving the low flow occurrence. The low flows of different return periods are compared for the inflow and outflow of a reservoir.

2.8.4 Drought Analysis

Drought is viewed here as the time when river discharges are below a specified flow, which is referred to as drought reference flow. A drought reference flow is used in defining stream flow drought and oftenly is considered to be the total demands that are supposed to be satisfied from a particular river. In this case rivers with same flows will have different magnitude of drought if the demands are different. This shows the difficulty of having a uniform way of defining stream flow drought for all rivers.

Statistical analysis of droughts is used in evaluation of usable water resources in a river basin for the purpose of water management [WMO, 1983]. Different parameters of stream flow droughts can be used in this analysis and the two important parameters are deficit and duration of stream flow drought [Zelenhasic and Salvai, 1987; Nazaar et al., 1980; WMO, 1983]

Deficit is defined as the magnitude of short fall of river discharge from drought reference flow while drought duration is the number of consecutive time period when the river discharge is below drought reference flow. The cumulative deficit is evaluated as detailed in equation (2.26)

$$D_{e(i)} = Q_r T_{(i)} - \int_{tb(i)}^{te(i)} Q_{(t)} dt \qquad (2.26)$$

Where $D_{e(i)}$ – Deficit of i^{th} drought (volume), Q_r – Drought reference flow (volume/Time), $t_{b(i)}$ and $t_{e(i)}$ – Starting and ending periods of i^{th} drought, $T_{(i)}$ – Duration of i^{th} drought and $Q_{(t)}$ – Flow during t^{th} time period of a year.

The two, that is drought duration and deficit, can be used to evaluate reservoir outflow reliability in the three complementary ways as described by Carthy and Cunane [1990].

(i) Occurrence based reliability

Here the reservoir outflow is analyzed to determine number of years when at least one drought event occurs. Any year when a drought occurs is considered a failure year and the reliability of reservoir is the total number of non-failure years expressed as percentage of total number of years in record.

(ii) Duration based reliability

The cumulative time periods when drought occurs is determined. This is used to compute for non-failure time periods. The reliability of reservoir here is considered as the total time period for non-failure periods expressed as a percentage of total number of time periods in the record.

(iii) Quantity based reliability

Reliability here is considered to be total water supplied expressed as a percentage of total water demand. The total water supplied is computed by getting the difference between the total demand and cumulative deficit.

2.9 RESERVOIR PERFORMANCE INDICATORS

2.9.1 System Reliability and Resilience

Reliability describe frequency (probability) that a system is in satisfactory state and resilience is a measure of how quickly a system is likely to recover (or jump back) from unsatisfactory state once unsatisfactory state occur [Loucks, et al., 1995 and Hashimoto, et al., 1982]. These two are defined empirically in equation (2.27) and (2.28).

$$Reliability = \frac{\text{No of times when a system is in a satisfactory state}}{\text{Total number of time}} \quad (2.27$$

$$Resilience = \frac{1}{\text{Average Length of Sequence in unsastifatory state}} \quad (2.28$$

IRAS model display these statistical parameters for any simulated variable which have threshold values assigned to it [Loucks et al., 1995]. Threshold values of a system define a boundary between satisfactory and unsatisfactory states for that particular system where unsatisfactory state occurs when flow is below a threshold and vise versa for flow, which is equal or above a threshold.

The assumption made while using these two parameters to describe performance of a water resources system is that the probability distribution that describe the output time series will not change within the planning horizon [Hashimoto et al., 1982].

2.9.2 Outflow and Inflow Hydrographs

This method compares outflow and inflow of a reservoir graphically. This graphical representation of outflow and inflow hydrograph displays the extent to which a reservoir has augmented the low flows of a river and reduced the high flows.

2.9.3 Low Flow Frequency Analysis of Reservoir Outflow and Inflow

Low flow frequency analysis for reservoir outflow and inflow will give the extent to which a reservoir has been able to improve the occurrence of low flow events of different magnitudes.

2.10 QUANTIFYING STREAMFLOW REGULATION BY A RESERVOIR

Just like most other water resources developments, reservoir development in a river basin system reduces the quantity of water in the basin [Nazaar, et al 1981] and the utility of developing it is mainly its function of regulating the streamflow which is turn improves the degree of assurance of water availability in the system both in time and space. A reservoir regulate the streamflow by retaining part of the flows during high flow periods and release it during low flow periods and this is the process of augmenting low streamflows and reducing the high peak flows by a reservoir.

Currently there is very little attempt of quantifying explicitly the ability of a reservoir to regulate streamflow and it is only used implicitly in determination of required reservoir capacity in the different methods used in reservoir size determination. Reservoir designers using mass curve method oftenly consider the critical period of the available data [Klemeš, 1979; Linsely and Franzini, 1979; Loucks et al., 1981] and the required size of the reservoir is fixed at that capacity that will just be depleted at this period. This method uses the cumulative difference between the inflow and the outflow of unconstrained (top-less) reservoir and its analytical procedure provide a suitable mean of quantifying streamflow regulation as it clearly shows when the reservoir is filling and when it is emptying and this could be used effectively in quantifying the amount of streamflow that is retained in the reservoir during high flows and the amount of flow released from the reservoir storage contents to augment low flows. However, for the method to be used in a constrained reservoir it will require to be modified such that the spills from the reservoir are separated from the draft and evaporation losses are put into consideration in the analysis.

With the advent of computer hardwares and softwares with much improved analytical capacity it is then much easier now to carry-out these analysis and compute precisely the ability of a reservoir to regulate streamflow and in this study a methodology has been formulated in section 3.5.2 where the spills, retained flows by the reservoir during high flows and released flows from storage contents of a reservoir to augment the low flows over the simulation period of the reservoir operation have been quantified. The low flow augmentation is considered the primary function of the two reservoirs being studied here and in that case the quantified releases from the storage contents of these reservoirs is used in establishing the optimum capacities.

3. MATERIALS AND METHODS

3.1 GENERAL

This research study aimed at evaluating the feasibility of developing reservoirs in the UENNRB at Archers Post and Kihoto to regulate the ENNR flow. In this respect the following activities were undertaken to achieve this aim;

i) Describing of topography, stream flow, sediment discharge and evaporation at the proposed reservoir sites.

ii) Simulation of reservoir operations using IRAS model.

iii) Analysis and discussion of results.

3.2 DEVELOPING OF TOPOGRAPHICAL MAPS FOR THE PROPOSED RESERVOIRS

3.2.1 General

The main purpose of developing topographical maps for the proposed reservoir sites was to form basis for identifying dam axis and developing depth-area-storage relationship for the sites. The location of a dam site is an important factor in the depth-area-storage relationship of a reservoir which is an important input in reservoir operation study particularly in simulation of reservoir water loss through evaporation and seepage.

3.2.2 Developing of contour map of the proposed reservoir sites.

3.2.2.1 General

The available topographic information for the two sites was in form of 1:50,000 topo-sheets with contour at 20m interval and at this interval the number of contours within the reservoir areas was not adequate to develop depth-area-storage relationship. Hence the need to develop contour maps for these sites with contours at 10m intervals at which interval at least 5 contour lines were developed for each site as recommended in Golzé [1977]. IRAS model requires reservoir volume and surface area at a maximum of 7 levels [Loucks et al., 1995].

Aerial photographs were used in developing of these maps since it was relatively cheap and the maps were produced faster at an acceptable accuracy. The aerial photographs were assembled from survey of Kenya. For Archers post area aerial photos of contract No. 255

with serial Nos. 62, 63, 64, 65, 89 and 90 were used while for Kihoto area those of contract No. 595 with serial Nos. 120 - 131 were used. The year of photography was 1977 for those covering Archers Post area and 1967 for those covering Kihoto area.

3.2.2.2 Reconstruction of Aerial Photograph Models

Reconstruction of aerial photograph models, from aerial photos stereo pairs, was done using *Wild* A8-Stereo Plotter, which was used to draw contour map from these models. The ground controls, which are very important in the reconstruction of these models, were determined from the 1:50,000 topo-sheets of the respective area. Features used for this purpose were those that were clear and sharp on the aerial photographs and could be identified on the 1:50,000 topo-sheets. Mostly river junctions were used since they are features that are not likely to change their co-ordinates with time. In each model at least three ground controls were identified.

3.2.2.3 Plotting of Contour Lines

After an aerial photograph model was reconstructed in the A8-Stereo Plotter contours were developed from it. A contour was plotted by first setting the elevation of the contour to be plotted in the machine and moving the floating mark in the machine until it touches the surface of the model. This machine height was fixed and then the floating mark was moved along the surface of a model such that it was in contact with the surface all the time. A plotting pencil on a plotting board attached to the Stereo Plotter simulated this movement and it traced this movement on a paper fixed on the board. This process continued until all areas on the model, which are of this height, were covered. The traced lines were marked after which the machine elevation was changed to the elevation of the next contour. The process was repeated until all possible contours were drawn.

The scale of the plotted map was fixed at 1:20,000, since the resulting map was found to be of a convenient size for this study especially while measuring the area enclosed by each contour within the reservoir area using of a planimeter. At this scale contours at intervals of 10m were well spaced and their number within the reservoir area was adequate to develop functional relationships relating both the reservoir volume and surface area to the height of the reservoir for use in IRAS model.

3.2.2.4 Plotting of Other Features

Other features that were plotted included river courses, roads, footpaths bridges, forests and other vegetation, buildings, settlement areas and any other important feature that was within the reservoir area. These features were plotted by moving the floating mark along them but the machine elevation was not fixed in this case.

3.2.3 Developing of Height-Area-Volume Relationship for the proposed reservoir sites

3.2.3.1 Locating the Dam Axis

Location of the reservoir was first identified using the information from the earlier studies [MOWD, 1992; MOWD, 1980]. Then the dam axis used in this study was identified by

i) Different possible dam sites were identified by selecting points along the river valley (near the location of the dam identified in MOW [1963], MOWD [1980] and MOWD [1992]) which were narrow and expanded in the upstream.

ii) The highest contour for each of the possible dam site was fixed at the elevation identified in MOWD [1992].

The possible dam site that had the highest storage capacity was selected for this study and the reservoir area was delineated as that area below the highest contour.

3.2.3.2 Height-Area Relationship

Area enclosed by each contour within the reservoir area was measured using a planimeter. The measured area for the two sites were plotted against reservoir depth and a power function was found to give a good relationship for the two sites and was adapted where a regression analysis was done to determine the best fitting equation of the form given in equation (3.1).

$$A = \alpha H^{\beta} \tag{3.1}$$

Where H-Reservoir height, A - surface area of reservoir, α and β - coefficients of a reservoir site.

3.2.3.3 Height-Volume Relationship

Volume of reservoir at different heights of the water was determined using equation (3.2).

$$V_n = 0.5h(a_0 + a_n + 2\sum_{j=1}^{n-1} aj) \tag{3.2}$$

Where V_n = volume of reservoir at the n^{th} contour, h = contour interval, a_j = Area enclosed by j^{th} contour, n= number of contours within the reservoir area at a given height of reservoir and j = 1,..n-1.

The computed volumes were plotted against reservoir height and just like in the case of the surface area a power function was found to give a good relationship relating reservoir volume to its height where a regression analysis was done to determine the best fitting equation of the form given in equation (3.3).

$$V = cH^m \tag{3.3}$$

Where V - volume of reservoir, H - Reservoir height and c and m - coefficients of reservoir site.

3.3 Time Series Data Used in Simulation

3.3.1 General

This study quantified reservoir inflows, releases, evaporation losses and sedimentation discharge. Reservoir inflows were simulated using the estimated flows at the proposed Archers Post and Kihoto Reservoirs respectively. These flows were estimated from 27 years (1960-1986) flow data at RGS 5E3 and RGS 5BC4 for Archers Post and Kihoto reservoirs respectively. This period had adequate data within NRM^3 database that was used to estimate reservoir inflows and carryout the other analyses that were done in this study. Sedimentation was also estimated from these flows together with sediment-rating equations derived from sediment samples collected in the period 1992-1996. Evaporation losses from these reservoirs were simulated using Elevation-Surface-Area function derived as detailed in section 3.2.3.2 together with evaporation rates estimated from daily pan evaporation data. Data from Archers Post Meteolorogical station was used for Archers Post reservoir while the average of Matanya (LRP) and Junction Meteolorogical stations was used for Kihoto Reservoir. Minimum release was set at different reservoir storage levels to simulate reservoir releases.

3.3.2 Time Period Used in Simulation

A time period of 7 consecutive days was used on the following basis;

i) This is a short period within which the high variation of ENNR flows can be captured

ii) The 7-day time period is widely used in low flow frequency analysis [Riggs et al., 1981; Linsely and Franzini, 1979] and

iii) By using 7 day time period there would be 52 within year time periods in a year, which is within the limits of IRAS model which require a maximum of 60 within year time periods.

A year was then divided into 52 weeks where 52^{nd} week was set to have 8 days in all years while 9^{th} week will have 8 days during leap years and 7 days during ordinary years. All other weeks were set to have 7 days.

Weekly averages for the flow data, which is the basic data in running of this model, were evaluated for all years in record. In the simulation of evaporation losses annual means are used and in this case annual weekly means of daily pan evaporation for Meteolorogical stations used were evaluated.

3.3.3 Reservoir Inflows

3.3.3.1 Estimation of Inflows at the Proposed Archers Post Reservoir

The proposed reservoir site is about 10km upstream of RGS 5E3. In the downstream of this site and upstream of the RGS there are 3 important source of flows. These are the Buffalo springs, Isiolo and Ngara Mara rivers. These sources are important both during wet and dry seasons since the Buffalo springs contribute the largest portion of flow at RGS 5E3 during low flow periods for instance in parts of 1984, 1985, 1986, 1994 and 1997 ENNR had dried-up in the upstream of these springs but there was still some flow at

RGS 5E3 from their discharges while Isiolo and Ngara Mara rivers flow contribution at RGS 5E3 during high and normal flow periods is high and can not be ignored.

It is then important to account for this increased flow in the ENNR to the downstream of the proposed reservoir site where equation (3.4) was used for this purpose and the assumption made while developing this equation is that the contribution of flow at RGS 5E3 by the catchment in the upstream and downstream of the proposed reservoir is proportional to their size since they exhibit similar climatic and environmental conditions and during extremely low flows the flow registered at the RGS is only from the springs.

$$\begin{cases} Q_d = f \times Q_r \ (\mathrm{m}^3/\mathrm{s}) & \text{for } (1-f)Q_r > Q_s \\ Qd = Q_r - Q_s \ (\mathrm{m}^3/\mathrm{s}) & \text{for } (1-f)Q_r \leq Q_s \\ Qd = 0 & \text{for } Q_r \leq Q_s \end{cases} \tag{3.4}$$

Where Q_d - Estimated flow at the proposed reservoir site, Q_r - Flow at RGS 5E3, Q_s - Discharge from the Buffalo Springs and f - Ratio of catchment area of the proposed reservoir to that of RGS 5E3

Discharge from Buffalo Springs was estimated from the flow at RGS 5E3 during the times when this river was known to have dried at the proposed reservoir site. This was based on the field visits of 25th February 1994 and 15th March 1997. Discharges at RGS 5E3 was 0.469 m^3/s and 0.168 m^3/s on 24th February, 1994 and 15th March, 1997 respectively. These flow values were applied to equation (3.4) and the resultant periods when it is estimated there was no flow at the proposed reservoir site are given in Table 5.

Table 5 Weeks when flow at the Proposed Archers Post Reservoir site is estimated to be zero.

Year	Periods when ENNR is Estimated to Dry-up for Different Estimated Discharge of Buffaloes Springs (Week)	
	Based on Discharge of 24th February 1994 (0.469 m^3/s)	Based on Discharge of 15th March 1997 (0.168 m^3/s)
1981	4th - 8th and 11th - 12 th	11th - 12 th
1984	22nd and 25th - 26th	non
1985	10th - 11th	non
1986	3rd - 9th	4th - 9th

The discharge of 0.468 m^3/s approximates well to what happened where the river had dried in 1984/85 and in 1986. Therefore this was assumed to be the spring discharge and was used in the analysis. There were few days without data whose mean discharges were estimated by interpolation.

3.3.3.2 Estimation of Inflows at the Proposed Kihoto Reservoir

In using flow data at RGS 5BC4 to estimate flows at Kihoto reservoir site two problems were encountered;

i) The years with complete data extended from 1960 to 1982 and had to be extended to cover the same period covered at Archers Post, that is upto 1986 and

ii) Downstream of this RGS and in the upstream of the dam site ENNR is joined by Segera/Suguroi River, which result to increased flow. Segera/Suguroi River is gauged at RGS 5BD5 and the available data for this RGS covered only the period 1992 to 1996.

Extension of this data was done by developing a regression relationship of flows of RGS 5BC4 with flows of nearby RGS whose data cover upto 1986 and the coefficient of determination for the regression equation had to be greater than 0.6. The nearby RGS considered were RGS 5D5 and RGS 5BE20. The RGS 5BE20 gauges flows in Nanyuki River that joins Ewaso Ngiro River in the downstream of RGS 5BC4 and in the upstream of RGS 5D5. This is shown schematically in Figure 6. These RGS had complete data in the period between 1965 and 1979 and a multiple linear regression for this data yielded a better correlation coefficient. A regression equation of the form given in equation (3.5) was then developed and it was used in extension of the data for RGS 5BC4 to cover upto 1986.

$$Q_{gt} = \beta Q_{ft} + \alpha Q_{jt} + \kappa \qquad (3.5)$$

Where Q_{gt} - Estimated flow at RGS 5BC4 on t^{th} time period, Q_{ft} - Measured flows at RGS 5BE20 on t^{th} time period, Q_{jt} - Measured flows at RGS 5D5 on t^{th} time period and β, α and κ - Coefficients derived from multiple regression of flows at the three stations.

Coefficient of determination (R^2) for the regression equation used to derive β, α and κ is 0.72 and their values are given in Table 6.

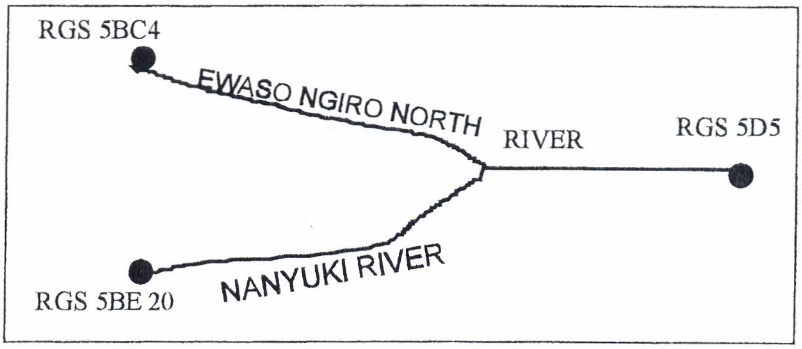


Figure 6 Schematic Presentation of Relative Location of RGS 5BC4, 5BE20 and 5D5

Table 6 Statistics of Regression of flows at RGS 5BC4 against flows at RGS 5BE20 and 5BD5.

	β	α	κ
Constants	0.287	0.0988	0.682

To account for the increased flow from Segera/Suguroi River equation (3.6) was used.

$$Q_{(t)} = f_{(t)} . Q_{(rt)} \qquad (3.6)$$

Where $Q_{(t)}$ - Estimated inflow into the proposed reservoir during t^{th} time period of the year, $Q_{(rt)}$ - Streamflow discharge at RGS 5BC4 during t^{th} time period of the year and $f_{(t)}$ - Factor used to estimate reservoir inflow using flows at RGS 5BC4 during t^{th} time period of the year

The factor, f (t), was derived from the weekly mean ratio of total flows of RGS 5BC4 and 5BD5 during the period 1992 to 1995 to the flows of RGS 5BC4 of the corresponding period. These ratios were raised by 2% to cater for lower than normal flows experienced during this period.

3.3.4 Estimation of Evaporation Losses from the surface of the Proposed Reservoirs

Daily pan evaporation data was used to estimate the evaporation rates at both reservoir sites. Data from Matanya (LRP), Junction and Archers Post Meteolorogical stations were used. The available data for Matanya (LRP) Meteolorogical station was for the period 1986 -1995 and that of Archers Post was for the period 1985 -1995. Mean evaporation rate for a given area is known not to vary a lot with time [W.M.O., 1983] and in this case the same was assumed when this data was used to simulate evaporation for the period 1960-1986.

Data for Matanya (LRP) and Junction Meteolorogical stations was used to estimate evaporation at Kihoto reservoir area. The Matanya (LRP) Meteolorogical station is 30km South of the proposed reservoir at altitude 1830m a.s.l and is within Agro-ecological zone (V) while the one at Junction is 30km north of the proposed Kihoto reservoir at an altitude of 1450m a.s.l and is within Agro-ecological zone (VI). The proposed Kihoto reservoir is in the transition zone between Agro-ecological zones (V) and (VI) and its lowest point is at altitude 1680m a.s.l. The annual weekly pan evaporation of Matanya (L.R.P) and Junction Meteolorogical stations exhibit similar trends but the rates are high at either of the station depending on the time of year. It was assumed that there is a gradient fall between the 2-stations and an average of the 2-stations was taken as the evaporation rate at Kihoto Reservoir site.

Archers Post Meteolorogical station data was used to estimate evaporation rate at Archers Post reservoir. This station is 10 km east of the reservoir site and the two are within the same Agro-ecological zone (VI) where they are estimated to experience the same climatic conditions hence the evaporation.

Evaporation from the surface of the reservoir was correlated to pan evaporation by use of pan-to-lake coefficients. The factors used were different for the 2 sites since they experience different climatic conditions. At Archers Post the climate is arid and experiences warm temperature conditions. Whereas in Kihoto area the climate is semi-arid and warm-temperate temperature conditions are experienced [MOA, 1980]. Then 0.6 and 0.7 were used to estimate evaporation losses for Archers Post and Kihoto reservoir respectively as recommended in WMO [1983].

3.3.5 Estimation of Reservoir Sedimentation of the Proposed Reservoirs

Sediment rating equation for RGS 5BC4 and RGS5E3 were developed using data from NRM3 database [Kihara, 1997]. These rating equations are in the form given in equation (3.7).

$$C_{st} = aQ_t^{\ b} \tag{3.7}$$

Where C_{st} - Sediment Concentration in g/l on t^{th} day of a year, Q_t - River flow discharge in m^3/s on t^{th} day of a year and a and b - Constants for a specific station.

The values for a and b for the two gauging stations are given in Table 7 below.

Table 7 Constants for Sediment Rating Equations for RGS 5E3 and RGS 5BC4.

RGS	a	b
5E3	0.2997	0.5615
5BC4	0.2439	0.5461

After Kihara [1997]

It was assumed that the sediment load sampled at the RGS was the sediment load at the respective reservoir site to simplify computation. At Archers Post it is expected that the estimated sediment load is just slightly higher than that experienced at the reservoir site. This is based on the assumption that the bulk of sediment load is assumed to come from the degraded catchment in the upstream of the reservoir site and the increased catchment area at the RGS 5E3 only contribute little sediment. At Kihoto it is expected that the estimated sediment load is slightly lower than that which is experienced at the reservoir site since the additional catchment is small in comparison with that of the sampling station and it contributes less flow.

Daily mean sediment concentration of river water at the proposed reservoir sites was evaluated by applying this equation to daily mean flows at each RGS. Weight of sediment in tons that is discharged daily at a reservoir site was computed using equation (3.8).

$$S_{d(t)} = 86.4Q_d^{\ 2}\,.C_{s(t)}\ \text{(tons)} \tag{3.8}$$

Where $S_{d(t)}$ - Daily sediment discharge in tons on t^{th} day of a year, $Q_{(t)}$ - Mean daily flow on t^{th} day of a year, $C_{s(t)}$ - Sediment concentration on t^{th} day of a year.

The sediment that is discharged annually at a reservoir site was computed using equation (3.9)

$$S_i = \sum_{j=1}^{n} S_{d(t)} \tag{3.9}$$

Where S_i - Total sediment discharge in the i^{th} year in tons, $S_{d(t)}$ - Sediment discharge in t^{th} day of the i^{th} year and n = 365 for ordinary years and 366 for leap years.

Weight of sediment that is retained in a reservoir annually was computed using equation (3.10).

$$S_{ri} = \gamma \, . S_i \tag{3.10}$$

Where S_{ri} - Sediment retained in a reservoir in the i^{th} year (tons), S_i - Sediment load discharge at reservoir site in the i^{th} year (tons/year) and γ - Trap efficiency of the reservoir.

Trap efficiency (γ) was evaluated from Brunes Chart (adopted from Linsely and Franzini [1979])

The volume occupied by sediment retained in the reservoir annually was computed using equation (3.11).

$$V_{ri} = S_{ri} / \rho \tag{3.11}$$

Where Vri - Volume of sediment retained in the reservoir in the i^{th} year (m^3), Sri - Weight of sediment retained in the reservoir in the i^{th} year (tons) and ρ - Average density of retained sediment ($tons/m^3$).

For each reservoir the average rate of sedimentation was computed using equation (3.12).

$$V_{ra} = (\sum_{i=1}^{N} V_{ri}) / N \tag{3.12}$$

Where V_{ra} - Average rate of sedimentation of the reservoir, V_{ri} - Volume of sediment retained in the reservoir in the i^{th} year (m^3) and N - Length of data used in years.

The average rate of sedimentation was used to compute dead storage of reservoir by multiplying it with the useful life of a reservoir.

3.4 SIMULATION OF RESERVOIR OPERATIONS USING IRAS MODEL.

3.4.1 General

The mass balance equation (2.4) is the one used in simulation of reservoir operations in IRAS model. Seepage loss, which is one of the losses considered in this equation, are assumed negligible for the purpose of this study and hence they are not simulated. Also precipitation is assumed to be included in the measured runoff and similarly it was not simulated. From this then equation (2.4) was reduced to equation (3.13) which was used in simulation of reservoir operations.

$$V_{(t+1)} = V_{(t)} + Q_{(t)} - D_{(t)} - E_{(t)} \tag{3.13}$$

Subject to

$V_{max} \geq V_{t+1} \geq V_{min}$

Where $V_{(t+1)}$, $V_{(t)}$ – Reservoir storage contents at the end and beginning of the t^{th} week respectively (m^3), V_{max} – Maximum Reservoir Capacity, V_{min} – Dead Storage, $I_{(t)}$ – Reservoir inflow during t^{th} week (m^3), $R_{(t)}$ – Reservoir releases during t^{th} week (m^3) and $E_{(t)}$ – Reservoir evaporation loss during t^{th} week (m^3)

3.4.2 Nodes and Links Models for the Proposed Reservoirs

The river and reservoir systems of the proposed reservoirs are shown in Figure 7. This system was modeled using nodes and links in IRAS model as shown in

Figure 8. The first node represents the RGS that was used to estimate inflows, from the river system, into the proposed reservoirs. The second node represents the proposed reservoirs while third node represent a point in the river system where the reservoir will be discharging its outflow. Links represent the river channels through which the water is flowing.

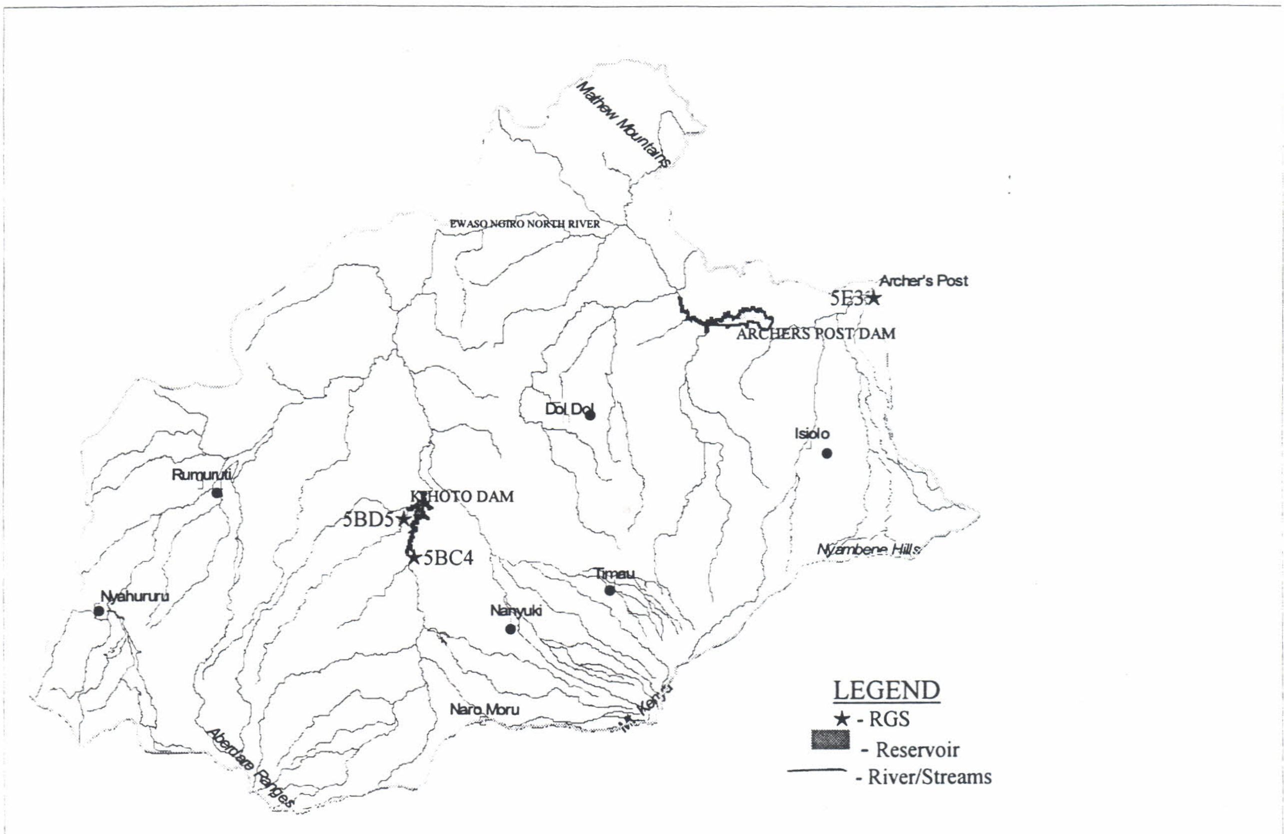


Figure 7. Reservoir and River System for Kihoto and Archers Post Dam

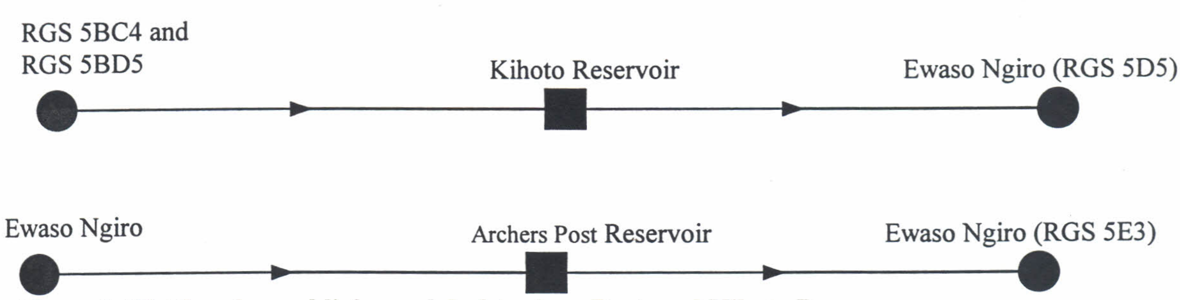


Figure 8. IRAS nodes and links model of Archers Post and Kihoto Dam

3.4.3 Reservoir Operating Policy

The main function of the proposed reservoir was assumed to be that of regulating river flow and the operating policy was set for this purpose. The reservoir contents were set to be released such that the remaining reservoir contents were adequate for a period of 14 weeks, which is the duration of consecutive weeks of low flows at Archers Post that is likely not to be exceeded with 90% probability.

The minimum release at dead storage was set at nil and that of the 50-million m^3 was set at 40% of the estimated mean flow at the proposed reservoir site. For the other storage levels the minimum release was set using equation (3.14). IRAS simulated releases by interpolating between the set minimum releases.

$$R_i = \left(\frac{S_i - S_{min}}{T\%}\right) + R_0 \qquad (3.14)$$

Where R_i - Release from reservoir at i^{th} reservoir storage level in m^3/s, R_0 - Release when active storage is 50 million m^3, S_i - Storage contents at i^{th} storage level in m^3 and S_{min} - Dead storage in m^3

T% - Critical time period in seconds during which time the reservoir contents should be able to supply water.

3.5 RESERVOIR PERFORMANCE EVALUATION

3.5.1 General

The purpose of this analysis is two folds, one determination of the required reservoir capacity at each of the reservoir site and then quantify the effect these reservoirs will have on the flow to their downstream.

The required reservoir capacity was determined by comparing inflow and outflow hydrographs for all reservoir capacities whose operations were simulated. Other analysis done were the evaluation of evaporation losses and spills from these reservoirs.

The impact of these reservoirs on the flow to their downstream was evaluated on the basis of their ability to improve the return period of low flow events and the reliability of flow.

At Kihoto the ability of the reservoir to regulate ENNR flow such that there will be a continuous flow upto Archers Post was evaluated.

3.5.2 Determination of Required Reservoir Capacity.

A visual model Figure 9 of flow through the reservoir- river system was made to describe parameters used here in selecting the required reservoir capacity.

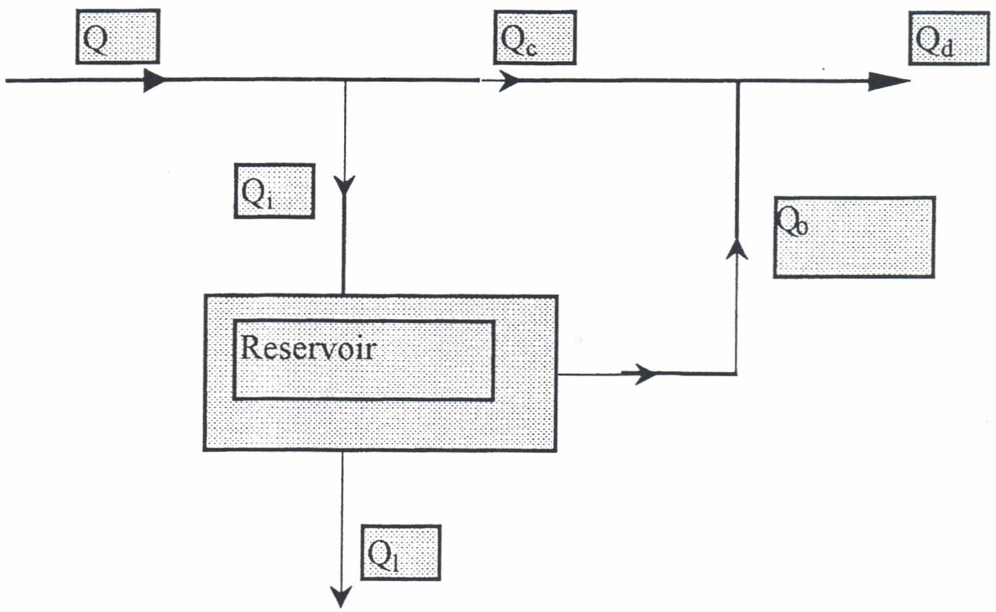


Figure 9. Conceptual Model for Flows through a Reservoir-River system

Q is the inflow from the upstream of the reservoir and is represented graphically by the inflow hydrograph in Figure 10 . Q_i is part of inflow that is delayed by reservoir for redistribution, which is the inflow that is in excess of outflow as demonstrated in Figure 10). Q_c is that part of inflow that flows directly to the outlets and will include spills (Q_{sp}). Q_l is part of the delayed inflow that is lost through evaporation and seepage due to the increased surface area of water while Q_o is the redistributed inflow, which is the outflow in excess of inflow as is demonstrated in Figure 10 . Q_d is the total discharge to the downstream of the reservoir and is presented graphically by the outflow hydrograph in Figure 10 .

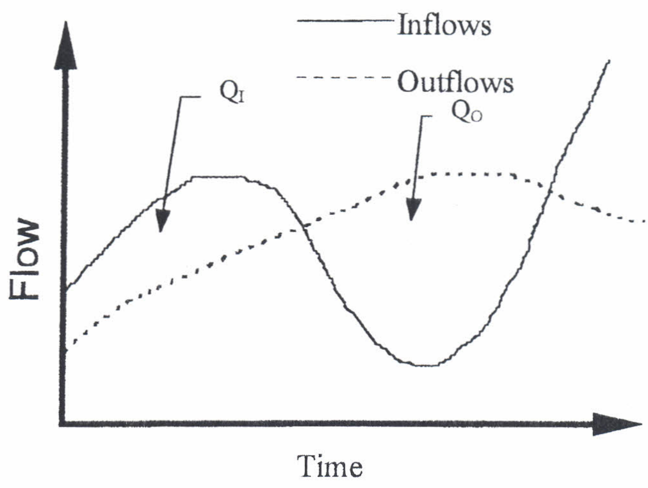


Figure 10. A Sample of Inflow and Outflow Hydrograph

Q_l is an output of the simulation as well as Q_d. Q is an input to the model and it is also given in the formatted output of the model. Q_i and Q_o were computed using equation (3.15) and (3.16) respectively while reservoir spills (Q_{sp}) were computed using equation (3.17).

$$\begin{cases} Q_i = Q - Q_d & \text{for} \quad Q > Q_d \\ Q_i = 0 & \text{Otherwise} \end{cases} \tag{3.15}$$

$$\begin{cases} Q_o = Q_d - Q & \text{for } Q_d > Q \\ Q_o = 0 & \text{Otherwise} \end{cases} \tag{3.16}$$

$$\begin{cases} Q_{sp} = Q_d - R_{max} & \text{for } V_{t+1} = V_{max} \\ Q_{sp} = 0 & \text{Otherwise} \end{cases} \tag{3.17}$$

Where R_{max} - Maximum release, V_{t+1} - Reservoir end storage content at t^{th} time period and V_{max} - Reservoir capacity

The total sums over the period of simulation for Q_i, Q_o, Q_l and Q_{sp} for all reservoir capacities that were simulated were computed and compared using graphs. A condition was set that when active storage capacity is increased by 50-million m^3 the resultant increase in reservoir storage outflow should be more than 15% and the first reservoir capacity that did not satisfy this condition was picked as the required reservoir capacity.

3.5.3 Reservoir Performance

3.5.3.1 Comparison of Reservoir Inflow and Outflow Hydrograph of the Proposed Reservoirs

The hydrograph for weekly mean reservoir inflow and outflow were plotted on the same axis to display the extent to which the reservoir has improved low flows. The impact of reservoir to annual weekly mean flows was evaluated by comparing the hydrograph of annual weekly mean reservoir outflow and inflow.

3.5.3.2 Low Flow Frequency Analysis

The purpose of this analysis was to evaluate the impact a reservoir will have on the return periods of low flow events at the proposed sites. The selected reservoir capacity at each of the proposed site was used in this analysis.

The lowest weekly mean flow events for each year were picked independently for both when there is a reservoir and when there is no reservoir. This data was ranked from the least to the highest and weibull plotting position was used to plot it on log-log graphs. The Log-Pearson III distribution fitted well with this data and was the one used to extend it beyond its limits. The return period for the situation when there is no reservoir and when there is one were compared graphically.

3.5.3.3 Comparing of Reliability and Resilience of Reservoir Inflow and Outflow

Threshold values were selected for each reservoir site. At Archers Post the highest threshold was set at 5.67m^3/s which was identified in MOW [1963] as the required flow at Archers Post to maintain a continuous flow upto Lorian swamps. Other threshold selected were 90%, 80% and 50% of this flow and 1.2m^3/s, which was considered to be the total water demand in the downstream of Archers Post by the same report. At Kihoto the highest threshold was set at 50% of mean reservoir inflow at the site, which is 2.07m^3/s. Other threshold that were selected were 40%, 30% and 20% of the mean reservoir inflow and 1.43m^3/s that was found to be the required flow at Junction to maintain continuous flow upto Archers Post.

Reliability and resilience were evaluated for all these thresholds, where for a particular threshold the system was considered to be in unsatisfactory condition if the flow is below it.

Reliability was evaluated by getting the number of weeks when the system was in unsatisfactory condition and then computes it as detailed in equation (3.18).

$$\text{Reliability} = \frac{\text{Total number of weeks} - \text{Number of weeks when the system is in unsatisfactory condition}}{\text{Total number of weeks}} \tag{3.18}$$

When the computed reliability was plotted against threshold it showed a close relationship in all cases. Functional relationships, which had a high correlation, were fitted to this data and they can be used to estimate reliability of other threshold within the range.

Resilience was evaluated by getting the number of times when unsatisfactory condition was followed by a satisfactory condition in a system and the total number of weeks when the system was in unsatisfactory state. Then it was computed as detailed in equation (3.19).

$$\text{Resilience} = \frac{\text{No of Times when Unsatisfactory Condition is Followed by a Satisfactory Condition}}{\text{Total Number of Weeks when the System is in Unsatisfactory Condition}} \tag{3.19}$$

3.5.4 Determination of Required Flow at Junction to Maintain Continuous Flow up to Archers Post

The flow at the confluence of Ewaso Narok River and Ewaso Ngiro North River (at Junction) is the sum of flow at RGS 5D5 and RGS 5AC8. The required flow at this point to maintain continuous flow upto Archers Post was determined for those weeks when it is expected that the area between Junction and Archers Post there will be no increased flow. These were identified as the weeks between 3rd and 13th week of the year.

The difference between the flow at Junction and flow at the proposed Archers Post reservoir site was determined for these weeks for the period 1960-1986. The differences that were above or equal 80% of the flow at Junction were picked and their average was considered to be the required flow.

To evaluate the impact of the proposed Kihoto reservoir in reducing the incidences of Ewaso Ngiro North River drying up in the upstream of Archers Post the number of weeks between 3rd and 13th week of the year for the period 1960-1986 when the weekly mean flow was below required flow were determined. In the case where there is a reservoir at Kihoto the flow made available by the reservoir was added to the weekly mean flow at Junction of the corresponding week. Then the number of weeks between 3rd and 13th week of the year for the period 1960-1986 when the weekly mean flow was below required flow were determined. The two situations were then compared to evaluate the impact of the proposed reservoir in regulating the flow so to reduce the number of incidences of the river drying in the upstream of Archers Post.

4. RESULTS AND DISCUSSIONS

4.1 GENERAL

This study have discussed the physical characteristics of the proposed reservoirs and the estimated time series data for reservoir inflow, evaporation and sedimentation at the proposed sites. Finally the study has discussed the reservoir operation simulation results.

4.2 PROPOSED RESERVOIR SITES CHARACTERISTICS

The contour maps developed for the 2 sites are presented in Figure 12 and Figure 13. From these maps sites were selected that were found suitable for dam development and their location is presented in Table 8. Both Table 9 and Figure 11 presents the cross section details of these sites. From this comparison Kihoto Reservoir has a narrower river valley which is V-shaped with a cross-section area of about 28,000m^2 while that of Archer Post Reservoir is wide and it is U-shaped with a cross-section area of about 93,600m^2.

Table 8. Co-ordinates of the Selected Dam site

Reservoir	Co-ordinates (Dam -Axis) X(m)	Y(m)
Archers Post	37 333,500	63,600
Kihoto	37 268,400	28,460

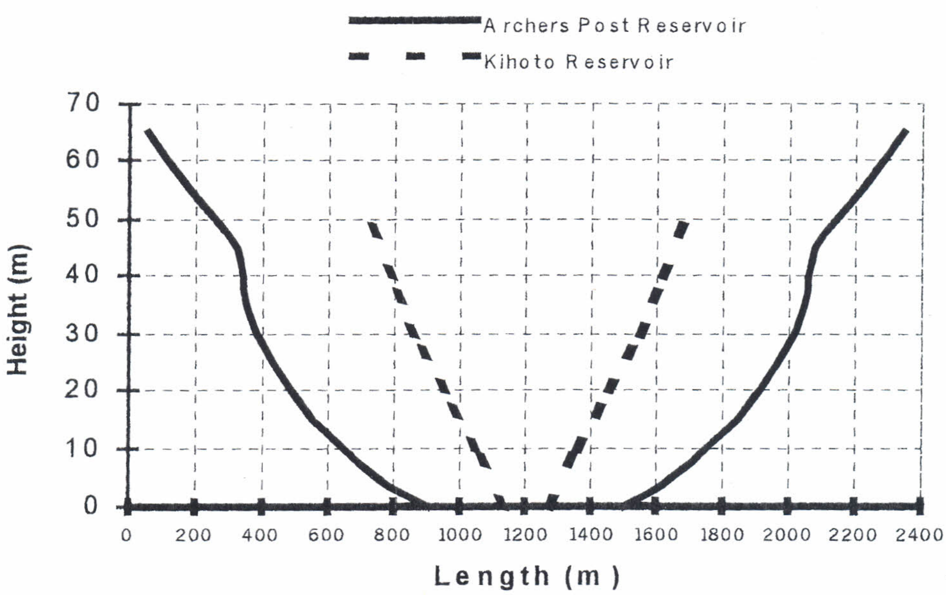


Figure 11 Cross-section of the Selected Dam site

Figure 13. Contour Map for the Proposed Archers Post Reservoir

Table 9 The Length of the Valley at Different Elevations of the Proposed Reservoirs.

ARCHERS POST			KIHOTO		
Altitude (m.a.s.l)	Reservoir Height (m)	Measured Valley Length (m)	Altitude (m.a.s.l)	Reservoir Height (m)	Measured Valley Length (m)
865	0	600	1690	0	140
870	5	920	1700	10	300
880	15	1300	1710	20	500
890	25	1540	1720	30	680
900	35	1700	1730	40	820
910	45	1760	1740	50	980
920	55	2040			
930	65	2280			

The measured area enclosed by contours at different elevations within the reservoir area is presented in Table 10 and Figure 15. Functional relationship relating areas to reservoir height is presented in Table 11. Computed reservoir volumes at different elevation is given in Table 10 and presented in Figure 16. The relationships relating reservoir volume to reservoir height for each of the reservoir site is presented in Table 11.

Kihoto Reservoir site has a higher storage capacity than Archers Post Reservoir site. Another advantage of Kihoto site is that the river valley at the site selected for dam development is narrower and this two makes it a better site for reservoir development in terms of topography.

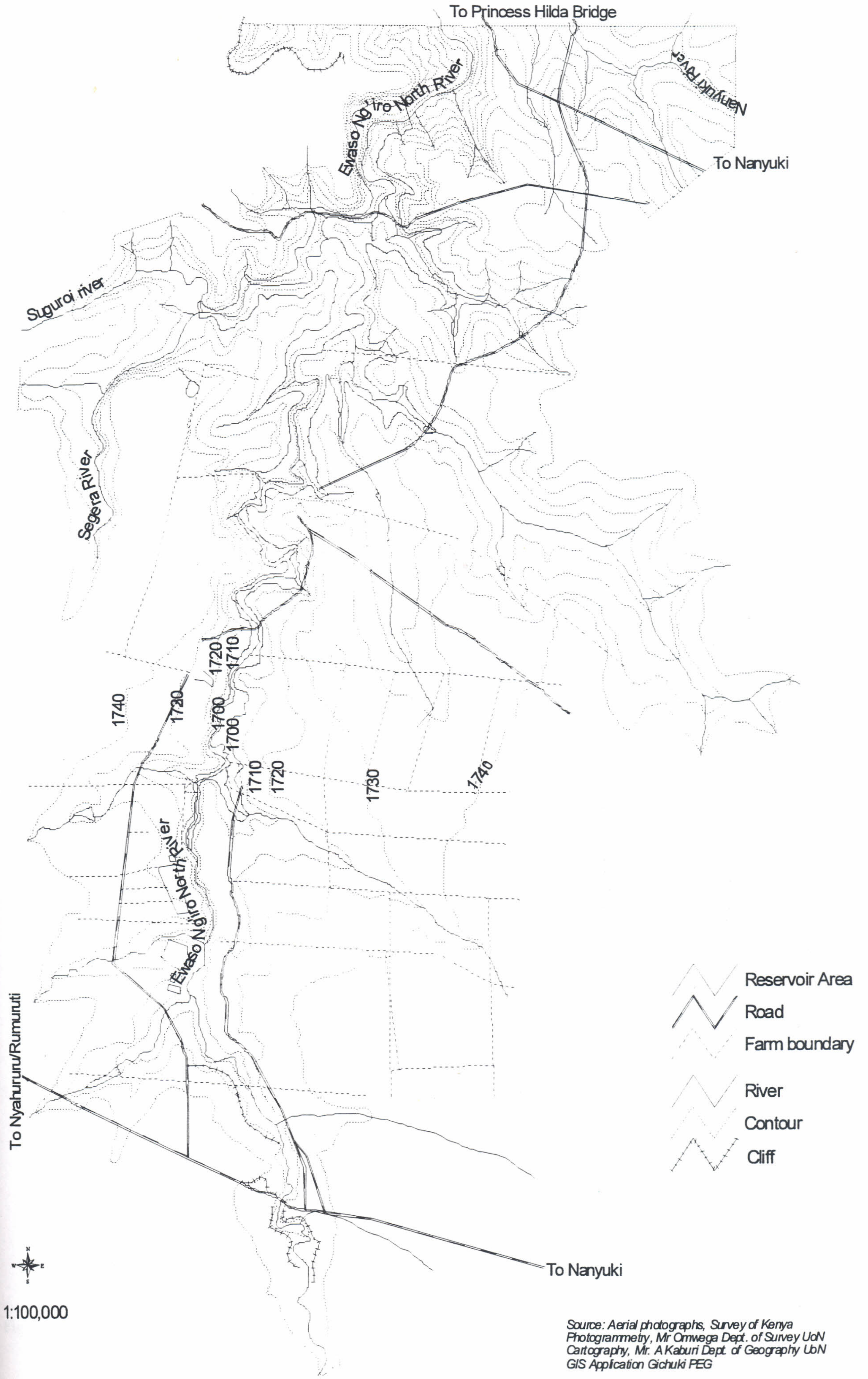


Figure 14 **Contour Map for the Proposed Kihoto Reservoir**

Table 10 Area Enclosed by Contours and Reservoir Volume at Different Elevation at the Proposed Reservoir Sites

..Archers Post Reservoir				Kihoto Reservoir			
Altitude (m.a.s.l)	Reservoir Height(m)	Measured Surface Area (million m2)	Reservoir Volume(million m3)	Altitude (m.a.s.l)	Reservoir Height(m)	Measured Surface Area (million m2)	Reservoir Volume (million m3)
865	0	0.84	0.00	1690	0	0.10	0.00
870	5	1.57	6.03	1700	10	3.19	16.44
880	15	2.69	27.33	1710	20	9.28	78.80
890	25	7.01	75.83	1720	30	20.14	225.92
900	35	12.89	175.32	1730	40	41.11	532.20
910	45	19.28	336.17	1740	50	70.92	1092.40
920	55	29.67	580.92				
930	65	41.11	934.83				

Table 11 The Height-Area-Volume Relationship of the Proposed Reservoirs

Reservoir	Area–HeightRelationship		Volume–HeightRelationship		
	Equation	R^2	Equation	R^2	H_{max}
Archers Post	$A = 126{,}753.6\ H^{1.32}$	0.94	$V = 179{,}000.6H^{1.98}$	0.98	65m
Kihoto	$A = 34{,}300H^{1.92}$	0.98	$V = 38{,}078.3H^{2.59}$	0.99	50m

Where A – Surface area of the reservoir (m^2), V – Volume of reservoir (m^3), H – Depth of reservoir (m), R^2 – Coefficient of determination, and H_{max} – Maximum height of the Dam (m).

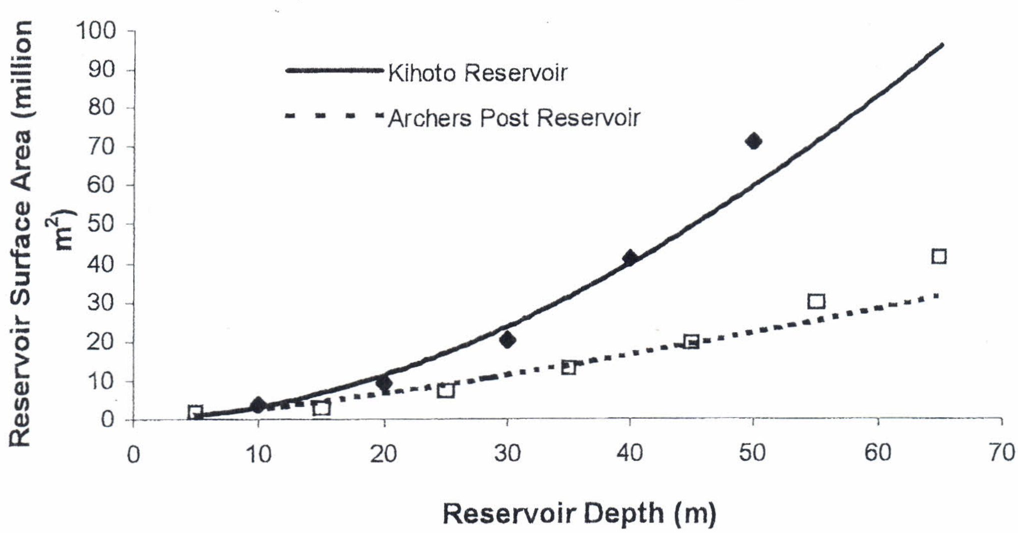
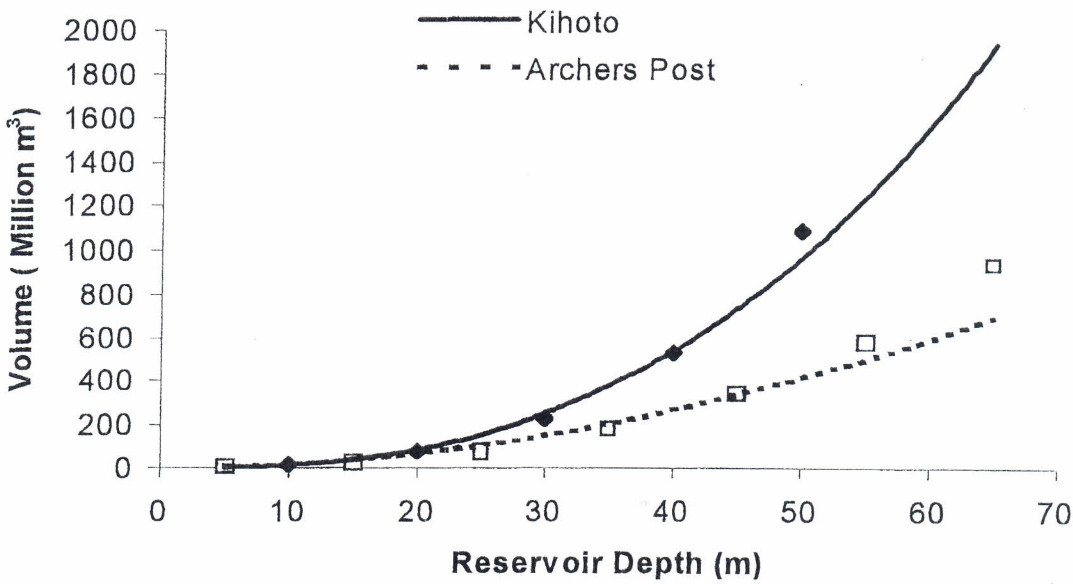


Figure 15. Change of Reservoir Surface Area with Depth



'igure 16 Change of Reservoir Volume with Height

4.3 Time Series Data Used in Simulation

4.3.1 Reservoir Inflows at the Proposed Reservoir Sites

4.3.1.1 The Proposed Archers Post Reservoir Site

A flow duration curve for the weekly average inflows at the proposed Archers Reservoir is presented in Figure 18 and the data used to develop it is given in appendix (1). The flow that is exceeded with a 95% probability is 0.57 m^3/s while those exceeded with 90% and 80% are 1.18 m^3/s and 2.98 m^3/s respectively. A hydrograph of the weekly mean reservoir inflow at the site for the period 1960 to 1986 is presented in Figure 17. This hydrograph depict a high variation of mean weekly inflows with some weeks registering very high flows of above 600 m^3/s and other weeks registering very low flows. Highest weekly average flow is 624.33 m^3/s and the river had dried in 1.3% of the time. The mean reservoir inflow is 17.04 m^3/s and standard deviation is 36.86 m^3/s (220% of mean inflow). This exhibits a high variation of inflows. The range, which is the difference between the highest and the lowest flow, is 624.33 m^3/s, which indicate that there is a big difference between high and low peaks.

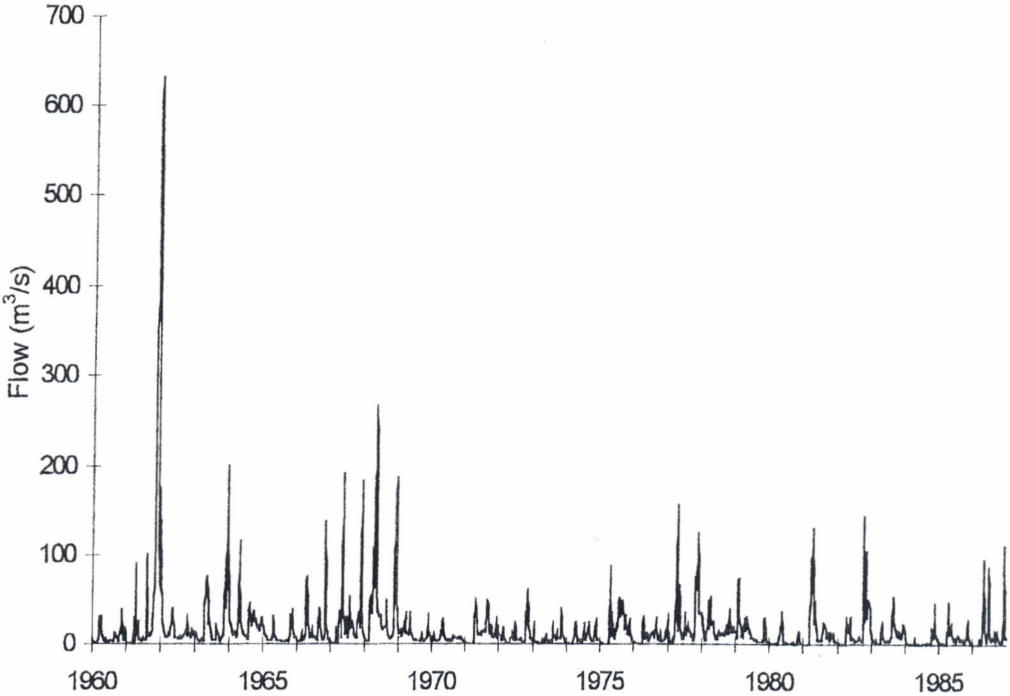


Figure 17 The Hydrograph for the weekly Mean Reservoir Inflow at Archers Post Site

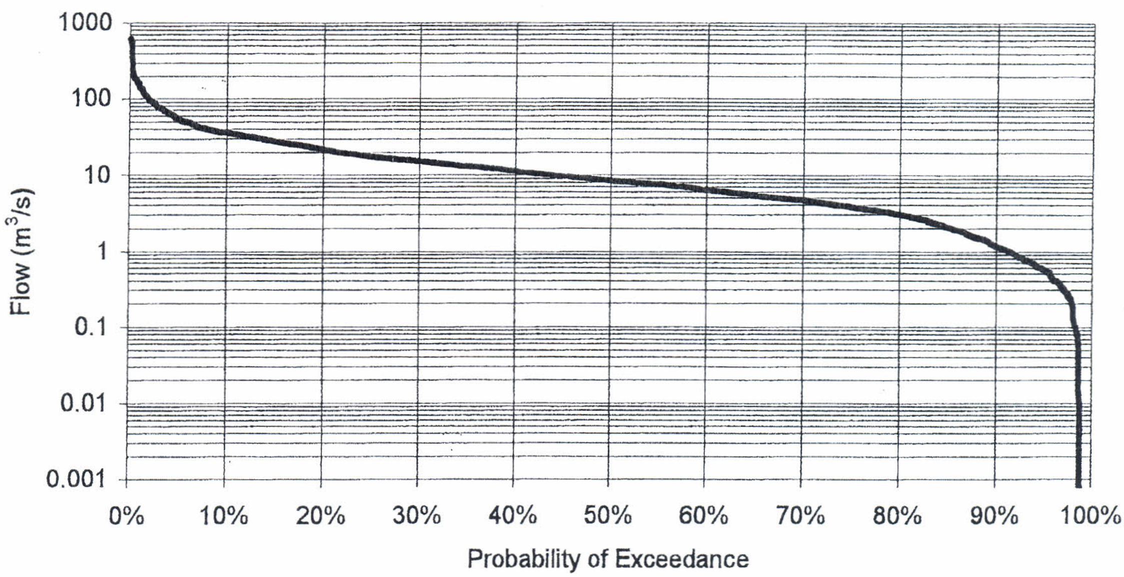


Figure 18 Flow Duration Curve for Reservoir Inflows at Archers Post Reservoir Site

The variation of flow from one year to the other is presented by the annual reservoir inflow hydrograph in Figure 19. The highest annual reservoir inflow is 2.04-billion m^3 that occurred in 1961 and the lowest is 153.3-million m^3 that occurred in 1984. In 1961 there was high rainfall in the country which was named Uhuru Rains [MOWD, 1992] and this is attributed to the high river flows in that year. In 1984 there was a severe drought in the country and this affected the river flows in the basin due to very low rainfall.

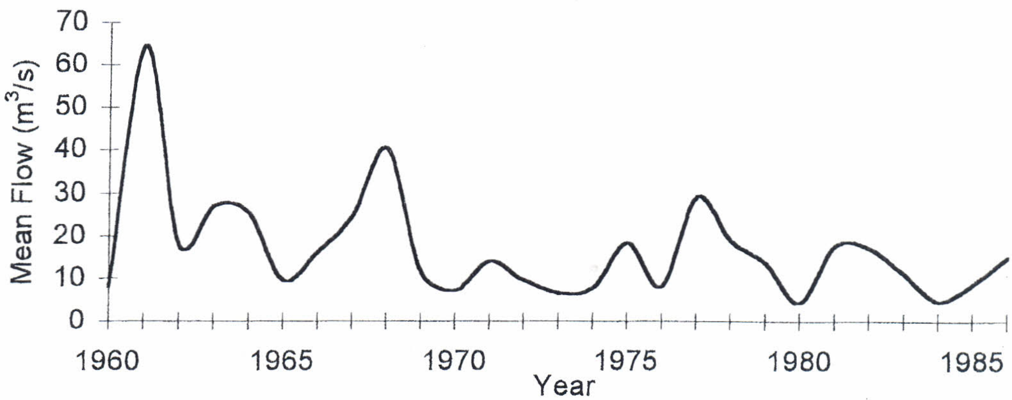


Figure 19 Hydrograph of Annual Reservoir mean Inflow at Archers Post Reservoir Site

4.3.1.2 The Proposed Kihoto Reservoir Site

A flow duration curve of the weekly mean inflows at the proposed Kihoto reservoir is presented in Figure 20 and data used to develop it is given in appendix (2). The maximum flow is 99.27 m^3/s while the minimum is 0.24 m^3/s. The mean flow is 4.11 m^3/s and the flow that is exceeded at 95% probability is 0.527 m^3/s. Flows that are exceeded with 90% and 80% probabilities are 0.69 m^3/s and 0.992 m^3/s respectively. There is high variation of inflows at this reservoir as is depicted by the large coefficient of variation of 182.5% and the big range of 99.04 m^3/s which indicates that the difference between high and low peaks is big compared to the mean flow. The hydrograph of weekly reservoir inflow at Kihoto site during the period between 1960 and 1986 (see Figure 21)) depict a high variation of mean weekly inflows with some weeks registering high mean inflows of above 90m^3/s and other weeks registering very low flows of below 0.25 m^3/s. The variation of annual inflows is depicted in the hydrograph of annual mean inflows given in figure (21). The maximum mean annual inflow is 13.05 m^3/s in 1961 when there was high rainfall in the country as reported in MOWD [1992]. The minimum mean annual inflow is 1.43 m^3/s in 1980 and low rainfall due to drought experienced that year can be attributed to this scenario.

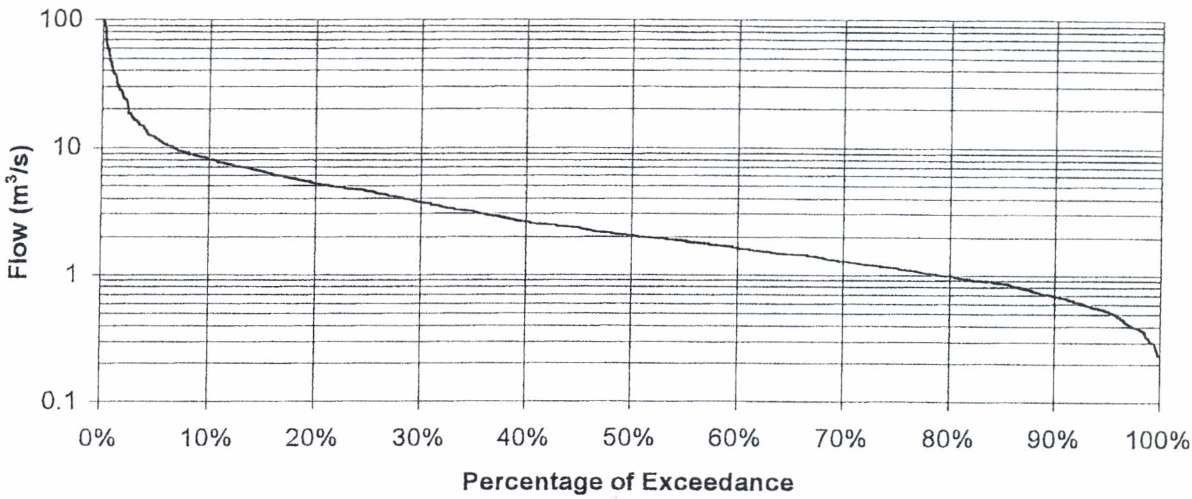


Figure 20. Flow Duration Curve for Reservoir Inflows at Kihoto Reservoir Site.

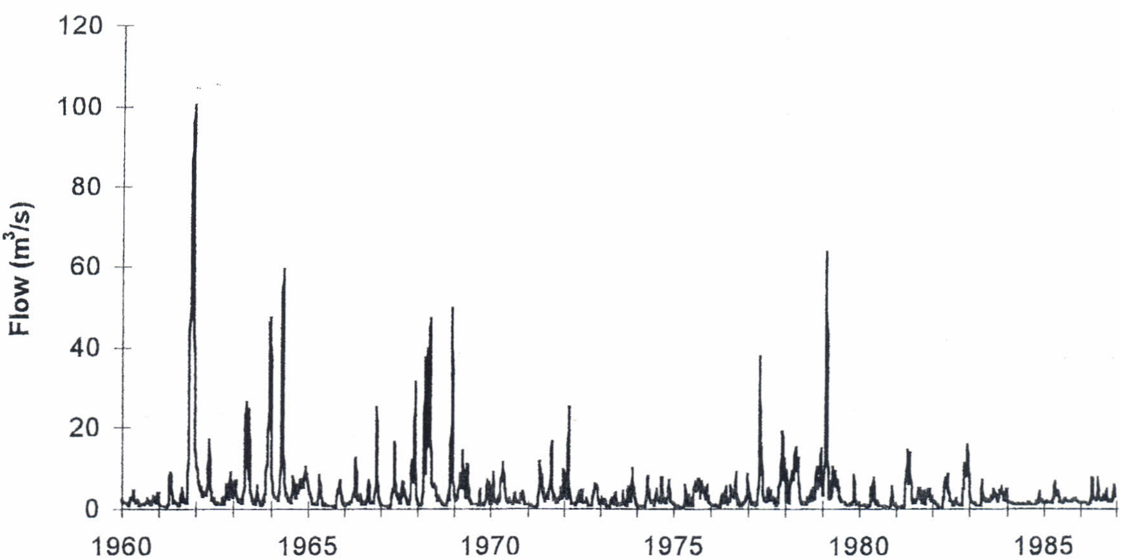


Figure 21. The Hydrograph for the weekly Mean Reservoir Inflow at Kihoto Site

4.3.2 Evaporation Loss from the Surfaces of the Proposed Reservoirs

Evaporation loss from the surface of the proposed Archers Post Reservoir was estimated to be 2018.24 mm annually while that of the Proposed Kihoto Reservoir is 1339.94 mm. The annual weekly mean evaporation at Archers Post and Kihoto Reservoirs are presented in Figure 23 and it is clear that evaporation loss at Archers Post is predominantly higher than that at Kihoto as expected.

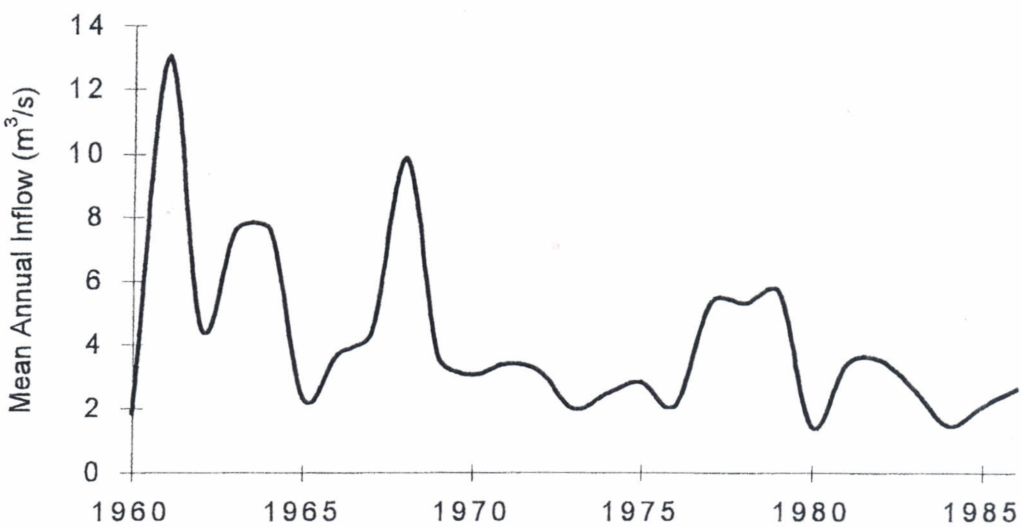


Figure 22 Hydrograph of Annual Reservoir mean Inflow at Kihoto Reservoir Site

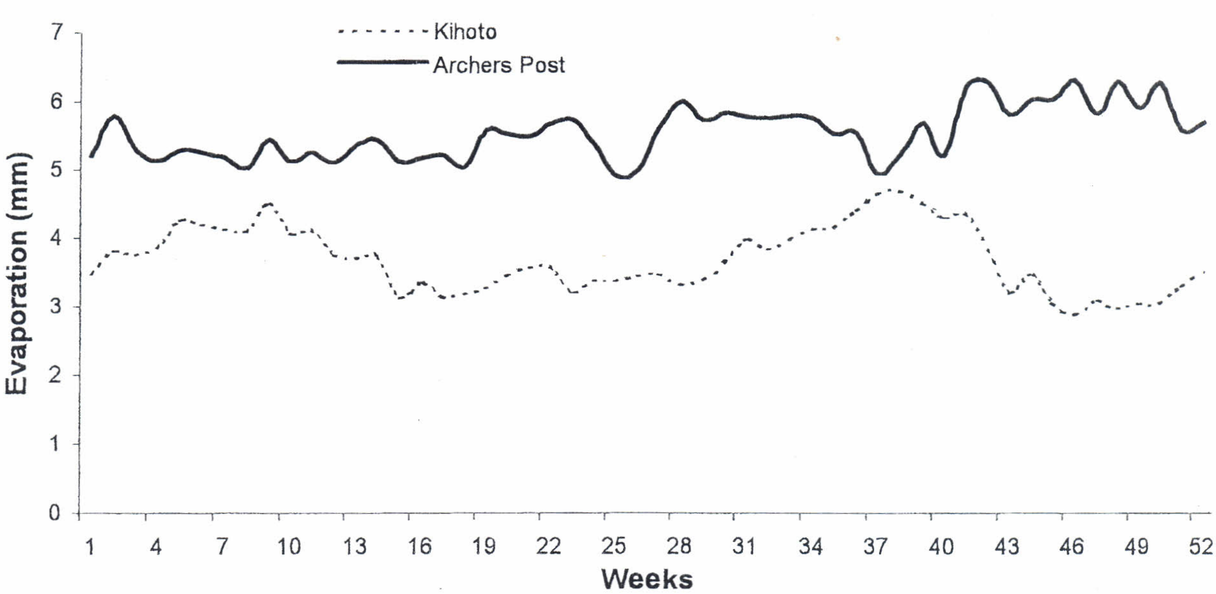


Figure 23 Estimated Evaporation Rates from the Proposed Reservoirs at Kihoto and Archers Post

4.3.3 Estimation of Sedimentation of the Proposed Reservoirs

Figure 24 and Figure 26 shows estimated volume of sediment trapped in the proposed Archers Post and Kihoto reservoirs within the simulation. Figure 25 and Figure 27 shows cumulative volume of sediment trapped in the 2 reservoirs over the simulation period. Over a period of 27 years reservoir sedimentation at Archers Post is estimated at 64.86-million m^3, which represent mean annual rate of 2.4-million m^3. A reservoir with dead storage of 200-million m^3 will be filled within a period of about 83 years at this rate of reservoir sedimentation, which is within the recommended range of between 50 and 100 years. If this reservoir was developed the Government will be required to commission soil conservation in the catchment area to check the high soil loss.

At Kihoto Reservoir the volume of sediments estimated to have been deposited in the reservoir is 2.81-million m^3 which represent an annual average of 0.104 million m^3. A dead storage of 11 million m^3 provided for this reservoir will fill in more than 100 years.

Figure 24 and Figure 26 show that the volume of sediment deposited in the reservoirs is temporal variable and is closely related to the annual mean flow. In 1961 alone the reservoir sedimentation at Archers Post is 22.63-million m^3, which account for 34.9% of the total reservoir sedimentation over the simulation period of 27 years. During this particular year the mean inflow into the proposed reservoir site was 64.59 m^3/s, which is

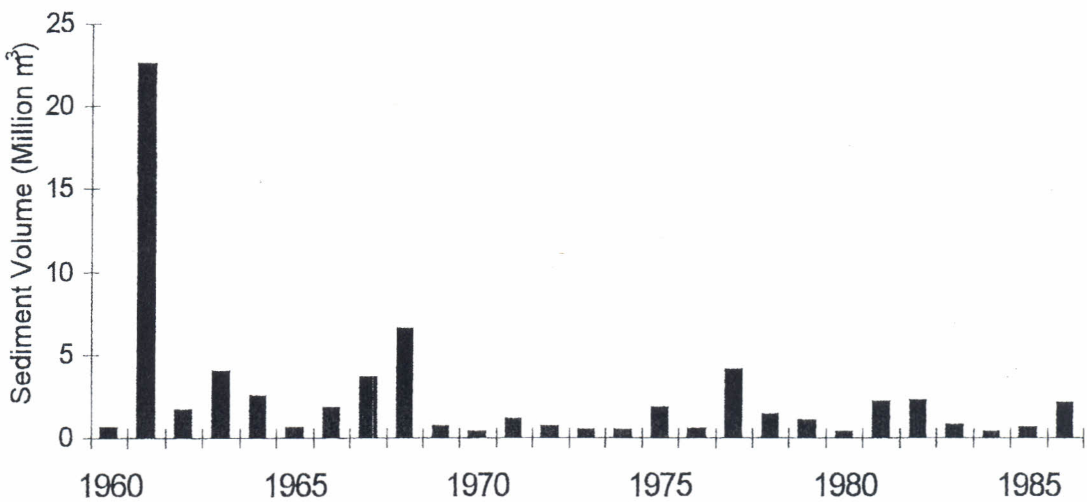


Figure 24 Estimated Annual Volume of Sediment Trapped in the Proposed Archers Post Reservoir

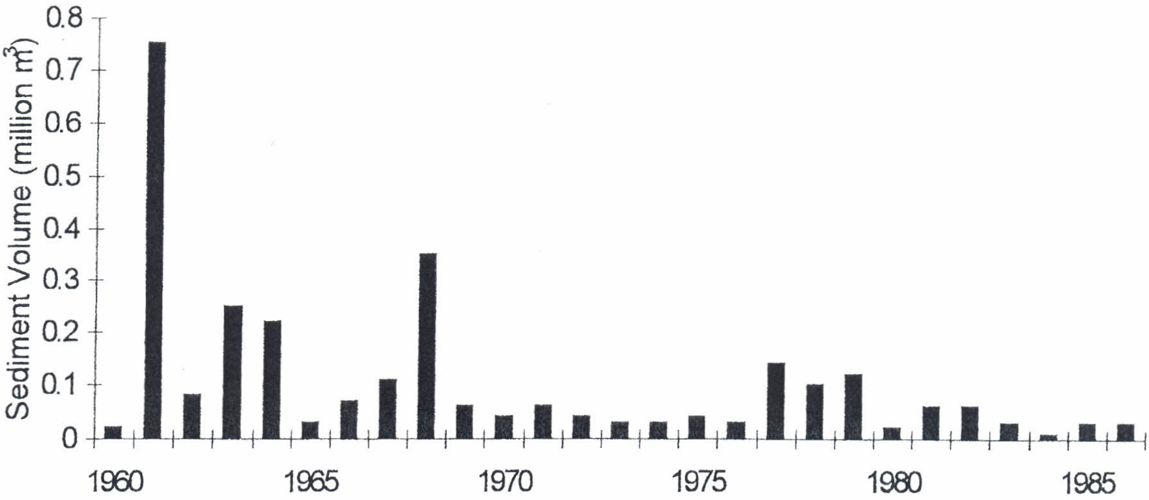


Figure 26. Estimated Annual Volume of Sediment Trapped in the Proposed Kihoto Reservoir

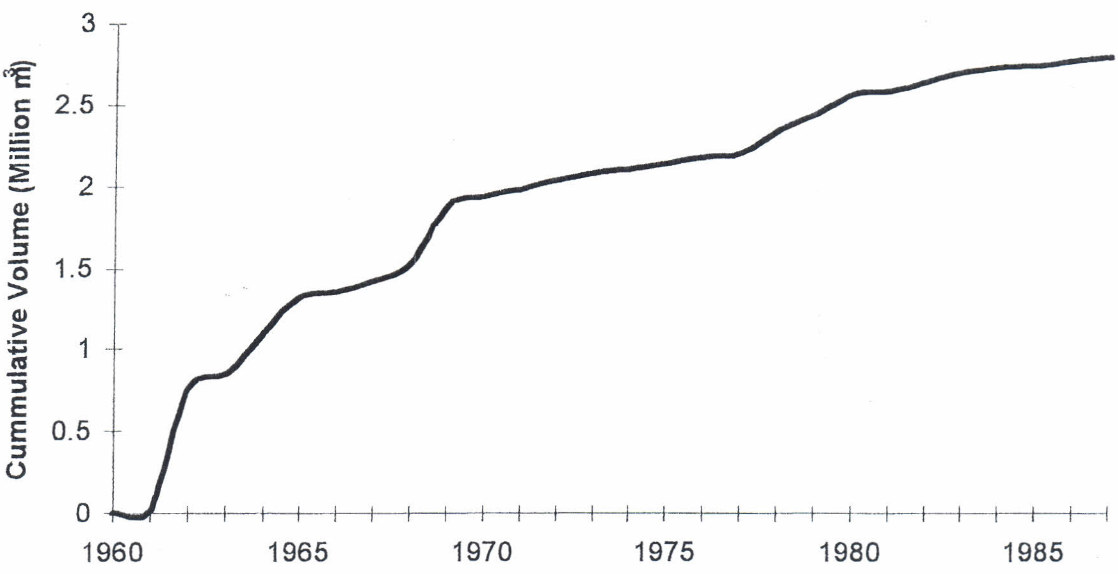


Figure 25 Estimated Cumulative volume of sediment Trapped in the Proposed Kihoto Reservoir

the highest annual mean flow.

From these analysis it is clear that there is a big difference in sediment deposition at the 2 sites. Archers Post exhibit higher rate of deposition than Kihoto as expected since it is to the downstream and the increased catchment area is more degraded than that in the

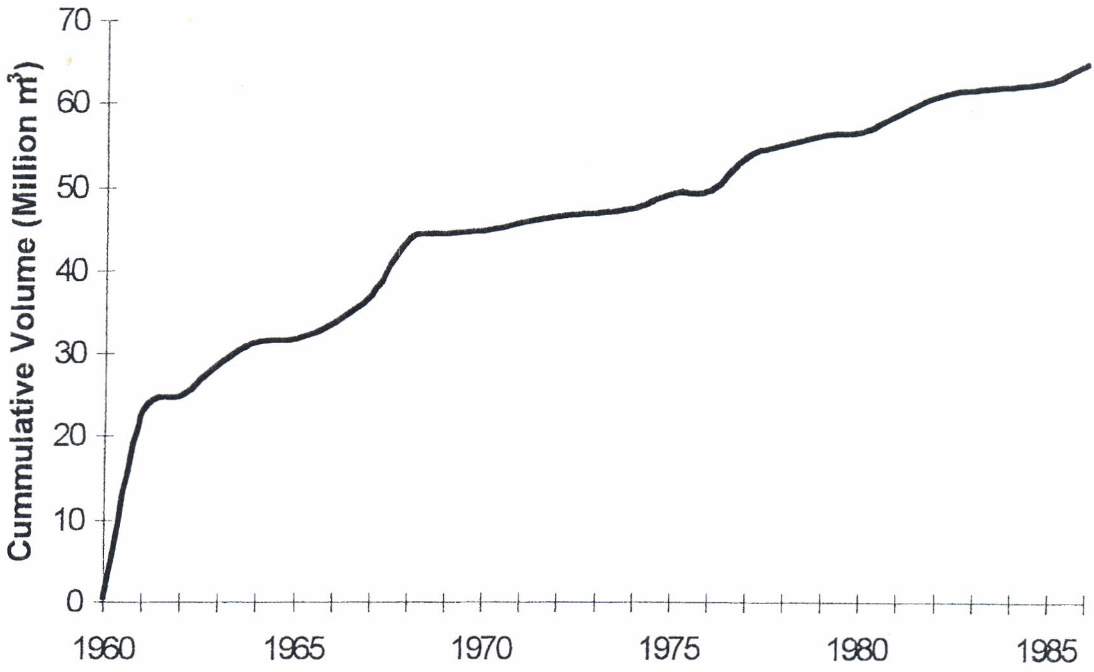


Figure 27 Estimated Cumulative volume of sediment Trapped in the Proposed Archers Post Reservoir

upstream of Kihoto. The volume to be provided for dead storage is much higher at Archers Post than in Kihoto and this makes Kihoto site a better site for reservoir development in regard to reservoir sedimentation.

4.4 RESERVOIR SIZING

4.4.1 Comparison of Mean and Standard Deviation of Reservoir Outflow for Different Reservoir Active Storage Capacities

Mean and standard deviation of reservoir outflow at the proposed Archers Post reservoir are given in Table 12 and presented in Figure 28. Mean flow ranges from 17.04 m^3/s when there is no reservoir to 15.74m^3/s for a reservoir with an active storage capacity of 350-million m^3. The standard deviation of the reservoir outflow varies from 36.86 m^3/s when there is no reservoir to 29.03 m^3/s for a reservoir with an active storage capacity of 350-million m^3. The decline in mean outflow with increase in reservoir capacity is very little but the decline in flow variation (standard deviation) as the reservoir capacity increases is comparatively high.

The decline in mean outflow as reservoir capacity increases is attributed to the increased water loss from the system through evaporation due to increased free water surface area that accompany increased reservoir capacity (see Figure 30). The water that is lost through evaporation over the 27 years of simulation is given in

Table 15. The decline in variation of outflow is linked to the increased capacity of the reservoir to delay more inflows during high flow periods and release it during low flow period. This is evident from the trend of spills (see Table 14 and Figure 30) where they are decreasing with increase in reservoir capacity.

Mean flow for different reservoir capacities at Kihoto reservoir site are given in

Table **13**. The mean reservoir outflow varies from 4.14 m^3/s when there is no reservoir to 3.75 m^3/s when reservoir active storage capacity is 200-million m^3 and above. Standard deviation of the outflow varies similarly from 7.56 m^3/s when there is no reservoir to 4.1 m^3/s when reservoir capacity is 300-million m^3 and above. This variation of the two statistical parameters is shown in Figure 29. Like in the case of Archers Post Reservoir the decline of mean reservoir outflow with increase in reservoir active storage capacity is small whereas the decrease in outflow variation (standard deviation) as the reservoir active storage capacity increases is comparatively high.

Table 12 Mean and Standard Deviation of Outflow at the Proposed Archers Post Reservoir.

	Active Storage (million m3)							
	0	50	100	150	200	250	300	350
Mean (m^3/s)	17.0	16.0	15.9	15.8	15.8	15.8	15.7	15.7
Standard Deviation (m^3/s)	36.9	36.3	35.0	33.9	32.8	31.6	30.3	29.0
Coefficient of Variation (%)	215.7	226.2	220.5	214.2	207.7	200.7	192.6	184.4

Table 13 Mean and Standard Deviation of Outflow at the Proposed Kihoto Reservoir.

	Active Storage (million m^3)							
	0	50	100	150	200	250	300	350
Mean (m^3/s)	4.11	3.79	3.76	3.76	3.75	3.75	3.75	3.75
Standard Deviation (m^3/s)	7.56	7.43	6.08	5.29	4.81	4.25	4.10	4.10
Coefficient of Variation (%)	182.8	196.0	161.7	140.7	128.1	113.2	109.2	109.2

From this comparison it is deduced that the increase in reservoir capacity has very little impact to the mean reservoir outflow but it is reducing variation of the outflow. The reduction of the outflow variation is the benefit accrued to the water resources management by the proposed reservoirs as the flow reliability is improved.

4.4.2 Comparison of Reservoir Evaporation Losses, Spills, Reservoir storage-Inflow and Reservoir Storage-Outflow

Evaporation loss from reservoirs is important in evaluating the increased water loss due to presence of a reservoir and increases with reservoir capacity at the 2 sites (see Figure 30 and Figure 31). At Archers Post total evaporation loss from a reservoir with an active storage capacity of 50-million m^3 is 850.34-million m^3 and it increases to 1008.42-million m^3 for a reservoir with an active storage capacity of 350-million m^3. Similarly evaporation loss from a reservoir with an active storage capacity 50-million m^3 at Kihoto is 241.64-million m^3 while that of reservoir with an active storage capacity of 300-million m^3 and above is 271.5-million m^3. Evaporation loss as a percent of reservoir storage-inflow for the 2 sites is given in Table 16. It decreases with increase in reservoir capacity.

Spills from reservoir decreases with increase in reservoir capacity (see Figure 30 and Figure 31). At Archers it decreases from 12880.87-million m^3 for a reservoir with an active storage capacity of 50-million m3 to 2303.7-million m^3 for reservoir with an active storage capacity of 350-million m^3. Similarly it decreases from 1903.37-million m^3 for a reservoir with an active storage capacity of 50-million m^3 at Kihoto site to 0 for a reservoir with an active storage capacity of 300-million m^3 and above. This indicates that a reservoir of capacity 300-million m^3 and above will not fill at Kihoto site. At Archers Post site more water would be harnessed by increasing the reservoir capacity beyond the 350-million m^3 maximum limit.

A reservoir reduces variation of flow by storing excess water during high flows to release it during low flows. Reservoir storage-outflow and storage-inflow (see section (3.3)) have been used to compare the extent to which reservoirs of different active storage capacities perform this task at the two sites under investigation. Results are given in Table 14 and Table 15 and presented in Figure 30 and Figure 31. Both the storage-inflow and storage-outflow increases with increase in reservoir capacity. At Archers Post reservoir site the storage-inflow and storage-outflow for a reservoir with an active storage capacity of 50-million m^3 are 1548.05 and 648.30 million m^3 respectively. They increase to 4901.81 and 3761.71 million m^3 for a reservoir with an active storage capacity of 350-million m^3.

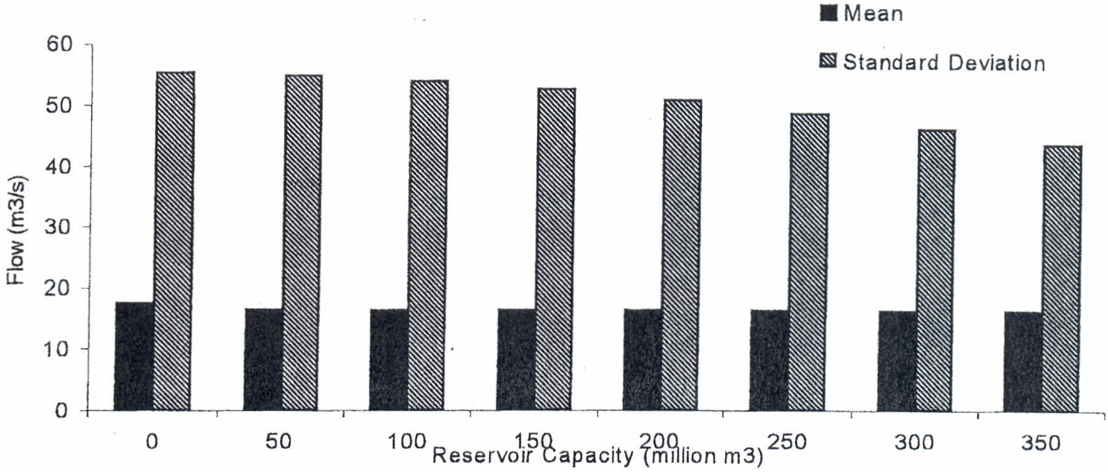


Figure 28 Mean and Standard Deviations of Reservoir Outflows at the Proposed Archers Post Reservoir Site for different Active Reservoir Capacities

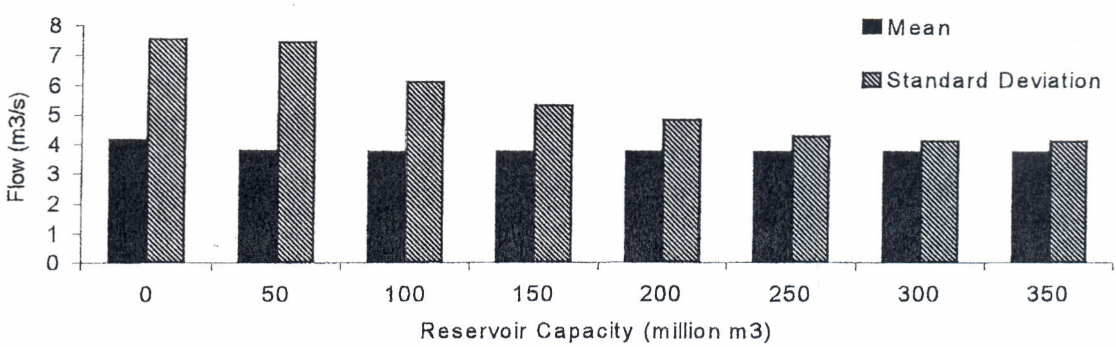


Figure 29. Mean and Standard Deviations of Reservoir Outflows at the Proposed Kihoto Reservoir Site for different Active Reservoir Capacities

At Kihoto reservoir storage inflow and outflow are 468.3 and 176.66 million m^3 respectively. These values increase to 1337.43-million m^3 and 1011.98-million m^3 respectively for a reservoir with an active storage capacity of 350-million m^3. The increase in reservoir storage outflow when reservoir active storage capacity is increased by

50-million m^3 from one capacity to the other is given in Table 16. The increase in reservoir outflow from a reservoir with an active storage of 50-million m^3 to one with an active storage of 100-million m^3 is 156.8% at Archers Post while at Kihoto is 331.53%. This gradually decreases to 5.2% when capacity change from 300 million m^3 to 350 million m^3 at Archers Post and 0% at Kihoto for the same change.

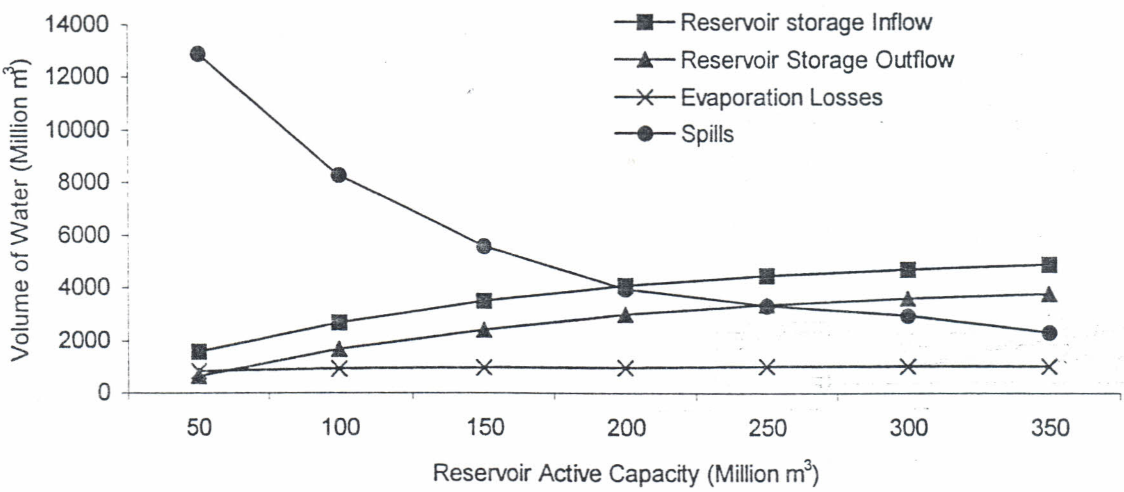


Figure 30. Reservoir Storage Inflow, Reservoir Storage Outflow, Evaporation Losses and Spills for Different Reservoir Active Capacities at Archers Post Site.

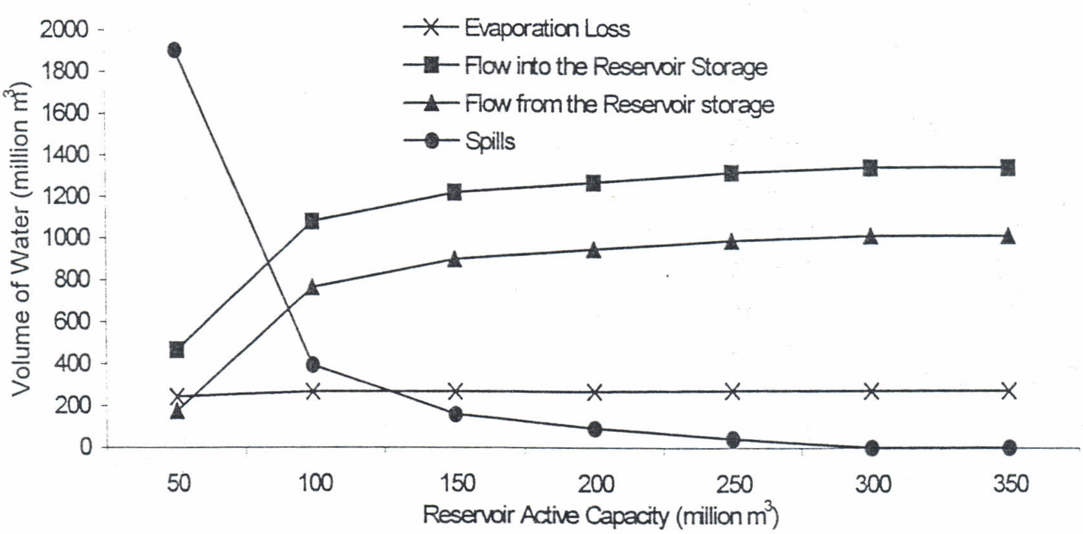


Figure 31. Reservoir Storage Inflow, Reservoir Storage Outflow, Evaporation Losses and Spills for Different Reservoir Active Capacities at Kihoto Site

Table 14. Reservoir Storage Inflow, Storage Outflow, Evaporation Losses and Spills for different Reservoir Active Capacities at Archers Post Site

	Active Storage (million m3)						
	50	100	150	200	250	300	350
Reservoir Storage-Inflow (million m^3)	1548.1	2675.5	3487.6	4111.8	4487.6	4711.1	4901.
Reservoir Storage-outflow (million m^3)	648.3	1665.1	2407.1	3001.4	3362.7	3577.4	3761.
Evaporation Loss(million m^3)	850.3	916.0	953.3	978.9	993.4	1002.0	1008.
Spills (million m^3)	12880.	8304.7	5552.9	3958.5	3323.3	2951.8	2303.

Table 15 Reservoir Storage Inflow, Storage Outflow, Evaporation Losses and Spills for different Reservoir Active Capacities at Kihoto Site

	Active Storage (million m^3)						
	50	100	150	200	250	300	350
Reservoir Storage Inflow (million m^3)	468.30	1080.55	1219.33	1266.41	1314.54	1337.43	1337.43
Reservoir Storage outflow (million m^3)	176.66	762.33	897.25	942.89	989.69	1011.98	1011.98
Evaporation Loss (million m^3)	241.64	264.29	268.14	269.58	270.92	271.50	271.50
Spills (million m^3)	1903.37	391.56	160.17	93.00	41.34	0.00	0.00

Table 16 Evaporation Loss As a Percentage of Reservoir Storage-Inflow

	Active Storage (million m^3)						
Proposed Reservoir	50	100	150	200	250	300	350
Archers Post	54.9%	34.2%	27.3%	23.8%	22.1%	21.3%	20.6%
Kihoto	51.6%	24.5%	22.0%	21.3%	20.6%	20.3%	20.3%

4.4.3 Selecting of the Required Reservoir Capacity

The use of a reservoir in water resources management is to reduce the flow variation in the system so as to improve the flow reliability and in particular to make water available during low flow periods which is referred to here as storage-outflow. This storage-outflow is actually the direct benefit of a reservoir to the water resources system and it is the parameter used here in the selection of the required reservoir capacity.

A desirable increase in reservoir storage-outflow when a reservoir active storage is increased by 50-million m^3 was set at 15%. The first reservoir active storage capacity that failed to meet this requirement was picked as the required active storage capacity. The increase in reservoir outflow at Archers Post when reservoir active storage is increased from 200-million m^3 to 250-million m^3 is 12% (see Table 17) and therefore the 200-million m^3 was selected as the required capacity. The required active storage capacity was added to the dead storage to come-up with the required gross storage of 400-million m^3.

Similarly at Kihoto the increase in reservoir active storage capacity from 150 million m^3 to 200 million m^3 is accompanied by an increase of reservoir storage outflow of 5.2% (see Table 17). Then an active storage of 150-million m^3 was picked as the required active storage and was added to the dead storage to get the required gross storage of 161-million m^3.

Table 17 Percentage increase of Reservoir Storage Outflow when Reservoir Active Storage is Increased by 50 million m^3

	Active Storage (millionm^3)					
Proposed Reservoir	50	100	150	200	250	300
Archers Post	156.8%	44.6%	24.7%	12.0%*	6.4%	5.2%
Kihoto	331.5%	17.7%	5.1%*	5.0%	2.3%	0.0%

* Selected Reservoir Active Storage

4.5 Evaluation of Reservoir Performance

4.5.1 General

The reservoir performance was assessed in terms of low flow augmentation and its ability to reduce the frequency of low flow events, meet target release and recover from failure. This performance assessment was done only for the required reservoir capacity at each site.

4.5.2 Comparing Hydrograph of Reservoir Inflow and outflow

This comparison was done to display the low flow augmentation by the reservoir. The hydrograph for reservoir inflow and outflow are plotted on the same axis in Figure 34 and Figure 35 for Archers Post and Kihoto reservoirs respectively. This displays clearly how the high and low inflows are moderated by the reservoir by having a controlled release.

4.5.3 Low Flow Frequency Analysis

The low flow frequency analysis for both situation when there is no reservoir and when there is a reservoir was carried out to analyse the impact of reservoir in reducing the occurrence of low flows. Results are given in Table 18 and presented in Figure 32 and Figure 33 for Archers post and Kihoto reservoirs respectively. These extreme values of low flow fitted Log Pearson III distribution as depicted in Figure 32 and Figure 33.

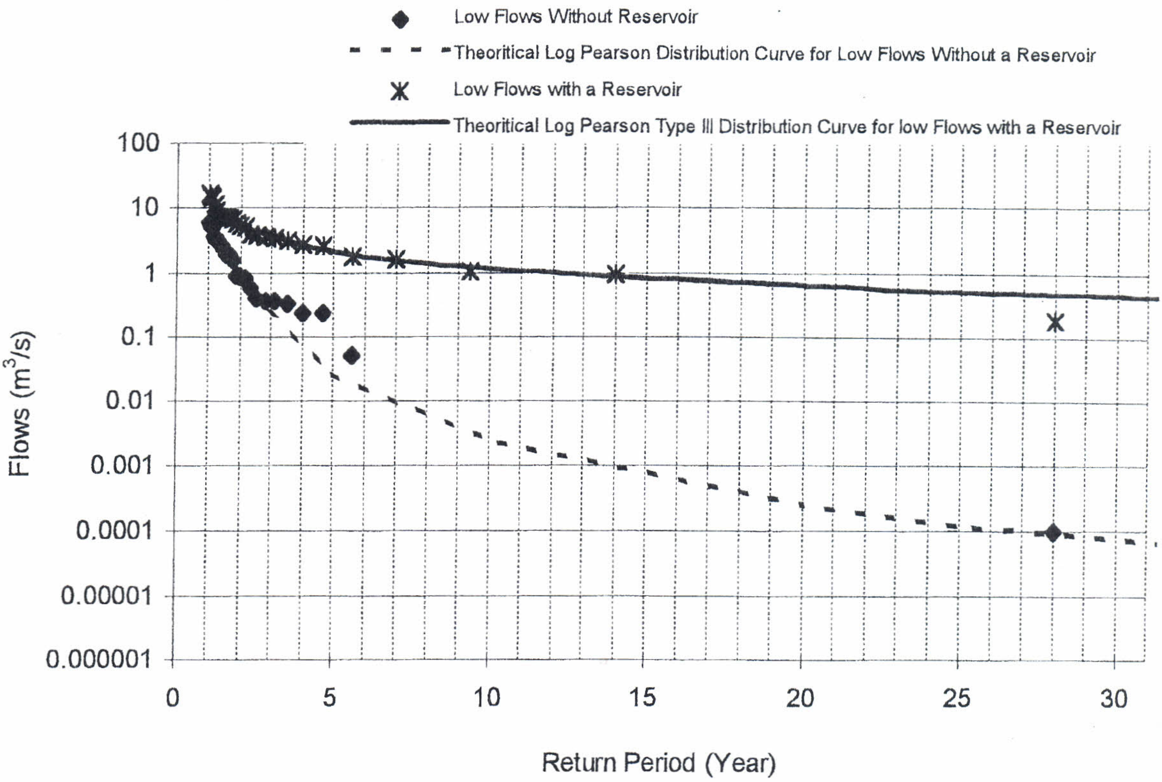


Figure 32 Frequency Curves for Low Weekly Flows at the Proposed Archers Post Reservoir Site

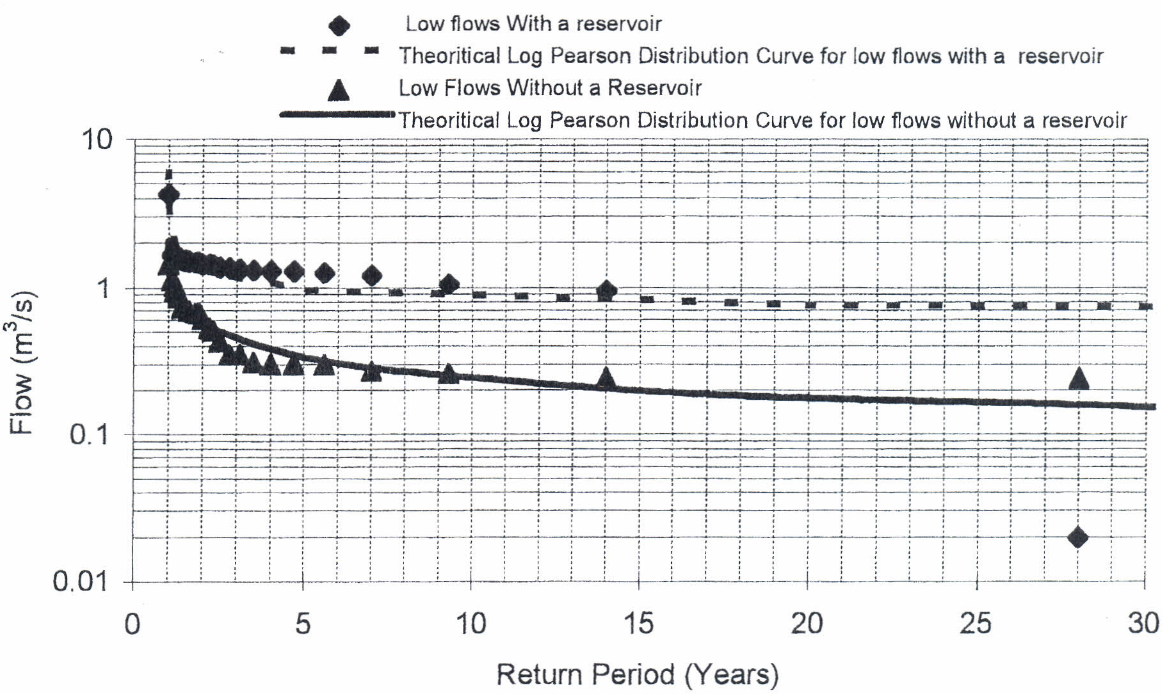


Figure 33 Frequency Curves for Low Weekly Flows at the Proposed Kihoto Reservoir Site

In both cases there is an improvement in return period of low flow events by having a reservoir. For instance at Archers Post a low flow event of $1 m^3/s$ or less occurs once every 3 years

without a reservoir but it occurs once every 10 years with a reservoir of 400-million m^3 gross storage. In the same way a low flow of similar magnitude occur every other year when there is no reservoir at Kihoto but it will occur once every 7 years with a reservoir of 161-million m^3 gross storage capacity.

Table 18 The Ranked Annual Minimum Weekly Mean Outflow at the Proposed Reservoir Site and Their Return Period.

	Archers Post Site		Kihoto Site	
Return Period	Without a reservoir	With a 400-million m3 Reservoir	Without a Reservoir	With 161-million m3 Reservoir
28	0.00	0.18	0.24	0.02
14	0.00	0.92	0.24	0.96
9.3	0.00	1.04	0.26	1.06
7.0	0.00	1.59	0.27	1.22
5.6	0.05	1.73	0.30	1.27
4.7	0.23	2.54	0.30	1.30
4.0	0.23	2.60	0.30	1.30
3.5	0.32	3.06	0.31	1.32
3.1	0.35	3.45	0.35	1.34
2.8	0.36	3.50	0.35	1.38
2.5	0.40	3.69	0.43	1.39
2.3	0.57	3.85	0.51	1.43
2.2	0.82	5.03	0.53	1.44
2.0	0.84	5.17	0.61	1.44
1.9	0.87	5.66	0.67	1.49
1.8	1.61	6.54	0.68	1.49
1.6	1.88	7.03	0.70	1.50
1.6	1.90	7.17	0.70	1.51
1.5	2.29	7.19	0.73	1.52
1.4	2.64	7.19	0.73	1.53
1.3	2.76	7.21	0.86	1.58
1.3	3.15	7.25	0.90	1.61
1.2	3.19	8.57	0.94	1.65
1.2	3.48	11.40	1.03	1.65
1.1	4.89	14.34	1.13	1.65
1.1	5.54	14.74	1.16	1.69
1.0	6.06	16.52	1.45	4.29

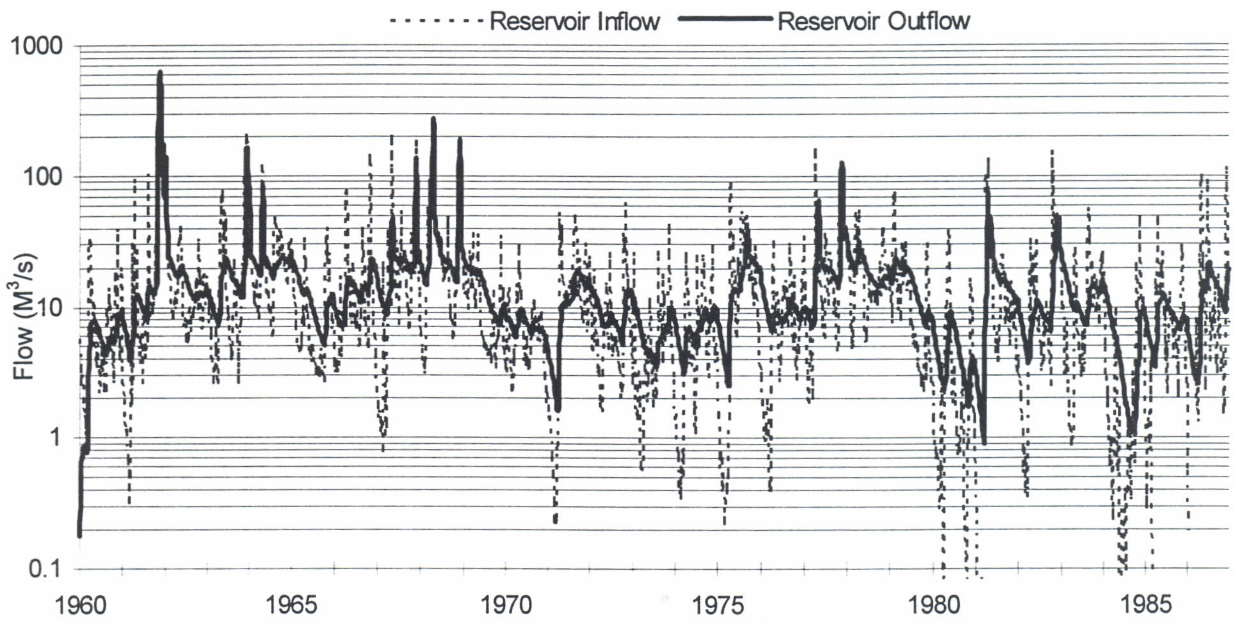


Figure 34 The Inflow and Outflow Hydrograph for the Proposed Archers Post Reservoir

4.5.4 Flow Reliability at the Proposed Reservoir Sites

The computed reliability of flow for different flows is given in Table 19 and Table 20 for the proposed Archers Post and Kihoto reservoirs respectively. There is a marked improvement in flow reliability at Archers Post when there is a reservoir as depicted in Figure 36. At Kihoto there is very little difference in flow reliability for relatively high flows when there is a reservoir and without one. But there is a significant improvement in reliability for moderate and low flows as depicted in Figure 37 with a reservoir.

Table 19 Flow Reliability at the Proposed Archers Post Reservoir Site

	Reliability (%)	
Threshold (m^3/s)	Without Reservoir	With a 400-million m^3 Reservoir
5.67	63.9%	84.8%
5.1	66.0%	87.2%
4.54	70.2%	89.5%
2.84	80.8%	95.5%
1.2	89.8%	98.6%

The reliability and weekly mean flow exhibited close relationship and trend-lines were fitted to the data. In all cases polynomial functions had the highest correlation coefficient and these functions are presented together with their correlation coefficients in table (21). Their correlation coefficients are all above 0.95, which indicates that they are important in estimating reliability of flow within these ranges

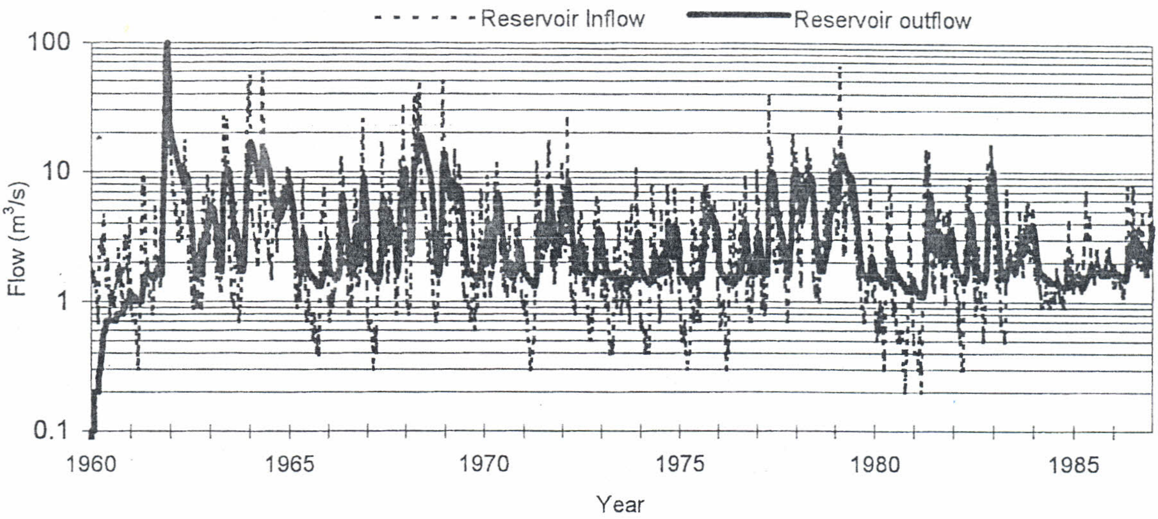


Figure 35 The Inflow and Outflow Hydrograph for the Proposed Kihoto Reservoir

Table 20 Flow Reliability at the Proposed Kihoto Reservoir Site

	Reliability (%)	
Threshold (m^3/s)	Without Reservoir	With a 161-million m^3 Reservoir
2.07	51.9%	54.2%
1.61	61.4%	70.7%
1.43	66.1%	88.6%
1.21	72.8%	94.5%
0.804	86.5%	97.3%

Table 21 Functions Relating Reliability of Outflow to Threshold

	Equations		
Reservoir	Without Reservoir	With Reservoir*	Range (m^3/s)
Archers Post	$Y=-0.0005Q^2-0.0565Q+0.96979$ $r^2=0.9975$	$Y=-0.0039Q^2-0.0047Q+0.9978$ $r^2=0.9993$	1.2 – 5.67
Kihoto	$Y=0.0654Q^2-0.4709Q+1.2017$ $r^2=1$	$Y=0.2852Q^2+0.4226Q+0.8288$ $r^2=0.9533$	0.8–2.01

*400-million m^3 for Archers Post and 161-million m^3 for Kihoto

4.5.5 Resiliency of Flow at the Proposed Reservoir Sites

Computed resiliency of flow, which is a measure of the ability of a water system to recover from unsatisfactory condition, is given in Table 22 and Table 23 for Archers Post and Kihoto reservoirs respectively.

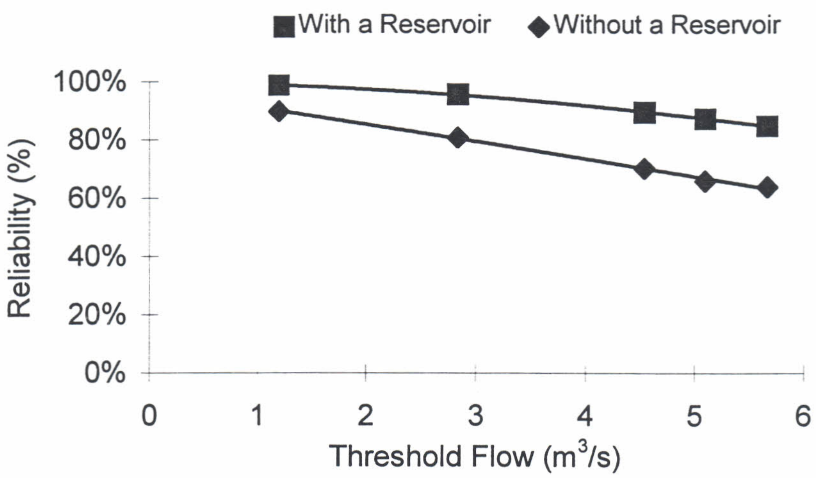


Figure 36 Reliability of Flow at the Proposed Kihoto Reservoir Site.

At the two sites the resiliency of the system with a reservoir is lower than the one without a reservoir. This indicates that once unsatisfactory condition occurs in a system where there is a reservoir it takes long to recover.

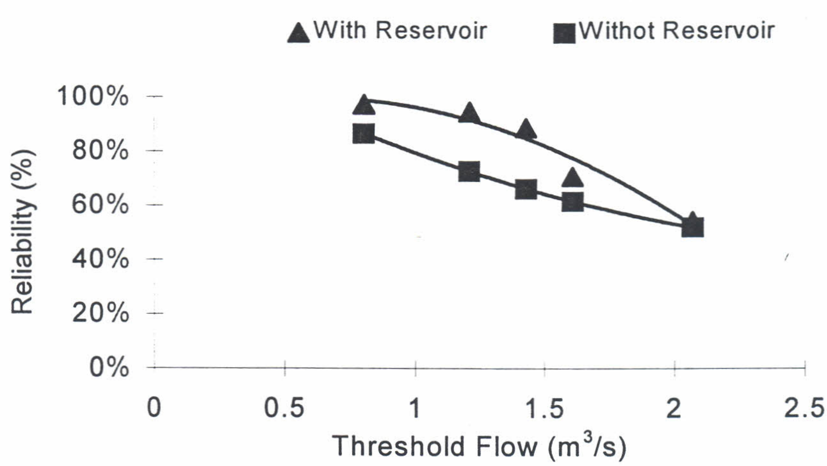


Figure 37 Reliability of Flow at the Proposed Archers post Reservoir Site.

Table 22 The Resilience of Outflow at the Proposed Archers Post Reservoir Site

	Resiliency (%)	
Threshold (m^3/s)	Without a reservoir	With a 400-million m^3 Reservoir
5.67	17.7%	8.5%
5.1	18.2%	8.3%
4.54	19.4%	9.5%
2.84	19.7%	14.3%
1.2	20.3%	15.8%

Table 23 The Resiliency Flow at the Proposed Kihoto Reservoir Site

	Resiliency (%)	
Threshold (m^3/s)	Without reservoir	With a 161-million m^3 Reservoir
2.07	18.8%	5.9%
1.61	18.3%	6.1%
1.43	20.6%	7.5%
1.21	24.3%	3.9%
0.804	26.5%	2.6%.

4.6 Availability of Water to Maintain Flow between Junction and Archers Post

The required flow at junction to maintain continuos flow to Archers Post during the period starting from 3rd week of a year to 13th week of a year was evaluated and found to be 1.43 m^3/s. Table (24) compares the number of weeks, out of the 297 weeks considered, when the flow at Junction is lower than the required flow for the case where there is no reservoir at Kihoto and when there is a reservoir. There is big reduction from 155 weeks where flow is lower than required when there is no reservoir to just 8 weeks when there is a reservoir. This shows the effectiveness of the proposed reservoir at Kihoto in flow regulation in the upper catchment.

Table 24 Number of Weeks When Flow at Junction is Less than Required Flow to Maintain Flow up to Archers Post

Required Flow at Junction (m^3/s)	Weeks With Flows Less Than Required Flow at Junction	
	Without a Reservoir	With a 161-million m^3 Reservoir at Kihoto
1.43	155	8

5. CONCLUSION AND RECOMMENDATIONS

5.1 Comparison of the Proposed Reservoirs in Kihoto and Archers Post

The proposed reservoirs at the two sites are compared in Table 25 and from this comparison the proposed Kihoto reservoir has small gross capacity due to low rate of sedimentation while the proposed Archers Post reservoir has a high mean outflow. Though the proposed Kihoto Reservoir looks attractive when the required capacity is considered it is disadvantaged by the little outflow, which to some extent will be adequate to guarantee continued flow upto Archers Post but not beyond there. Proposed Archers Post reservoir has a high outflow but it has the problem of high rate of sedimentation. If the present conditions in the catchment prevail in the future the proposed reservoir with an active storage capacity of 200 million m^3 and a dead storage of similar size will have a useful life-span of around 80 years which is a short period of a reservoir of its size.

Table 25 Comparison of Reservoir Parameters for the Selected Reservoirs at the Proposed Sites of Kihoto and Archers Post.

Parameter	Kihoto Reservoir	Archers Post Reservoir
Gross Capacity (million m^3),	161	400
Dead Storage (million m^3),	11	200
Active Storage (million m^3),	150	200
Dead Storage as % of Gross Storage,	6.8	50.0
Mean Outflow (m^3/s),	3.79	16.45
Coefficient of Variation of the Outflow (%),	140.7	308
Total Flow made available by the Reservoir (million m^3)	897.25	2904.97

5.2 Conclusion

This study explored the possibility of developing reservoirs at the proposed sites viz. a viz. the problem of sedimentation, evaporation loss and availability of a topographical suitable site.

The rating equations and streamflow data for RGS 5E3 and RGS 5BC4 were used to estimate the sedimentation rate at Archers Posts and Kihoto reservoir sites respectively. It was established that the average annual sedimentation rate at Archers Post Reservoir site is 2.4 million m^3 while at Kihoto Reservoir site is 0.104 Million m^3. This implies that the selected dead storage of 200 million m^3 at Archers Post site will fill within a period of 83 years while at Kihoto site a reservoir with an expected life span of 100 years will require a dead storage of about 10.4 million m^3 and hence the selected dead storage of 11 million m^3 for this site will suffice.

The streamflow data at Archers Post site was estimated from data for RGS 5E3 and it was establish that the mean flow at the site is 17.04 m^3/s which is 81% of the mean flow at the RGS 5E3. The flow at Kihoto site was estimated from the data for RGS 5BC4 which is in its upstream and the mean flow at the site was found to be 4.14 m^3/s and it is 126% of the mean flow at the RGS 5BC4.

The reservoir operation study was done using IRAS simulation model and at each site reservoir with active capacity ranging from 50 million m^3 to 350 million m^3 at interval of 50 million m^3 were studied. From the study it was shown that reservoir mean outflow reduces with increase in reservoir capacity and this is attributed to increased evaporation losses from the reservoir. The mean outflow at Archers Post site ranges from 16.0 m^3/s for a reservoir with an active capacity of 50 million m^3 to 15.7 m^3/s for one with 350 million m^3 active capacity. The evaporation losses over the 27 years simulation period from the reservoir at this site increases from 850.3 million m^3 for a reservoir of 50 million m^3 active storage capacity to 1008.4 million m^3 for one with 350 million m^3 active storage. While at Kihoto site the mean outflow ranges from 3.79 m^3/s for a reservoir with a active capacity of 50 million m^3 to 3.75 m^3/s for one with an active storage of 200 million m^3 and above. The reservoir evaporation losses at the site increases from 241.64 million m^3 for a reservoir with active storage of 50 million m^3 to 271.50 million m^3 for one with active storage capacity of 300 million m^3 and above.

The ability of the reservoir to regulate river flow by holding high flows and releasing them during low flows is evaluated in this study, where the ability to hold high flows is computed as the reservoir storage inflow and increases with increase in reservoir capacity. Part of this storage inflow is lost through evaporation from the reservoir free water surface and the rest is released during low flow periods which is the actual augmentation of low flows by the reservoir and it was considered in this study to be the reservoir storage outflow. The storage outflow was used to compare the various reservoir capacities in order to come up with an optimum reservoir size. Over the 27 year simulation period at

Archers Post site a reservoir with 50 million m^3 active storage capacity held a total of 1548.1 million m^3 of reservoir storage inflow and released 648.3 million m^3 of it as the reservoir storage outflow, while a reservoir with an active storage capacity of 350 million m^3 held a total of 4901.8 million m^3 storage inflow and released 3761.71 million m^3 of this as storage outflow. The difference between the reservoir storage inflow and outflow is largely composed of the evaporation losses which are 54.9% of the storage inflow for the reservoir with an active storage capacity of 50 million m^3 and 20.6% of the same for the reservoir with an active storage capacity of 350 million m^3. Similarly, at Kihoto site the storage inflow for a reservoir of 50 million m^3 active storage capacity is 468.30 million m^3 while a reservoir with an active storage capacity of 300 million m^3 and above have storage inflow of 1337.43 million m^3 and 1011.98 million m^3 of storage outflow. The evaporation losses expressed as a percentage of storage inflow varies from 51.6% for a reservoir of 50 million m^3 active storage capacity to 20.3% for a reservoir with an active storage capacity of 300 million m^3 and above. It is deduced from this comparison that there is no significant difference to evaporation losses at the two sites when these losses are expressed as a proportion of the storage inflow.

A marginal increment of 15% of the reservoir storage outflow as a result of increasing active storage capacity by 50 million m^3 was used in identifying the optimum reservoir capacity and at Archers Post site the increase of the active storage capacity from 200 million to 250 million m^3 resulted to a 12% increase of storage outflow which is less than the marginal value of 15% while the preceding increase from 150 million to 200 million m^3 results to a 24.7% increase in reservoir storage outflow. Therefore, at this site a reservoir with an active storage capacity of 200 million m^3 was found to be the optimal capacity and thus, when the 200 million m^3 dead storage is considered the reservoir gross capacity becomes 400 million m^3. In this case the dead storage of the proposed reservoir will be 50% of the gross storage. Similarly at Kihoto site increase of reservoir active storage capacity from 150 million m^3 to 200 million m^3 resulted to 5.1% increase of storage outflow while the preceding increase resulted to a 17.7% increase in reservoir storage outflow and hence the 150 million m^3 was selected as the optimal active storage capacity at the site. Therefore, when the 11 million m^3 dead storage is considered the optimal gross storage at the site becomes 161 million m^3 which implies that dead storage is only 6.8% of the gross storage.

The selected reservoir capacity at the two sites reduces the frequency of low flow events occurrence and improve the flow reliability at the sites. At Archers Post site the flows lower or equal to 1 m^3/s has a return period of 2 years without a reservoir but the return period increases to about 10 years with a reservoir at the site and similarly at Kihoto flows lower or equal to $1m^3/s$ occurs every other year but the return period increases to about 7 years with a reservoir. There is a marked improvement in flow reliability at Archers Post site when there is a reservoir and at Kihoto there is very little difference in reliability of high thresholds values for the situations when there is no reservoir and when there is one but there significant improvement in reliability for the moderate and low threshold values.

Neither of the proposed reservoirs has been found to have an overwhelming advantage over the other and it is concluded that each will be required for its own purpose. Kihoto reservoir will mainly be required to regulate river flow in the upper part of the basin so as to ensure continued flow upto Archers Post in the year round. Whereas Archers Post reservoir will be required to regulate flow to the lower parts of the basin and the improved flows could be used to tap the irrigation and livestock production potential in the area.

5.3 RECOMMENDATIONS

This study did not address the possibility of diverting flood flow from Nanyuki River system to the proposed Kihoto reservoir so as to utilise the unutilised reservoir capacity at the site. This could enhance the potential of this site and hence it is recommended that a detailed investigation be carried-out to evaluate this possibility.

Also from this study two things are clear. One is that the required flow at Junction to maintain continuous flow upto Archers Post has not been determined precisely and it is recommended here that a detailed study be done to establish it. Secondly the required flow at Archers Post that will guarantee some flow upto Lorian Swamps has not been determined precisely and similarly a detailed study is recommended to establish it plus the water demand that can be met from the river water.

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Appendix 4.31 Reservoir Inflow at the Proposed Archers Post Reservoir

Year

Week	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986
1	5.6	3.5	139.3	12.4	77.8	21.8	5.1	2.4	8.2	18.4	3.4	2.2	14.6	2.6	1.5	2.6	3.1	34.4	36.7	14.2	2.3	0.5	3.6	13.1	13.9	1.2	0.5
2	3.4	2.2	72.6	7.7	35.5	19.2	3.7	1.8	6.3	14.4	3.2	1.7	10.8	4.6	1.5	1.5	2.9	7.6	28.2	12.2	1.4	0.1	2.7	9.6	4.8	0.5	0.2
3	2.5	1.9	43.3	10.0	24.2	14.0	3.2	1.5	4.7	12.6	5.4	1.5	9.0	25.9	1.3	1.0	1.9	14.0	22.7	7.7	0.9	0.0	2.2	10.2	3.3	0.3	0.0
4	3.0	1.5	31.4	13.6	18.9	12.2	3.3	1.6	4.1	12.6	12.0	1.5	5.0	3.9	0.8	0.6	1.5	3.9	31.1	8.5	0.7	0.0	2.0	5.9	2.9	1.4	0.0
5	2.5	1.1	21.2	6.0	14.9	10.6	3.9	1.4	3.5	18.1	14.0	1.1	4.7	2.1	0.7	0.8	1.2	3.0	9.2	67.8	0.9	0.0	1.5	3.1	2.2	0.9	0.0
6	2.0	1.0	17.1	4.2	13.1	9.2	3.2	1.1	3.1	12.4	6.0	0.8	6.0	1.5	0.5	0.7	1.0	2.0	4.9	74.6	0.8	0.0	0.8	3.1	1.8	3.7	0.0
7	1.4	1.1	14.2	4.0	11.9	7.0	3.7	1.3	3.5	9.6	3.4	0.7	19.4	1.4	0.4	0.4	1.1	1.9	8.0	41.6	0.6	0.0	0.5	4.0	1.2	2.0	0.0
8	1.4	0.6	11.2	2.9	10.4	5.6	3.6	0.8	4.7	12.9	3.0	0.5	10.2	2.4	0.6	0.5	1.2	9.5	5.1	25.5	0.6	0.1	0.8	7.5	0.6	0.4	0.0
9	1.1	0.5	9.3	2.7	14.9	4.9	17.1	20.1	43.9	25.7	2.3	0.3	5.2	3.2	0.4	0.2	2.0	7.0	17.2	13.8	0.3	0.1	0.4	2.4	0.6	0.1	0.0
10	0.8	0.3	8.3	3.4	15.7	4.6	4.3	5.8	56.2	22.7	2.4	0.2	5.7	1.9	0.8	0.3	1.1	4.1	17.7	9.7	0.5	0.0	0.7	1.5	1.0	0.0	2.0
11	6.8	0.9	8.5	4.6	9.3	4.6	5.5	1.8	51.2	14.0	4.6	0.2	3.2	1.0	0.7	0.3	0.7	2.1	50.3	14.0	0.8	0.0	0.6	1.2	1.0	0.0	8.2
12	27.1	30.3	11.2	6.9	8.2	4.8	6.8	1.0	35.2	15.6	3.8	0.3	3.4	0.6	0.5	0.4	0.4	10.1	35.4	21.1	3.4	76.1	0.4	1.2	0.6	6.5	1.6
13	31.4	1.9	12.9	2.8	12.3	5.1	9.8	1.3	41.0	36.0	5.8	0.8	2.1	0.6	12.0	0.4	0.4	10.1	18.2	13.5	0.1	98.5	0.4	1.0	0.3	15.7	1.4
14	31.3	5.1	9.8	2.7	11.9	5.8	9.3	37.4	110.0	18.3	12.4	0.9	1.9	0.7	17.8	0.5	1.3	71.1	38.9	8.0	0.4	80.0	17.7	0.9	2.8	12.7	1.5
15	11.4	91.2	10.9	4.5	35.5	10.2	60.6	31.2	77.8	11.4	14.7	9.5	1.6	1.1	26.0	5.3	16.1	158.3	54.8	29.6	4.4	59.6	4.3	1.0	4.1	47.9	24.5
16	15.6	17.0	15.0	34.8	116.2	7.4	76.5	23.4	65.2	12.7	11.1	50.8	1.6	6.2	13.7	64.7	31.5	54.9	29.6	24.0	10.2	132.8	31.5	1.1	9.1	45.2	6.6
17	9.1	10.7	23.8	55.3	84.5	31.8	29.3	34.4	262.6	10.0	20.2	29.1	1.9	5.6	13.3	86.7	12.2	15.7	21.9	15.2	5.2	26.8	16.1	19.3	0.8	12.5	66.7
18	10.4	14.2	31.5	63.9	41.3	20.7	22.4	10.7	177.7	18.8	28.5	41.3	7.5	4.0	5.7	10.7	10.2	57.6	24.0	19.1	15.3	24.6	24.1	27.6	0.7	14.1	96.4
19	6.7	27.3	40.8	75.0	26.0	12.8	19.3	192.8	70.8	35.4	13.2	20.9	7.5	1.9	3.8	8.8	6.4	66.9	26.1	31.2	37.3	49.4	15.5	12.5	0.6	17.1	22.4
20	4.1	7.8	34.6	41.2	20.6	9.7	10.6	49.1	42.9	23.5	13.2	15.6	7.2	2.8	4.2	35.4	10.4	31.7	27.6	22.8	28.7	39.6	13.1	7.9	1.6	18.8	20.9
21	3.6	4.5	24.1	22.5	16.3	8.6	8.8	30.8	35.9	15.1	8.6	12.2	15.2	2.9	3.6	15.6	12.8	19.7	14.8	13.5	27.1	18.8	24.3	6.4	0.2	25.7	11.3
22	7.5	3.8	12.7	50.5	13.2	6.0	7.6	23.8	35.7	11.6	7.8	9.8	8.8	11.7	5.2	17.3	9.7	11.7	8.8	15.8	7.1	10.4	32.0	5.1	0.0	14.7	6.7
23	4.2	5.4	9.1	40.6	12.8	4.8	8.5	14.9	37.4	7.5	6.4	11.7	9.5	5.5	6.4	8.3	7.7	10.0	5.5	15.3	4.0	7.0	12.5	5.0	0.3	9.0	2.4
24	3.3	6.2	7.2	18.9	13.1	4.6	20.5	11.7	30.7	6.4	3.9	13.1	4.8	4.8	2.3	6.0	2.8	7.4	5.7	15.8	2.8	5.0	6.7	4.2	0.3	4.0	14.6
25	2.2	2.6	7.9	11.7	10.1	3.9	16.0	31.6	33.8	5.6	4.4	10.0	17.3	3.3	1.7	12.2	4.1	6.3	6.5	12.6	1.9	4.5	8.4	2.9	0.0	9.4	87.0
26	2.5	8.2	7.3	8.1	9.5	5.0	8.1	28.0	27.8	5.4	3.6	11.3	25.1	1.8	1.1	23.9	7.1	5.9	8.1	7.5	3.5	6.2	8.1	4.9	0.0	3.3	60.5
27	5.0	8.4	5.6	10.2	9.8	5.0	7.6	9.4	21.3	4.9	4.8	14.3	7.7	3.8	10.8	15.1	7.5	35.4	16.6	7.7	3.5	5.5	5.4	6.2	0.1	2.2	27.3
28	3.6	4.3	5.5	7.1	7.5	4.0	6.9	7.5	20.3	5.3	5.6	14.1	4.4	4.4	24.7	21.2	14.2	31.5	16.3	6.2	1.9	3.9	4.9	3.2	0.4	4.5	8.0
29	3.3	4.2	5.8	4.9	6.1	4.0	8.7	19.5	16.0	4.7	3.4	15.9	5.8	2.9	9.3	37.9	13.0	13.7	11.5	5.3	1.5	6.9	3.3	4.8	0.1	5.8	3.9
30	4.8	4.9	7.5	4.7	6.3	3.7	9.2	53.6	16.7	5.0	3.4	15.5	4.6	2.1	5.0	52.9	12.0	10.7	11.0	5.5	1.1	9.2	3.9	6.3	1.7	12.5	7.7
31	2.3	6.0	9.9	4.1	28.6	3.3	5.1	23.0	19.6	4.3	3.7	13.0	6.0	2.2	9.5	38.9	15.7	18.7	12.0	4.9	0.8	17.0	3.9	9.0	1.0	9.8	9.4
32	2.3	101.8	6.7	4.3	46.8	3.1	5.2	18.5	19.6	4.7	4.8	13.3	5.2	25.5	8.3	30.3	11.9	26.1	13.0	6.1	1.0	25.0	7.3	8.5	1.6	8.0	6.5
33	2.3	16.2	8.3	22.2	41.6	3.1	6.6	19.4	18.8	4.6	8.3	35.9	5.2	11.5	7.7	34.8	10.7	20.4	12.6	5.3	0.9	24.6	6.3	15.1	1.3	7.7	8.0
34	9.4	13.8	8.6	18.4	32.9	3.5	9.0	26.1	19.3	4.7	8.7	49.6	7.8	7.9	14.9	48.6	7.2	15.6	13.2	4.8	0.8	17.5	8.3	54.0	0.6	4.1	16.6
35	7.5	8.7	8.4	15.0	21.5	3.3	38.6	26.6	49.0	3.7	10.5	47.3	5.0	8.0	24.8	34.7	18.1	16.4	11.1	3.7	1.8	18.3	10.8	35.7	0.4	6.6	7.9
36	13.5	9.0	9.6	12.9	26.2	2.9	32.0	19.9	12.3	4.0	11.1	40.2	4.5	10.5	18.8	49.7	30.1	13.6	13.7	3.4	1.4	17.7	8.0	32.9	3.0	6.1	4.2
37	11.3	9.2	10.7	10.7	27.2	4.7	25.2	14.7	9.5	9.8	7.2	23.2	3.3	6.8	18.4	39.7	19.3	8.9	10.7	3.1	2.7	7.8	5.7	20.1	1.9	6.9	3.2
38	10.1	13.8	12.4	5.6	38.2	3.9	14.0	10.7	8.7	13.4	8.3	13.1	2.5	4.0	10.0	25.3	12.6	11.5	13.0	2.9	1.0	8.6	5.4	21.5	1.7	5.4	16.5
39	6.8	10.6	14.2	4.4	36.9	3.1	8.3	9.1	7.8	6.3	7.7	12.3	2.1	16.9	6.5	16.3	9.1	9.1	16.4	2.5	0.3	13.9	3.9	15.6	1.5	5.0	11.3
40	5.9	15.3	15.8	2.6	28.1	3.1	7.6	7.0	7.8	4.4	8.0	23.2	2.3	11.1	4.6	32.5	4.9	6.0	22.4	3.0	0.3	14.8	2.5	10.0	0.7	2.1	7.2
41	4.3	17.7	9.6	7.1	25.0	2.8	5.2	7.3	6.0	3.5	5.6	14.8	3.5	5.6	3.6	28.5	6.0	4.7	13.3	2.5	0.1	8.3	2.6	17.3	19.9	3.5	3.4
42	2.7	45.2	32.4	7.7	28.9	10.2	5.0	13.3	6.1	4.8	8.4	9.8	35.6	7.5	2.9	17.0	17.1	5.1	12.4	2.3	0.1	5.2	145.3	15.2	10.7	6.8	3.2
43	4.4	86.4	16.8	7.9	22.3	33.5	10.4	36.4	8.3	5.6	6.7	9.1	31.2	7.0	6.9	12.7	3.8	20.9	15.7	5.7	1.1	8.5	26.5	9.4	10.5	8.0	3.8
44	17.5	137.8	15.9	21.3	21.1	22.1	137.6	36.5	12.0	6.4	5.5	8.4	22.4	6.7	7.3	16.9	2.8	75.3	37.6	3.0	0.9	15.4	107.3	9.8	6.9	7.4	4.4
45	12.6	560.4	10.3	12.2	18.8	39.5	58.0	28.2	13.0	6.0	9.1	7.7	61.9	41.3	16.3	10.5	4.5	63.5	39.9	25.3	15.7	8.4	34.7	15.7	35.8	28.6	1.5
46	38.5	567.8	6.9	95.8	22.6	15.9	32.9	18.1	15.8	8.9	9.0	8.8	18.2	24.9	22.9	18.7	5.2	120.3	17.9	30.2	12.1	6.9	36.5	12.0	47.0	16.7	1.8
47	13.0	624.3	7.9	61.8	16.2	25.4	16.0	38.5	47.9	34.1	7.0	13.9	16.3	14.4	28.7	28.8	4.5	126.0	14.7	8.8	13.7	13.0	29.9	7.6	12.1	7.4	9.7
48	6.1	410.4	16.8	42.1	15.2	16.7	11.3	183.7	158.4	18.1	5.3	30.5	14.8	8.5	5.9	19.2	8.3	89.1	13.9	5.6	8.3	8.7	50.5	8.1	9.7	10.6	33.9
49	4.6	105.2	17.7	198.6	27.5	10.7	7.7	45.0	186.3	9.8	4.7	14.4	10.1	6.4	4.6	7.5	7.3	40.2	16.3	4.0	2.1	5.0	48.5	22.0	9.4	4.8	112.7
50	4.1	175.1	14.9	165.3	29.8	6.9	5.9	28.5	68.0	13.9	2.9	6.6	6.1	4.0	3.8	6.9	16.0	34.5	20.8	2.9	1.6	4.1	48.9	24.5	11.3	3.5	21.8
51	19.2	88.4	10.7	61.6	30.1	8.8	4.6	16.4	33.6	5.8	2.4	7.4	6.2	2.7	5.3	4.6	8.2	35.6	20.4	2.7	1.5	3.2	33.2	17.5	6.2	2.7	11.8
52	9.5	71.9	17.0	92.7	32.6	7.9	3.6	12.8	25.6	4.7	1.9	23.7	6.5	2.2	4.2	3.5	11.5	41.9	18.7	2.6	1.0	4.2	24.0	26.2	2.7	2.2	7.8

Appendix 4.32 Reservoir Inflow at the Proposed Kihoto Reservoir

Week	Year																										
	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986
1	2.2	1.2	11.8	5.4	28.8	6.3	1.0	0.8	1.5	4.1	1.3	1.0	9.1	1.1	0.7	1.0	0.9	2.9	6.3	5.3	1.5	0.5	2.1	5.3	4.7	1.2	1.3
2	1.7	0.9	9.8	5.5	10.4	6.2	0.7	0.7	1.0	2.2	1.1	0.8	6.6	1.7	0.6	0.9	1.1	6.0	3.6	3.4	0.7	0.4	2.1	2.7	2.4	1.3	1.3
3	1.4	0.8	7.1	6.8	6.3	4.2	0.7	0.6	0.9	1.9	4.0	0.8	6.3	1.6	0.7	0.6	0.8	2.2	2.1	3.3	0.6	0.4	1.2	2.1	1.7	1.4	1.4
4	1.6	0.7	6.6	7.0	4.4	4.3	0.9	0.7	0.8	2.4	6.0	1.1	2.9	3.0	0.6	0.5	0.6	2.2	9.6	3.7	0.5	0.4	1.4	1.5	1.5	1.4	1.4
5	1.7	0.7	6.7	4.7	3.9	3.6	1.2	0.7	0.9	2.8	9.0	0.8	2.4	1.3	0.6	0.5	0.7	1.7	4.3	16.5	0.6	0.4	1.6	1.5	1.7	1.8	1.7
6	1.5	0.7	4.7	2.2	3.3	3.7	1.0	0.7	0.9	3.3	4.4	0.7	3.5	0.9	0.5	0.6	0.7	1.4	2.1	63.4	0.8	0.4	0.9	1.3	1.6	1.9	1.6
7	1.3	0.8	4.9	2.5	3.0	2.1	0.9	0.5	1.5	2.7	2.0	0.5	24.8	0.9	0.4	0.5	0.8	0.9	1.3	27.6	0.6	0.3	0.6	1.5	1.4	2.0	1.6
8	1.4	0.7	5.4	1.8	2.9	1.6	1.1	0.5	4.5	6.7	1.6	0.5	8.5	2.4	0.6	0.7	1.0	1.0	1.9	16.7	1.0	0.5	1.0	1.6	1.5	2.0	1.7
9	1.0	0.5	4.1	1.5	2.7	1.0	1.2	0.3	11.9	8.7	1.2	0.4	3.7	1.6	0.4	0.4	0.8	0.9	9.4	8.5	0.6	0.6	0.6	1.5	1.3	1.6	1.4
10	0.9	0.4	3.3	1.9	2.9	1.0	1.4	0.6	34.7	8.6	1.1	0.3	4.0	0.9	0.4	0.4	0.8	1.5	6.4	4.7	0.6	0.3	0.5	0.9	1.2	1.4	1.3
11	0.7	0.3	2.9	1.7	1.9	0.9	1.5	0.5	37.0	7.5	3.1	0.3	1.7	0.5	0.4	0.4	0.5	0.8	8.6	2.3	0.9	0.2	0.4	0.7	1.1	1.3	1.3
12	2.6	0.6	4.3	2.5	2.0	1.3	1.7	0.6	9.5	5.1	1.5	0.4	1.3	0.4	0.4	0.3	0.3	0.9	8.5	5.2	0.6	0.3	0.3	0.6	0.9	1.2	1.2
13	2.5	1.0	4.2	1.3	5.1	1.0	4.0	0.4	17.7	14.3	1.8	0.4	1.1	0.4	0.5	0.4	0.3	0.9	7.0	6.9	0.3	2.9	0.3	0.6	0.9	1.3	1.1
14	2.4	1.0	3.5	1.2	3.5	1.6	2.8	1.0	15.4	7.5	5.9	0.5	1.2	0.5	2.9	0.4	0.5	1.6	10.5	2.5	0.4	8.7	0.3	0.6	0.9	2.0	1.1
15	3.0	7.2	3.1	1.2	15.0	1.0	3.3	1.7	39.5	4.7	8.1	0.5	0.9	0.4	7.1	0.4	0.4	8.2	15.3	4.1	0.6	8.1	2.5	0.6	1.0	2.5	1.2
16	2.3	8.8	5.7	7.2	41.1	1.7	11.3	5.0	18.5	5.5	6.9	1.1	0.9	0.5	7.9	1.2	3.4	37.6	11.1	10.5	1.7	10.5	2.2	0.5	1.0	3.8	1.5
17	3.9	9.0	9.2	17.6	58.8	4.3	12.6	4.4	37.5	4.9	9.5	6.3	1.0	2.5	4.9	6.0	2.0	12.9	9.3	9.2	5.3	14.5	4.2	1.3	1.3	7.0	2.4
18	4.6	8.6	12.5	26.1	20.9	8.4	10.7	4.3	46.8	3.0	11.5	11.7	0.8	1.5	4.7	3.5	1.7	8.5	9.3	7.5	1.9	6.7	7.7	7.1	1.4	3.3	7.7
19	2.4	6.8	17.2	11.2	8.1	4.7	8.1	16.2	24.2	11.0	7.1	8.3	1.5	1.2	3.3	0.8	1.4	15.1	12.5	5.6	2.9	6.2	7.6	4.3	1.0	3.1	7.6
20	1.8	2.5	10.7	12.9	7.0	2.6	2.9	14.2	10.1	10.8	6.2	6.5	2.6	1.7	1.5	0.7	1.3	12.7	7.6	9.5	7.7	13.9	5.1	2.9	1.0	3.2	3.5
21	1.3	1.3	7.3	7.7	4.8	2.7	2.1	6.5	6.8	6.8	3.5	8.3	2.1	0.9	1.4	3.2	2.2	9.1	5.4	5.6	7.9	10.0	6.0	2.0	1.0	3.8	2.5
22	1.8	2.1	4.2	24.6	3.2	1.3	1.9	5.9	5.7	4.7	2.6	3.5	1.9	3.0	1.9	3.6	5.6	4.9	3.3	5.1	4.9	5.4	8.9	2.5	0.9	4.5	2.7
23	2.1	1.8	2.5	10.4	3.3	0.8	2.0	4.9	6.1	3.0	2.4	4.9	4.1	4.0	1.8	3.1	3.8	2.4	2.1	7.1	2.0	2.9	6.3	2.5	0.9	2.4	2.2
24	1.9	1.6	1.8	5.9	4.0	0.7	3.7	2.4	5.1	2.0	1.7	3.4	1.9	3.3	1.6	1.0	1.4	2.0	1.5	4.4	1.3	2.0	3.0	2.3	1.2	2.0	2.0
25	1.3	1.1	2.6	3.3	2.5	0.7	2.5	3.5	6.1	1.6	1.8	1.7	1.6	1.3	1.1	0.7	1.0	2.3	1.3	5.1	0.9	1.6	2.0	1.9	1.0	1.7	3.3
26	0.7	1.2	2.5	2.0	2.1	1.2	1.2	2.5	5.4	1.2	1.5	1.8	4.7	0.8	0.7	1.1	1.9	1.7	1.2	3.5	0.8	1.3	2.1	1.7	1.1	1.8	7.8
27	1.2	1.2	1.5	2.1	2.2	0.9	0.8	1.5	3.3	1.1	1.3	2.1	4.5	0.8	1.7	2.6	1.5	1.7	1.1	2.4	1.9	2.1	1.7	2.3	1.1	1.5	4.4
28	1.2	0.9	1.2	1.4	2.0	0.8	0.8	1.4	3.2	1.0	2.5	2.2	1.9	1.4	4.9	0.7	2.0	4.2	1.5	2.6	1.2	1.2	1.5	1.9	1.1	1.4	2.4
29	1.0	0.8	1.0	1.1	1.6	0.8	0.9	1.8	2.3	1.0	1.2	3.9	1.4	1.0	4.4	4.0	5.7	5.2	1.2	1.7	0.7	1.3	1.1	1.6	1.0	1.6	2.2
30	1.2	0.9	0.9	1.0	1.5	0.8	1.2	6.8	2.0	1.0	1.2	3.9	1.2	1.0	1.8	5.4	4.2	2.0	1.1	1.5	0.6	2.8	0.8	2.3	1.4	2.0	2.2
31	1.0	1.3	1.6	1.0	5.4	0.6	1.8	3.0	3.4	1.1	1.2	2.4	1.5	1.0	2.3	6.8	2.8	2.5	1.0	1.6	0.5	3.4	1.0	3.2	1.8	2.5	2.4
32	0.9	4.9	1.2	0.9	8.3	0.6	0.9	6.5	2.5	0.9	1.1	2.7	1.4	1.9	2.6	5.2	2.8	3.7	2.9	1.5	0.5	4.2	1.4	2.5	1.3	2.1	1.9
33	0.9	5.3	1.0	5.2	6.5	0.5	2.6	3.8	2.0	0.9	2.2	7.1	1.2	4.1	1.9	4.3	4.7	4.7	1.8	1.6	0.6	5.1	1.8	2.2	1.5	1.9	2.1
34	1.7	2.8	0.9	5.7	4.2	0.8	3.5	4.4	2.0	0.9	3.0	12.3	2.6	1.5	1.4	7.5	2.6	2.1	1.8	1.3	0.5	5.1	2.6	3.3	1.2	1.9	3.7
35	1.4	1.6	1.0	2.4	4.2	0.6	7.1	3.2	1.7	0.7	3.7	16.7	1.2	0.9	7.5	6.0	2.6	2.0	2.1	0.9	0.5	2.1	1.5	4.9	1.1	1.8	3.5
36	1.5	2.2	0.9	3.1	3.3	0.6	6.9	2.7	1.2	0.7	1.5	7.4	0.7	1.5	7.7	4.6	8.9	2.8	1.9	0.9	0.6	2.9	1.0	3.8	1.0	1.7	2.3
37	2.5	1.6	2.7	1.1	2.5	0.5	5.6	2.2	0.9	2.1	1.9	3.9	0.6	1.1	6.0	7.6	6.6	1.9	4.5	0.9	0.6	2.4	1.2	3.3	1.1	1.8	1.8
38	1.7	1.6	1.7	0.7	6.1	0.4	1.9	1.1	0.9	4.7	1.5	2.2	0.5	0.7	2.9	4.0	2.1	1.3	2.5	0.7	0.6	1.3	0.9	3.0	1.1	1.8	2.8
39	1.3	1.3	3.0	0.7	3.9	0.5	1.3	1.1	0.8	1.3	1.2	1.9	0.5	3.3	1.6	2.6	1.2	2.7	1.9	0.7	0.3	3.1	0.9	2.0	1.1	1.8	4.8
40	1.8	1.4	1.7	0.9	5.6	0.6	1.3	0.8	0.7	0.8	1.7	3.6	0.8	5.4	1.9	6.7	1.0	1.0	5.1	1.0	0.3	4.6	0.5	1.6	0.9	1.7	2.1
41	1.4	3.2	0.9	1.0	4.9	0.4	1.5	1.2	0.7	0.6	1.2	2.3	0.6	1.5	1.3	6.5	2.3	0.7	3.1	1.2	0.3	2.5	0.6	3.5	1.0	1.6	1.8
42	0.9	15.3	4.0	2.6	7.0	1.9	1.4	2.5	0.8	0.6	3.5	1.5	1.5	2.5	1.2	5.3	0.9	0.8	3.2	1.2	0.2	1.3	0.7	4.6	2.1	2.1	1.9
43	0.9	23.5	6.2	1.7	5.0	4.8	2.3	8.0	0.9	1.4	2.1	1.4	3.8	2.4	2.5	4.0	0.7	1.5	2.3	1.0	0.4	1.0	4.8	4.3	1.4	1.8	1.6
44	2.7	64.6	5.8	3.6	7.6	4.8	5.1	12.0	3.3	0.9	2.4	2.4	4.0	1.5	2.5	4.5	0.7	4.6	4.6	1.5	0.5	4.6	4.9	3.8	1.9	2.5	2.4
45	3.2	76.9	3.9	3.7	5.4	4.4	24.3	10.1	2.8	0.9	3.4	1.9	4.7	1.9	3.3	4.2	0.7	6.1	10.4	1.7	0.6	4.8	11.4	5.3	1.4	2.0	2.0
46	2.5	93.4	2.1	6.0	7.5	4.6	18.7	6.5	4.8	2.6	4.4	2.5	6.2	10.2	7.2	2.4	1.7	7.4	7.0	8.3	3.2	3.4	8.0	5.6	2.1	2.5	1.6
47	3.1	87.3	3.2	13.4	6.2	7.3	7.1	9.4	2.4	2.9	3.6	2.2	5.8	6.9	5.0	4.1	1.0	14.3	4.8	5.6	5.5	4.8	9.3	2.7	4.2	2.6	2.3
48	1.9	99.3	9.0	10.9	5.1	5.7	5.8	31.1	31.6	7.1	2.7	4.5	5.9	4.8	4.3	5.8	1.5	19.0	7.4	3.3	2.3	5.0	8.0	2.5	2.2	2.1	2.0
49	1.5	59.1	8.6	28.9	10.4	3.5	3.1	17.3	49.6	4.7	1.5	5.1	5.0	4.1	2.3	3.4	3.1	16.6	5.8	1.7	1.6	2.4	16.0	2.1	1.8	2.0	5.8
50	1.4	32.9	5.6	46.0	8.3	1.9	2.2	5.5	25.0	6.4	1.1	2.2	2.1	2.0	1.7	3.2	1.8	8.6	9.3	1.2	0.9	2.2	12.5	2.9	1.5	1.8	5.0
51	3.6	18.7	4.5	28.4	8.0	3.1	2.1	2.9	9.1	2.2	1.0	2.7	2.0	1.5	1.5	1.9	2.9	7.2	15.1	1.1	1.2	1.6	10.3	4.7	2.3	1.8	4.9
52	4.3	14.2	2.8	54.1	10.1	1.8	1.3	2.6	5.8	1.6	0.9	11.2	2.4	1.0	1.4	1.3	10.2	14.0	13.3	1.3	0.7	3.7	9.9	5.1	1.6	1.7	3.3

Appendix 4.33 Reservoir Outflow at the Proposed Archers Post Reservoir

Week	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986
1	0.2	8.3	137.9	12.4	76.5	23.1	10.4	16.8	22.3	24.0	8.7	4.9	14.6	11.2	7.7	8.0	16.4	9.3	35.4	16.7	7.0	3.1	9.3	23.7	14.4	8.0	6.6
2	0.5	7.9	71.1	12.2	34.0	22.9	9.9	15.7	21.1	23.4	8.2	4.6	14.4	10.6	7.2	7.5	15.4	10.1	26.7	16.4	6.5	2.8	8.8	22.8	14.0	7.4	6.0
3	0.6	7.4	42.0	11.9	24.5	22.4	9.4	14.6	20.0	22.6	7.9	4.3	14.0	10.8	6.7	7.0	14.4	10.0	24.4	15.9	6.0	2.5	8.3	21.8	13.3	6.8	5.4
4	0.7	6.9	30.1	11.8	24.2	21.6	8.9	13.7	18.8	21.9	7.9	4.0	13.5	11.1	6.2	6.5	13.5	9.9	26.7	15.3	5.5	2.2	7.8	20.8	12.5	6.3	4.9
5	0.8	6.4	24.4	11.6	23.6	20.8	8.4	12.7	17.7	21.3	8.1	3.7	12.8	10.4	5.7	6.0	12.5	9.4	24.0	16.6	5.1	2.0	7.3	19.6	11.7	5.8	4.5
6	0.9	5.9	23.9	11.1	22.9	20.0	8.0	11.9	16.6	20.8	8.2	3.4	12.2	9.8	5.2	5.5	11.7	8.9	22.7	20.3	4.7	1.8	6.8	18.4	11.0	5.4	4.1
7	0.9	5.5	23.3	10.5	22.1	19.1	7.6	11.1	15.6	20.1	7.9	3.1	12.1	9.1	4.8	5.0	10.9	8.3	21.5	22.9	4.3	1.6	6.3	17.2	10.3	5.2	3.7
8	0.9	5.0	22.5	10.0	21.2	18.1	7.3	10.3	14.7	19.3	7.5	2.9	12.3	8.6	4.4	4.6	10.1	8.0	20.4	23.6	3.9	1.4	5.8	16.4	9.5	4.8	3.3
9	0.8	4.6	21.5	9.4	20.5	17.2	7.4	10.2	15.2	19.2	7.1	2.6	11.9	8.1	4.0	4.2	9.5	8.0	19.6	23.3	3.6	1.2	5.3	15.5	8.9	4.4	3.0
10	0.8	4.2	20.6	8.9	20.1	16.2	7.6	10.4	17.5	19.5	6.7	2.4	11.4	7.6	3.7	3.8	8.9	7.8	19.4	22.4	3.3	1.1	4.8	14.5	8.2	3.9	2.7
11	0.9	3.9	19.6	8.5	19.5	15.3	7.3	9.8	20.0	19.4	6.4	2.1	10.8	7.2	3.3	3.5	8.2	7.4	20.3	21.5	3.0	0.9	4.4	13.5	7.7	3.6	2.8
12	2.1	4.6	18.9	8.2	18.7	14.5	7.2	9.2	21.5	19.0	6.1	1.9	10.2	6.6	3.1	3.1	7.6	7.2	21.8	21.2	2.8	3.6	4.0	12.6	7.1	3.5	3.0
13	4.2	5.6	18.3	7.9	18.0	13.8	7.2	8.6	22.6	19.3	6.0	1.7	9.7	6.1	3.2	2.9	7.1	7.3	22.1	20.8	2.7	9.7	3.7	11.7	6.6	4.0	2.8
14	6.2	5.3	17.8	7.5	17.5	13.1	7.3	9.2	90.1	19.8	6.1	1.6	9.0	5.6	4.1	2.6	6.5	9.4	22.4	20.1	2.4	15.1	4.0	10.9	6.1	4.7	2.6
15	7.3	8.2	17.2	7.2	17.8	12.7	9.0	10.8	76.6	19.4	6.6	1.8	8.5	5.1	5.4	2.5	6.6	16.4	31.4	19.8	2.3	18.9	4.5	10.1	5.8	6.5	3.3
16	7.7	11.5	16.8	7.9	29.5	12.4	13.0	11.9	63.9	18.8	7.0	3.9	7.9	4.9	6.5	4.9	7.7	22.8	28.3	20.3	2.7	73.7	5.4	9.4	5.8	9.3	4.3
17	8.0	11.6	16.9	10.3	83.2	12.7	15.8	13.0	261.2	18.2	7.5	6.7	7.4	4.9	6.9	9.9	8.7	23.7	24.4	20.1	3.0	25.4	6.8	9.4	5.6	10.7	6.5
18	8.0	11.6	17.5	13.6	40.0	13.6	16.4	13.6	176.4	17.8	8.6	8.6	7.2	4.8	7.1	12.7	8.8	39.5	24.2	19.8	3.5	24.5	7.7	10.3	5.2	10.8	11.7
19	8.0	12.1	18.7	17.4	24.6	13.8	16.7	36.9	69.4	18.4	9.4	10.1	7.1	4.6	6.9	12.4	8.7	65.5	24.2	20.1	5.2	46.8	8.5	10.9	4.8	11.0	15.1
20	7.7	12.4	19.9	20.2	24.4	13.5	16.5	47.7	41.5	19.1	9.6	10.6	7.1	4.4	6.5	13.0	8.6	30.3	24.3	20.5	7.2	36.2	8.8	10.8	4.4	11.4	15.4
21	7.4	11.9	20.5	21.0	23.9	13.2	15.9	29.4	34.5	19.0	9.6	10.8	7.3	4.2	6.3	13.8	8.7	24.3	24.0	20.3	8.6	24.3	9.4	10.5	4.0	12.1	15.4
22	7.2	11.3	20.3	21.9	23.1	12.7	15.3	24.5	34.3	18.5	9.5	10.7	7.5	4.3	6.0	13.9	8.8	23.7	23.0	19.8	9.2	23.5	10.6	10.1	3.7	12.6	14.9
23	7.0	10.8	19.6	23.4	22.3	12.1	14.7	24.0	35.9	17.8	9.2	10.7	7.6	4.6	5.9	13.8	8.7	22.7	21.9	19.4	8.8	22.4	11.4	9.6	3.3	12.5	14.1
24	6.7	10.4	18.7	23.8	21.6	11.5	14.6	23.2	29.3	17.0	8.9	10.7	7.5	4.6	5.7	13.2	8.4	21.6	20.6	19.0	8.4	21.2	11.2	9.2	3.0	12.0	13.6
25	6.3	9.9	17.8	23.2	20.8	10.9	14.8	22.9	32.5	16.2	8.5	10.7	7.6	4.5	5.4	12.9	8.0	20.5	19.5	18.6	7.9	20.0	10.9	8.8	2.7	11.5	15.9
26	6.0	9.5	17.1	22.2	20.0	10.4	14.6	23.4	26.5	15.4	8.1	10.6	8.5	4.2	5.0	13.1	7.8	19.4	18.6	18.0	7.5	18.9	10.6	8.4	2.4	11.1	19.9
27	5.7	9.3	16.3	21.2	19.2	10.0	14.0	23.0	24.4	14.6	7.8	10.7	9.0	4.1	5.0	13.5	7.7	19.4	18.1	17.2	7.1	17.9	10.3	8.1	2.2	10.5	21.6
28	5.5	9.1	15.4	20.2	18.4	9.5	13.5	21.9	24.0	13.8	7.5	10.9	8.7	4.0	5.9	13.7	7.8	20.3	17.9	16.4	6.8	16.9	9.8	7.8	1.9	9.9	21.2
29	5.3	8.7	14.7	19.2	17.5	9.1	13.0	21.2	23.5	13.1	7.2	11.1	8.4	3.9	6.7	14.7	8.1	20.4	17.5	15.6	6.3	16.0	9.4	7.5	1.7	9.5	20.1
30	5.1	8.3	14.0	18.1	16.6	8.6	12.6	22.1	22.9	12.5	6.9	11.3	8.1	3.7	6.7	16.7	8.4	19.7	17.0	14.8	5.8	15.4	8.9	7.2	1.6	9.4	19.0
31	4.9	8.0	13.6	17.0	16.5	8.2	12.2	23.2	22.5	11.9	6.6	11.4	7.8	3.5	6.6	18.7	8.7	19.3	16.5	14.0	5.3	15.1	8.5	7.2	1.5	9.4	18.2
32	4.6	10.9	13.1	16.1	17.9	7.8	11.6	22.9	22.2	11.3	6.3	11.5	7.6	4.2	6.7	19.7	9.0	19.4	16.2	13.4	4.9	15.4	8.2	7.2	1.4	9.3	17.4
33	4.3	14.3	12.6	15.7	19.6	7.4	11.1	22.5	21.9	10.7	6.2	12.2	7.4	5.3	6.7	20.5	9.0	19.5	15.8	12.7	4.5	16.0	8.0	7.5	1.4	9.1	16.6
34	4.4	14.3	12.3	16.0	20.7	7.0	10.8	22.4	21.6	10.2	6.3	14.2	7.2	5.6	7.0	21.8	9.0	19.4	15.5	12.1	4.1	16.2	7.9	9.2	1.3	8.8	16.2
35	4.6	14.0	11.9	15.9	21.1	6.7	11.6	22.6	22.3	9.7	6.5	16.5	7.1	5.7	7.8	23.1	9.1	19.0	15.2	11.5	3.8	16.3	7.9	11.6	1.1	8.5	15.9
36	5.0	13.6	11.6	15.7	21.2	6.3	13.1	22.6	22.9	9.3	6.7	18.3	6.9	5.9	8.7	34.0	10.1	18.7	14.9	10.9	3.6	16.3	8.0	13.1	1.1	8.3	15.1
37	5.5	13.2	11.5	15.4	21.5	6.0	14.2	22.1	22.0	9.0	6.9	19.2	6.6	6.0	9.3	38.5	11.0	18.1	14.7	10.3	3.4	16.0	7.8	14.0	1.1	8.1	14.3
38	5.9	13.0	11.4	14.8	22.2	5.8	14.5	21.4	21.0	9.1	6.9	19.1	6.2	5.9	9.6	24.5	11.3	17.5	14.4	9.7	3.2	15.4	7.6	14.4	1.1	7.9	13.8
39	6.0	12.9	11.4	14.1	23.1	5.6	14.2	20.5	20.0	9.1	6.9	18.6	5.8	6.2	9.4	24.2	11.2	16.9	14.3	9.2	2.9	15.0	7.3	14.6	1.1	7.7	13.8
40	6.0	12.8	11.6	13.3	23.7	5.3	13.7	19.6	19.1	8.8	6.9	18.4	5.5	6.7	9.1	24.1	10.9	16.2	14.6	8.7	2.7	14.9	7.0	14.4	1.0	7.3	13.4
41	5.8	13.0	11.6	12.6	23.8	5.0	13.1	18.6	18.2	8.4	6.8	18.4	5.2	6.8	8.7	24.4	10.4	15.3	14.8	8.2	2.4	14.6	6.6	14.2	1.7	6.9	12.8
42	5.5	14.1	12.1	12.1	23.9	5.0	12.4	17.9	17.2	8.0	6.7	17.8	6.1	6.6	8.2	24.2	10.4	14.5	14.5	7.7	2.1	13.9	10.8	14.3	2.7	6.7	12.0
43	5.3	17.4	12.9	11.7	23.9	6.2	12.0	18.2	16.4	7.7	6.7	17.1	8.0	6.6	7.9	23.4	10.3	14.3	14.4	7.3	1.9	13.3	16.2	14.0	3.2	6.6	11.3
44	5.6	72.8	13.0	11.8	23.6	7.7	15.9	19.4	15.8	7.5	6.5	16.5	9.2	6.5	7.7	22.7	9.7	16.4	15.1	7.0	1.7	13.1	19.4	13.6	3.6	6.6	10.7
45	6.3	558.9	13.0	12.1	23.3	9.2	21.6	20.2	15.5	7.3	6.5	15.8	11.3	7.6	7.9	22.0	9.2	20.0	16.6	7.4	2.1	13.0	23.0	13.5	4.8	7.3	10.1
46	7.5	566.1	12.6	14.7	23.0	10.4	23.2	20.3	15.3	7.2	6.6	15.2	13.3	9.3	8.6	21.3	8.8	77.4	17.4	8.7	3.0	12.5	23.8	13.4	7.5	8.3	9.4
47	8.8	622.9	12.1	19.1	22.6	11.0	23.2	20.7	16.3	8.1	6.6	14.8	13.5	10.0	9.7	21.4	8.5	124.5	17.2	9.4	3.7	12.2	24.3	13.1	9.0	8.5	9.1
48	8.8	408.7	12.1	21.3	22.1	11.7	22.5	137.5	58.0	9.3	6.5	15.2	13.6	10.0	10.2	21.5	8.2	87.5	16.9	9.2	4.2	12.1	48.6	12.6	9.1	8.5	9.8
49	8.4	103.7	12.3	162.5	21.9	11.7	21.5	43.5	184.9	9.6	6.3	15.6	13.4	9.8	9.8	20.9	8.1	38.7	16.7	8.6	4.2	11.6	47.0	12.7	9.0	8.3	13.9
50	8.1	173.4	12.5	163.8	22.2	11.5	20.4	26.9	66.4	9.6	6.0	15.2	13.0	9.4	9.3	19.8	8.3	32.9	16.7	8.4	4.0	11.1	47.3	13.3	9.0	8.0	17.8
51	8.2	87.0	12.5	60.1	22.7	11.1	19.2	24.2	32.2	9.6	5.7	14.6	12.4	8.9	8.9	18.8	8.5	34.1	16.9	7.9	3.7	10.5	31.8	13.8	8.9	7.6	17.6
52	9.8	70.2	14.2	91.0	26.3	12.3	20.6	26.7	27.9	10.5	6.0	16.5	13.6	9.5	9.7	20.1	9.7	40.3	19.3	8.5	3.9	11.3	27.8	16.2	9.7	8.1	19.3

Appendix 4.34 Reservoir Outflow at the Proposed Kihoto Reservoir

Week	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986
1	0.0	1.1	18.1	4.3	16.5	7.3	2.0	3.7	8.0	11.4	2.1	1.7	4.9	2.5	2.0	2.0	2.5	3.0	9.2	8.4	1.7	1.3	2.6	8.5	3.6	1.4	1.7
2	0.1	1.1	17.6	4.5	16.7	7.0	1.7	3.0	6.5	10.8	1.9	1.6	5.5	2.2	1.7	1.8	2.1	3.3	8.8	7.5	1.6	1.3	2.4	7.5	3.5	1.4	1.6
3	0.1	1.1	16.9	4.8	16.1	6.5	1.7	2.4	5.2	10.1	1.9	1.6	5.6	2.0	1.7	1.7	1.8	3.4	7.6	6.5	1.6	1.3	2.2	6.3	3.2	1.4	1.6
4	0.1	1.1	16.1	5.2	15.3	5.9	1.6	2.0	4.2	9.5	2.5	1.6	5.4	2.0	1.6	1.6	1.7	3.1	7.1	5.8	1.6	1.2	1.9	5.3	2.8	1.4	1.6
5	0.1	1.1	15.4	5.2	14.4	5.4	1.6	1.7	3.4	9.0	3.5	1.6	4.7	2.0	1.6	1.6	1.6	2.8	7.0	6.6	1.6	1.2	1.8	4.4	2.4	1.4	1.6
6	0.2	1.1	14.6	4.8	13.6	5.0	1.6	1.6	2.8	8.0	4.1	1.5	4.2	1.7	1.6	1.6	1.6	2.4	6.1	10.3	1.5	1.2	1.7	3.6	2.2	1.4	1.6
7	0.2	1.0	13.9	4.2	12.8	4.4	1.6	1.6	2.3	6.8	3.9	1.5	6.2	1.7	1.5	1.5	1.6	2.1	5.0	12.9	1.5	1.2	1.6	3.1	2.0	1.4	1.6
8	0.2	1.0	13.2	3.7	12.1	3.8	1.6	1.6	2.4	6.2	3.3	1.5	8.5	1.7	1.5	1.5	1.6	1.8	4.2	13.6	1.5	1.1	1.6	2.7	1.8	1.4	1.6
9	0.2	1.0	12.5	3.1	11.3	3.2	1.5	1.6	3.5	6.4	2.8	1.5	7.9	1.7	1.5	1.5	1.5	1.7	4.4	13.5	1.5	1.1	1.6	2.3	1.7	1.4	1.6
10	0.2	1.0	11.9	2.7	10.7	2.6	1.5	1.5	7.4	6.8	2.4	1.4	6.9	1.6	1.5	1.5	1.5	1.6	5.1	13.0	1.4	1.1	1.6	2.0	1.7	1.4	1.6
11	0.2	1.0	11.2	2.5	10.0	2.2	1.5	1.5	10.4	7.0	2.2	1.4	5.9	1.6	1.4	1.4	1.5	1.6	5.5	12.2	1.4	1.1	1.5	1.7	1.6	1.4	1.6
12	0.3	1.0	10.6	2.3	9.4	1.9	1.5	1.5	11.4	6.8	2.2	1.4	4.9	1.6	1.4	1.4	1.5	1.6	6.1	11.6	1.4	1.1	1.5	1.7	1.6	1.3	1.6
13	0.3	1.0	10.1	2.2	9.0	1.7	1.5	1.5	11.5	7.3	2.0	1.4	4.0	1.5	1.4	1.4	1.4	1.6	6.4	11.1	1.4	1.1	1.5	1.6	1.6	1.3	1.5
14	0.3	1.0	9.6	1.9	8.2	1.7	1.6	1.4	11.8	8.0	2.3	1.3	3.3	1.5	1.4	1.4	1.4	1.6	6.8	10.7	1.4	1.1	1.5	1.6	1.6	1.3	1.5
15	0.4	1.0	9.1	1.7	8.3	1.6	1.6	1.4	12.9	7.5	3.3	1.3	2.8	1.5	1.4	1.3	1.4	1.6	8.0	10.1	1.3	1.3	1.4	1.6	1.6	1.4	1.5
16	0.4	1.1	8.6	2.2	10.0	1.6	2.0	1.5	14.1	6.9	4.1	1.3	2.3	1.5	1.6	1.3	1.4	5.6	8.9	9.8	1.3	1.4	1.5	1.5	1.5	1.4	1.5
17	0.5	1.3	8.3	4.3	12.8	1.7	4.0	1.5	15.0	6.5	4.9	1.3	1.9	1.5	1.7	1.4	1.4	9.2	9.0	9.8	1.4	1.9	1.5	1.5	1.5	1.5	1.5
18	0.6	1.4	8.7	7.9	14.8	2.4	5.6	1.6	16.9	5.9	6.1	1.5	1.7	1.5	2.1	1.4	1.4	9.3	9.0	9.7	1.4	3.4	1.6	1.6	1.5	1.5	1.6
19	0.6	1.6	9.2	9.3	14.8	3.2	6.4	2.4	18.3	6.0	6.7	1.7	1.7	1.5	2.4	1.4	1.4	9.4	9.1	9.4	1.4	4.0	1.9	1.7	1.5	1.6	1.9
20	0.6	1.6	9.5	9.5	14.2	3.3	6.1	5.0	18.2	7.0	6.6	2.6	1.7	1.5	2.4	1.4	1.4	9.7	9.1	9.3	1.5	5.2	2.7	1.8	1.5	1.6	2.5
21	0.7	1.6	9.5	9.5	13.6	3.1	5.3	6.1	17.4	7.3	6.2	3.6	1.7	1.4	2.1	1.4	1.4	9.8	8.9	9.1	1.6	6.6	3.2	1.9	1.5	1.6	2.6
22	0.7	1.6	9.2	9.9	12.9	2.8	4.5	6.0	16.6	6.9	5.4	4.0	1.7	1.5	2.0	1.5	1.5	9.6	8.1	8.8	2.0	6.8	4.1	1.9	1.5	1.7	2.5
23	0.7	1.6	8.5	10.5	12.1	2.3	3.9	5.8	15.8	6.2	4.7	4.0	1.8	1.5	1.9	1.5	1.5	9.1	6.8	8.2	2.2	6.2	4.8	1.9	1.4	1.8	2.4
24	0.7	1.6	7.1	10.3	11.5	1.9	3.6	5.3	15.0	5.3	4.1	4.0	2.0	1.5	1.8	1.5	1.5	8.1	5.7	7.6	2.0	5.3	4.7	2.0	1.4	1.9	2.3
25	0.7	1.6	6.0	9.9	10.9	1.7	3.4	4.7	14.3	4.5	3.5	3.6	1.9	1.5	1.7	1.5	1.5	6.8	4.7	6.9	1.8	4.5	4.2	2.0	1.4	1.8	2.3
26	0.7	1.6	5.2	9.3	10.2	1.7	3.0	4.3	13.6	3.8	3.1	3.2	2.1	1.5	1.7	1.5	1.5	5.6	3.9	6.3	1.7	3.8	3.6	1.9	1.4	1.7	2.9
27	0.7	1.6	4.4	8.6	9.6	1.6	2.5	3.7	12.9	3.1	2.6	2.8	2.6	1.5	1.6	1.5	1.5	4.7	3.2	5.5	1.7	3.2	3.2	1.8	1.4	1.7	3.6
28	0.7	1.6	3.7	7.1	9.0	1.6	2.1	3.2	12.2	2.6	2.4	2.6	2.7	1.5	1.7	1.5	1.5	4.3	2.7	4.8	1.6	2.9	2.8	1.8	1.4	1.7	3.5
29	0.7	1.5	3.1	5.8	7.8	1.6	1.8	2.8	11.5	2.2	2.2	2.6	2.4	1.5	2.1	1.5	1.5	4.3	2.4	4.1	1.6	2.4	2.4	1.8	1.4	1.7	3.2
30	0.7	1.5	2.5	4.7	6.4	1.6	1.7	3.0	10.8	1.9	2.0	2.9	2.1	1.5	2.3	1.5	1.6	4.1	2.1	3.5	1.6	2.3	2.0	1.7	1.4	1.7	2.9
31	0.7	1.5	2.2	3.8	5.6	1.5	1.7	3.4	10.1	1.7	1.7	2.9	1.9	1.4	2.2	1.6	1.6	3.6	1.8	3.0	1.6	2.4	1.7	1.9	1.4	1.7	2.7
32	0.7	1.5	1.9	3.1	5.8	1.5	1.6	3.6	9.6	1.7	1.7	2.7	1.7	1.4	2.1	1.9	1.7	3.4	1.7	2.6	1.5	2.6	1.7	2.0	1.4	1.7	2.5
33	0.7	1.6	1.7	3.0	6.1	1.5	1.6	3.9	9.0	1.6	1.7	3.1	1.7	1.5	2.1	2.4	1.9	3.5	1.8	2.3	1.5	3.0	1.7	2.0	1.4	1.7	2.3
34	0.7	1.6	1.7	3.5	5.8	1.5	1.7	3.8	7.8	1.6	1.7	4.4	1.7	1.5	1.9	3.1	2.2	3.4	1.7	2.1	1.5	3.4	1.7	2.1	1.4	1.7	2.4
35	0.8	1.7	1.6	3.5	5.4	1.5	2.2	3.8	6.5	1.6	1.9	6.5	1.7	1.5	2.4	3.8	2.2	3.0	1.7	1.8	1.5	3.4	1.7	2.5	1.3	1.7	2.6
36	0.8	1.6	1.6	3.3	5.0	1.4	3.1	3.5	5.3	1.6	2.0	7.6	1.6	1.5	3.4	4.1	2.9	2.8	1.7	1.7	1.4	3.1	1.6	2.8	1.3	1.7	2.6
37	0.8	1.6	1.6	3.0	4.4	1.4	3.7	3.2	4.3	1.6	1.8	7.1	1.6	1.5	4.1	4.4	3.9	2.6	1.9	1.6	1.4	2.9	1.6	2.9	1.3	1.7	2.4
38	0.8	1.6	1.6	2.4	4.3	1.4	3.7	2.8	3.4	1.6	1.7	6.2	1.6	1.4	4.1	4.6	3.9	2.3	2.2	1.6	1.4	2.6	1.6	2.9	1.3	1.6	2.3
39	0.8	1.6	1.6	2.0	4.4	1.4	3.1	2.3	2.8	1.6	1.7	5.2	1.6	1.4	3.6	4.3	3.3	2.2	2.1	1.6	1.4	2.4	1.6	2.7	1.3	1.6	2.5
40	0.8	1.6	1.6	1.7	4.4	1.3	2.7	1.9	2.3	1.6	1.7	4.6	1.5	1.5	3.1	4.2	2.8	2.0	2.3	1.6	1.3	2.7	1.6	2.4	1.3	1.6	2.6
41	0.8	1.6	1.6	1.7	4.5	1.3	2.3	1.7	1.9	1.6	1.6	4.1	1.5	1.5	2.7	4.7	2.4	1.7	2.6	1.6	1.3	2.8	1.5	2.4	1.3	1.6	2.4
42	0.8	2.6	1.6	1.7	4.7	1.3	2.0	1.7	1.7	1.5	1.7	3.6	1.5	1.5	2.3	4.9	2.2	1.7	2.7	1.5	1.3	2.5	1.5	2.7	1.3	1.6	2.2
43	0.8	6.1	1.9	1.7	4.9	1.3	1.9	2.3	1.6	1.5	1.7	3.1	1.5	1.5	2.2	4.7	1.8	1.6	2.6	1.5	1.3	2.2	1.5	3.0	1.3	1.6	2.0
44	0.9	10.5	2.6	1.7	5.2	1.4	2.2	3.9	1.6	1.5	1.7	2.7	1.6	1.5	2.2	4.6	1.7	1.7	2.7	1.5	1.2	2.2	1.6	3.1	1.3	1.6	2.0
45	0.9	14.8	3.1	2.0	5.4	1.5	4.8	5.4	1.7	1.5	1.9	2.6	1.6	1.5	2.3	4.5	1.6	2.2	3.7	1.5	1.2	2.7	2.1	3.4	1.3	1.6	2.0
46	0.9	70.2	3.0	2.5	5.5	1.5	8.3	6.0	2.0	1.5	2.2	2.4	1.8	1.6	2.8	4.2	1.6	3.1	4.7	1.6	1.2	2.9	3.6	3.8	1.3	1.6	1.9
47	1.0	86.8	2.9	4.0	5.8	1.6	9.0	6.3	2.3	1.5	2.6	2.4	2.5	2.5	3.5	3.9	1.6	4.7	4.9	1.8	1.3	3.1	4.7	3.8	1.3	1.7	1.9
48	1.0	98.7	3.5	5.7	5.7	1.8	8.7	8.6	5.3	1.6	2.6	2.5	3.2	3.2	3.7	4.1	1.6	7.2	5.1	2.2	1.3	3.5	5.5	3.5	1.4	1.7	1.9
49	1.0	58.6	4.6	8.3	6.0	2.3	7.8	10.1	10.1	1.7	2.5	3.0	3.6	3.4	3.6	4.1	1.6	9.0	5.4	2.2	1.3	3.5	6.8	3.2	1.4	1.7	2.2
50	1.0	32.3	5.1	10.8	6.7	2.4	6.6	10.2	12.1	2.3	2.2	3.1	3.5	3.3	3.2	3.9	1.6	9.3	5.8	2.0	1.3	3.2	8.4	3.0	1.4	1.7	2.9
51	1.0	19.0	5.0	12.7	7.0	2.3	5.6	9.7	12.5	2.7	1.9	2.9	3.2	2.9	2.8	3.6	1.6	9.2	7.1	1.8	1.3	2.8	8.9	3.1	1.4	1.7	3.3
52	1.2	21.3	5.3	16.7	8.3	2.6	5.3	10.4	13.8	2.8	1.9	4.1	3.3	2.8	2.7	3.5	2.5	10.5	9.6	1.9	1.5	3.1	10.2	3.9	1.6	1.9	3.8