ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

OPERATION MODELING FOR TENDAHO RESERVOIR USING HEC-ResSim

Thesis Submitted to the School of Graduate Studies in Partial Fulfillment of the Requirements for the Degree of

Master of Science

In

Civil Engineering

By

Biruk Asrat Woldeyohanis

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CERTIFICATION

I, the undersigned, certify that I read and here by recommend for acceptance by Addis Ababa University a dissertation entitled "**OPERATION MODELING FOR TENDAHO RESERVOIR USING HEC-ResSim**" in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

____

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Supervisor

____

Date

DECLARATION AND COPYRIGHT

I, Biruk Asrat Woldeyohanis, declare that this is my own original work and that it has not been presented and will not be presented to any other University for similar or any degree award.

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Above all, I want to thank God for through Him I had my well being and passed every hurdle in my study time and in my life at all.

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ABSTRACT

In this study it has been intended to develop operation modeling for Tendaho reservoir using HEC-ResSim software. The input data considered for the model are daily inflow, monthly irrigation and environmental diversion flow requirements, monthly reservoir surface evaporation and Tendaho dam and reservoir physical data.

The reservoir operation modeling has been done for two inflow source options. These are upstream and Tendaho station inflow source options. In the upstream inflow source option the ten years (1992-2001) daily flow records of Awash River at Adaitu and Mille River at Mille Bridge have been routed to the reservoir. Moreover, for the ungauged catchments downstream of the hydrological gauging stations considered, daily flows were generated using SCS-CN method. However, in the Tendaho gauging station inflow source option, the ten years daily flows of Tendaho gauging station has been used as source of inflow to the reservoir.

The SCS-CN method was calibrated for the ungauged catchments using five years (1992-1996) flow data transferred from the Logiya catchment and areal rainfalls computed from Adaitu, Mille, Bati and Dubti stations. The method was then validated using another five years (1997-2001) data. The parameter considered was the CN.

For the two inflow source options considered, operation modeling has been made for three spillway gate setting alternatives: Low level (1m), Average level (4m) and Maximum level (6.9m) gate openings. Incase of the upstream inflow option the maximum reservoir releases are 282.35m^3/s, 937.98m^3/s and 1015.40m^3/s for low level, average level and maximum level gate settings respectively. Incase of Tendaho inflow the maximum reservoir releases are 259.82m^3/s, 267.73m^3/s and 349.11m^3/s for low level, average level and maximum level gate settings respectively. The irrigation outlet releases are similar for the three gate setting alternatives in both inflow options.

The wet season reservoir operation rule curves have been developed for the minimum gate opening alternative of both the inflow options. Following the ten years wet season pool level curves, three Operation Guide Curves have been developed. These are Upper Bound Guide Curve, Lower Bound Guide Curve and Average Guide Curve.

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LIST OF ABBREVIATIONS

%	Percent
ASL	Above sea Level
AVG	Average
BM^3	Billion metric cubes
CUM	Cumulative
DEM	Digital elevation Model
DSS	Data Storage System
El	Elevation
FRL	Full Retention Level
EVA	Evaporation
GIS	Geographic information system
GMT	Greenwich Mean Time
GUI	Graphical user interface
HA	Hectares
HEC	Hydrologic Engineering Center
HEC-ResSim	Hydrologic Engineering Center Reservoir System Simulation
HFL	High Flood Level
ILRI	International Livestock Research Institute
ITCZ	Inter tropical convergent Zone
KM	Kilo meter
Km^2	Kilometer square
M	Meter
M/S	Meter per Second
M^3/S	Cubic meter per second
MM/H	Millimeter per hour
MDDL	Minimum Draw Down Level

MM^3	Million metric cubes
MMC	Million Meter Cubes
MOWR	Ministry of water resource
NMA	National meteorological Agency
PPT	Precipitation
ResSim	Reservoir Simulation model
SCS-CN	Soil Conservation Service-Curve Number
TEMP	Temperature
TMAX	Maximum temperature
TMEAN	Mean temperature
USBR	United States Bureau of Reclamation
WWDSE	Water Works Design and Supervision Enterprise

1. INTRODUCTION

1.1. General

Water resources planning and management involve the development, control, protection and beneficial use of surface and ground water resources. Consequently, it addresses a broad spectrum of applications including agricultural, industrial and municipal use with regards to pollution prevention, hydroelectric power generation, erosion and sedimentation control and the proper design of storm water drainage and flood waters control to reduce damages due to extreme precipitation events.

Mathematical modeling plays an important role in these fields of application. From the scientist's and the researcher's perspective, the role of mathematical models is to provide a better understanding of real world processes. From the water manager perspective, mathematical modeling is a way to generate quantitative information in support of decision making activities (Skoulikaris C., 2008).

During the last three decades, much progress has been made in the field of mathematical modeling applied to water resources planning and management. A diversity of software packages have been developed and play nowadays an effective role in all aspects of water management. In addition, the progress in computer technology facilitates the use of even the most complex mathematical models. Powerful computers are no longer the privilege of well endowed organizations, but can be easily obtained by individuals.

Even though the computer models provide more accurate results and additional information, they do not solve decision and implementation problems by themselves. In particular, the user is responsible for choosing among the large number of software packages. Moreover, the model selection must be made carefully because the water resources models results are often limited by the accuracy of data which is used as input for their calibration. This is an important factor which conditions the ultimate quality of the results. Consequently, a researcher's expertise is often welcomed in reviewing these results and interpreting correctly their significance and limitations.

Reservoir operation is a complex problem that involves many decision variables, multiple objectives as well as considerable risk and uncertainty (Loucks, D.P. Stedinger, J.R. and Haith, D.A., 1981). In addition, the conflicting objectives lead to significant challenges for operators when making operational decisions. Traditionally, reservoir operation is based on heuristic procedures, embracing rule curves and subjective judgments by the operator. This provides general operation strategies for reservoir release according to the current reservoir level, hydrological conditions, water demands and the time of the year. Establishing rule curves, however, do not allow a fine-tuning of the operations in response to changes in the prevailing conditions. Therefore, it would be valuable to establish an analytic and more systematic approach to reservoir operation, based not only on traditional probabilistic or stochastic analysis, but also on the information and prediction of extreme hydrologic events and advanced computational technology in order to increase the reservoir's efficiency for balancing the demands from the different uses (Long Le Ngo, 2006).

1.2. Statement of the problem

Unlike most other river basins in Ethiopia, the surface water resources of the Awash River have been highly developed with respect to irrigation. Currently, even more areas are brought under irrigation. The Tendaho Sugar plantation in the lower plain area, the Kessem project in the middle valley, expansion projects of the Wonji and Metehara sugar cane plantations and the Fentale Irrigation project owned by the regional state of Oromiya in the upper valley are the irrigation development projects in the Awash basin which are under implementation.

The increasing abstraction of water for irrigation with the decreased water impounding capacity of the Koka reservoir will cause water resource balance problems in the river basin as the amount of flow during the dry season is being minimized (Halcrow, 2005). The expansion projects of Wonji and Metehara sugar plantations, which make use of the release from the Koka dam, have an impact on the availability of water for the Tendaho reservoir. Moreover, tremendous amount of water is depleted to the atmosphere by evaporation process from the Awash basin, especially at Gedebassa swamp. As a result, the availability of water for the lower plain area of the basin during the dry season would no longer be taken for granted.

The Awash River is also known for causing serious flooding problems during the wet seasons of the year. Irrigation development in the river basin is located in the flood plains on the either side of the Awash River. It is estimated that in the Awash Valley almost all of the area delineated for irrigation development is subjected to flood (Kefyalew A., 2003). The lower plain area of the basin, where Tendaho project is located, has been frequently affected by flooding. High economic damage occurs during flooding along this river basin. The remarkable flooding problems occurred in the lower Awash River basin in 1993, 1994 and 1995 were the causes of displacement of 144,400, 154,900 and 145,700 people respectively (Shimelis Behailu, 2004).

The problems mentioned above would be partially alleviated as a result of the construction of the Tendaho reservoir. The irrigation demand for the Tendaho sugar cane plantation will be met and the reservoir can also absorb any flooding from Mille and River Awash itself. However, in order to effectively utilize the scarce water resources of the development area and to alleviate the adverse effects of the floods arriving at Tendaho, the reservoir has to be wisely operated. The wet season floods arriving at the Tendaho reservoir need to be well managed through implementation of operation rules so as to avoid the possible scarcity of water for the irrigation and ecological purposes during the dry seasons and flooding problems during the wet seasons that will occur as a result of inappropriate storages, spillages and releases.

1.3. Objectives of the study

The main objective of this research work was to establish wet season release rule for Tendaho reservoir on a daily basis operation. The specific objectives connected to the main objective were:-

- To estimate the daily residual inflows to the reservoir,
- To analyze the capability of Tendaho reservoir to meet irrigation, environmental and flood control demands,
- To analyze the flow conditions of Awash River reach downstream of Tendaho dam.

1.4. Description of the study area

1.4.1. Location and accessibility

The Awash basin is part of the Great Rift Valley in Ethiopia located from 8.5^0 N to 12^0N. It covers a total area of 110,000 km^2 of which 64,000 km^2 comprises the Western Catchment, drains to the main river or its tributaries. The remaining 46,000 km^2, most of which comprises the so called Eastern Catchment, drains into a desert area and does not contribute to the main river course.

The highest sources of the Awash lie in a mountain range lying near the southern edge of the high plateau of Ethiopia near Ginchi town ,some 80km West of the capital Addis Ababa, at an altitude of about 3000m above sea level(Halcrow,1989). The river then flows along the rift valley into the Afar triangle, and terminates in salty Lake Abe on the border with Djibouti, being an endorheic basin. The total length of the main course is some 1,200 km.

The Awash Basin covers the central and northern part of the rift valley and is bounded to the west, southeast and south by the Blue Nile, the Rift Lakes and the Wabi Shebelle Basins respectively.

The main highway from Addis Ababa to Assab traverses the entire length of the Awash Valley. The other highway runs up to Awash where it continues eastward to Dire Dawa and further across the Wabi Shebelle watershed to Harar and on to Jijiga. An alternative route linking with the Assab road at Mile runs along the western escarpment from Addis Ababa, serving the larger industrial centers of Kombolcha and Desse. Furthermore, the main railway links Addis Ababa and the port of Djibouti.

The Awash Basin has been traditionally divided into four distinct zones. These are; Upper Basin, Upper Valley, Middle Valley and Lower Valley.

Designation	*From*	*To*
Upper basin	Head waters	Koka dam
Upper Valley	Koka dam	Awash station
Middle Valley	Awash Station	Tendaho
Lower Valley	Tendaho	Lake Abe

Table 1.1.Divisions of the Awash river basin *(Source: Halcrow, 1989).*

The study area, Tendaho Dam and Reservoir site, is located in the Afar regional state, North-East of Ethiopia, some 580km from the capital Addis Ababa. It is located in the lower valley of the Awash River basin.

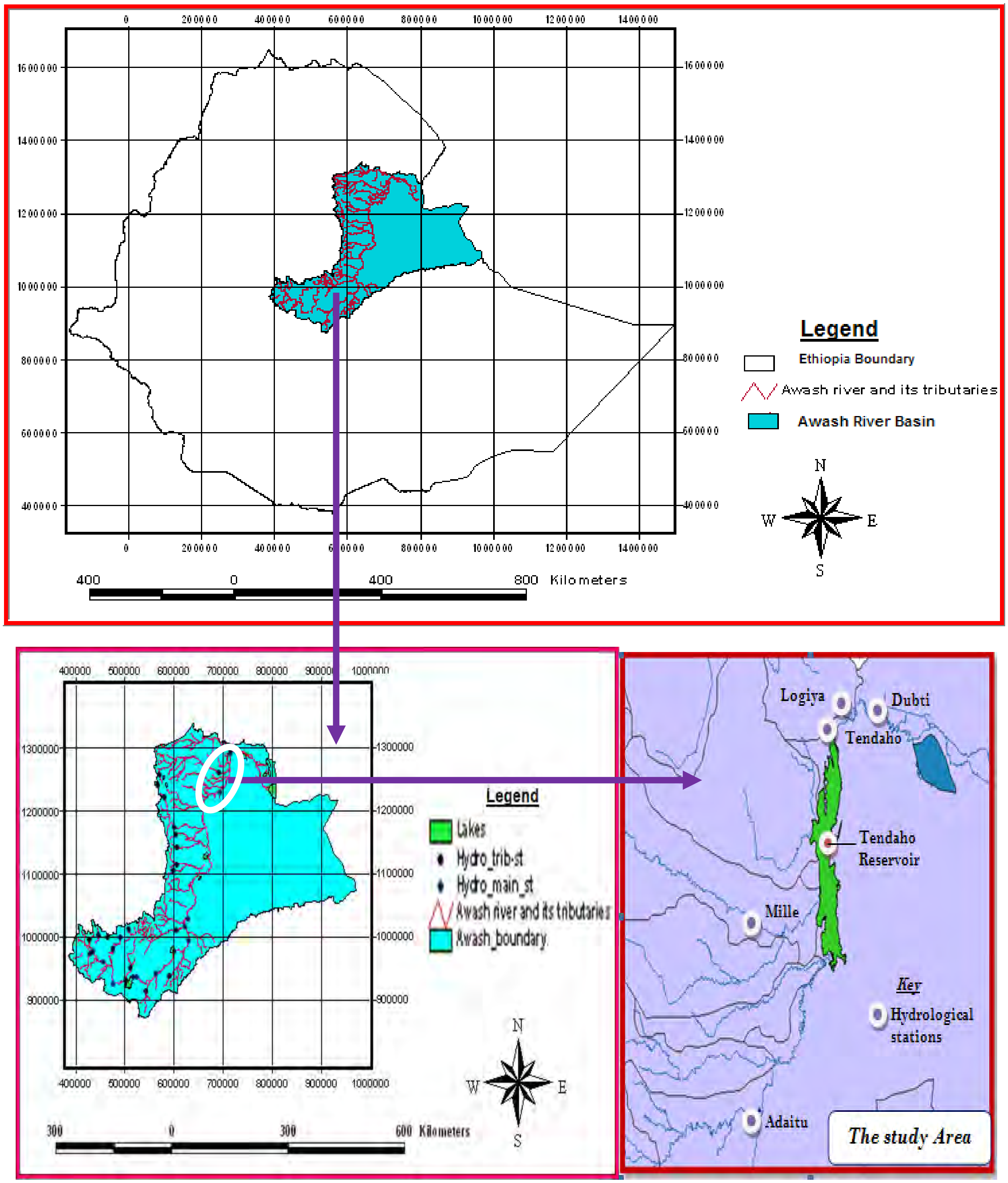

Figure 1.1 Awash River basin and the study area.

1.4.2. Physiography and Geology

The topography of the Ethiopian Plateau is generally flat with elevations ranging generally from 2,500 m down to 2,000 m. The lowest elevation of the plateau, or uplands, is commonly considered to be at 1,500m. Geologically, the plateau comprises the denuded surface of Precambrian basement rocks on which lies the near horizontal Mesozoic sediments, covered in turn by the Tertiary flood basalt extrusions. Exceptions on this flat plateau area are the deeply incised river valleys and the volcanic masses, the latter rising to over 3,000m. A series of fault scarps leads from the plateau to the floor of the Rift Valley which slopes north-east form the elevation of nearly 2,000 m at Lake Ziway to less than 400 m where it becomes the Afar Triangle. The flat floor of the Rift Valley is frequently broken by fault scarps and the effects of Pleistocene and Holocene volcanic activity. On the south-eastern side of the valley fault steps lead to the Ogaden Plateau. This slopes south east, draining towards the Indian Ocean and for that reason the Awash receives most of its tributaries from the western, Ethiopian Plateau, side.

The lower Awash Valley comprises the deltaic alluvial plains in the Tendaho, Asayita, Dit Behri area and the terminal lakes area. The Rift Valley part of the Awash River basin is seismically active. The international border region of South-Western Djibouti and North - Eastern Ethiopia also named the Afar Depression or the Afar Triangle or the Danakil desert, is a result of the separation of three tectonic plates (Arabian, Somali and African). Shared between Djibouti, Ethiopia and Eritrea, this region is an arid to semi-arid region.

1.4.3. Climate

The climate of the Awash Basin comes under the influence of the Inter-Tropical Convergence Zone (ITCZ). This zone of low pressure marks the convergence of dry tropical easterlies and the moist equatorial westerlies. The seasonal rainfall distribution within the basin results from the annual migration of the ITCZ. In March, the ITCZ advances across the basin from the south, bringing the small or spring rains. In June and July it reaches its most northerly location beyond the basin which then experiences the heavy or summer rains. It then returns southwards during August to October, restoring the drier easterly airstreams which prevails until the cycle repeats itself in March (Halcrow, 1989).

The annual rainfall distribution resulting from this cycle is exhibited most clearly in the two distinct rainy periods which are characteristic of the northern plains of the basin. Moving southwards the more prolonged exposure to the moist air-stream is evident in the tendency for the two dominant rainy periods to merge into a more contiguous distribution. On the high plateau to the west of Addis Ababa the rainfall distribution shows a continuous increase from the spring rains to the summer peak rainfall. The distribution of rainfall over the highland areas is modified by orographic effects and is significantly correlated with altitude. The mean annual rainfall varies from about 1, 600 mm at Ankober, in the highlands north east of Addis Ababa to 160 mm at Asayita on the northern limit of the Basin. Addis Ababa receives 90% of its annual rainfall during the rainy period March to September. At Dubti the same overall proportion is received during the two rainy periods, distributed 30% and 60% respectively. The mean annual rainfall over the entire Western Catchment is 850 mm and over the headwaters of the Awash, as gauged at Melka Hombole, is 1216 mm. Over the Eastern Catchment the mean annual rainfall is estimated to be 456 mm (Halcrow, 1989).

The annual and monthly rainfalls are characterized by high variability. Potential evapotranspiration (PET) is also significantly correlated with altitude. At Wonji, in the Upper Valley, the mean annual PET is 1,810 mm, over twice the mean annual rainfall, with average monthly rainfall exceeding average monthly evapotranspiration only in July and August. At Dubti, in the Lower valley, the mean annual PET is 2,348 mm which is over ten times the mean annual rainfall.

Mean annual temperatures range from 20.8 ^{0}C at Koka to 29 ^{0}C at Dubti, with the highest mean monthly temperatures at these stations occurring in June, at 23.8 ^{0}C and 33.6 ^{0}C respectively.

Mean annual wind speeds at Dubti averages 148.3 km/d, the windiest month being March with mean monthly value of 187.1km/d.

1.4.4. Surface water resources and river configuration

The Awash River rises on the high plateau to the west of Addis Ababa, at an altitude of about 2,500 m. It then flows eastwards, through the Becho Plains area and is joined by several small tributaries before entering Koka reservoir, created by a dam, commissioned in 1960. At the outlet to Koka the total catchment area is some 11,500 km^2. The Awash then descends into the Rift Valley and flows north-eastwards to Awash Station, being joined by smaller tributaries draining from the eastern section of the watershed. Beyond Awash Station the river trends northwards receiving contributions from a number of sub catchments from the western escarpments, most notably the Kessem and Kebena. The large expanse of catchment to the east of the river, accounting for some 40% of the total Basin area, does not contribute any surface runoff to the river. This area, of some 47,000 km^2 extent, known as the Eastern Catchment, loses all runoff into the vast expanse of desert plains which stretches from the escarpment northwards to the terminal lakes.

The main river channel passes adjacent to the Gedebassa swamp in a low flow channel with the main river floods passing into the swamp and partially returning, attenuated and reduced in amount, into the river channel downstream extending to Tendaho, which has a much reduced capacity. The main contributing sub-catchments in the lower part of the river are the Mile River at up stream of Tendaho and Logiya River at the down stream. There are also a number of ungauged catchments contributing heavy floods to the Awash River at upstream of Tendaho during rainy times.

After Tendaho, the Awash turns abruptly eastwards and flows in a river rich in meandering stage. The river carries considerable amount of silt and very often changes course. After flowing through the changed course, it becomes dry and another new course at another place develops. In this process several islands, like those near Boyale or Ferretie are formed. Below Asayita, the river enters the delta stage and the main course of the river splits into several streams, which join the lakes and some of them have become dry. Finally, Awash River terminates in the salt lake Abe at an elevation of 250 masl.

The drainage area at the out-let of the Tendaho Dam is some 63,485km^2 and the estimated annual flow over the reference period of 1962 to 2003 is 2334 MM^3 (WWDSE , 2005). At Tendaho some 75%-80% of the mean annual runoff arrives during the periods March to May and July to October.

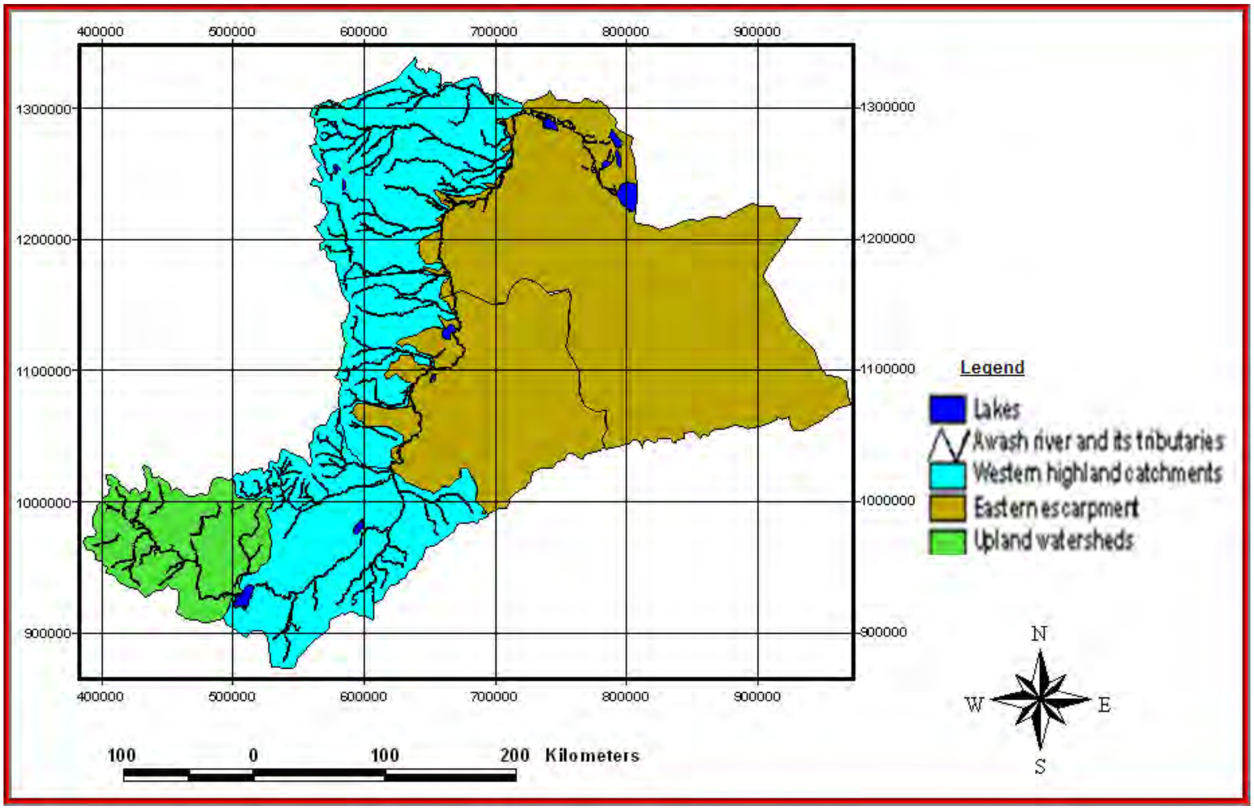

Figure 1.2. The major watershed classifications of Awash River basin.

1.4.5. Irrigation developments in the Awash River basin

Awash is one of the large river basins in Ethiopia which carry quite a big amount of water annually. The whole river basin of Awash lies within the borders of Ethiopia and therefore no international agreements regulating the use and management of the water resources. Due to its strategic location, Awash River basin is currently the most developed part of Ethiopia in terms of its irrigated agriculture. Almost 74 % of the irrigated area served by large and medium schemes is located in the Awash River basin (Dessaleng Chemeda, 2005).

Many of the large irrigation schemes in the Awash River Basin were established during the early 1960’s by local and foreign investors after the construction of Koka dam at Koka in 1960. Thousands of hectares of land have been irrigated since then. A total of 48,000 ha of land were irrigated under different schemes, out of which 13,000 ha was abandoned later on (Halcrow, 2005).

Recently, the government of Ethiopia has constructed two dams, one on the Kessem River in the middle valley and the other on the Awash River in the lower valley, to irrigate additional

80,000 hectares of land. Moreover, the Wonji and Metahara sugar plantations are attempting to irrigate 18,400 and 4,500 hectares of additional areas respectively. The Fentale Irrigation project which accounts for an area of 20,000ha is also being implemented by the regional state of Oromiya at the upper valley of the river basin.

The Tendaho irrigation project is the largest project in the Awash River basin which is being implemented with the aim to irrigate sugarcane plantation on an area of about 60,000 hectares.

Location	***Area currently under irrigation(ha)***	***Initially developed(ha)***	***Development in progress(ha)***
Upper valley	30,146	2,700	32,540
Middle valley	16,151	260	20,055
Lower valley	5,300	21,258	50,007
Total irrigation potential of the basin			***200,000 ha***

Table 1.2. Summary of irrigation potential of Awash river basin *(Source: Halcrow, 2005)*.

1.4.6. Tendaho Dam and Reservoir

The Tendaho dam has been constructed with the aim to harness the inflows of River Awash at Tendaho for irrigating sugar cane plantation in an area of about 60,000 hectares in the lower plain area of the Awash River basin. The project will help in setting up sugar factories having target production of 500,000 tones of sugar per annum, as part of development of Lower Awash Valley (WWDSE, 2005).

The engineering works for impounding rainy season inflows of river Awash and diverting the water into the network of canals mainly comprise of a storage dam across river Awash at Tendaho, a chute spillway required for passing any flood that may impinge when reservoir is at full retention level so as to avoid overtopping of the dam, and an Irrigation outlet for drawing the water from the reservoir to the distribution system of canal network.

The Tendaho dam is zoned earth fill and has embankment height of 42.5m and crest length of 421.1m. The volume of the fill material is 1.37 MM^3. The reservoir capacity is 1.86 BM^3 and the surface area of the reservoir accounts for 17,000 hectares. The fetch length of the reservoir is 37 km.

The side spillway comprises an ogee weir of 37.5m crest length surmounted by three radial gates of 10.5m width and 9m height. The design discharge over the spill way is 1,700m^3/s.

The irrigation outlet is provided with 247.36m long horse shoe type tunnel of 6m diameter with a regulated maximum discharge capacity of 78m^3/s. The out let flow is regulated by a slide gate operated by hydraulic hoist. The Irrigation Outlet is also required to function as an outlet for controlled rise of the reservoir during initial filling as well as for emergency depletion of the reservoir when need so arises.

Significant features of Tendaho dam and appurtenant works

Dam

Top of wave wall	El. 413.5
Embankment crest	El. 412.0
Crest length	412.5 m
Crest width	10.0 m
General Riverbed	El. 369.5
Deepest foundation	El. 359.0
Height above riverbed	42.5 m
Height above deepest foundation	53.0 m

Reservoir

Maximum water level	El. 410.4
Full Retention level	El. 408.0
Minimum Draw Down Level	El. 396.0
Capacity	1860 Mm^3
Surface area	17,000 ha
Fetch length	37 km
Live storage at zero year	1416 MM^3

Live storage at 25 years	1100 MM^3

Irrigation Outlet

Center Line Level	El. 391.0
Intake size	4m (W)*5m (h)
Intake capacity	$78m^3/s$

Side spillway

Crest Level	El. 400.0
Tail Water Level	El. 383.0
Design Discharge	$1700m^3/s$
Crest Length	37.5m

1.5. Outline of the study

This study presents the reservoir operation modeling of Tendaho reservoir for safe flood withdrawal through the irrigation outlet and over the side spillway during the wet season of the year. The study considers two reservoir inflow source options and a number of interrelated activities such as the analysis of contribution of the Logiya River flow to the downstream reach of Awash River and the ecological flow requirements for the end lake complexes.

The study is divided into seven sections. Following this a review of literatures pertinent to the study under consideration is presented. Section 3 presents the general descriptions of the methodology followed. In section 4, data collection from different sources and data analysis is presented. In section 5, modeling or the application of HEC- ResSim for reservoir operation is presented. In section 6, the results of the reservoir operation simulation and result analysis are presented. Section 7 comes up with a brief conclusion and recommendations of the study.

2. LITERATURE REVIEW

This section presents a general review of literatures relevant to the research work. It tries to describe some of the theoretical background of the computational methodologies employed and review of previous related studies in the project area. The major concerns are the hydrological methods, metrological methods, different reservoir operation models and HEC-ResSim model description.

2.1. Hydrometrology

A clear understanding of the hydrometrological conditions of the area is one of the basic requirements of any water management study. In this study, basic statistical approaches were used to analyze the variability of the watershed hydro metrological variables, particularly rainfall and stream flow.

2.1.1. Rainfall

The major analytic approaches carried out on the rainfall records are the following:

2.1.1.1. In-filling the daily missing records

Records of hydrologic process such as rain fall and stream flow are usually short and often have missing observations. Therefore, one of the first steps in any hydrologic data analysis is to fill in missing values and to extend short records. The gaps in rainfall records can be filled using data from the surrounding gauging station located within the basin on the assumption that the group of stations has a metrological similarity (Maidment, 2002).

There are different techniques applicable to fill the missing rainfall records. Some of the widely used statistical methods are the arithmetic mean, the normal ratio method, the regression method and the inverse distance methods.

Arithmetic mean method

This method is used when the normal annual rainfall of the missing station is within 10% of the normal annual rainfall of the surrounding stations. The surrounding index stations are necessarily required to be evenly spaced around the missing station and ideally as close as possible. The method yields a good estimate in flat area if the gauges are uniformly distributed and the individual gauges catch does not vary widely from the mean, (Garg, 1989).

The arithmetic mean is given by:

$$p_x = \frac{1}{n} * \sum_{i=1}^{n} p_i \text{ .... (3.1)}$$

Where: P_X=the station with missing record (mm),

Pi= index station (mm),

n=number of index stations.

Normal ratio method

The method is used when the normal annual precipitation of the index station differs by more than 10% of the missing station. The rainfall of the surrounding index stations are weighted by the ratio of normal annual rainfall by using the following relation, (Garg, 1989).

$$P_x = \frac{N_X}{n}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + - - - + \frac{P_n}{N_n}\right] \text{ .... (3.2)}$$

Where:

P_1, P_2, P_n =rain fall data of index stations (mm/d),

N_1, N_2, N_n =normal annual rainfall of index stations (mm),

P_X, N_x =the corresponding values of the missing station x,

n = number of stations surrounding the station x.

2.1.1.2. Data homogeneity test

In order to select the representative meteorological station for the analysis of areal rainfall, checking the homogeneity of the selected stations is important, provided that two or more stations among the group are going to be used.

The homogeneity test can be conducted using non-dimensionalized monthly rainfall records of the stations using the equation:

$$P_i = \frac{\overline{P_i}}{\overline{\overline{P}}} * 100 \text{ ………………………………(3.3)}$$

Where:-

Pi=Non dimensional Value of rainfall for the month i,

$\overline{P_i}$ =Over years averaged monthly precipitation for the station i,

$\overline{P}$ =The over years average yearly rainfall of the station under consideration.

2.1.1.3. Data consistency checking

Rainfall data reported from a station may not always be consistent. Over the period of observation of rainfall record, there could be unreported shifting of the gauge site by as much as 10km spatially and 30m in elevation, change in the observation procedure incorporated from a certain period or significant construction work ,heavy forest fire, earth quick and land slide might have taken place in that area. Such changes at any station are likely to affect the consistency of data (Patra, 2002).

The double mass curve is used to check and correct the consistency of the record. The method is based on the graphical representation. The graph is plotted taking the cumulative rainfall of the group of station as abscissa and cumulative rainfall of the affected station as ordinate. The deviation of the curve from the straight line shows inconsistency of the record. The slope of the line of the group of the station is used to correct the record (Patra, 2002).

2.1.1.4. Computation of areal rainfall

Determination of the average amount of rain which falls on a watershed during a given storm is a fundamental requirement for many hydrologic studies. Of practical necessity, rainfall is measured at a number of sample points, and the amounts recorded at these points are utilized to form an estimate of mean areal rainfall for the storm of interest.

There are many ways of determining the aerial rainfall over the catchments from the point rain gauge measurement.

The most common are:

A) Arithmetic mean method

B) Isohyetal method

C) Thieson polygon method

Arithmetic mean method

This is the simplest objective method of calculating the average rainfall over an area. The simultaneous measurements for a selected duration at all gauges are summed and the total divided by the number of gauges. The rainfall stations used in the calculation are usually those inside the catchment area, but neighboring gauges outside the boundary may be included if it is considered that the measurements are representative of the nearby parts of the catchment.

$$P_{avg} = \frac{1}{n}\sum_{i=1}^{n} p_i \quad \text{...} (3.4)$$

Where: P_{avg}= aerial rainfall (mm),

P_i = Rainfall recorded by ***n*** number of metrological gauging stations located within the basins (mm).

The arithmetic mean method gives a very satisfactory measure of areal rainfall under the following conditions:

a. The catchment area is sampled by many uniformly spaced rain gauges,
b. The area has no marked diversity in topography so that the range in altitude is small and hence variation in rainfall amounts is minimal. The arithmetic mean is readily used when short-duration rainfall events spread over the whole area under study and for monthly and annual rainfall totals.

Thieson polygon

This is also an objective method. The rainfall measurements at individual gauges are first weighted by the gauges, and then summed. On a map of the catchment with the rain gauge stations plotted, the catchment area is divided into polygon by lines that are equidistant between pairs of adjacent stations.

The areal rainfall over an area by Thieson polygon method is given by:

$$P_{avg} = \sum_{i=1}^{n} \frac{P_i * a_i}{A} \quad \text{..} (3.5)$$

Where: P_i = the point rainfall measurement of the i station (mm),

a_i= the area of the polygon (km^2),

A= the total area of the catchment (km^2).

Isohyetal method

This is considered one of the most accurate methods, but it is subjective and dependent on skilled, experienced analysts having a good knowledge of the rainfall characteristics of the region containing the catchment area.

In this method areas between the isohyets and the catchment boundary are measured with a planimeter. The areal rainfall is calculated from the product of the inter-Isohyetal areas and the corresponding mean rainfall between the isohyets divided by the total catchment area.

The relation is given by the formula:

$$P_{avg} = \frac{A_1 * \left[\frac{P_1 + P_2}{2}\right] + A_2 * \left[\frac{P_2 + P_3}{2}\right] + - - - + A_{n-1} * \left[\frac{p_{n-1} + p_n}{2}\right]}{A_1 + A_2 + A_3 ------- A_n} \quad \text{.....} (3.6)$$

Where: P_{avg} = Aerial rainfall (mm),

P_1,P_2 = Isohyets(mm) and A_1,A_2 = Area between the isohyets(km^2).

2.1.2. Stream flow

Stream flow measures the part of precipitation excess from a watershed. In water resource projects, the availability of long-term continues stream flow data is essential to study the economics or the pattern of demands from the project. However, if the data is very short and not continuous it is common practice to fill the gaps and extend the records with the reliable stream flow generating methods. The various methods used for the runoff analyses are extension of flow data, rainfall-runoff correlation and runoff simulation models.

2.1.2.1. In-filling missing flow records

It is generally observed that discharge data in many cases are found missing for some days and even for months. Attempts are made to fill-in missing links using standard methods to make a continuous record. For small gaps up to three days, arithmetic method can be applied to in-fill the missing data. The major gaps can be filled by conducting rainfall-runoff or runoff-runoff correlation between concurrent records of the hydrological stations.

2.1.2.2. Data consistency check

The consistency of flow records of a station can be checked by applying differential method over the time series data of the gauging station. The analysis considers the changes over each observation interval of the given time resolution. Abrupt changes in the differential plot are regarded as errors and require some amendments.

2.1.3. Evaporation

Estimation of evaporation from water surface is of great importance in hydrological studies and is useful in determining the operation procedure for reservoir system. Direct measurement of evaporation from large water bodies is not possible at present. However, several indirect methods have been developed that give acceptable result. Some of the common measuring methods are:

- Measurement using evaporimeters (i.e., pan evaporimeters and atmometers),
- Empirical equations,
- Water balance method,
- Energy budget method,
- Mass transfer method.

Out of all the above methods measurement using an evaporimeters are the most reliable. Evaporation records of pans are frequently used to estimate evaporation from lakes and reservoirs. Many different types of evaporation pans are in use. Among the various types, the US class A was recommended by WMO. Its performance has been studied under a range of climatic conditions within wide limit of latitude and elevation (WMO, 1994).

The other common evaporation-measuring instrument is the piche atmometer. The piche atmometer has, as an evaporating element, a 32mm diameter disk of filter paper attached to the underside of an inverted graduated cylinder tube graduated from zero to 300mm closed at one end, which supplies water to the disk. Successive measurement of the volume of water remaining in the graduated tube will give the amount lost by evaporation in any given time. Water replacement occurred whenever the water level was above 200mm.

The evaporimeters still have some inconvenience due to heat diffusion in small pan cross section compared to large water bodies, due to insertion of water droplets at the time of rainfall, due to dust accumulation on the filter paper etc. Therefore, the value obtained cannot be used with out adjustment to arrive at reliable estimate of lake or reservoir evaporation. To relate the daily piche and the pan records, and pan and the reservoir surface evaporation, different experiments and regression analysis were made. The most applicable and widely used coefficient is that Pan Evaporation rate is 0.752 times the Piche records and pan Evaporation rate is 0.7 times lake or reservoir evaporation.

2.1.4. Flow generation

Many times situation arises when the discharge observations are not available to be used in water resources developments. In these cases flow data generation is of importance. There are a number of data generating techniques applicable to generate flows from ungauged catchments. In this study, the widely used SCS-CN method is considered to generate the daily flows from the ungauged catchments. Mean annual ratio method was also used to transform flows from the Logiya catchment to the ungauged catchments to calibrate the runoff curve number.

2.1.4.1. SCS-CN method

The Soil Conservation Service Curve Number (SCS-CN) method is one of this techniques widely used to estimate flows from ungauged catchments. In this study this method was selected to estimate the flows generated from the ungauged catchments contributing flows to the main rivers downstream of the hydrological stations.

The Soil Conservation Service Curve Number (SCS-CN) method is widely used for predicting direct runoff volume for a given rainfall event. This method was originally developed by the US Department of Agriculture, Soil Conservation Service and documented in detail in the National Engineering Handbook (SCS, 1993). Due to its simplicity, it soon became one of the most popular techniques among the engineers and the practitioners, mainly for small catchment hydrology.

The main reason for its success is that it accounts for many of the factors affecting runoff generation including soil type, land use and land treatment, surface condition and antecedent moisture condition, incorporating them in a single CN parameter (K. X. Soulis, J. D. Valiantzas, N. Dercas, and P. A. Londra, 2009).

Although the SCS-CN method was originally developed in the United States and mainly for evaluation of storm runoff in small agricultural watersheds, it soon evolved well beyond its original objective and was adopted for various land uses such as urbanized and forested watersheds.

The SCS-CN method is based on the water balance equation:

$$\boldsymbol{P = I_a + F + Q} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ (1)$$

And on the fundamental assumption that the ratio of runoff to effective rainfall is the same as the ratio of actual retention to potential retention:

$$\frac{Q}{P - I_a} = \frac{F}{S} \ \ldots\ldots\ldots\ldots\ldots\ldots \textbf{.(2)}$$

Where: ***P***= the total rainfall (mm),

$\boldsymbol{I_a}$= The initial abstraction (mm),

F= the cumulative infiltration excluding $\boldsymbol{I_a}$,

Q= the direct runoff (mm) and

S= the potential maximum retention (mm).

The combination of equations (1) and (2) above yields the basic form of the SCS-CN method:

$$Q = \frac{(P - I_a)^2}{P - I_a + S}$$ Which is valid for $P \geq I_a$; otherwise Q=0.

By study of results from many small experimental watersheds an empirical relation was developed (Ven T. Chow, 1988).

$\boldsymbol{I_a = 0.2S}$

On this basis

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

The retention parameter ***S*** varies:

a. Among watersheds because soils, land use, management, and slope all vary, and
b. With time because of changes in soil water content.

Determination of Curve Numbers for the ungauged catchments

The parameter CN (runoff curve number or hydrologic soil-cover complex number) is a transformation of S, and it is used to make interpolating, averaging, and weighing operations more nearly linear (K. X. Soulis, J. D. Valiantzas, N. Dercas, and P. A. Londra, 2009).

The transformation is:

$$S = 254 * \left(\frac{100}{CN} - 1 \right) \text{ .... } \mathbf{(3)}$$

The constant, 254 in equation (3) gives S in mm.

The following section will explain how to determine factors affecting the CN.

The curve numbers used in the above formula apply for normal antecedent moisture condition (AMC II) implies average condition of moisture. For dry (AMC I) low runoff potential and wet condition (AMC III) high runoff potential, equivalent curve number has been computed by the following formulas.

$$CN(I) = \left(\frac{4.2\,CN(II)}{10 - 0.058\,CN(II)} \right)$$

and

$$CN(III) = \left(\frac{23\,CN(II)}{10 + 0.13\,CN(II)} \right)$$

The range antecedent moisture condition for each class is shown in (Table 3.1).

Curve number has been tabulated by the Soil Conservation Service on the basis of soil type and land use. Four soil groups are defined as follow:

Group A: Deep sand, deep loess, aggregated silts, high infiltration rate (>7.6mm/hr).

Group B: Shallow loess, sandy loam, moderate infiltration rate (3.8 to 7.6 mm/hr).

Group C: Clay loams, shallow sandy loam, soils low in low in organic content, and soils usually high in clay, low infiltration rate (1.3 to 3.8 mm/hr).

Group D: Soils that swell significantly when wet, heavy plastic clays, and certain saline soils, very low infiltration rate (<1.3 mm/hr).

	Total 5-days antecedent rainfall(mm)	
AMC group	Dormant season	Growing season
I	less than 12.7	less than 35.6
II	12.7 to 28.0	35.6 to 53.3
III	over 28.0	over 53.3

Table 2.1. Classification of antecedent moisture class (AMC) for SCS method of rainfall abstractions *(Source: Applied hydrology, Ven Te Chow, et.al. 1988).*

There should be understanding that the assumptions reflected in the initial abstraction term and ascertain assumption applies to their situation. The initial abstraction term which consists of interception, initial infiltration, surface depression storage, evapotranspiration, and other factors, was generalized as 0.2S based on data from agricultural watersheds. This approximation can be especially important in an urban application because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The opposite effect of greater initial loss can occur if the impervious areas have surface depressions that store some runoff. To use a relationship other than initial abstraction, Ia=0.2S one must use rainfall- runoff data to establish new S or CN relationships for each cover and hydrologic soil group.

The CN procedure is less accurate when runoff is less than 12.7mm (Ven Te Chow, 1988). Therefore, another procedure to determine runoff from the ungauged catchments was estimated based on the guide lines given in Table 3.1. The SCS runoff procedures apply only to direct surface runoff; do not overlook large sources of subsurface flow or high groundwater levels that contribute to runoff. These conditions are often related to hydrologic soil group and forest areas that have been assigned relatively low CN values (Maidment, 1993).

Daily flow data generation

SCS-CN method is widely applied to estimate peak floods from a known rainfall hyetograph together with the SCS triangular unit hydrograph. However, this method could be also applied to estimate the flow generated from a multi-day rainfalls (SCS, 1984).

2.1.4.2. Flow data transposition

When long term flow measurement data of a site on the same stream or adjoining stream is available, it can be transposed to the proposed site in proportion to the catchment areas of the two sites (IIT, 2008). Flow data can also be transferred to ungauged catchments in proportion to the mean annual rainfall of the catchments. This technique gives more acceptable estimation of flow data.

2.2. Flood routing

Flow routing is a procedure to determine the time and magnitude of flow (i.e. the flow hydrograph) at a point on a water course from known or assumed hydrographs at one or more points up stream(Chow, 1989). In a broad sense, flow routing may be considered as an analysis to trace the flow through a hydrologic system, given the input. The two types of hydrologic routing systems are lumped and distributed models. In a lumped system model, the flow is calculated as a function of time alone at a particular location, while in a distributed system routing the flow is calculated as a function of space and time throughout the system. Routing by lumped system methods is some times called hydrologic routing, and routing by distributed-system methods is sometimes referred to as hydraulic routing.

There are a number of routing techniques applicable in hydrology. The Muskingum method is a commonly used hydrologic routing method for handling a variable discharge-storage relationship. This method models the storage volume of flooding in a river channel by a combination of wedge and prism storages (Fig 2.1). During the advance of a flood wave, inflow exceeds outflow, resulting in a negative wedge. In addition, there is a prism of storage which is formed by a volume of constant cross section along the length of prismatic channel.

Assuming that the cross-sectional area of the river is proportional to the discharge at the section, the volume of prism storage is equal to KQ where K is a proportionality coefficient, and the volume of prism storage is equal to *KX(I-Q),* where *X* is a weighting factor having the range $0 \leq X \leq 0.5$. The total storage is therefore the sum of two components,

S = KQ + KX (I-Q)

Which can be rearranged to give the storage function for the Muskingum method,

S = K [XI + (1-X) Q]

And represents a linear model for routing flow in streams.

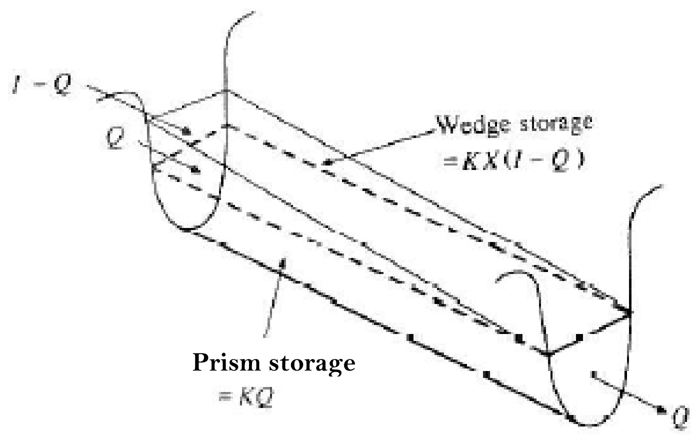

Figure 2.1. Prism and wedge storage in a stream reach.

The value of *X* depends on the shape of the model wedge storage. In natural streams, *X* is between 0 and 0.3 with a mean value near 0.2. Great accuracy in determining *X* may not be necessary because the results of the method are relatively insensitive to the value of this parameter. The parameter *K* is the time of travel of the flood wave through the channel rich. The values of *K* and *X* are assumed to be specified and constant throughout the range of flow.

The hydrological stations of sources of in-flow to the Tendaho reservoir are at remote distance from the reservoir inlet point. These stations were connected to the reservoir inflow point by routing reaches in HEC-ResSim model. Reservoir Network module of HEC-ResSim requires the routing parameters of these reaches as input data.

In this study Muskingum hydrologic routing method was selected for each reach connecting the inflow sources at remote distance to the reservoir inlet. The maximum value of 0.5 was taken for *X (*attenuation coefficient), which means direct translation of the hydrograph to the downstream points and the value of *K* (travel time of the flood wave from the out most distant point to the outlet) was approximated using the Kirpich's formula.

$$Tc = 0.00312*L^{0.77}*S^{-0.385}$$

Where:

T_c= Time of concentration (minutes),

L= length of Channel from head water to the outlet (m),

S= Average water shade slope (m/m).

The reaches length (L) and slope (S) were extracted from the topographic map produced using Arc view GIS.

2.3. Reservoir operation models

Reservoir operation is the method used to allocate water stored in the reservoir among different upstream and downstream users. It takes into account the water uses for water supply, irrigation, power generation requirement and environmental releases for downstream ecosystem, the needs of aquatic habitats and possible health impacts. These large numbers of complex and interrelated activities may create a demand conflict that requires an optimal operation rule and strong decision support system.

Numerous researchers have developed computer models for the operation of the reservoirs and river system. Currently, the vast majority of the reservoir planning and operation is undertaken using simulation and optimization models. A brief review of the two categories of reservoir operation techniques that have been, or could be, applied to dam operation is presented below.

2.3.1. Optimization techniques

Although a simulation model can accurately represent system operation and is useful for examining long-term reliability of operation that exists in coordinated system operation, optimization models are often used to systematically determine the optimal solution under specified objectives and constraints (McCartney, 2007). In all mathematical optimization techniques, the problem of reservoir operation is formulated as a problem the objective of which is to maximize or minimize a set of benefits over time, subjected to a set of constraints. The most widely used techniques are: Linear Programming, Dynamic Programming, Non-linear Programming, Optimal Control Theory, Fuzzy logic and Artificial Neural Network.

2.3.2. Simulation techniques

Despite the development and growing use of optimization models, the vast majority of reservoir planning and operation studies are based predominantly or exclusively on simulation modeling, and thus require intelligent specification of operating rules (Jay R. Lund and Joel Guzman, 1999). Simulation models represent an abstraction of reality and replicate the physical behavior of the system on a computer. The key characteristics of the system (i.e., the main system process and variability) are reproduced by mathematical or algebraic descriptions. Simulation is different from the mathematical programming techniques, which find an "optimal solution" for the system operation meeting all system constraints while maximizing or minimizing some objective (Yeh, 1985). In contrast, simulation models provide the response of the system to specified input under given conditions or constraints. Hence, the simulation model enables the analyst to test the alternatives scenarios (e.g. different operating rules) and examine the consequence before actually implementing them. The main drawback of simulation is that it requires prior specification of the system operation policy. In consequence, the only way to locate optimal policy is through subsequent trials. Many researchers have employed optimization methods within simulation models. These techniques do not result in an optimal solution but rather facilitate compliance with the predefined operating rules (Everson and Moseley, 1971; Ginn and Houck, 1981).

Simulation models for the operation of reservoirs have been applied for many years. Many models are customized for a particular system. However, more recently, the trend has been to develop general simulation models that can be applied to any basin or reservoir system (McCartney, 2007). Some of the common and the most applicable reservoir operation models are described below:

RIVERWARE

RIVERWARE is a general software tool modeling river basin operation developed at the center for advanced decision support for water and environmental system at the University of Colorado. It is optimization software that uses an object-oriented approach for modeling river basins;

different parts of the basins are modeled as at different classes of objects. Each class of the object has knowledge of its physical process, modeling features and data. The essential decision variables in this optimization model are the reservoir release at each time- period. However, the other variables are included in the model to represent constraints and objectives. The previous models were considered useful to the agencies but they had two noticeable limitations. It was dedicated to particular river bases and Policy was imbedded in the code and largely inaccessible to reservoir operators. Currently RIVERWARE has improved the modeling in terms of these limitations.

RRFM

RRFM is the Reservoir release-forecasting model that used to test different reservoir operation alternative. Developed by the Water Resource Development Act of 1999, the software can be used both for operational and planning mode. Under the deterministic operational mode the RRFM captures varies input variables, including inflow forecasts from the appropriate models. Through application of the flood control and downstream release rule, it provides forecasts of release rates and reservoir refill mechanism.

RESOP

RESOP is a Reservoir Operation Study Computer Program developed by SCG. It assists the planning, design, and evaluation of reservoirs, which must meet water supply and demand requirements. RESOP is used to compute monthly water balance for a reservoir system based on inflow, outflow and reservoir storage data. The RESOP out put contains detailed information on each of the water balance aspect for each reservoirs and year of operation such as storage and deficit at the end of the month, spill from the reservoir at the end of the month and the optimized demand computed by the program.

WEAP

WEAP is Water Evaluation and Planning model. It is a simulation model developed to evaluate planning and management issues associated with water resource development. WEAP can be applied to both municipal and agricultural systems and can address a wide range of issues including sectoral demand analysis, water conservation, water right and allocation priorities, stream flow simulations reservoir operations and project cost benefit analysis (McCartney, 2007).

HEC-5

HEC-5 is simulation of Flood Control and Conservation Systems software developed by the Hydrologic Engineering Center of US Army of Corps of Engineering. It is designed to perform a sequential reservoir operation based on a specified project demand and constraints. The simulation is performed with the specified flow data in the time interval for simulation. The simulation software determines the reservoir release at each time steps and the resulting downstream flows. It can be applied for both single and a system of reservoirs operating to reduce the downstream flooding. For the system of reservoirs on parallel streams operated for the common downstream points and tandem operation, the model effectively applied to control the release according to the constraint made (HEC, 2003).

HEC-ResSim

HEC-ResSim has been designed and developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers specifically to perform Reservoir System Simulation. It is a general simulation models that can be applied to any basin or reservoir system and designed to perform reservoir operation modeling at one or more reservoirs.

HEC-ResSim is widely used public domain simulation software, the successors to HEC-5 developed by the Hydrologic Engineering Center of the US Army of Corps of Engineers. This model was applied to simulate the Tendaho reservoir operation. The software used is version-3, September 2007 release that used to simulate reservoir operations including all characteristics of the reservoir network. The simulation module allows the users to define the alternatives and run their simulations simultaneously to compare results (D. Klipsch and Marilyn B. Hurst, 2007). The details of this model are discussed in the next sub-section.

2.4. HEC-ResSim model concepts

HEC-ResSim has been designed and developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers to perform Reservoir System Simulation. It is intended to meet the needs of real-time reservoir regulators for a decision support tool, as well as the needs of modelers doing reservoir projects studies.

As HEC's newest "Next Generation" (NexGen) software package, HEC-ResSim is intended to eventually replace HEC-5 for performing reservoir operation modeling at one or more reservoirs for a variety of operational goals and constraints. The first releases (Versions 1.0 and 2.0) of ResSim were within the water control management community of the Corps. Their reviews, experience, and feature requests have helped make ResSim a powerful tool for reservoir operations modeling.

2.4.1. Features of HEC-ResSim

HEC-ResSim has been developed by the Hydrologic Engineering Center of the US Army Corps of Engineers to aid engineers and planners performing water resources studies in predicting the behavior of reservoirs and to help reservoir operators plan releases in real-time during day-to-day and emergency operations. The following describes the major features of HEC-ResSim:

I. Graphical User Interface

Designed to follow Windows software development standards, HEC-ResSim's interface doesn't require extensive tutorials to learn to use. Familiar data entry features make model development easy, and localized "mini plots" graph the data entered in most tables so that errors can be seen and corrected quickly. A variety of default plots and reports, along with tools to create customized plots and reports and HEC-DSSVue, facilitate output analysis.

II. Map-Based Schematic

HEC-ResSim provides a realistic view of the physical river/reservoir system using a map-based schematic with a set of element drawing tools. Also, with the hierarchical outlet structure, the modeler can represent each outlet of the reservoir rather than being limited to a single composite outlet definition.

Schematic - The program's user interface allows the user to draw the network schematic either as a stick figure or an overlay on one or more geo-referenced maps of the watershed. The network of rivers and streams, called a stream alignment, is created in the watershed setup module. This stream alignment is used as a back bone on which the reservoir network schematic is developed.

Drawing Tools - HEC-ResSim represents a system of reservoirs as a network composed of four types of physical elements: junctions, routing reaches, diversions, and reservoirs. By combining these elements, the HEC-ResSim modeler is able to build a network capable of representing anything from a single reservoir on a single stream to a highly developed and interconnected system.

A reservoir is the most complex element of the reservoir network and is composed of a pool and a dam. HEC-ResSim assumes that the pool is level (i.e., it has no routing behavior) and its hydraulic behavior is completely defined by an elevation-storage-area table. The real complexity of HEC-ResSim's reservoir network begins with the dam.

Alternatives-Alternatives are developed to compare results using different model schematics (physical properties), operation sets, inflows, and/or initial conditions.

Hierarchical Outlet Structure - The dam is the root of an outlet hierarchy or "tree" which allows the user to describe the different outlets of the reservoir in as much detail as is deemed necessary. There are two basic and two advanced outlet types. The basic outlet types are controlled and uncontrolled. An uncontrolled outlet can be used to represent an outlet of the reservoir, such as an overflow spillway, that has no control structure to regulate flow. Controlled outlets can be used to represent any outlet capable of regulating flow, such as a gate or valve. The advanced outlet types are power plant and pump, both of which are controlled outlets with additional features to represent their special purposes. The power plant has the ability to compute energy production. The pump is even more specialized because its flow direction is opposite that of the other outlet types, and it can draw water up into the reservoir from the pool of another reservoir. The pump outlet type was added to enable the user to model pump-back operation in hydropower systems, although hydropower is not required for its operation.

III. Rule-Based Operations

Most reservoirs are constructed for one or more of the following purposes: flood control, power generation, navigation, water supply, recreation, and environmental quality. These purposes typically define the goals and constraints that describe the reservoir's release objectives. Other factors that may influence these objectives include: time of year, hydrologic conditions, water temperature, current pool elevation (or zone), and simultaneous operations by other reservoirs in a system. HEC-ResSim is unique among reservoir simulation models because it attempts to reproduce the decision making process that human reservoir operators must use to set releases. It uses an original rule-based description of the operational goals and constraints that reservoir operators must consider when making release decisions. As HEC-ResSim has developed, advanced features such as outlet prioritization, scripted state variables, and conditional logic have made it possible to model more complex systems and operational requirements.

IV. Module elements

HEC-ResSim offers three separate sets of functions called modules that provide access to specific types of data within a watershed. These modules are Watershed Setup, Reservoir Network, and Simulation. Each module has a unique purposes and an associated set of functions accessible through menus, toolbars, and schematic (D. Klipsch and Marilyn B. Hurst, 2007). Figure 2.2 illustrates the basic modeling features available in each module.

Watershed Setup Module

The purpose of the watershed setup module is to provide a common framework for watershed creation and definition among different modeling applications. This module is currently common to HEC-ResSim, HEC-FIA, and the CWMS CAVI.

A watershed is associated with a geographic region for which multiple models and coverages can be configured. A watershed may include all of the streams, projects (e.g., reservoirs, levees), gage locations, impact areas, time-series locations, and hydrologic and hydraulic data for a specific area. All of these details together, once configured, form a watershed framework.

When a new watershed is created, ResSim generates a directory structure for all files associated with the watershed.

In the Watershed Setup module, items that describe a watershed’s physical arrangement are assembled. Once a new watershed is created, it is possible to import maps from external sources (e.g. Arc View-GIS), specify the units of measure for viewing the watershed, add layers containing additional information about the watershed, create a common stream alignment, and configure elements.

Reservoir Network Module

The purpose of the Reservoir Network module is to isolate the development of the reservoir model from the output analysis. In the Reservoir Network module, network schematic is built, the physical and operational elements of the reservoir are described and the alternatives that are required to be analyzed are developed. Using configurations that are created in the watershed setup module as a template, the basis of a reservoir network will be created. Then, routing reaches and possibly other network elements will be added to complete the connectivity of the network schematic. Once the schematic is complete, physical and operational data for each network element are defined. Also, alternatives are created that specify the reservoir network, operation set(s), initial conditions, and assignment of pathnames (time-series mapping).

Simulation Module

The purpose of the Simulation module is to isolate output analysis from the model development process. Once the reservoir model is complete and the alternatives have been defined, the simulation module is used to configure the simulation. The computations are performed and results are viewed within the Simulation module.

When Simulation is created a simulation time window, a computation interval, and the alternatives to be analyzed must be specified. Then, ResSim creates a directory structure within the rss folder of the watershed that represents the simulation. Within this simulation tree will be a copy of the watershed, including only those files needed by the selected alternatives. Also created in the simulation is a DSS file named simulation.dss, which will ultimately contain all the DSS records that represent the input and output for the selected alternatives. Additionally, elements can be edited and saved for subsequent simulations.

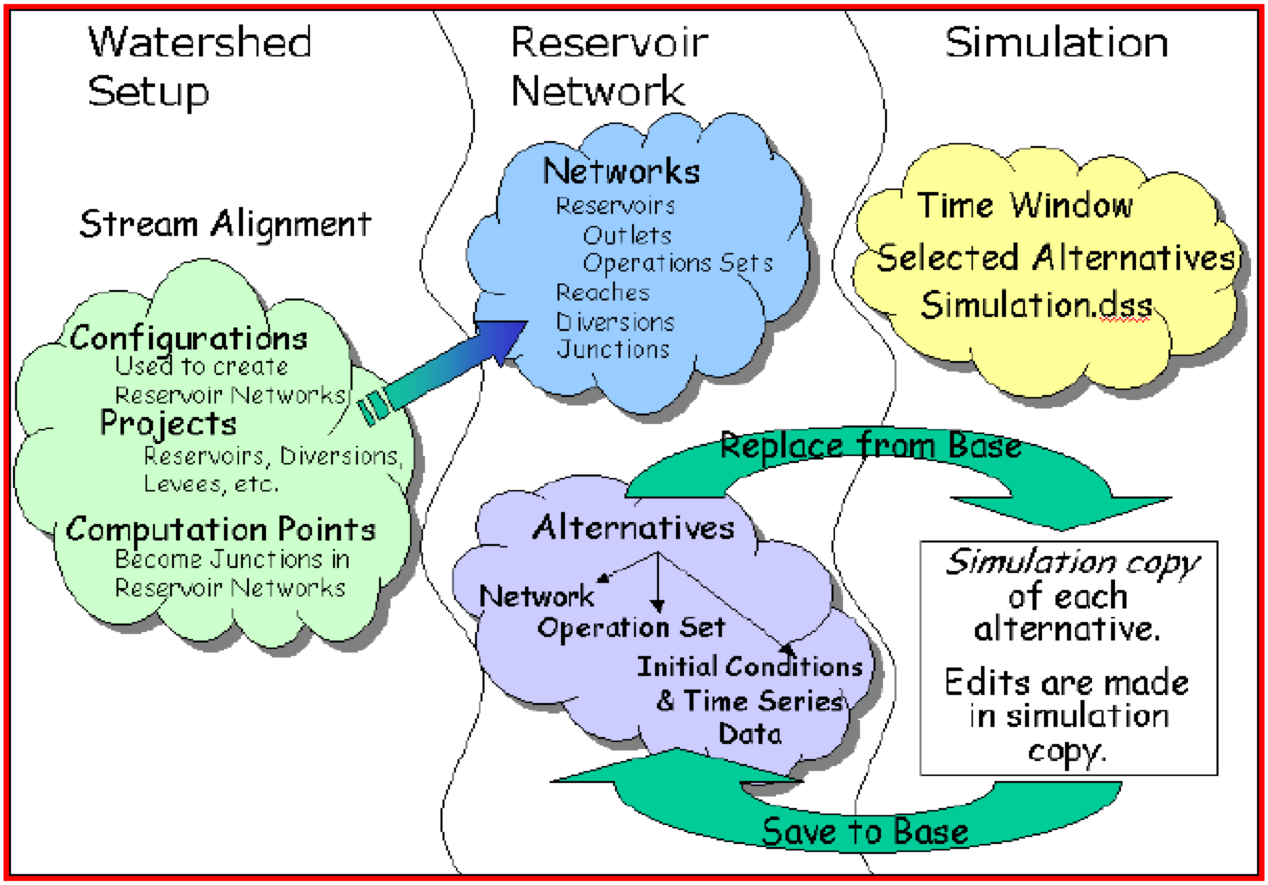

Figure 2.2. ResSim Modules concepts (Source: Joan D. Klipsch and Marilyn B. Hurst, 2007).

2.4.2. Operation rule concepts

All reservoir system simulation models require the specification of reservoir operating rules. These rules determine the release and storage decisions for each reservoir at each time-step during the simulation (Jay R. Lund, 1996). HEC-ResSim also requires defining operating rules to perform reservoir operation modeling.

The standard operating policy for a single reservoir providing water supply down stream , the most straightforward operating policy is to provide as much of the target demand as possible. If insufficient water is available to meet target demands, the reservoir is emptied to make releases as close to the target as possible. Water available in excess of the target release is stored. Water exceeding both the release target and available storage capacity is spilled downstream.

For a single reservoir whose sole purpose is to provide flood protection, the downstream flood peak is minimized by keeping a maximum amount of flood storage available, except for the capture and slow release of flood peaks. Typically, storage is released from a flood control reservoir at the rate of the downstream channel's maximum flow which incurs little or no flood damage. By releasing at or near this critical discharge level, a reservoir has greatest ability to prevent downstream flooding by freeing storage for the control of later hydrograph peaks.

When reservoirs serve multiple purposes it becomes difficult to develop simple rules to define the operating policy. Dividing the storage capacity into zones provides a mechanism to deal with the various purposes of the reservoir. Typically, each zone has a specific function and rules associated with it. Each zone may be assigned a particular function, such as flood control, water supply, hydropower, navigation and recreation. Multi-purpose zone-based operations implement a specific set of rules varying with the level of current reservoir pool level, RPL_t. This can be implemented generically:

If RPL_t is with in Zone 4, then implement Zone 4 rules

else if RPL_t within Zone 3 then implement Zone 3 rules

else if RPL_t within Zone 2 then implement Zone 2 rules

else implement Zone one rules.

The Tendaho reservoir has been planned for two main purposes: water supply for irrigation and flood control. The three main reservoir storage zones considered in the operation modeling by HEC-ResSim are Flood control, Conservation and Inactive. Operation rules can be applied to all the zones except the inactive storage zone.

2.5. HEC-DSS tool

HEC-DssVue (HEC-Data Storage System Visual Utility Engine) is a tool for transferring time series data from HEC-DSS database storage to a working space or it allows to access data stored in HEC-DSS database. The tool is comprised of two visual basic executable that utilize an object library and object classes within the database structure (DSS catalogs) and contains all relevant records and descriptors to automatically transfer the time series data during simulation process. When converting time series data in to HEC-DSS format it is important that the time series be continuous, without gaps in the dates. After converting the time series data in the HEC-DSS file format it can be used in the simulation by setting the path to DSS-path for each inflow points to the reservoir in the alternative editors.

A key step in transferring a time series data from the database storage in to the DSS format is the creation of DSS catalog in side the database. DSS catalog is the object class table within the database that contains the information related to the DSS data and its pathname. The DSS pathname consists of six parts in the following format.

A/B/C/D/E/F

Where:

- **A** - Group name for the data such as a watershed name, study name, or any identifier which allows the records to be recognized as belonging to a group.
- **B** - The location identifier for the data. The location identifier may be a site name or organization ID such as NMSA gage name.
- **C –** The parameter of the data such as **Flow, Precipitation, Storage, Evaporation, Volume, Flow-In, Flow-Out.**
- **D –** The start date of the time series.
- **E –** The time interval for regular data or the block length for irregular interval data.
- **F –** An optional descriptor that can be used for additional information about the data.

2.6. Related studies

WWDSE Detail Design Reports (2005)

As Awash River basin is the most intensively developed basin in Ethiopia in terms of irrigation, a number of studies have been conducted with respect to the development strategies to be followed to utilize the water resources of the basin. Survey of the Awash basin by Sogreah-FAO (1965), Feasibility study of the Lower Awash Basin by Gibb and Hunting (1975), Feasibility Study, Proposal and Estimate of Cotton Development on the area of 60,000 ha in the Lower Awash Valley by UZBEK (1985) and Master Plan for the Development of Surface Water Resources in the Awash Basin by Halcrow (1989) and the latest Tendaho Dam and Irrigation Project Detail Design Reports (2005) are among the most important ones. All these studies have tried to show the irrigation development opportunities attainable in the lower valley of the basin with the construction of Tendaho dam at Tendaho. They generally dealt with proposition of the type and features of the dam to be constructed at Tendaho and all share almost similar approaches, except the latest study conducted under the consultancy service of WWDSE. Thus, in this research the study conducted by WWDSE (2005), which is related to this study, is reviewed.

The Tendaho Dam Hydrology and Awash River Basin Modeling are the two most important study works which have been conducted as parts of the Tendaho Dam and Irrigation Project Design Report. The study reports included revisions of all the study works presented in earlier times. The existing situations of the Awash River basin as a whole have also been considered in the study reports.

In the Awash River basin modeling, simulation was made for the whole Awash River basin to evaluate the availability of water for the Tendaho reservoir under different level of water abstractions at the upper reaches of the Awash River. The simulation was done by HEC-5 software under four irrigation development level scenarios. The simulation result showed that except the Kessem irrigation development, all the other irrigation projects necessitating abstraction of water from the Awash River, especially Wonji and Metahara sugar plantation expansions, have an impact on the amount of water flowing to the Tendaho reservoir.

In the Tendaho Dam Hydrology Report a comprehensive reservoir simulation model was done for Tendaho. In the study, as reservoir operation study requires a long-term series data, Thomas-Fiering synthetic monthly flow generation model was used to generate long-term flow data using the monthly flow data at the Tendaho dam site over the period 1982 to 2003 as a base data.

A 10-day water balance model was constructed to estimate the probability of meeting the 10-day gross irrigation demand and a constant environmental release of $5m^3/s$ as well as losses through evaporation from the reservoir. As a first approximation, reservoir seepage was ignored. The simulation was run for three stages of the reservoir life:

1. Year 0, initial condition,
2. Year-25, half design life of the reservoir and
3. Year-50, at the end of the reservoir.

The simulation was done under two irrigation development scenarios:-Irrigating 60,000 hectares and irrigating 48,000 hectares.

Based on the storage available and the release reduction for the whole 10-days, reduction in irrigated area would be planned or series of water saving mechanisms should be used if the whole 60,000 hectare area is essentially required.

The irrigation project successes frequency and the flow to the lakes down stream of the reservoir were also simulated. The 10-days reservoir simulation was done for release reduction up to 50 percent at the three life stages of the reservoir: initial, half life and end of life. The simulations were done for different reservoir levels under the two development scenarios.

The study showed the number of years with spill out of fifty years operation for different reservoir conditions and at different reservoir life stages. The environmental demands for the lake complexes down stream of the Tendaho dam have been also quantified. The effect of the unavoidable sedimentation on the reservoir capacity and in turn on the irrigation demand has been considered to simulate the reservoir operation.

In the hydrology report of the Tendaho reservoir, a part of the Tendaho Dam design report, great works have been done with the aim to achieve two main goals: Reservoir sizing and ensuring availability of water for Tendaho reservoir in the coming fifty years under different hydrological and physical conditions from the time the reservoir is left open for use. In the study, a 10-day basis reservoir simulation has been done with a 50 years synthetic monthly flow data. However, due to considerable progress and invention of new reservoir operation models, still with the advancement of the knowledge and needs of reservoir operation decision support tools, it is required to develop a more reliable operation policy to alleviate the hydrologic impacts most likely to occur on the Tendaho dam as well as the development areas down stream.

3. METHODOLOGY

As it was indicated in the first chapter, the objective of this research was to establish wet season operation rule for Tendaho reservoir. The methodology adopted in this study to simulate operation of the reservoir was HEC-ResSim model. Other methods were also used to prepare the model input data. To estimate the flows from the ungauged catchments SCS-CN was used. The procedures followed are discussed and the schematic representation is given in Figure 3.1.

Generally the study involves the following procedures:

- Collection of relevant data such as:
  - Metrological data of the selected stations(daily rainfall and monthly data of other metrological elements),
  - Hydrological data(daily flow records) of the key hydrological stations,
  - Soil and land use land cover data of the ungauged catchments,
  - Tendaho dam physical data,
  - Irrigation and ecological flow demand data,
- Rainfall data analysis:
  - In-filling missing records,
  - Data homogeneity test,
  - Data consistency checking,
  - Computing areal rainfall over the ungauged catchments and Tendaho reservoir,
- Flow data analysis:
  - In-filling missing records,
  - Data consistency checking,

- Preparing soil and land use land cover data of the ungauged catchments,
- Generating daily flows from the ungauged catchments by SCS-CN method,
- Transposing flow data to ungauged catchments from Logiya catchment in proportion to the mean annual rainfall,
- Calibration and validation of SCS-CN method,
- Monthly irrigation and ecological flow demand computation,
- Tendaho dam and appurtenant structures data analysis,
- Estimation of the reservoir evaporation using the pan evaporation data available,
- Computing routing parameters for river reaches carrying flows from remote distances,
- HEC-ResSim model setup,
- Adding data to HEC-ResSim model:
    - Adding reservoir physical and operational data through the model's window editor,
    - Adding the daily inflow data through HEC-DSS tool,
- Reservoir operation simulation process for the ten years data and different levels of spillway gate setting alternatives,
- Developing wet season operation guide curves based on the ten years HEC-ResSim model computation result,
- Developing wet season guide curves using the ten years flow records of Tendaho hydrological station for comparison purpose.

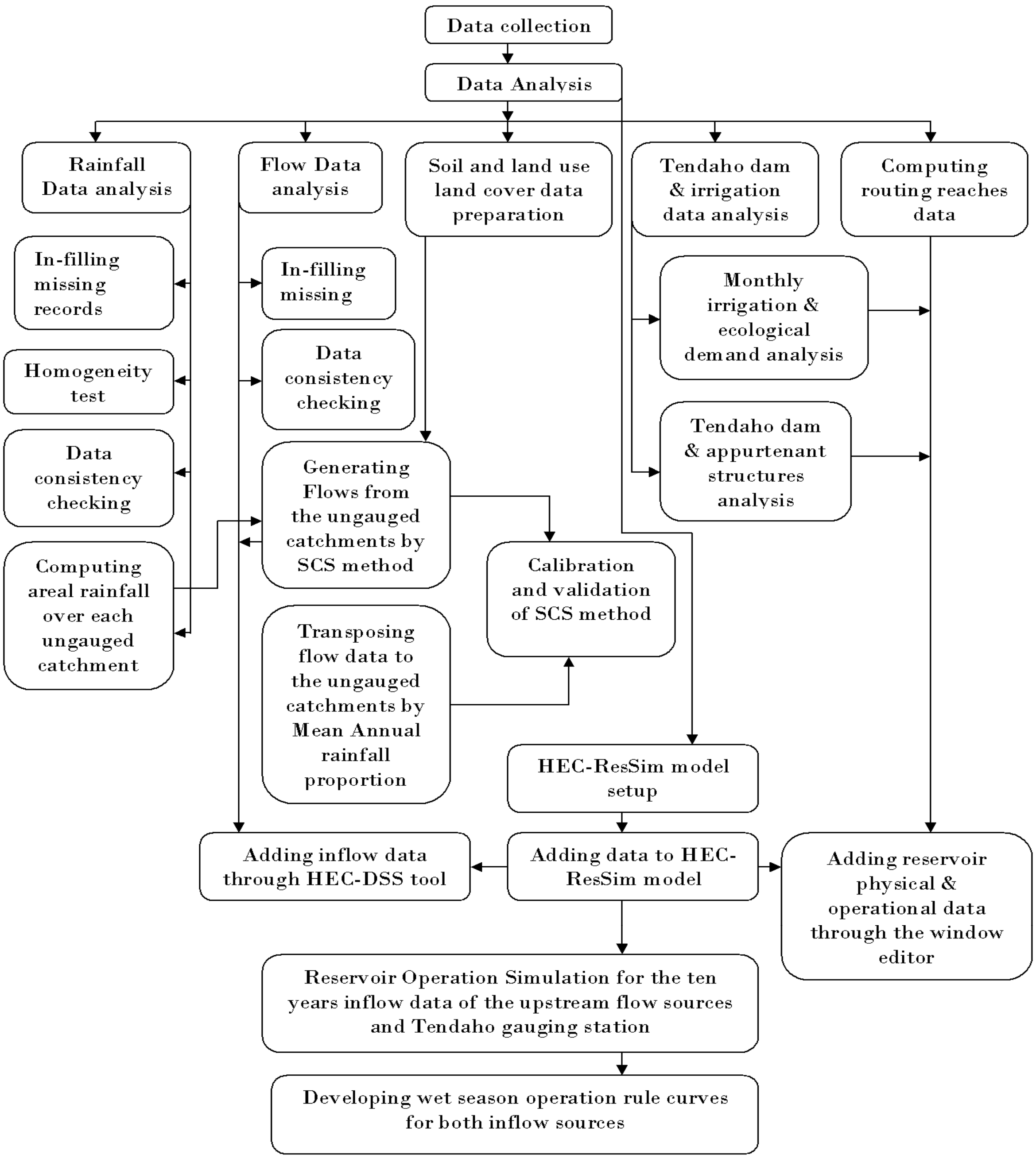

Figure 3.1. Schematic representation of the methodologies followed.

4. DATA COLLECTION AND ANALYSIS

4.1. Data availability

4.1.1. Hydrological Data

The most important input data for reservoir simulation by HEC-ResSim software is the reservoir inflow data. To estimate the inflow to the Tendaho reservoir, the immediate upstream hydrological stations were considered as they are the main sources. These two hydrological stations (Mille and Adaitu) represent the flow of Mille River and residual flow of Awash River respectively. The hydrological data of the Tendaho station were also collected to in-fill the missing records of Adaitu hydrological station and to be used as input data to the model to simulate operation of Tendaho reservoir for comparison with the simulation made using the upstream inflow sources. The daily flow records of Logiya station were also collected to analyze the flow conditions downstream of Tendaho dam. The daily flow data of these stations were collected from Ministry of Water Resources, Department of Hydrology.

The daily flow data obtained at Mille station was a record from 1976 to 2003 and that of Adaitu was from 1981 to 2001. The flow data of the Tendaho hydrological station recorded from 1969 to 2003 and that of Logia started in 1977 and extended through 2001.

Though long years daily flow records of the selected stations could be obtained, only a ten-years record(1992-2001) were considered for analysis as these duration were observed to have relatively less missing records.

For the ungauged catchments contributing flows to the main rivers downstream of the gauging stations, a rainfall-runoff modeling was applied to generate daily flow data from the areal rainfall data over the catchments. The reservoir inflow sources are schematically represented as in Figure 4.1.

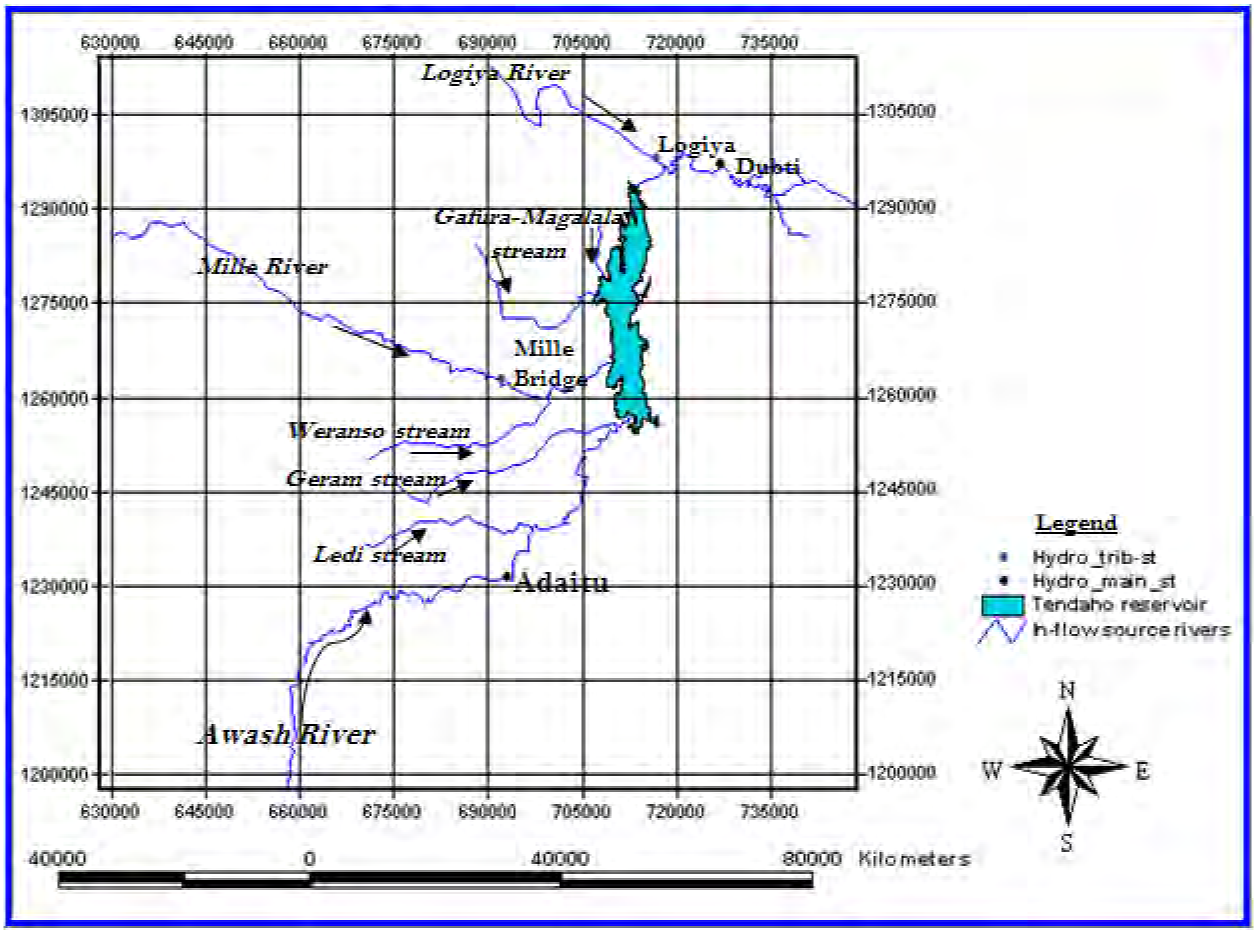

Figure 4.1. Tendaho Reservoir inflow Source Rivers.

4.1.2. Metrological data

The metrological data required in this study are the daily rainfall data and monthly mean evaporation data of the selected metrological stations. Rainfall data were collected from the selected stations (Adaitu, Bati, Mille and Dubti) to compute direct run-off over the reservoir and the run-off generated from the unguaged catchments contributing flow downstream of the hydrological gauged points. The daily rainfall data of the selected metrological stations that could be obtained from the National Metrological Agency were only of ten years records (1992-2001).

The other metrological data (sunshine hour, humidity, wind speed and mean temperature) were directly taken from the hydrology report of Tendaho Dam and Irrigation project to estimate the evaporation rate from the reservoir surface.

4.1.3. Tendaho Dam and Irrigation Project data

The other basic data to be used as inputs to HEC-ResSim model are monthly irrigation and environmental flow demand data, Tendaho reservoir capacity curve data, Tendaho dam appurtenant structures data, routing reaches data and downstream river reach flow data. Some of these data were obtained from the Tendaho Dam and Irrigation Project Irrigation and Drainage system design report and the others were computed.

4.2. Data analysis

4.2.1. Rainfall data analysis

Among the stations considered as rainfall data sources, Adaitu, Bati, Dubti and Mille were found to be relatively appropriate stations. The ten years daily time step rainfall data collected from these metrological stations were analyzed for missing records, data quality and consistency. The data analyses conducted are discussed in this section.

4.2.1.1. In-filling the daily missing records

Almost all the metrological stations considered have missing daily records. As the normal annual rainfall of all the stations considered differed by more than10% of the missing station, the normal ratio method was used to fill the missing rainfall data of the selected stations.

4.2.1.2. Data homogeneity test

In this study homogeneity test for two groups of gauging stations were conducted. The two groups of gauging stations considered are: Adaitu-Dubti-Mille and Adaitu-Bati-Mille. The first group of gauging station was considered to compute the areal rainfall on the reservoir and ungauged catchment contributing flow directly to the reservoir. The other group was analyzed for the computation of areal rainfall on the ungauged catchments contributing flow to Mille River and Awash River downstream of the hydrological gauging stations.

The homogeneity test was conducted using non-dimensionalized monthly rainfall records of the selected stations and the graphical output is presented as in Figures 4.2 and 4.3.

As the plots of the two groups of stations show bimodal pattern of the monthly rain fall of the stations, the selected stations in both groups were homogeneous.

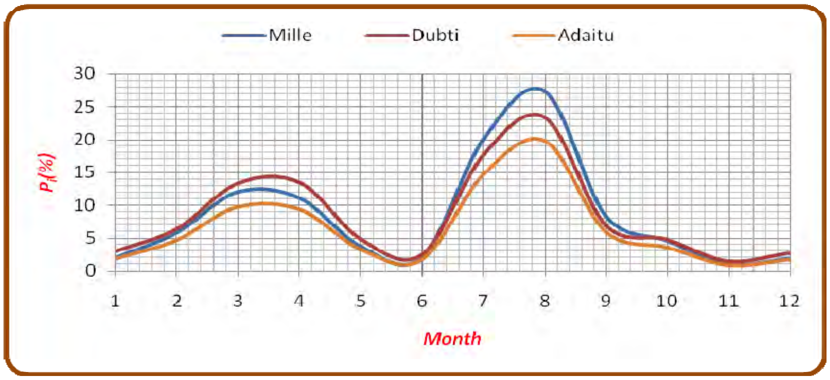

Figure 4.2. Homogeneity test for Adaitu-Dubti-Mille stations.

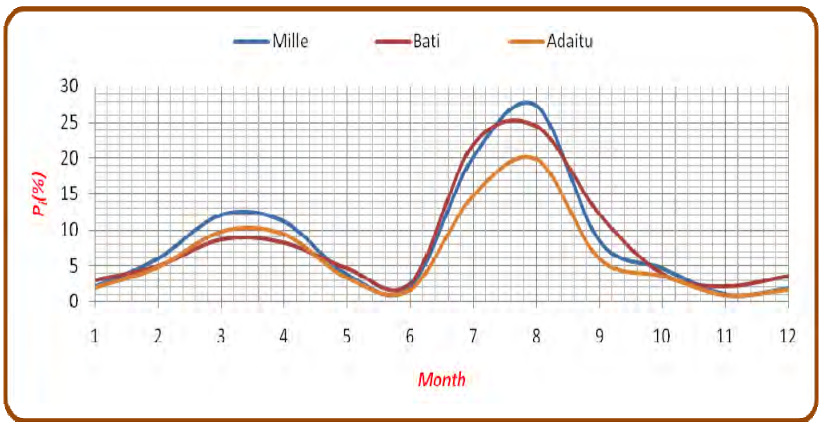

Figure 4.3. Homogeneity test for Adaitu-Bati-Mille stations.

4.2.1.3. Data consistency checking

In this research Double mass curve analyses were made between two groups of gauging stations categorized based on the homogeneity test previously made: Adaitu-Dubti-Mille and Adaitu-Bati-Mille. The cumulative mean annual rainfall of the two gauging stations, i.e., all stations except the one station to be analyzed, and the cumulative annual rainfall of the same station was evaluated. The deviation of the curve from the straight line was considered as an error and the proper adjustment was made. The annual correction factor was distributed to all the daily records by the ratio of the previous to the adjusted values.

The double mass analysis results made at Adaitu metrological station (Figures 4.4 and 4.5) are presented for clarity. The results of the other stations are presented in Appendix-1 and the mean monthly rainfall of the stations after correction is given in Appendix-2. The cumulative mean annual rainfall of the two gauging stations, i.e., all stations except the Adaitu station, and the cumulative annual rainfall of Adaitu station were evaluated.

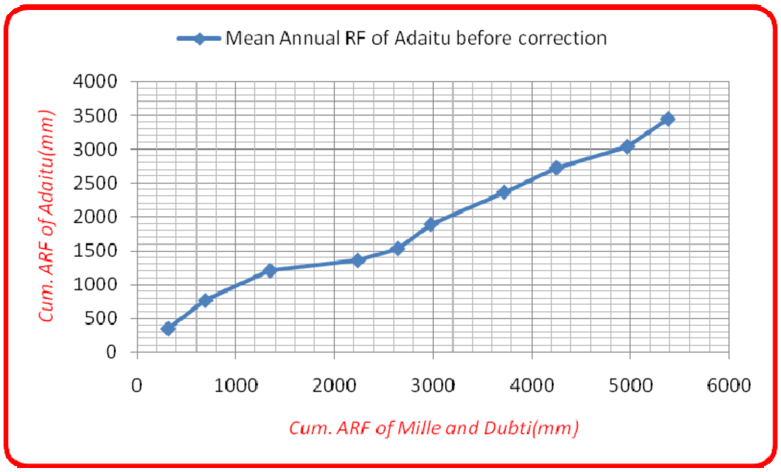

Figure 4.4. Adaitu Mean Annual RF before double mass analysis.

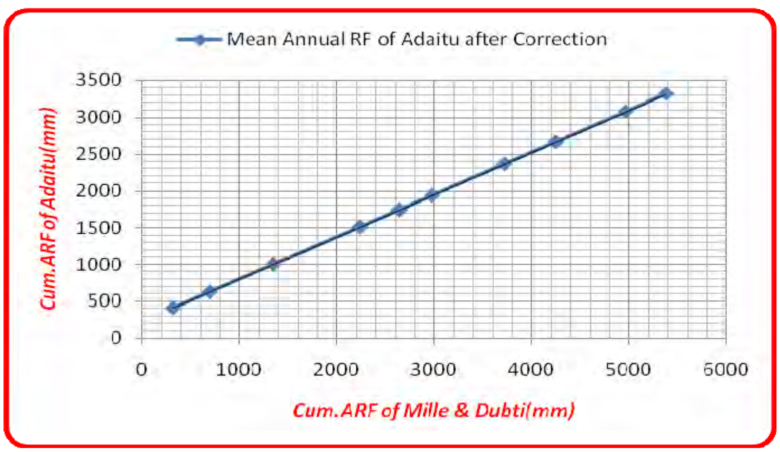

Figure 4.5. Adaitu Mean Annual RF after double mass analysis.

4.2.1.4. Determination of areal rainfall

In this study, two groups of only three rainfall stations located around the catchments could be arranged for the two ungauged catchment groups. The locations and the number of the selected stations were such that, defined Thieson polygons could not be created for each ungauged catchment. Thus, this method could not be adopted to estimate the areal rainfall on the ungauged catchments considered.

Isohyetal method could not be also applied in this study to compute the daily areal rainfall as the Isohyetal map obtained was of isohyets of mean annual rainfall of the Awash River basin.

As the two methods discussed above could not be applied for the respective reasons mentioned above, simple arithmetic method was adopted as a last resort to compute the areal rainfall over the ungauged catchments.

Depending on the locations of the ungauged catchments, two groups of rainfall stations were considered for the two groups of the catchments. For the ungauged catchments terminating near the inlet of the reservoir (Figure 4.6), Adaitu-Bati-Mlle station group was considered. The catchments treated under the influence of Adaitu-Bati-Mille station group are Ledi, Geram, Weranso and the intervening catchment downstream of Mille hydrological station. The other rainfall station group considered to calculate the areal rainfall over the catchment contributing floods directly to the reservoir near its middle is Adaitu-Dubti-Mille. In addition to the ungauged catchment, Gafura-Magalala, this station group was also used to compute the direct runoff on the reservoir.

The ungauged catchment contributing flow to Mille River is Weranso and of 1062Km^2. The intervening catchment has an area of 186Km^2. The two ungauged catchments contributing flow to Awash River downstream of Adaitu station are Ledi and Geram sub catchments with catchment areas of 1083km^2 and 278km^2 respectively. The catchment contributing flows directly to the reservoir (Gafura-Magalala) has got an area of 1083 km^2.

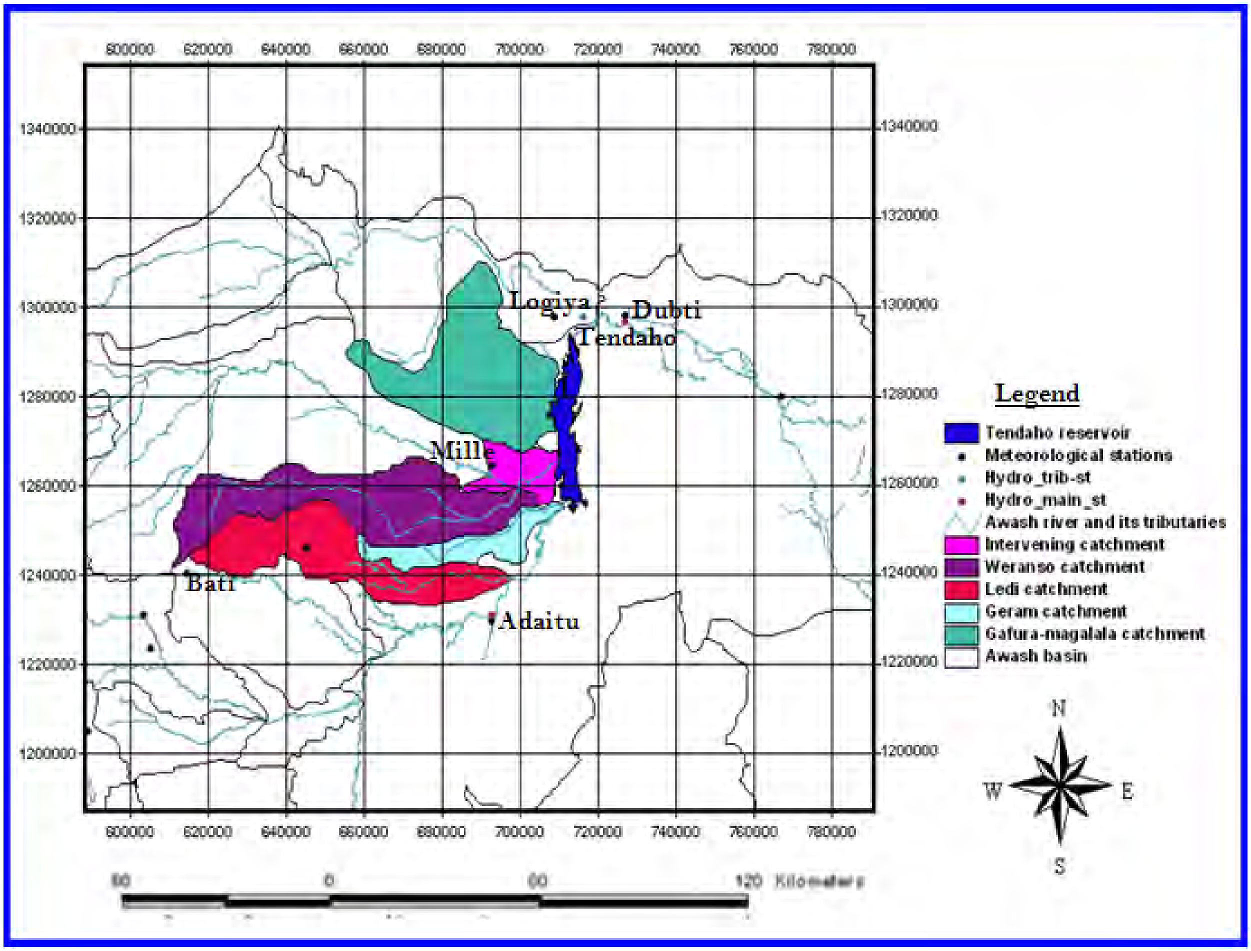

Figure 4.6. The ungauged catchments and locations of the key metrological stations.

4.2.2. Hydrological data analysis

In-filling missing flow records

In the daily flow records of the key hydrological stations considered in this study, there were a number of record gaps that required filling. The small gaps up to three days were filled by arithmetic mean methods. The major gap filling analyses were made by conducting correlation between the hydrological stations. Some missing records for which correlation could not give a solution, were filled by computing the average values of the over years flows of the same days.

Correlation between Adaitu and Mille gave poor result($r<0.4$). Hence, missing flow records of the Adaitu station were filled by linear correlation of Tendaho up on Adaitu station using the daily flow records of the years 2000 and 2001, the only complete concurrent data available. The coefficient of determination (R^2) obtained was *0.58* and correlation coefficient (r) of *0.61*. The same relation was also used to make the daily flow records of Tendaho hydrological station complete. As the missing flow records of Mille station was of short duration, up to maximum of three days, simple arithmetic mean method was used to in-fill the gaps. The daily flow records of Logiya station are complete for the duration considered.

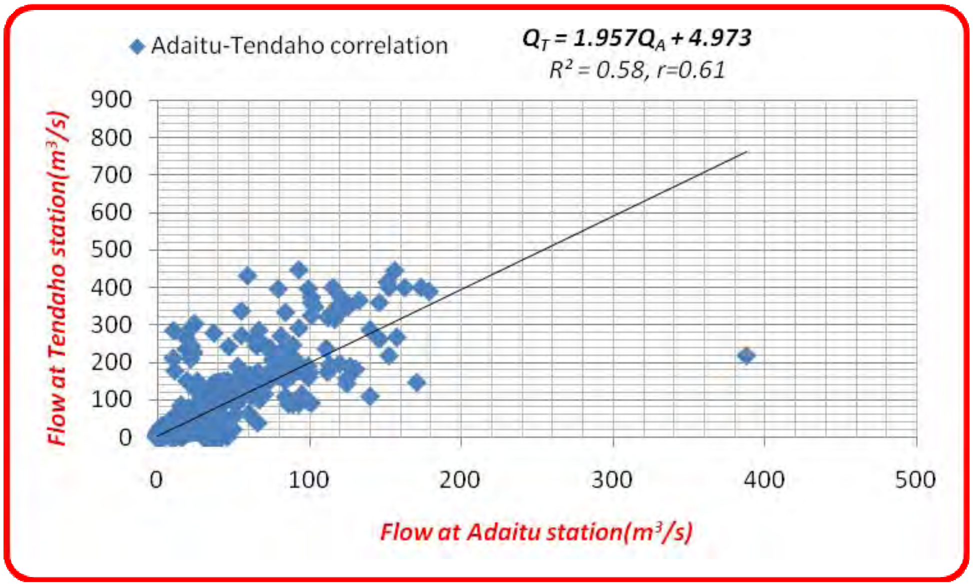

Figure 4.7. Correlation between Adaitu and Tendaho gauging stations.

The daily flows of Adaitu station for the months November and December of years 2000 and 2001 were totally missing. As the flow records of Tendaho for the same duration were similarily missing, it was not possible to apply correlation. Hence, average records of historical flows corresponding to the days with the missing values were used to make the time series data of both Adaitu and Tendaho hydrological stations complete.

Data consistency check

The differential method was applied over the time series data of the Mille, Adaitu and Tendaho hydrological stations. Some daily records of the Mille station were very large values much more than the normal maximum increment and hence correction was made to reduce those values in the acceptable limits.

4.2.3. Soil and land use land cover data

The soil data for the ungauged catchments was generated from a soil map produced by Arc View GIS (Figure 4.8) using the shape files of soils of the whole Awash River basin obtained from Ministry of Water Resources, department of GIS and Remote Sensing. From the map produced, the soil units predominantly existing on the ungauged catchments considered were collected. As the map produced presents only the soil unit names, a research report of soil research stations in the Awash River basin and the FAO soil data base from Global Soil Data Products available in the thesis entitled Stream flow Simulation for the Upper Awash Basin (Shimelis Behailu, 2004) was used to relate the soil unit and the corresponding soil property. Among the soil properties, hydrologic group of the soil was used as an input to determine the curve number for SCS method of rainfall-runoff analysis.

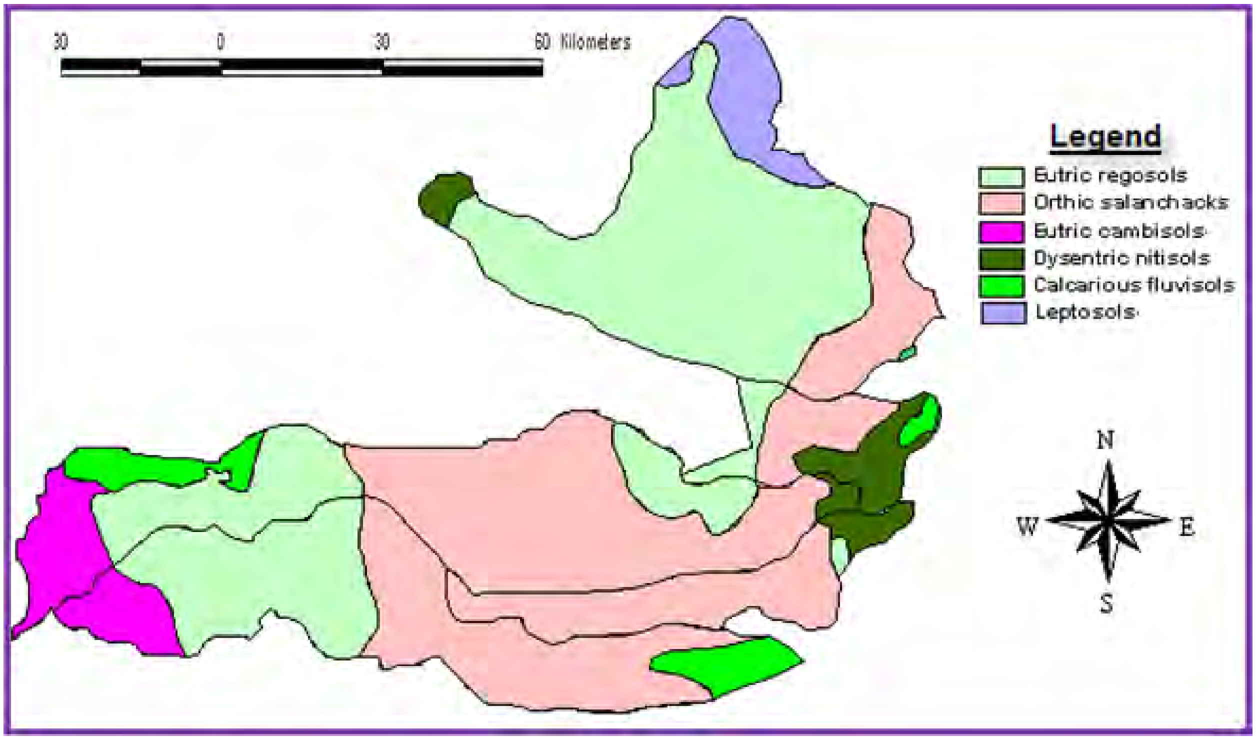

Figure 4.8. Soil map of the ungauged catchments.

The land use land cover map of Awash River basin was also produced by Arc View GIS using the land use land cover shape files obtained from Ministry of Water Resources. The land use land cover data of the ungauged catchments considered (Figure 4.9) was collected from the map developed. The land use data and the corresponding hydrologic soil group of the ungauged catchments were used to estimate the curve number.

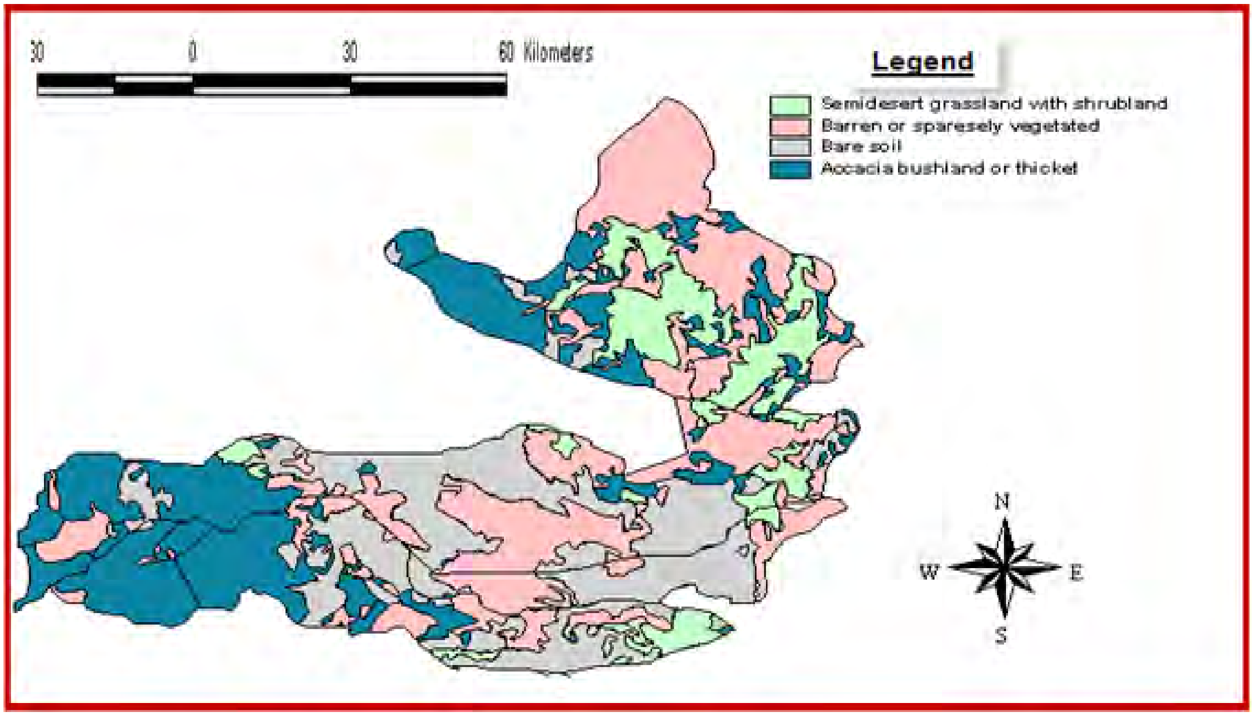

Figure 4.9. Land use land cover of the ungauged catchments.

4.2.4. Flow generation from the ungauged catchments

SCS-CN method

A number of hydrological stations were installed on Awash River and its tributaries during the Sogreah-FAO study on Survey of the Awash Basin over the period 1962-1965(Halcrow, 1989). Additional stations have been also installed since then. However, there are still ungauged catchments contributing huge floods to the main rivers during the rainy seasons. The catchments contributing flow to the Awash River downstream of Adaitu and Mille stations and directly to the Tendaho reservoir have no gauging stations recording the stream floods. Thus for such cases, it is unavoidable to use the rainfall data to estimate the surface runoff flows to the corresponding river and Tendaho reservoir from each catchment.

The flow contributions were calculated using SCS-CN method of estimating direct runoff based on the ten years daily rainfall data analyzed and the soil and land use land cover data of the ungauged catchments presented in sections 4.2.2 and 4.2.3.

The CN for each ungauged catchments considered in this study was read from the table showing the runoff curve number for different land use types and corresponding hydrologic soil group(Appendix-4). For the catchments made up of several soil types and land uses composite CNs were calculated.

The daily flow data generated from the ungauged catchments under consideration were calculated by applying SCS-CN technique of generating flows from multi-day rainfalls. The detail of the calculation procedure is presented in Appendix-4. The flows generated after calibration were compiled on monthly basis and presented for illustration in Table 4.1. The monthly flow hydrograph of Ledi catchment has been also presented as in Figure 4.10 for illustration and that of the other catchments are placed in Appendix-5.

The rainfall over the surface of the Tendaho reservoir at each daily time step was changed to flow data using the reservoir surface area.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	0.31	0.00	0.04	0.00	0.00	0.00	2.66	5.11	0.34	0.00	0.00	0.55
1993	0.62	0.73	0.00	2.74	0.22	0.00	11.14	0.00	3.27	1.62	0.00	0.00
1994	0.00	0.00	2.80	0.00	0.00	0.00	14.86	2.31	0.88	0.00	0.00	0.00
1995	0.00	0.00	4.23	0.18	0.00	0.00	15.29	3.01	0.00	0.00	0.00	0.00
1996	0.00	0.00	2.32	0.00	0.83	0.02	1.55	0.00	0.01	0.00	0.07	0.00
1997	0.00	0.00	0.03	0.01	0.00	0.00	1.38	0.44	0.00	1.61	5.66	0.00
1998	0.00	2.27	1.21	0.00	0.00	0.00	5.34	1.50	0.09	0.10	0.00	0.00
1999	0.00	0.00	0.00	0.00	0.00	0.04	1.64	5.38	0.00	1.27	0.00	0.00
2000	0.00	0.00	0.00	0.00	9.70	0.00	4.53	6.05	0.00	0.38	3.13	0.43
2001	0.00	0.00	0.48	0.62	0.26	0.00	4.81	0.83	0.08	0.00	0.00	0.08

Table 4.1. Mean monthly flow generated from Ledi catchment.

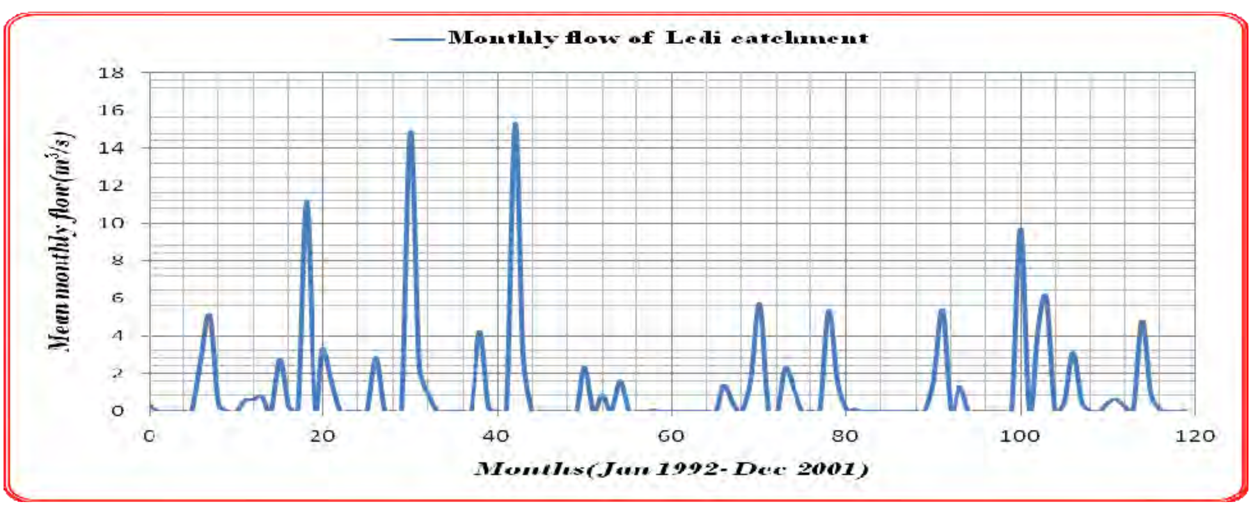

Figure 4.10. Ledi catchment mean monthly flow hydrograph.

Flow data transposition

To generate the daily flows of the ungauged catchments contributing floods downstream of the selected hydrological stations, Logiya river flow data were used as both Logiya and the ungauged catchments exhibit similar physical and hydrological Characterstics. The ten years daily flow records of Logiya catchment were used to generate the flows of major ungauged catchments. The flow records were transferred to the ungauged catchments in proportion to the mean annual rainfall of the catchments. The mean annual rainfalls of the catchments were computed by Isohyetal method from the isohyets map obtained from the ministry of water resources. The mean annual rainfalls and the corresponding areas are presented in Table 4.2.the flow data generated were used to calibrate the SCS-CN parameter, CN.

Catchment	*Area(ha.)*	*Mean annual rainfall(mm)*
Logiya	344,711.00	502.02
Ledi	81,258.00	438.23
Geram	27,828.00	174.03
Weranso	106,804.00	398.56
Gafura-Magalala	108,269.00	313.52

Table 4.2. The ungauged catchments and their mean annual rainfalls.

4.2.5. Calibration and validation

Calibration is a process of standardizing estimated values, using deviations from observed values for a particular area to derive correction factors that can be applied to generate predicted values that are consistent with the observed values. Such empirical corrections are common in modeling, and it is understood that every hydrologic model should be tested against observed data, preferably from the watershed under study, to understand the level of reliability of the model. The calibration process can provide important insight into both local conditions and model performance; if correction factors are large or inconsistent across several study areas; this suggests that some significant component of the hydrologic system or its controls is being neglected.

The most important parameter that affects SCS method is runoff curve number (CN). Thus, this parameter has to be derived from calibration against the observed output or stream flow. In this study SCS method was applied for ungauged catchments. Therefore, for calibration of SCS method flow data were transposed from Logiya catchment in proportion to the mean annual rainfall. The Logiya flow records were chosen because the physical and hydroclimatological features of both Logiya and the ungauged catchments are similar.

The statistical measures used in this thesis to assess the performance of SCS-CN method, as well as highlight its strengths and weaknesses, are commonly used measures referenced in literatures. Theses are: the correlation coefficient (r), Root Mean Square (RMS), Mean Absolute Error (MAE), Maximum Absolute Error (MaxAE), Bias or mean daily error (MDE) and Nash-Sutcliff measure (R^2). As all the ungauged catchments have similar physical and hydroclimatological properties, calibration of only Ledi catchment was made and the standardized CN was used for all the other ungauged catchments (Appendix-3). The calibration process was purely manual and was made by trial and error.

Three calibration and verification tests were designed to evaluate the method. In the first test, data from 1992 to 1996 were used for calibration and data from 1997 to 2001 were used to verify the model. In the second test, data from 1997 to 2001 were used for calibration and 1992 to 1996 were used to verify the model. In the third test, the dataset was divided into odd years and even years and odd years were used for calibration and the even years were used to verify the model. Once the initial data preparation was completed, a modified version of the SCS-CN method was used to compute daily direct runoff values for the period 1991 to 2001. The statistical parameters of the default and calibrated results for all the tests made are given as in Table 4.3.and the graphical representations of the observed, calibrated and verified simulation flow results for the test with good result (Test-I) are presented as in Figures 4.11 and 4.12. The hydrographs are produced on monthly basis as the daily hydrographs produced are not visible. The verification statistical results are also presented in Table 4.4.

For a relatively satisfactory visual fit of the hydrographs produced on monthly basis, statistical performance investigation were further carried out and the best simulation parameters were selected based on the Nash and Sutcliff goodness of fit (R^2).

The correlation between the transposed and simulated stream flow values are with in the range of 0.25 –0.64, indicating that the model has not captured the pattern of the transposed flow.

The root mean square (rms) value measures the models adequacy in matching the peak flows. The rms value obtained for the best run of Test-I is less than that of the other two tests.

The maximum absolute error searches for the highest deviation of the historic record from the simulated flow. The three tests showed different results as shown in Table 4.3.

Bias from the mean value simply indicates the variation of the mean of historical record and the simulated data. The SCS method at its best calibration has given a lower average value for all the tests hence a positive value.

The Nash and Sutcliff goodness of fit criteria was the main statistical measure in the calibration. Higher values were obtained for tests II and III and a lower value for test I. From visual observation, Test-I has given the best fit of daily extreme floods, while the other two cases produce a smooth hydrograph with better R^2 value but missing the daily variation.

Test	Test-I(1992-1996)		Test-II(1997-2001)		Test-III(Odd years)	
Parameters	Default	Calibrated	Default	Calibrated	Default	Calibrated
Qomean	1.760	1.760	4.541	4.541	3.400	3.400
Qsmean	1.315	1.209	0.999	0.905	1.174	1.073
Correlation	0.450	0.570	0.640	0.620	0.280	0.250
RMS	13.740	13.363	18.499	18.327	16.241	15.984
MAE	2.793	2.703	5.023	4.966	4.966	4.137
MaxAE	189.668	186.985	289.111	279.627	189.668	181.915
Bias from mean	0.446	0.551	3.542	3.635	2.226	2.328
Nash and Sutcliff	0.130	0.450	0.150	0.220	0.110	0.150

Table 4.3. Statistical results of default and calibrated SCS method for the three tests.

Test	Test-I(1997-2001)	Test-II(1992-1996)	Test-III(Even years)
Qomean	4.541	1.760	3.502
Qsmean	0.905	1.209	1.071
Correlation	0.620	0.570	0.340
RMS	18.327	13.363	17.469
MAE	4.966	2.703	4.106
MaxAE	279.627	186.985	279.627
Bias from mean	3.635	0.551	2.431
Nash and Sutcliff	0.220	0.450	0.340

Table 4.4. SCS method Verification Statistical results for the three tests.

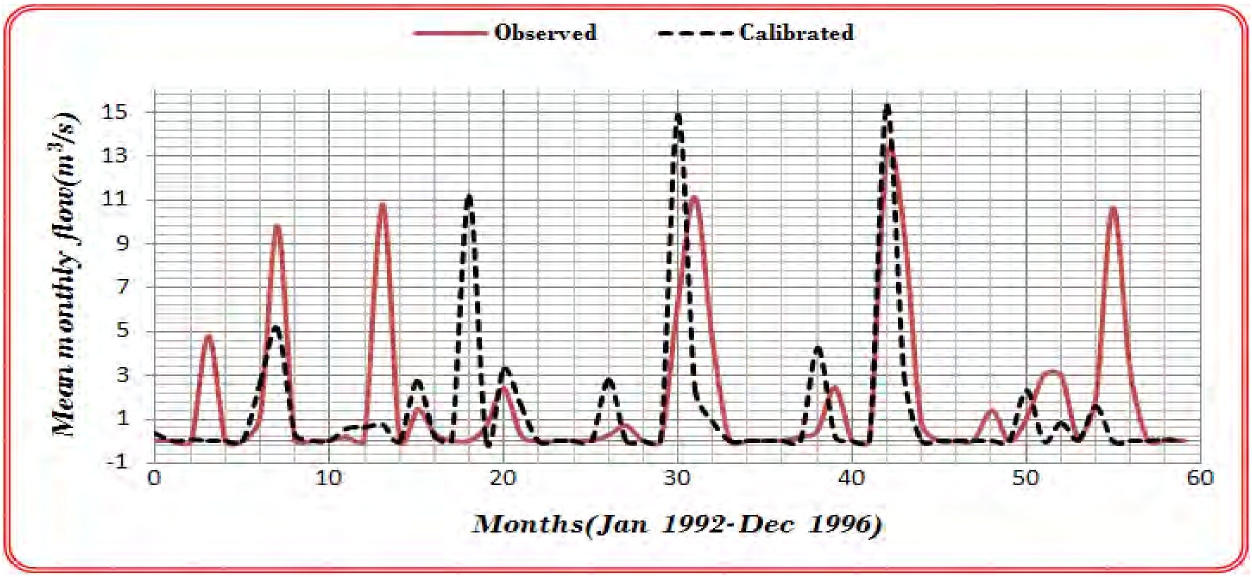

Figure 4.11. Ledi catchment mean monthly observed flow record and calibrated result.

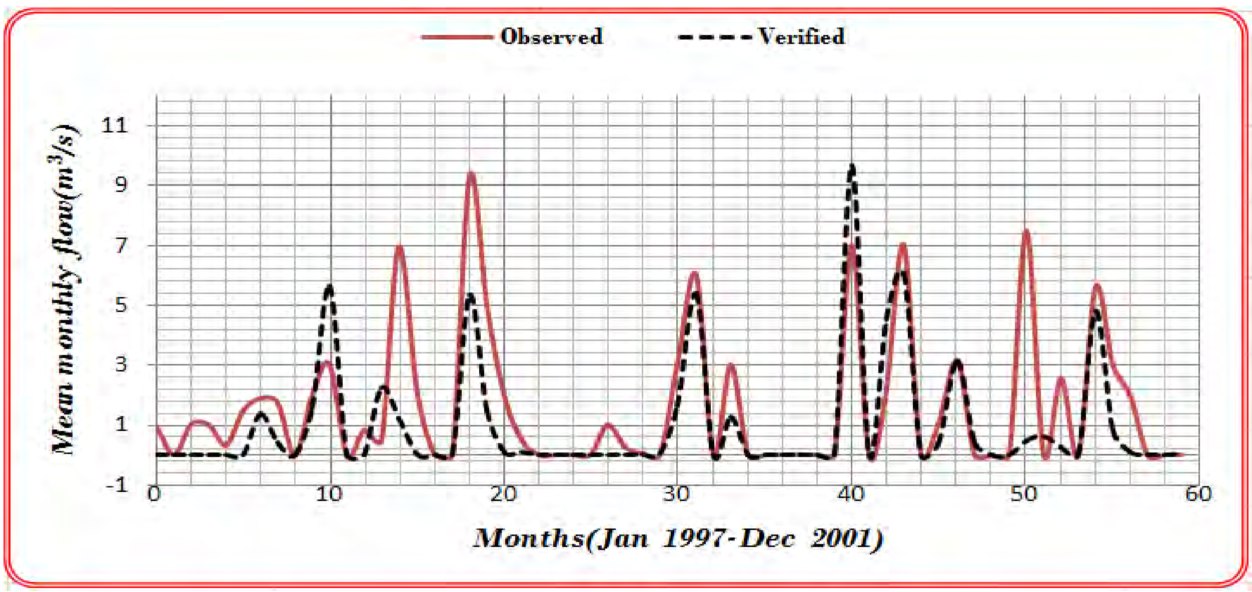

Figure 4.12. Ledi catchment mean monthly observed flow record and verified result.

4.2.6. Evaporation data analysis

The monthly evaporation data of the reservoir area is one of the basic input data of HEC-ResSim reservoir simulation model. The pan evaporation data recorded at Dubti metrological station for the duration 1979 to 2002 was used to evaluate the evaporation rate from the reservoir surface. The common and widely acceptable conversion factor of 0.7 was used to transfer the pan evaporation data to the reservoir. The computed reservoir evaporation data computed was presented as in Table 4.5 below.

Month	*Pan evaporation(mm)*	*Reservoir evaporation(mm)*
Jan	243.40	170.38
Feb	229.37	160.56
Mar	276.00	193.20
Apr	255.73	179.01
May	343.96	240.77
Jun	342.22	239.55
Jul	375.42	262.79
Aug	282.54	197.78
Sep	266.75	186.73
Oct	248.54	173.98
Nov	229.78	160.85
Dec	228.64	160.05

Table 4.5. Evaporation data of Tendaho reservoir.

4.2.6. Tendaho irrigation and ecological flow demand data

Command area

Tendaho dam and irrigation project has been implemented with the aim to cultivate sugarcane plantation on 60,000 ha of command area. The command area commences from about 18km downstream of the Tendaho Dam and continues up to the tail end of Awash River below Asayita where it merges into Afambo, Gamari and Abe lakes.

The command area is scattered at different locations along the river course both on left and right banks. In the upstream reach the command area comprises Dubti farm north of the left bank of Awash River and Boyale farm located south of Dubti farm on right side of Awash River. In the middle reach, the area comprises Dit Bahari farm, Bokaitti and Galifage areas whereas the downstream area comprised the large cultivated area located below Assayita town extending up to Gamari and Afambo lakes (WWDSE, 2005). The detail command area distribution of each scattered area is given in Table 4.6.

S.No.	**Locations**	**Command Area (ha)**	
		Gross	Net
1	Dubti	12,126.00	9,367.00
2	Galifage	3,119.00	2,409.00

3	Boyale	6,092.00	4,706.00
4	DitBahari	12,205.00	9,428.00
5	Interable -Bokaitti	4,303.00	3,324.00
6	Assayita	26,893.00	20,773.00
Total		***64,738.00***	***50,007.00***

Table 4.6. Tendaho Command area distribution *(Source: WWDSE, 2005).*

Irrigation requirement

As of Tendaho Dam and Sugar Project, it is aimed at national level to cultivate sugarcane in the entire command (60,000 ha) and establish sugar factory to manufacture sugar. Thus the intensity of sugarcane is 100% in the command area.

For this study the crop water requirement result computed by WWDSE was used with some arrangements. The crop water requirement for selected varieties of sugarcane was calculated by using the climatic data available at Dubti. It was calculated in accordance with Penman-Montieth approach as given in FAO Irrigation and Drainage paper No. 46 CROPWAT. The crop characteristics like crop co-efficient (KC) were adopted in accordance with the recommendations available in FAO Irrigation and Drainage paper No 24: Crop Water Requirement and No. 33: Yield Response with suitable modification in light of local conditions.

The crop water requirement computation result shows that net annual water requirement for the principal crop, sugarcane, is 2000mm. Application efficiency of 0.85 is considered appropriate in view of scarcity of water, the canal system up to tertiary is lined and hence a conveyance efficiency of 0.75 has been adopted. This gives an over all efficiency of (0.75 X 0.85) = 0.6375 say 0.65. Thus, the gross requirement of water at the head of canal is 3000mm. The gross annual requirement for the entire command is thus 1800Mm^3. The annual rainfall in the command area is in the order of 200mm and negligible compared to the crop water requirement computed. Thus, the crop water requirement is approximately equal to the crop irrigation demand.

The average crop water requirements calculated on monthly basis shows that the peak value at the field level is 0.78 l/s/ha and occurs in June whereas the minimum value corresponding to the month November is 0.31 l/s/ha.

For the purpose of planning and simulation study, the water requirement for 10 – day period was calculated. The 10-day water requirement at the intake level in m^3/sec and MMC were calculated including 10% flexibility factor. A project efficiency of 0.65 was used to account for all the losses. In the study under consideration, the average monthly irrigation requirement at intake level presented in Table 4.7 was used as input for HEC-ResSim.

Month	Diversion Requirement(m^3/s)	Diversion Requirement(MMC)
Jan	36.74	95.22
Feb	37.46	97.04
Mar	40.65	105.35
Apr	41.75	108.21
May	73.21	189.75
Jun	78.49	203.43
Jul	69.74	180.76
Aug	73.26	189.90
Sep	67.87	175.92
Oct	58.19	150.82
Nov	50.86	131.82
Dec	44.18	114.53
Annual		***1742.75***

Table 4.7. Monthly irrigation diversion at the dam intake (*Source: WWDSE, 2005*).

Irrigation outlet

Irrigation outlet of the Tendaho dam is the appurtenant structure through which regulated flow for irrigation and ecological demands are provided. It also enables controlled filling and emergency depletion of the reservoir. Irrigation outlet is the only water way provided through the dam body.

It was designed for peak irrigation demand release of value 78m^3/s. This discharging capacity is expected to control the rise of reservoir during initial filling to permissible limits. Similarly, the depletion of reservoir up to El 400.0 would be affected by allowing the flow over the spillway crest in addition to the release through Irrigation Outlet. The depletion of reservoir below El 400.0 would be achieved solely by release through the outlet. The reservoir capacities at this stage per meter depth of the reservoir are relatively low compared to the capacities above El 400.0 and hence, provision of the outlet of 78 m^3/s capacity is considered to be adequate (WWDSE, 2005). The intake has a bell mouthed entry. The opening at tunnel intake is rectangular of 4.0m (width) x 5.0m (height) in size. This joins to the 5.0m diameter circular opening of the irrigation outlet tunnel by means of appropriate transition of 6-metre length.

The Minimum Draw Down Level (MDDL) is set at El 396.0. The height of intake opening is 5.0 m. Hence, the centerline of intake is placed at El 391.0, which provides a minimum water cover of 5.0 m above the centerline. The invert of tunnel intake is placed at El 388.0. The general ground level in front of the tunnel intake is higher than El 388.0. An entrance channel was excavated with bed at El 388.0 so that the irrigation outlet tunnel is able to withdraw water from the reservoir without any obstruction.

Based up on the reservoir and irrigation outlet data, elevation versus maximum capacity for the irrigation outlet can be computed by orifice formula:

$$Q = CA\sqrt{2gH}$$

Where:

A = Cross-sectional area of the intake tunnel (m^2),

C = Contraction coefficient (usually taken as 1 for bell mouth entry),

H = Head of water above the center line of the intake (m) and

g = Acceleration due to gravity (m/s^2).

In the Tendaho Dam and Irrigation Design Report, velocity of flow through Irrigation outlet has been restricted to 4m/s as the water drawn by the outlet may have considerable suspended sediment load. Accordingly, a circular tunnel of 5.0 m diameter has been provided for the outlet, to restrict the velocity of flow to 4.0 m/s. Using all these information and the above equation, maximum capacity of the tunnel at the corresponding elevation could be calculated.

The minimum available head in the reservoir can make a flow velocity value twice of the allowable velocity in the intake tunnel and in turn a discharge twice of the predetermined outlet capacity of $78m^3/s$. Hence, the maximum irrigation outlet capacity of $78m^3/s$ was taken for any reservoir pool level above the MDDL of 396 masl and these data were used as input for HEC-ResSim.

Ecological flow demand

In planning reservoir construction, consideration of water release requirements for the downstream environment is of great importance. As far as Tendaho Dam construction and irrigation development is concerned, the terminal lakes are of great interest. These lake complexes occupying the lower Awash basin area are six in number and are of different sizes. These lakes are Gefu, Abe, Adobed, Afambo, Gamari and Suwata. All the lakes, except the end salt Lake Abe, are fresh in nature. They are replenished by the residual flow from the Awash River, the only source of inflowing water.

All the lakes, except Abe, and the swamps around them provide important grazing and watering points for the Afar. The environmental assessment report of the Tendaho Dam and Irrigation Project shows that the lakes are also home for some varieties of fishes and hippopotamus. The allocation of water to the lakes also serves the fundamental requirement of maintaining an acceptable level of salinity in the watercourse and in providing the facility for the passage of the heavily silt laden waters at the onset of the wet season through the reservoir thereby increasing its operational life (WWDSE, 2005).

Thus, allocation of adequate volume of water to the lakes is unquestionable. The amount of water to be released for ecological reasons was estimated in different studies conducted related to the construction of dam at Tendaho. All the studies have proposed that regulated constant flow of $5m^3/s$ is adequate to replenish the lakes and of aquatic life in them to be kept alive. In the study under consideration, this value of estimated environmental flow demand was used for the reservoir operation analysis.

4.2.7. Tendaho reservoir data

The volume-elevation curve and area- elevation curve of Tendaho reservoir were conducted by WWDSE in 2005 as input for Tendaho Dam and Irrigation project-Dam detail design work. These data were used in this study to analyze the reservoir operation.

Elevation(m)	Area(km^2)	Volume(MMC)
373	0.001	0.000
374	0.026	0.014
375	0.102	0.078
376	0.220	0.238
377	0.244	0.470
378	0.388	0.786
379	0.799	1.380
380	1.673	2.617
381	2.684	4.795
382	3.131	7.702
383	6.834	12.685
384	8.918	20.561
385	12.718	31.378
386	15.111	45.293
387	17.649	61.673
388	20.160	80.577
389	22.014	101.664
390	23.713	124.528
391	25.362	149.066
392	47.924	185.709
393	55.263	237.302

394	64.472	297.169
395	73.705	366.257
396	81.100	443.660
397	86.969	527.695
398	92.590	617.475
399	97.868	712.704
400	106.375	814.826
401	112.357	924.192
402	118.592	1039.667
403	124.768	1161.346
404	130.950	1289.206
405	133.725	1421.543
406	144.291	1560.552
407	149.860	1707.627
408	154.567	1859.841
409	160.598	2017.423
410	166.060	2180.752
411	174.503	2351.033
412	179.243	2527.906
413	185.889	2710.472

Table 4.8. Characterstics curve data of Tendaho reservoir.

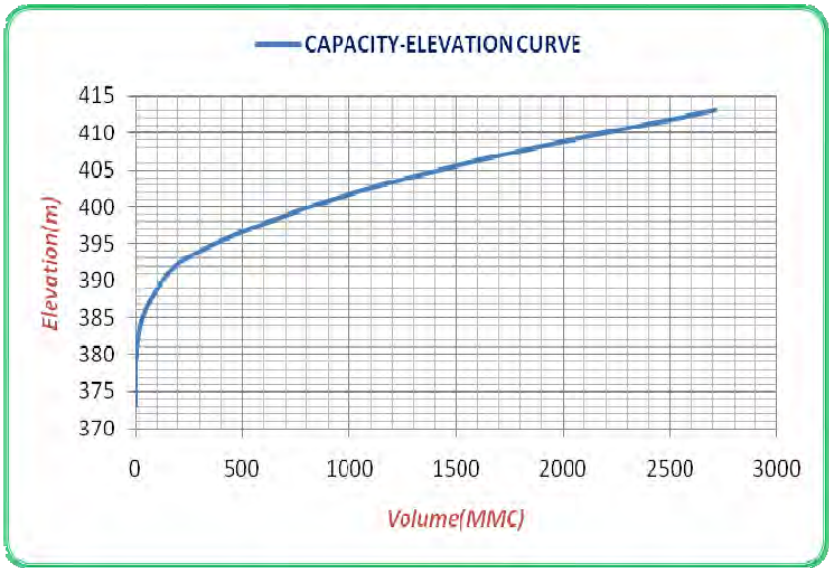

Figure 4.10. Tendaho reservoir Capacity-Elevation curve.

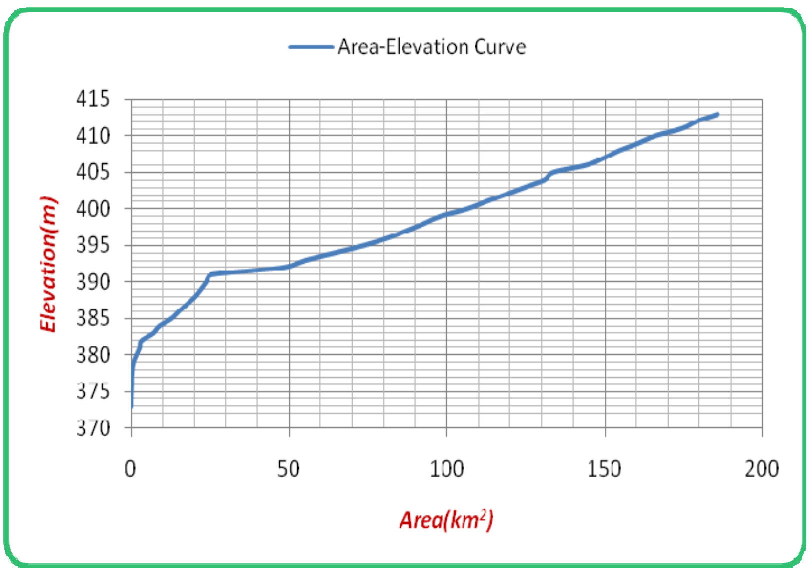

Figure 4.11. Tendaho reservoir Area-Elevation curve.

4.2.8. Spillway and downstream river reach data

Side Spillway

As Tendaho dam is an earth fill dam, spillway of sufficient capacity has been provided for the safety of the dam. The spillway has such a capacity that it is expected to contain the inflow design flood within the storage space above the full retention level of the reservoir and prevents the dam to be overtopped in the event of occurrence of design flood.

The side spillway structure provided comprises an approach channel leading the water to the control structure, a control structure and an inclined chute terminating into a stilling basin. The control structure is an ogee weir of 37.5m crest length surmounted by three radial gates of each 10.5m width and 9m height. The crest level of the weir is 400.0 masl. The design discharge over the spill way is 1,700m^3/s.

As the side spillway has been designed to control the inflow design flood within the storage space above the full retention level of the reservoir (408 masl), only the flow through the opening of the spillway gates for reservoir pool above this level need to be considered. The side spillway and its control structure have been designed in such a way that, the nature of the flow over the spillway is entirely submerged.

Based up on the reservoir and spillway data available, elevation versus maximum capacity for the spillway for reservoir pool level above 408 masl and different levels of gate openings was estimated by the following relation and the result was used as input for HEC-ResSim.

$$Q = \frac{2}{3}\sqrt{2}\, g^{1/2} bCd1\left(H^{3/2} - H_1^{3/2}\right)$$

Where:

b = Effective crest length of the spillway (m),

Cd_1 = Coefficient of discharge (usually taken as 0.6),

H = Head of water above the spillway crest level (m),

H_1=Head of water above the bottom edge level of the gates (m) and

g = Acceleration due to gravity (m/s^2).

The computation result and the stage-discharge curve data developed are presented in Table 4.9 and Figure 4.11 respectively.

Maximum capacity(m^3/s)							
Reservoir Level(masl)	***For gate setting(m)***						
	1.0	***2.0***	***3.0***	***4.0***	***5.0***	***6.0***	***6.9***
408.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
408.1	259.79	501.87	724.84	926.89	1105.57	1257.32	1365.94

408.5	266.54	515.86	746.66	957.33	1145.71	1308.77	1429.98
409.0	274.75	532.82	773.06	994.03	1193.87	1370.04	1505.33
409.5	282.72	549.26	798.58	1029.38	1240.05	1428.43	1576.44
410.0	290.47	565.22	823.30	1063.54	1284.50	1484.34	1644.06
410.2	293.52	571.48	832.98	1076.89	1301.84	1506.08	1670.24
410.4	296.53	577.67	842.54	1090.07	1318.94	1527.49	1695.99

Table 4.9. Spillway rating curve data.

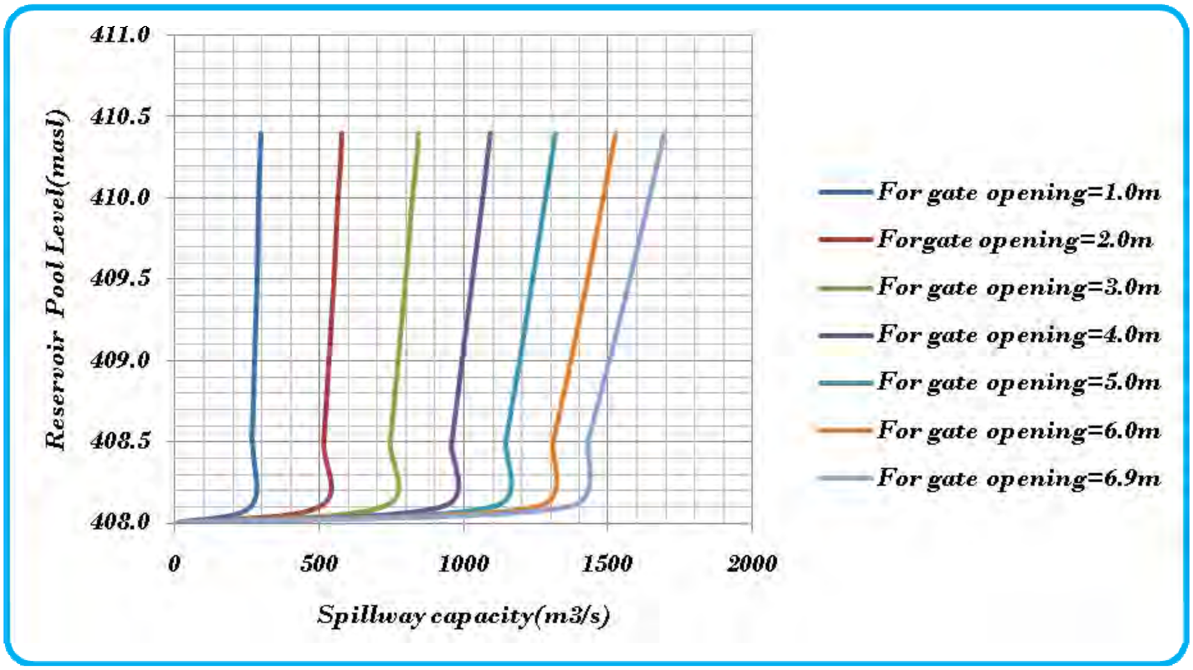

Figure 4.11. Spillway stage-Discharge curves for different gate settings.

Downstream river reach

The command area of Tendaho Sugar plantation is flood prone. Therefore flood protection works in the form of embankment have been planned. Thus, a total length of 172.2 km of embankment has been planed to be provided to protect the command area and irrigation system against the flood (WWDSE, 2005).

The downstream river reach data is concerned with the maximum flood that the river reach downstream of Tendaho dam can accommodate so that no flooding of the dykes, and in turn, irrigation development areas allocated along the river is going to occur.

The design flood for which the protection dykes have been designed could not be obtained in the Tendaho Dam and Irrigation Project Design Reports. Thus, assumption has been made to account for the maximum flood that Awash River downstream of Tendaho reservoir can accommodate.

The assumption made in this study was that, in hydrologic design of the protection dikes, the sum total of half of the spillway design discharge that was taken for the design of the spillway stilling basin, the irrigation outlet maximum release and the average wet season flood magnitude of Logiya river should have been taken as the maximum flood that the downstream river riches could accommodate. Hence, the sum total of these flow magnitudes was considered in this study to identify the release constraints in reservoir operation simulation made by HEC-ResSim.

4.2.9. Routing reaches data

The routing reaches parameters computed using Kirpich's formula for the river channels conveying flows from remote distances to the reservoir inlet to be used as inputs for Muskingum method of flow routing technique are as presented in the table below.

The computed K (T_c) and X values of each routing reach (Table 4.10) were used as inputs to the reach editor of HEC-ResSim model.

Reach		*Reach length(m)*	*Slope(m/m)*	*K(min.)*	*K(hr)*	*X*
From	*To*					
Adaitu station	Ledi-Awash confluence	9000	0.004	29	0.48	0.5
Ledi-Awash confluence	Geram-Awash confluence	21400	0.00096	98	1.63	0.5
Mille	Weranso-Mille confluence	8130	0.0038	27	0.46	0.5
Weranso-Mille confluence	Reservoir inlet	12200	0.0029	41	0.69	0.5

Table 4.10. Summary of input for Muskingum method.

5. MODELING

5.1. General

When a new watershed is created, ResSim generates a directory structure and stores all files associated with the watershed inside that structure. ResSim creates a new watershed directory in the base directory. The watershed directory is named according to the name given to the watershed. The watershed directory stores all of the base data for the watershed, including maps, schematic elements, base model data, and simulation data and results. When a new simulation is created, ResSim generates a simulation directory named according to the date and time of the simulation. When a simulation is created, ResSim automatically copies all of the base data for created watershed (except for maps) into the simulation directory. This facilitates archiving of simulation information and ensures consistency in model results.

The modeling was done for two inflow source options to compare the guide curves to be developed. The two inflow conditions considered were upstream inflow sources (Mile River, Awash River at Adaitu gauging station and the flows generated from the ungauged catchments) and only the flow recorded at Tendaho hydrological station. The details of application of HEC-ResSim model for the two inflow conditions are discussed in the following sections.

5.2. Operation modeling for upstream inflow sources

5.2.1. Tendaho reservoir Watershed Setup

The background image that describes the Geo-referenced area of the watershed was imported from external source of Arc View GIS (Fig 5.1). The unit to be used in the system and the international time zone of the area was set to be SI unit and GMT+ 3 respectively. Items that describe the water shade’s physical arrangement such as Streams, gauging stations, Computation points, reservoir and irrigation diversion were drawn using their respective mouse tools provided in the watershed set up module of the software.

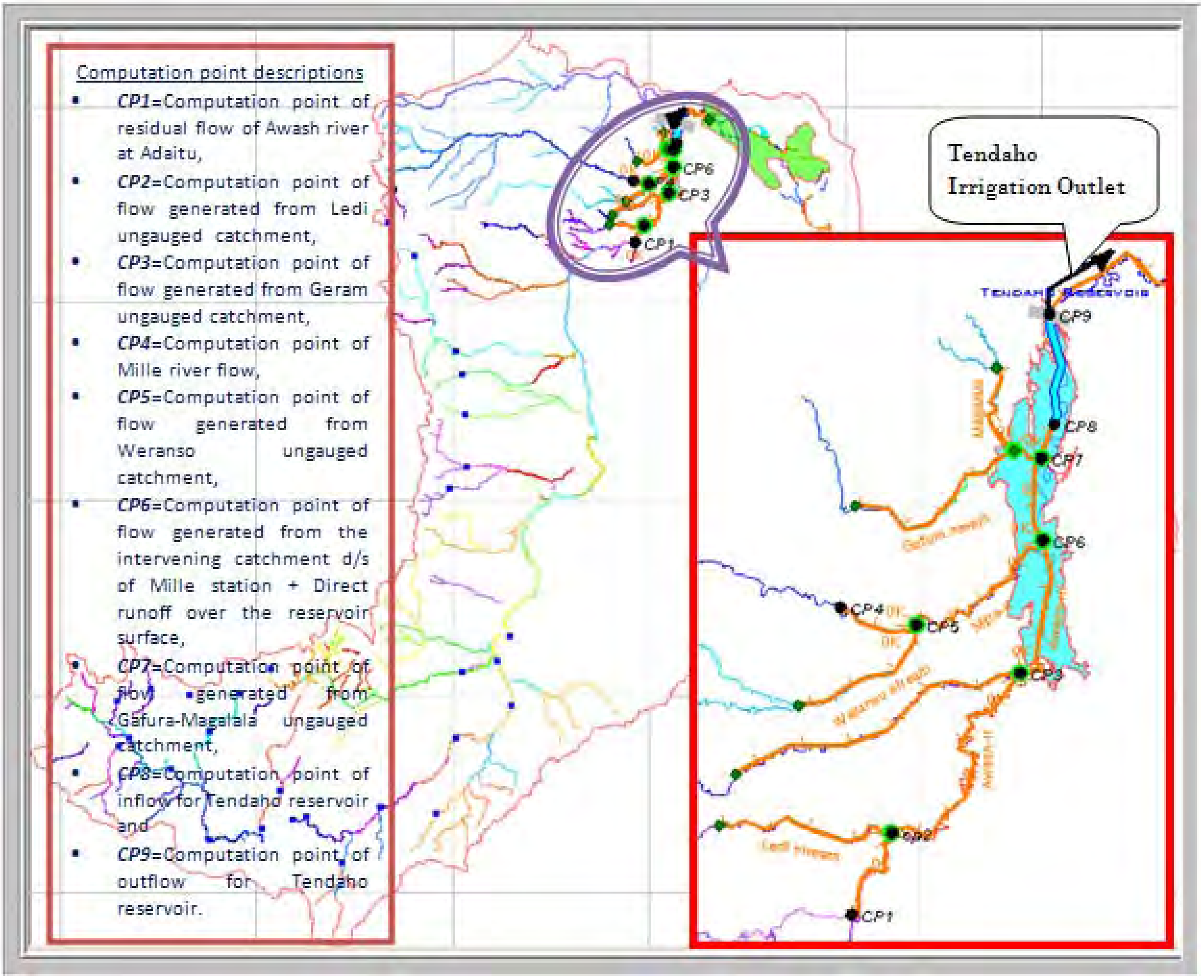

Figure 5.1. Watershed setup of Tendaho reservoir.

5.2.2. Tendaho Reservoir Network and model components

The purpose of the Reservoir Network module is to isolate the development of the reservoir model from the output analysis. In the Reservoir Network module, one will build the river system schematic, describe the physical and operational elements of the reservoir model, and develop the alternatives that to be analyzed. Using configurations that are created in the Watershed Setup module as a template, the basis of a reservoir network has been created. Then the routing reaches have been added and other network elements to complete the connectivity of the network schematic as well. Once the schematic is complete, physical and operational data for each network element were defined. Also, alternatives were created that specify the reservoir network, operation set(s), initial conditions, and assignment of DSS pathnames (time-series mapping). The full model components created for Tendaho reservoir are the following.

1. **River nodes (CP_1 to CP_7)**: are the points used to add the daily flow records (i.e. flow time series, 1992-2001) of the gauged rivers, the daily flow generated for the ungauged catchments and the direct runoff over the reservoir surface.
2. **Tendaho irrigation outlet (CP9)**: was the node that used to add the monthly irrigation requirement for Tendaho Sugar Plantation and ecological demand (Table 4.7).
3. **Tendaho reservoir physical component**: was used to add the mean monthly rate of evaporation (Table 4.5) and the characteristics curves of the reservoir (Table 4.8).
4. **Tendaho dam physical components**: was the control points used to add the dam component like, irrigation outlet and spillway capacity curves, dam crest length and elevation of top of the dam.

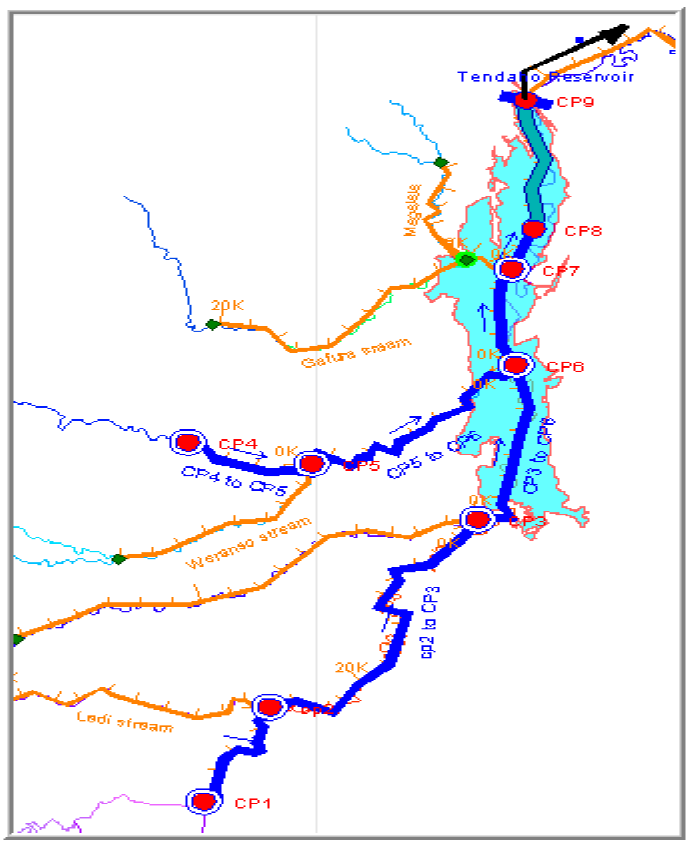

Figure 5.2.Tendaho reservoir network.

5.2.3. Operation parameters of the reservoir

After HEC-ResSim elements are geographically placed including main stream segments, inflow points, the dam with its connections to the main river stream as well as irrigation outlet, the next step of the HEC-ResSim set-up is the definition of the physical and operation parameters of the dam (Appendix-6). These parameters are geometric properties of the pool, the capability of the irrigation outlet and the side spillway and the operation in conditions of flooding.

There are four reservoir storage zones defined in the Tendaho reservoir operation modeling: Flood control zone, Conservation zone, Spillway crest storage zone and Inactive zone. For each of the zones generated in the operation set, operation rules are provided.

Flood control mode of operation

Two rules were used in order to define the Flood control mode which is triggered when the reservoir pool just exceeds 408.0 masl. The first rule which was called "Max.IO release" simply states that the irrigation outlet continues to operate with a maximum intake capacity of 78m^3/s to relief the reservoir. The second rule is called "Max.Spills over SSW" and specifies that the side spillway is operated for floods arriving in excess of the maximum irrigation outlet capacity to withdraw the floods.

Normal (Conservation) mode of operation

For the normal (Conservation) mode of operation, the only rule provided is "Sustaining Irr. & ecol.demand" which reveals that the irrigation outlet is operated throughout the year to sustain the irrigation requirements for the irrigated land and ecological flow demand for the downstream river regime and terminal lake complexes. The Conservation mode is triggered when the level of the reservoir is situated between the top of conservation zone of 408.0 masl and the empty level reached at 396.0 masl. The spillway crest conservation zone is included in the conservation zone of the reservoir storage and defined only to show the side spillway crest level in HEC-ResSim. Operation of the dam in this zone is also determined by the rule defined for the normal operation mode. The normal mode rule is defined so that the minimum flow equal to the requirements of the irrigated land (Table 4.7) and the monthly ecological demand flow of 5.0m^3/s are maintained.

As the side spillway is a gated ogee spillway, simulation was done for three gate opening levels. The gate setting considered are opening of the gate to a minimum height of 1m, average height of 4m and maximum height of 6.9m.

5.2.4. Simulation procedure and results

The purpose of the Simulation module is to separate output analysis from the model development process. Once the reservoir model is complete and the alternatives have been defined, the Simulation module is used to configure the simulation. The computations are performed and results are viewed within the Simulation module.

Due to the difficulty of showing the very large number of data points in tabular form, the out puts of the model have been shown graphically in the next chapter. During the creation of the simulation model it was a must to specify a simulation time window, a computation interval, and the alternatives to be analyzed. The time widows given for present case was starting, lock back, and end time of the simulation. Then, ResSim creates a directory structure within the rss folder of the watershed that represents the "simulation". Within this "simulation" tree will be a copy of the watershed, including only those files needed by the selected alternatives. Also created in the simulation is a DSS file called simulation.dss, which will ultimately contain all the DSS records that represent the input and output for the selected alternatives. Additionally, elements can be edited and saved for subsequent simulations.

5.2.5. Transferring the time series data to HEC-DSS tools

The only time series data required storing in HEC-DSS file for this study were the daily in flows to the reservoir, from the year 1992 to 2001, analyzed in section 4. Once the time series data are converted in to HEC-DSS file format, it can be used in the simulation module of HEC-ResSim by setting the path to DSS-path for each inflow points to the reservoir in the alternative editors. The HEC-DSS time series data transfer process is schematically presented in Figure 5.3.

The computation results of HEC-ResSim model are also stored in HEC-DSS format and could be accessed by the mouth tools found in the simulation module after computation. These data were converted to spreadsheet format for further processing.

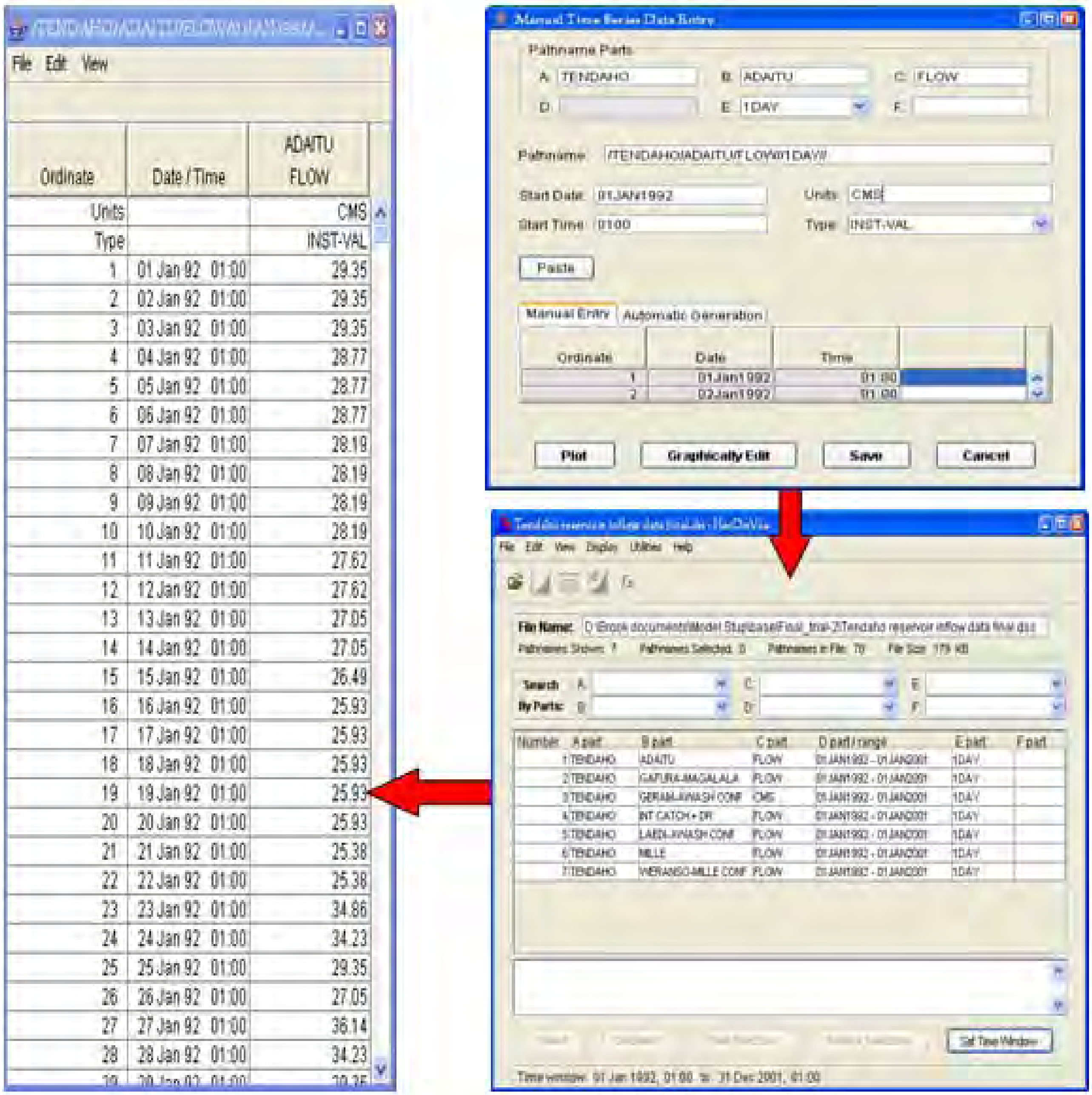

Figure 5.3. HEC-DSS time series data transfer process.

5.2.6. Reservoir initial condition

In the reservoir network module of HEC-ResSim model, after the routing reaches and junctions are edited and reservoir physical and operation data are fed, the last task is concerning the alternative editor. In the alternative editor defining the operation set, the look back or initial condition and time-series mapping is required. In this modeling the reservoir initial condition or look back assumed was concerned with putting the assumed reservoir parameters in the alternative editor:

- Tendaho reservoir pool elevation **405.0** masl,
- The spillway release **0.0** m^3/s,
- The irrigation outlet release **41.74m**^{3}/s.

5.2.7. Reservoir Operation Simulation Steps

The modeling activities mainly include building the modules, the time series conservation, entering the reservoirs network data and performing the HEC-ResSim simulation. The model consists of the stream alignments that define the flow coming in to the Tendaho reservoir and a representation of Tendaho Reservoir (Figure 5.2). A simple operational simulation was done for the years 1992 to 2001 based on the daily time series data. The operation simulation follows the following steps or procedure:

1 On the launched watershed setup module, Arc view GIS stream line shape files are imported and used as a basis to align the river network.

2 From the watershed module of HEC-ResSim drawing of the river network, Tendaho reservoir, River junctions, Computational points and Irrigation diversion.

3 Storing the compiled time series stream flow data in to the HEC Data Storage System, (i. e in the HEC-DSS).

4 From the reservoir network module, Draw reaches (including the reach properties), add junction properties and adding the irrigation diversion data.

5 Add the physical and operational data of the reservoir and dam components through the reservoir editor menu.

6 Set alternatives and access the input time-series data from HEC-DSS and fix at the required junctions of the rivers.

7 Simulate the reservoir operation for each alternative set.

8 After analyzing the computation result repeat step 6 and 7 until the best alternative result for evaluation of Spillage loss, minimum operating storage level ,maximum flood controlling storage level and water availability for the minimum irrigation and environmental demands is achieved.

9 Determine the final selected gate operation alternative based on all gate setting pattern and formulate the gate operation rule for the best get setting alternatives.

10 Finally, the computation result of the selected alternative is processed in the excel sheet to select the wet season operation rules for each year of the computation period. Then, the daily basis wet season gate operation rules of Tendaho reservoir is re-established based up on the ten years operation rule results of the selected alternative made by HEC-ResSim.

5.2. Operation modeling for Tendaho station inflow source

Following all the modeling procedures discussed in the above sub-sections, except the initial condition, operation modeling was done considering the Tendaho flow data as the inflow source of the reservoir. Ten years daily flow data of Tendaho hydrological station was used for operation simulation and the results are presented in the next section. In this case the reservoir initial condition or look back assumed was:

- Tendaho reservoir pool elevation **407.0** masl,
- The spillway release **0.0** m^3/s,
- The irrigation outlet release **41.74m**^{3}/s.

6. RESULTS AND DISCUSSIONS

6.1. General

In this research, simulation was made for the period of 1992-2001 using the daily inflow data of upstream flow sources and Tendaho hydrological station, monthly irrigation diversion data, monthly evaporation data and the minimum monthly ecological release demand of the lake complexes downstream of Tendaho reservoir as inputs to HEC-ResSim model. The simulation was done for three levels of side spillway gate settings selected. The reservoir operation simulation results are discussed in this section.

6.2. Inflow to Tendaho reservoir

Operation modeling of the reservoir has been done considering two inflow sources. One of the inflow sources are Awash residual flow recorded at Adaitu, Mille river flow recorded at Mille bridge, the flow generated from the ungauged catchments downstream of the gauging stations and the direct run-off over the reservoir surface. The other source of flow considered for alternative use of operation modeling was the daily flows recorded at Tendaho gauging station. In the first case HEC-ResSim has computed the total inflows by routing the flow data at considerably remote distances from the reservoir inlet and finally adding all the flow time series data defined in the alternative editor and in the second case the inflow to the reservoir was directly applied. The inflow output obtained from the simulation of the model for the two alternative inflow sources considered are summarized in Table 6.1 below. The monthly average in-flow values summarized using spreadsheet (Table 6.2 and 6.3) and the daily in-flow hydrograph produced by HEC-ResSim for the two inflow alternatives (Figure 6.2and 6.3) are presented. The ten years mean monthly inflow to Tendaho reservoir is also graphically represented in Figure 6.1.

Flow source alterative	**Min.flow(m^3/s)**	**Max.flow(m^3/s)**	**Avg.flow(m^3/s)**
Upstream flow sources	3.00	1055.00	82.00
Tendaho gauging station	0.00	590.35	72.47

Table 6.1. Inflow summary for the two alternative inflow sources.

Year	Month											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	35.20	41.19	39.55	37.67	21.21	13.02	21.24	133.58	219.43	224.28	109.27	46.04
1993	23.40	60.30	33.32	92.81	80.23	62.18	115.12	83.71	121.91	112.73	79.98	42.41
1994	25.86	19.43	54.27	36.26	17.41	11.35	210.46	243.53	205.21	114.20	41.56	23.27
1995	16.90	20.40	42.05	49.54	45.31	22.17	129.97	205.69	123.38	89.06	32.50	14.35
1996	23.35	12.18	61.36	31.76	41.14	30.12	44.39	176.44	135.82	116.64	55.04	30.02
1997	26.79	6.79	38.08	58.57	29.00	20.21	69.19	121.67	94.21	108.66	126.97	104.19
1998	69.23	82.16	129.31	69.36	41.14	22.33	246.95	349.92	250.13	214.12	99.61	47.70
1999	35.45	26.71	33.08	28.22	21.49	16.45	140.04	320.83	274.23	202.70	121.32	60.47
2000	37.88	29.16	22.33	24.14	63.01	16.67	54.05	149.52	110.44	148.61	62.84	18.95
2001	41.46	26.70	52.71	44.63	33.62	17.62	118.69	146.33	54.61	23.66	75.79	46.72
Mean	*33.55*	*32.50*	*50.61*	*47.30*	*39.36*	*23.21*	*115.01*	*193.12*	*158.94*	*135.47*	*80.49*	*43.41*

Table 6.2. Mean monthly inflow (m^3/s) to Tendaho reservoir for upstream flow sources.

Year	Month											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	28.69	49.55	38.26	41.72	32.64	16.06	44.50	196.05	214.07	232.66	92.69	54.00
1993	50.32	97.53	56.19	105.87	34.06	95.59	89.47	103.37	141.83	122.54	108.96	82.15
1994	28.15	19.40	42.41	42.38	35.77	31.97	72.03	160.61	167.71	111.69	55.11	41.09
1995	36.57	37.03	42.07	51.72	55.22	38.78	60.03	131.21	118.84	93.06	48.41	32.51
1996	37.52	7.37	24.70	10.94	16.47	17.13	7.08	213.62	59.79	44.16	57.13	21.89
1997	12.68	13.13	55.39	43.12	22.18	22.39	34.49	74.32	73.15	119.50	114.60	73.45
1998	30.62	46.09	51.14	8.31	1.10	16.88	184.71	448.99	193.32	88.51	18.38	9.77
1999	45.20	32.47	49.26	23.72	12.69	10.49	144.01	198.50	123.68	81.31	63.56	64.42
2000	36.36	23.39	17.16	11.77	23.50	5.29	122.04	316.77	112.26	60.16	28.47	1.29
2001	4.06	13.18	67.42	72.01	80.66	64.50	145.91	185.88	99.69	57.05	4.67	0.00
Mean	*31.02*	*33.91*	*44.40*	*41.15*	*31.43*	*31.91*	*90.43*	*202.93*	*130.43*	*101.06*	*59.20*	*38.06*

Table 6.3. Mean monthly inflow (m^3/s) to Tendaho reservoir for Tendaho flow source.

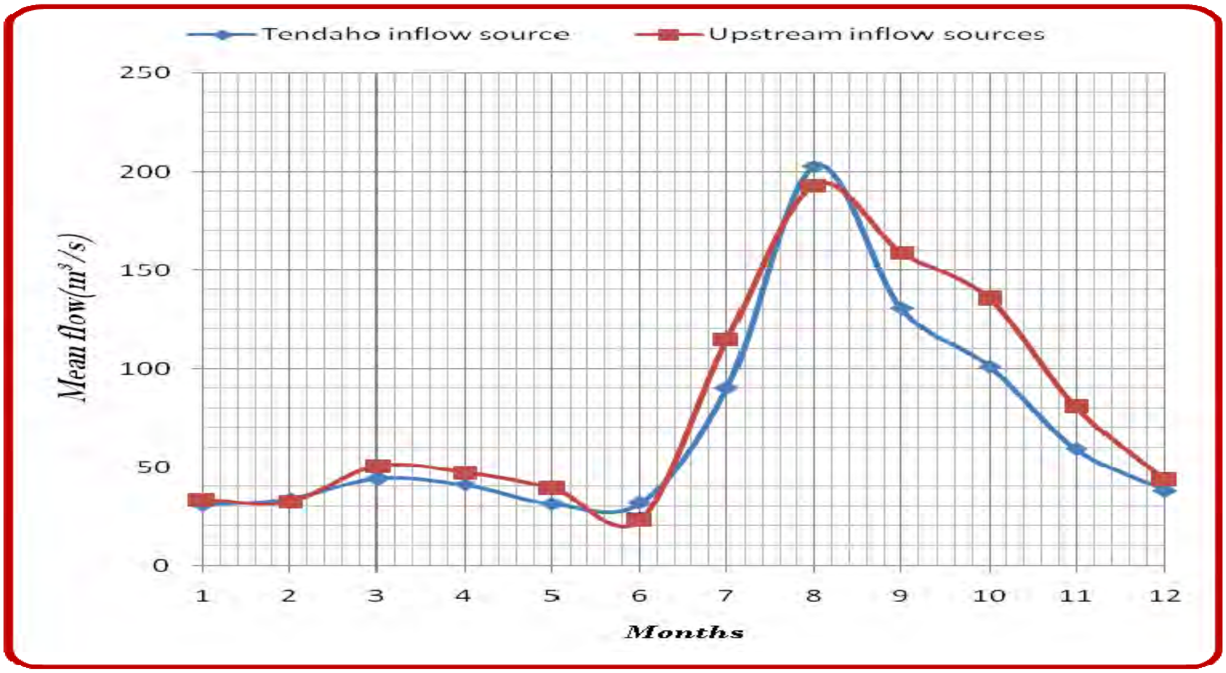

Figure 6.1. The ten years mean monthly inflow hydrograph to Tendaho reservoir.

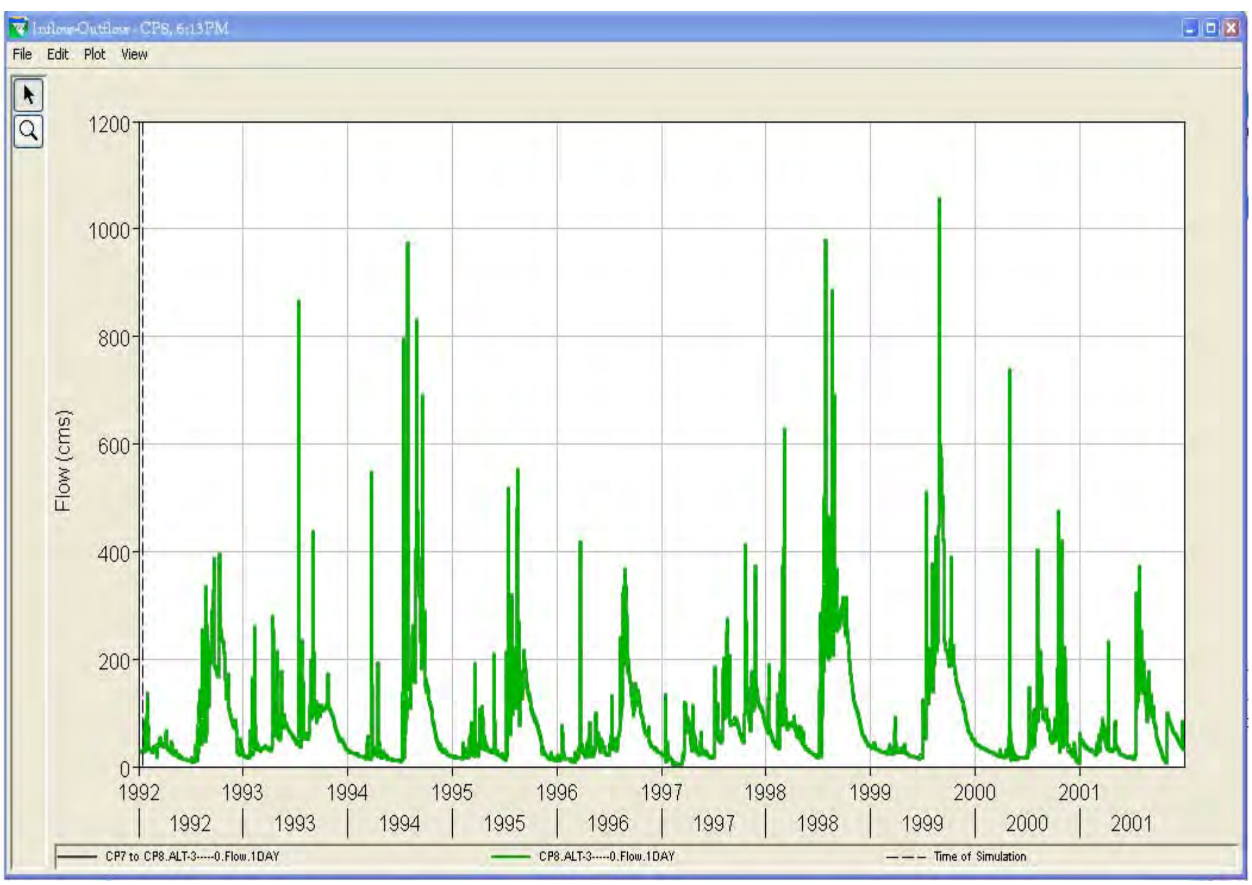

Figure 6.2. Daily inflow hydrograph to Tendaho reservoir for the upstream flow source.

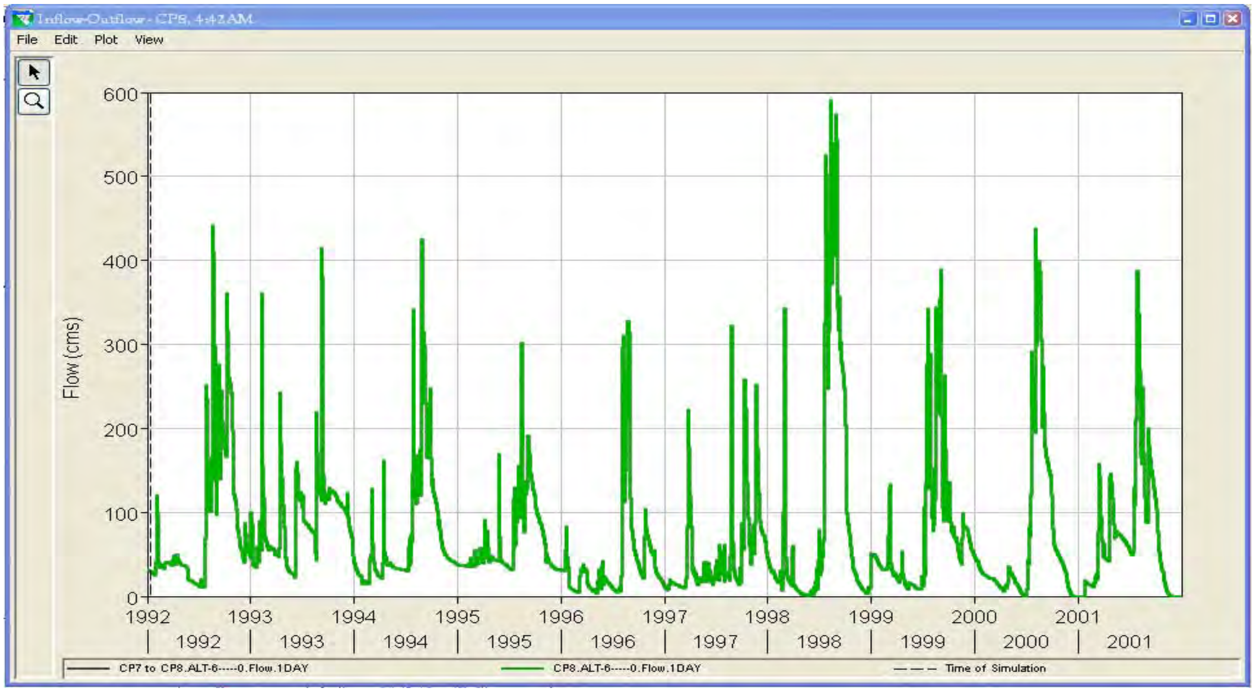

Figure 6.3. Daily inflow hydrograph to Tendaho reservoir for the Tendaho flow source.

6.3. Tendaho reservoir storages

Reservoir storage for minimum level (1.0m) gate opening

Simulations of reservoir operation have been done for the ten years of time series data of the two alternative inflow options. The results show that the reservoir receives floods of magnitudes in excess of the storage capacity of the conservation zone of the reservoir twice every five years for the upstream inflow source option (Figures 6.4) and trice every five years for the Tendaho recorded inflow source option (Figure 6.5). Thus, there is a need for operation of both the side spillway and irrigation outlet for these wet durations. The reservoir stores water above the MDDL of the reservoir every year incase of the upstream inflow option. However, storage of the reservoir drops to the dead storage zone incase of Tendaho inflow source option for the year 1997 and 1998. The pool storage summary of the reservoir for the two inflow source options is as given in Table 6.4 below.

Flow source alterative	**Minimum pool storage(BM^3)**	**Maximum pool storage(BM^3)**	**Average pool storage(BM^3)**
Upstream flow sources	0.70	2.06	1.47
Tendaho gauging station	0.45	1.88	1.38

Table 6.4. Summary of the reservoir pool storages for the two inflow options and 1m gate opening alternative.

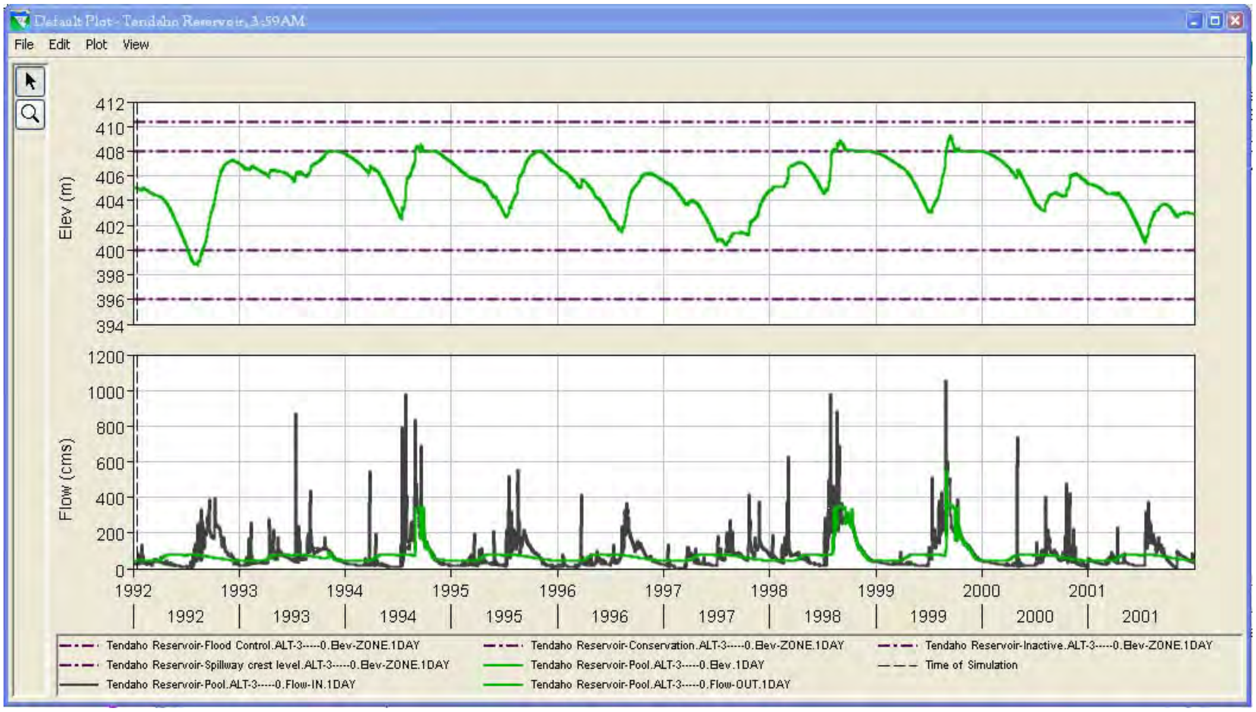

Figure 6.4. Tendaho reservoir pool level and inflow-outflow rate for upstream inflow option and maximum gate opening of 1m.

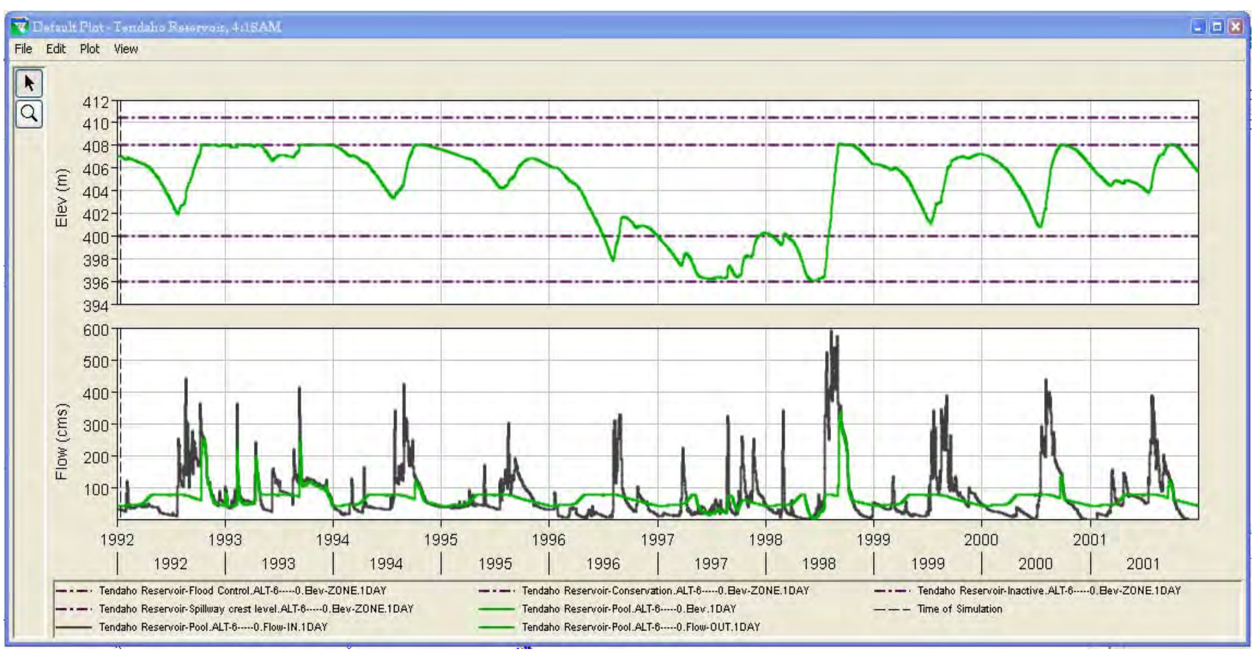

Figure 6.5. Tendaho reservoir pool level and inflow-outflow rate for Tendaho inflow option and maximum gate opening of 1m.

Reservoir storage for average level (4.0m) gate opening

For this gate setting alternative and upstream inflow source option, the reservoir pool level just crosses the bottom level of the flood control zone twice every five years (Figure 6.6) and hence operation of the side spillway, in addition to the irrigation outlet, becomes necessary. In this case also the reservoir stores water above the MDDL every year. However, the reservoir pool storage is decreased compared to the previous case. For Tendaho inflow source option storages similar to that of the 1m gate opening alternative occurs with a little variation and the graphical representation of the inflow-outflow and storages is presented in Figure 6.7. The pool storage summary is as given below.

Flow source alterative	Minimum pool storage(BM^3)	Maximum pool storage(BM^3)	Average pool storage(BM^3)
Upstream flow sources	0.70	1.90	1.47
Tendaho gauging station	0.45	1.88	1.38

Table 6.5. Summary of the reservoir pool storages for the two inflow options and 4m gate opening alternative.

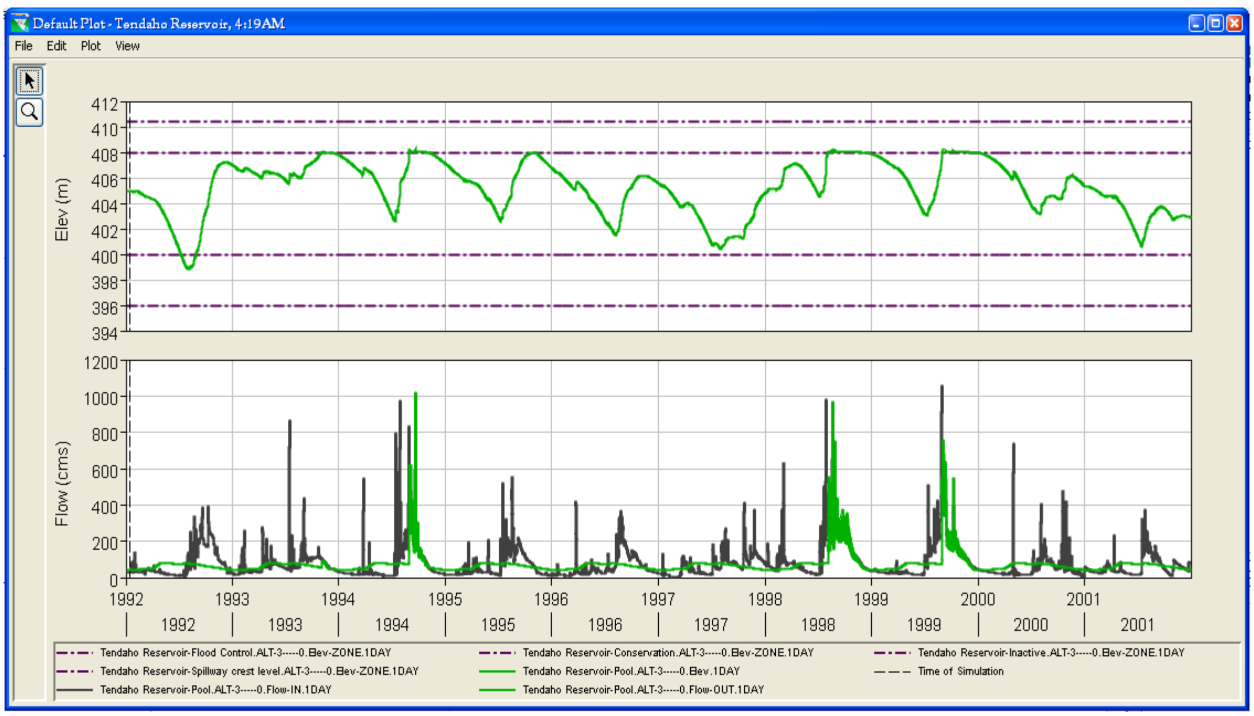

Figure 6.6. Tendaho reservoir pool level and inflow-outflow rate for upstream inflow option and maximum gate opening of 4m.

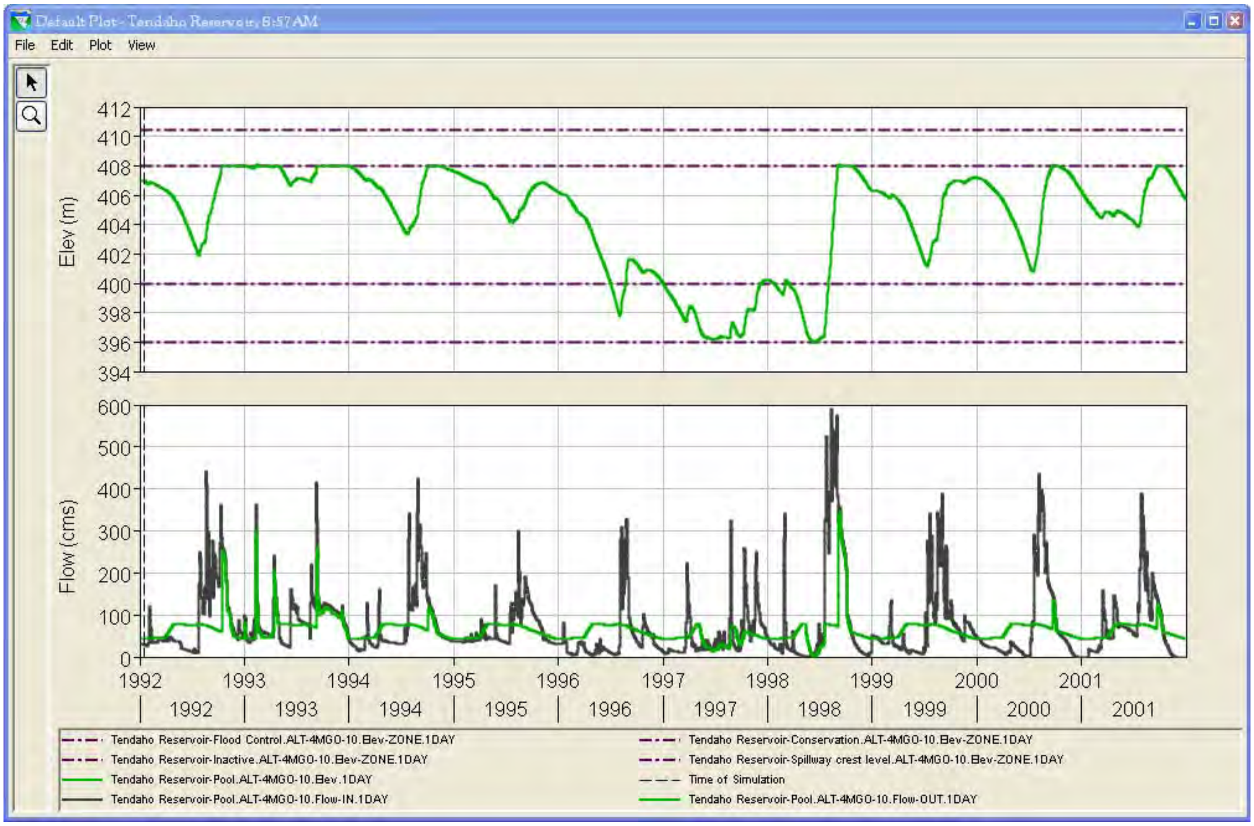

Figure 6.7. Tendaho reservoir pool level and inflow-outflow rate for Tendaho inflow option and maximum gate opening of 4m.

Reservoir storage for maximum level (6.9m) gate opening

This gate opening level is the maximum height the side spillway gate is raised to spill the design discharge of 1,700m^3/s. In this case also the reservoir pool level just crosses the bottom level of the flood control zone twice every five years (Figure 6.8) and hence operation of the side spillway, in addition to the irrigation outlet, becomes necessary. In this case also the reservoir stores water above the MDDL every year. However, the reservoir pool storage is minimized compared to the two previous cases. For the Tendaho inflow source option, the storages for the two previous alternatives are repeated (Figure 6.9)The pool storage summary is as given below.

Flow source alterative	Minimum pool storage(BM^3)	Maximum pool storage(BM^3)	Average pool storage(BM^3)
Upstream flow sources	0.70	1.90	1.47
Tendaho gauging station	0.45	1.88	1.38

Table 6.6. Summary of the reservoir pool storages for the two inflow options and 6.9m gate opening alternative.

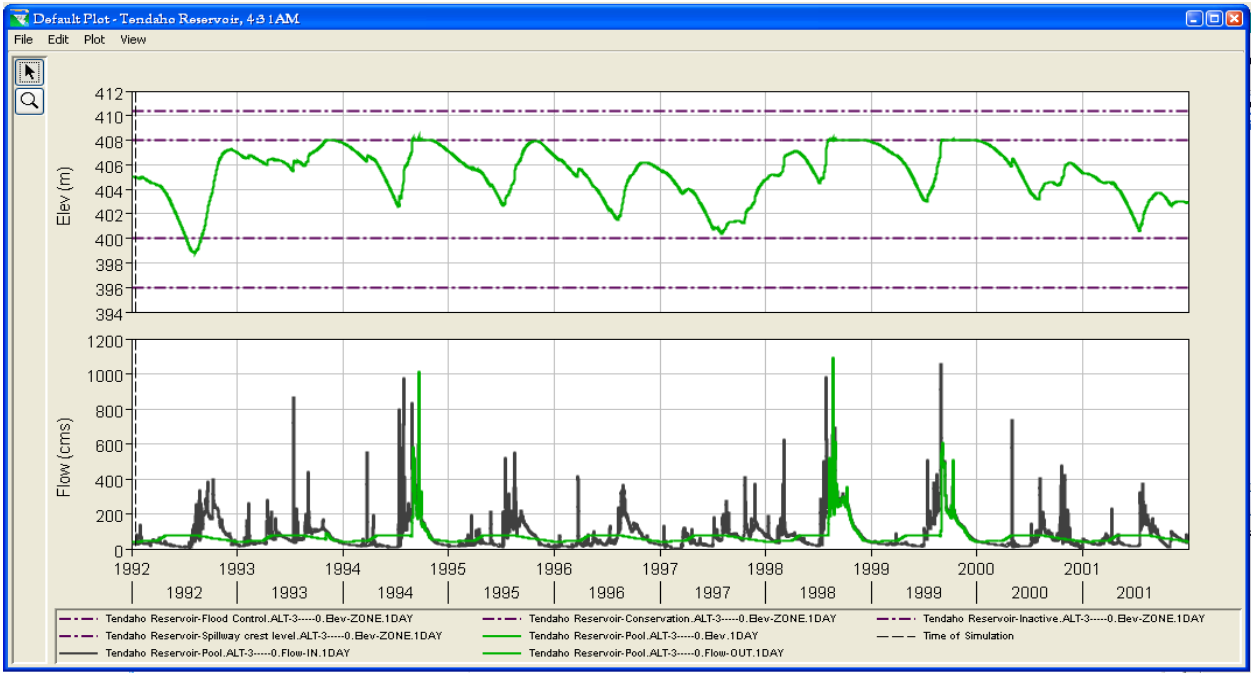

Figure 6.8.Tendaho reservoir pool level and inflow-outflow rate for upstream inflow option and maximum gate opening of 6.9m.

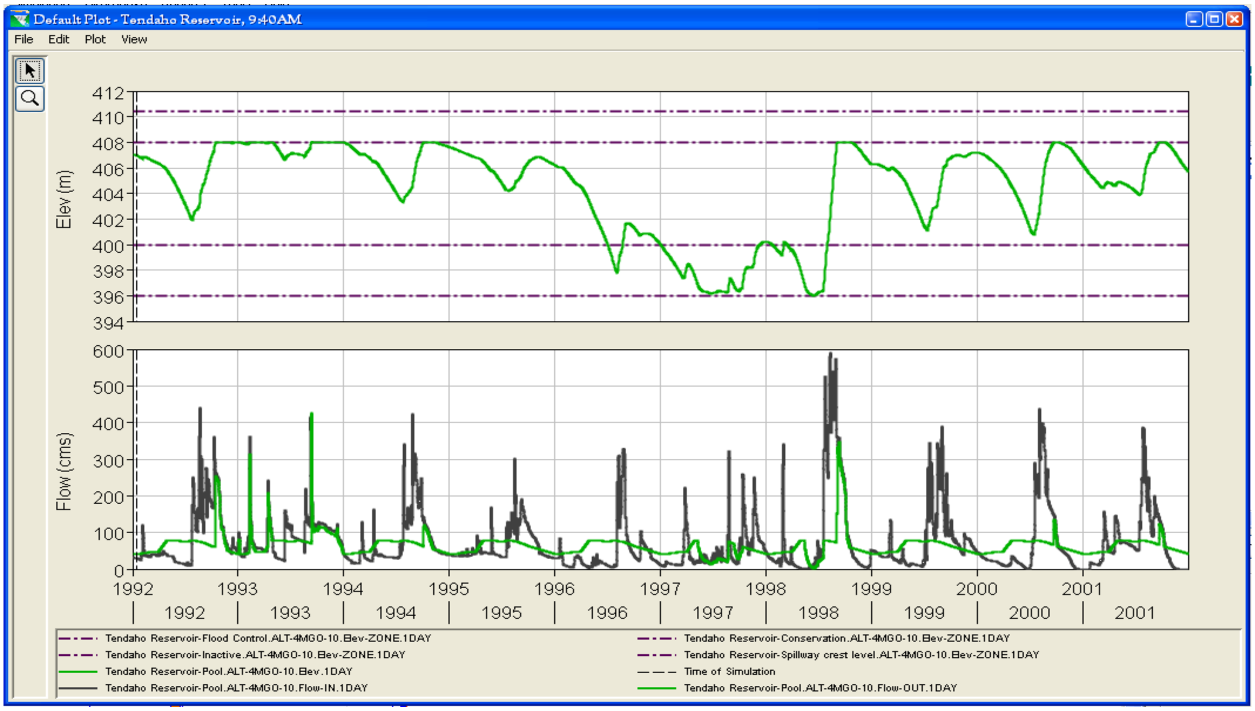

Figure 6.9.Tendaho reservoir pool level and inflow-outflow rate for Tendaho inflow option and maximum gate opening of 6.9m.

6.4. Tendaho reservoir releases

For the two inflow options considered in this study, the minimum and maximum releases through the irrigation outlet are similar for the three gate settings and follows similar pattern (Figure 6.10 and 6.11). The maximum and minimum releases through the irrigation outlet for the upstream inflow condition are 41.74m^3/s and 78m^3/s respectively. The values for Tendaho inflow option are 1.86m^3/s and 78m^3/s respectively.

In both the inflow options the spillway releases vary according to the gate settings considered. Incase of upstream inflow option, the spillway released for four wet seasons in the ten years simulation time. In these times, though the magnitude varies, the spillway releases for the three gate settings (Figures 6.12, 6.13 and 6.14). The release summary for the three gate settings applied is presented in Table 6.7. Incase of Tendaho gauging station inflow option, the spillway released for six wet seasons of the simulation period and for the three gate setting alternatives selected(Figure 6.15, 6.16 and 6.17). The release summary for this inflow option for the three gate settings selected is presented in Table 6.8.

The reservoir total releases for the two inflow options are also summarized as in Tables 6.9 and 6.10.

Gate setting	*Minimum release(m^3/s)*	*Maximum release(m^3/s)*
Opening 1.0m (14.5%)	0.00	282.35
Opening 4.0m (58.0%)	0.00	937.98
Opening 6.9m (100%)	0.00	1015.40

Table 6.7. Summary of spillway releases for the three gate settings and upstream inflow option.

Gate setting	*Minimum release(m^3/s)*	*Maximum release(m^3/s)*
Opening 1.0m (14.5%)	0.00	259.82
Opening 4.0m (58.0%)	0.00	267.73
Opening 6.9m (100%)	0.00	349.11

Table 6.8. Summary of spillway releases for the three gate settings and Tendaho inflow option.

Gate openings	*Minimum release(m^3/s)*	*Maximum release(m^3/s)*	*Average release(m^3/s)*
Opening 1.0m (14.5%)	41.74	360.35	73.62
Opening 4.0m (58.0%)	42.74	1015.98	72.60
Opening 6.9m (100%)	42.74	1093.40	72.60

Table 6.9. Summary of reservoir releases for the three gate settings and upstream inflow option.

Gate setting	*Minimum release(m^3/s)*	*Maximum release(m^3/s)*	*Average release(m^3/s)*
Opening 1.0m (14.5%)	1.86	337.82	63.43
Opening 4.0m (58.0%)	1.86	345.73	63.43
Opening 6.9m (100%)	1.86	427.11	63.43

Table 6.10. Summary of reservoir releases for the three gate settings and Tendaho inflow option.

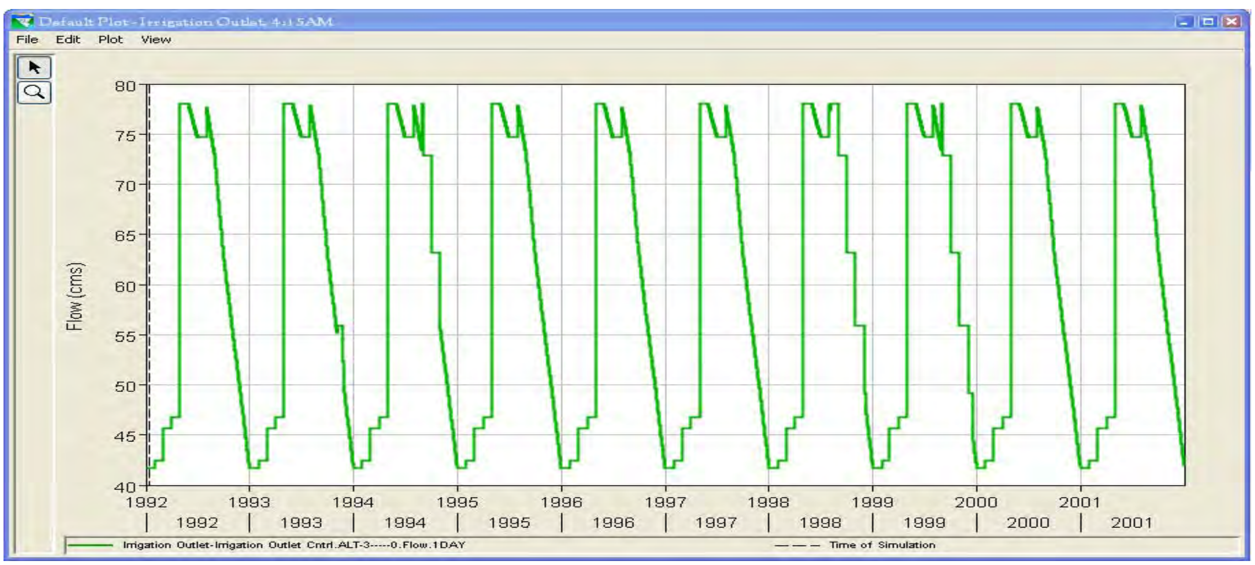

Figure 6.10. Irrigation outlet releases for upstream inflow options.

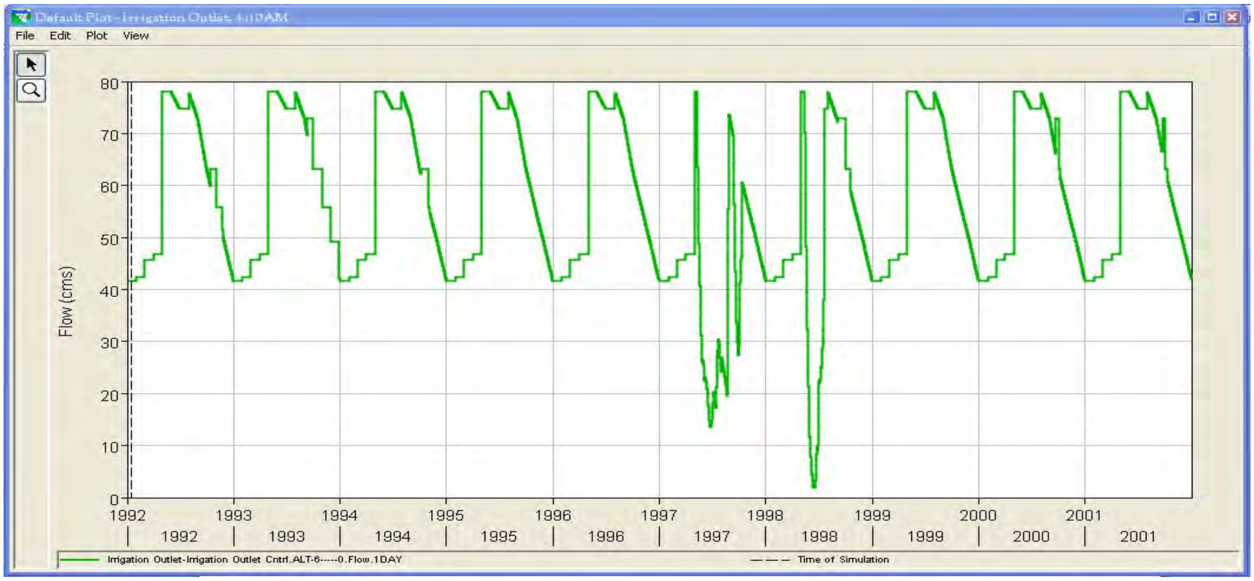

Figure 6.11. Irrigation outlet releases for Tendaho inflow options.

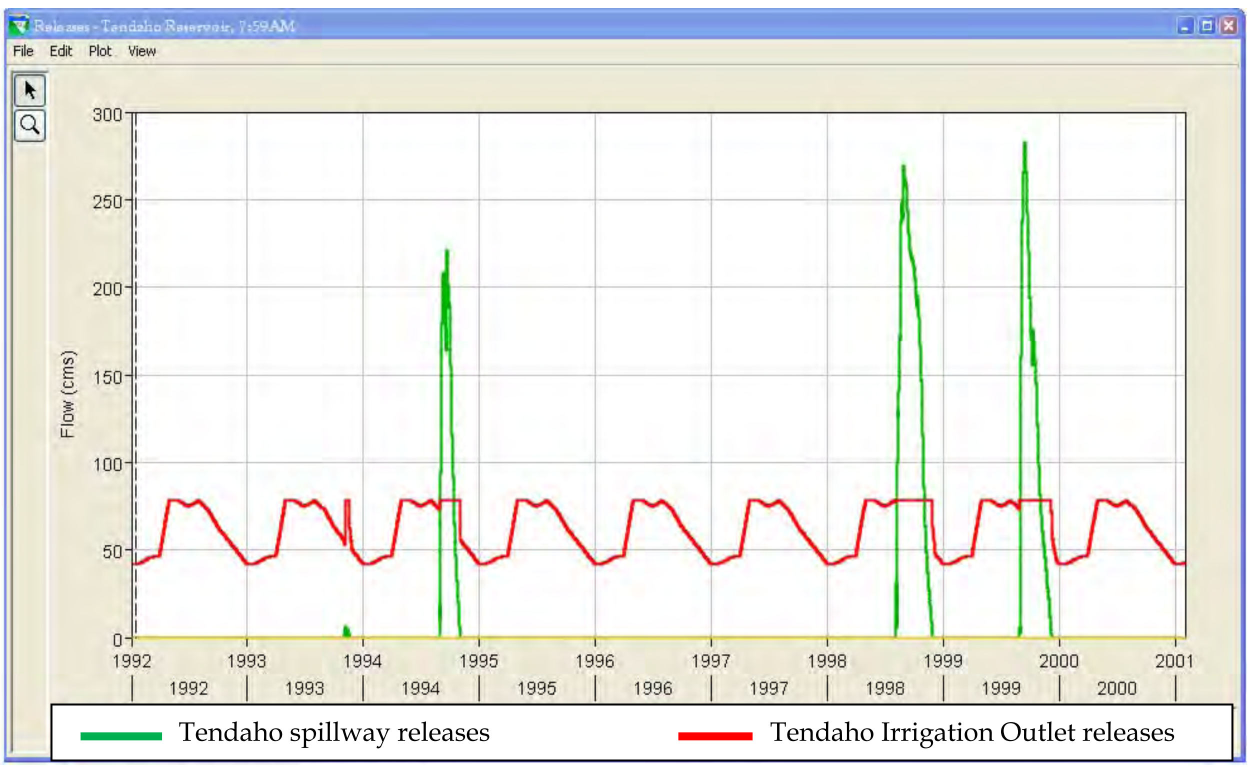

Figure 6.12. Reservoir releases for minimum level (1m) gate setting and upstream inflow.

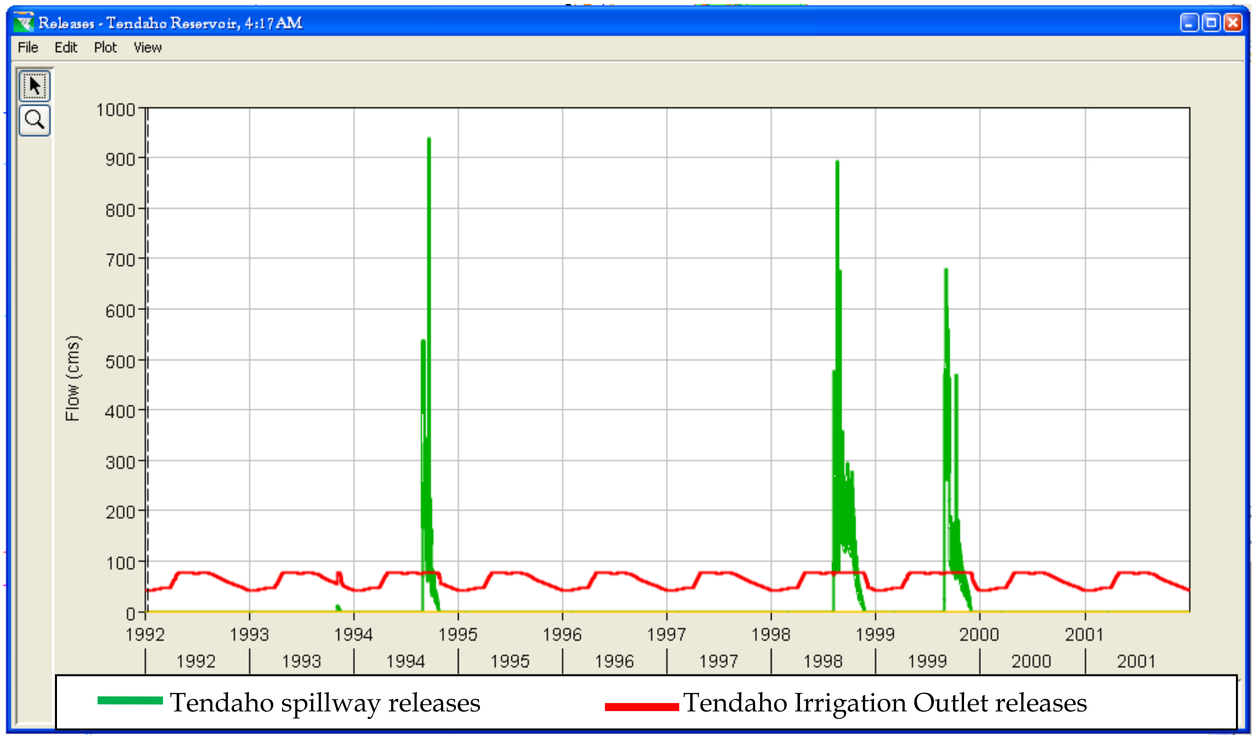

Figure 6.13. Reservoir releases for average level (4m) gate setting and upstream inflow.

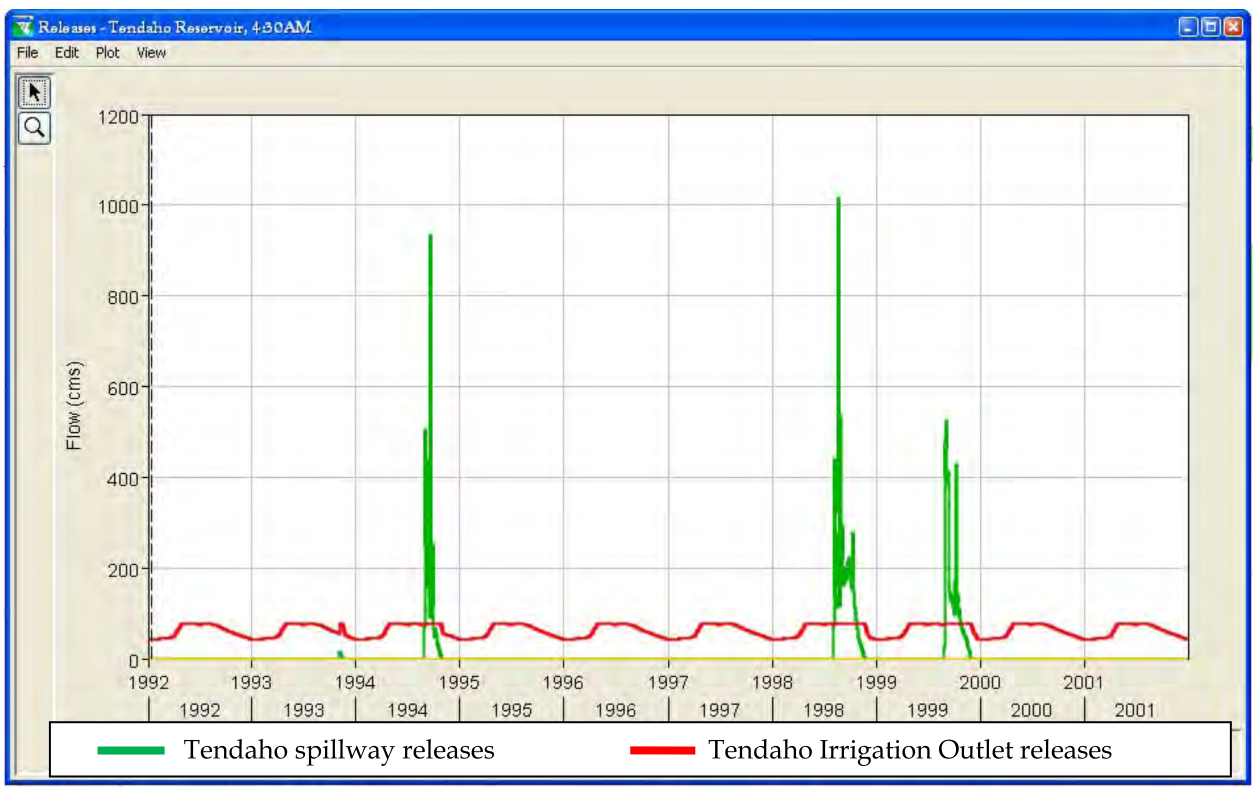

Figure 6.14. Reservoir releases for maximum level (6.9m) gate setting and upstream inflow.

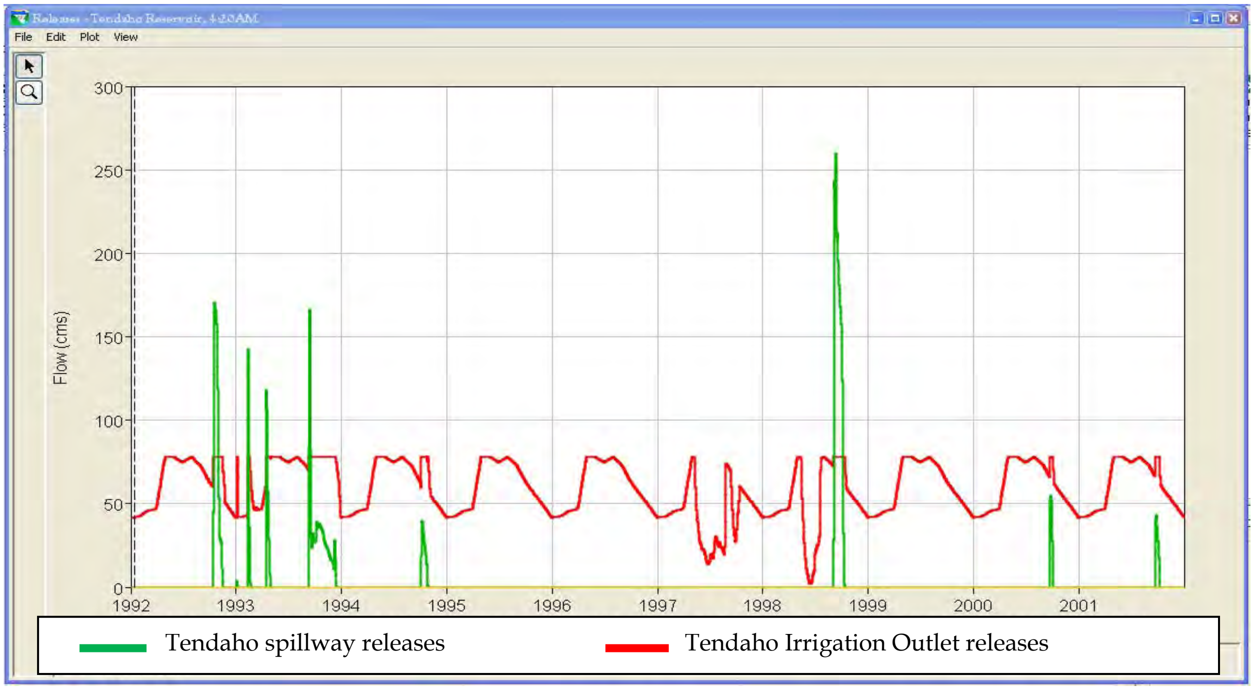

Figure 6.15. Reservoir releases for minimum level (1m) gate setting and Tendaho inflow..

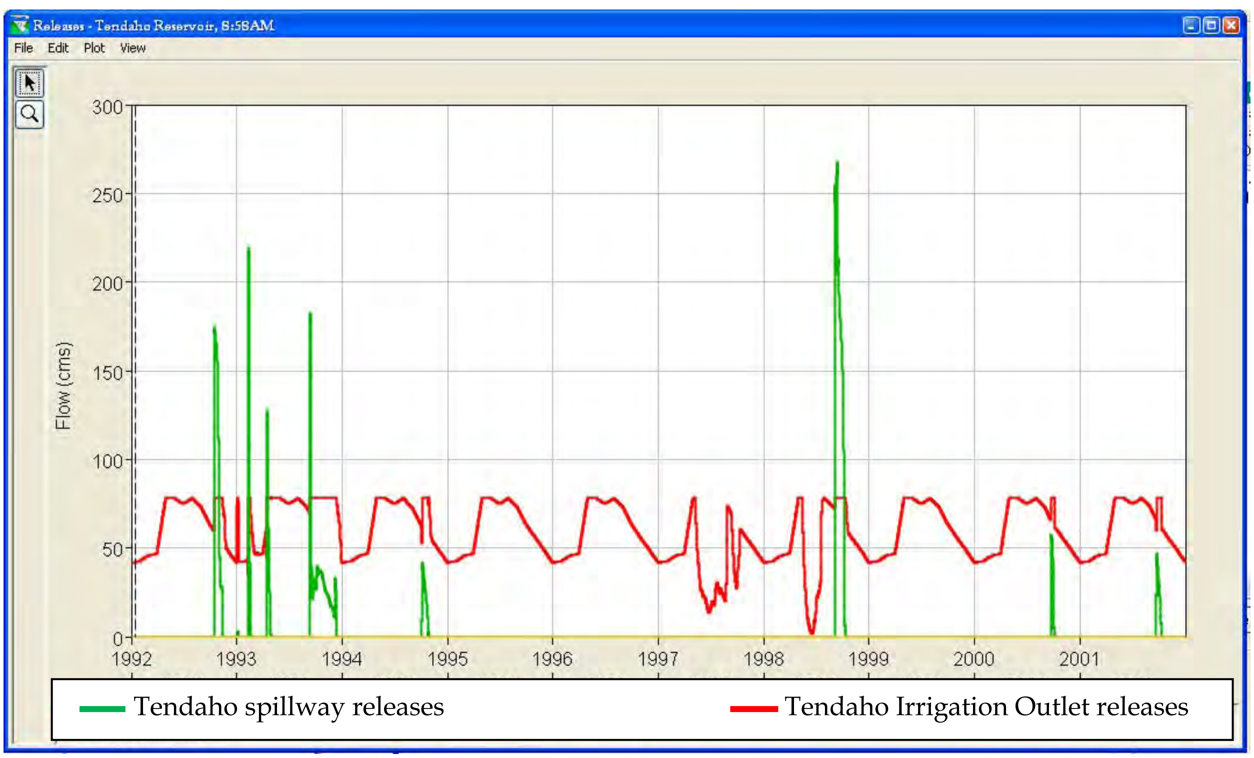

Figure 6.16. Reservoir releases for average level (4m) gate setting and Tendaho inflow.

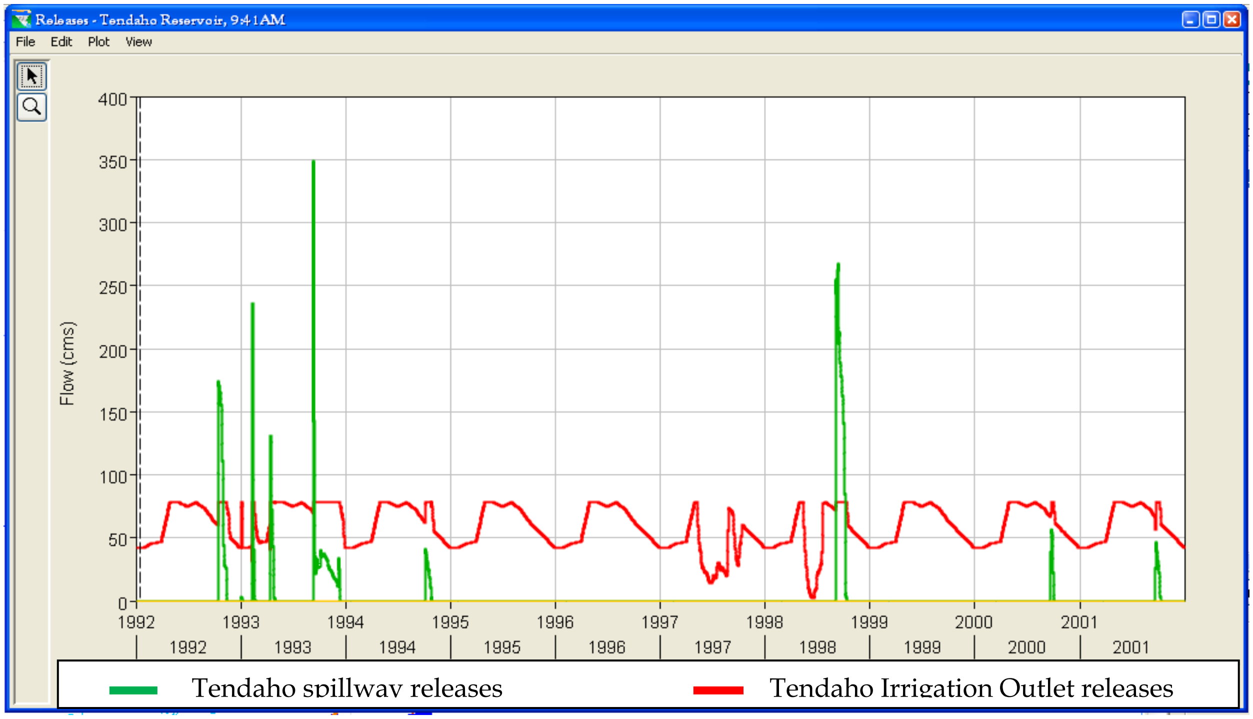

Figure 6.17. Reservoir releases for maximum level (6.9m) gate setting and Tendaho inflow.

As the reservoir has been planned to harness the wet season floods for utilization in the dry months of the year, situations increasing the reservoir storage and minimizing spillages are accepted. Thus, among the spillway gate settings considered, the minimum level gate opening alternatives of both the inflow source options are selected for further discussion as the maximum reservoir storages and minimum spillages occur in these cases.

6.5. Tendaho reservoir water balance

Reservoir water balance is defined by water balance equation that satisfies the basic continuity equation which implies the water inflow to system must be equal to the outflow from the system plus any change in storage within the system. Inflow is largely made up of rainfall and river inflow while outflow consists mainly of reservoirs outflow or release, and evaporation from open water surface. Groundwater flow is the component of inflow and outflow, but in this study this parameter has been ignored as it is negligible compared to the river or surface flow amounts. In this study reservoir water balance has been done for the two inflow source options considered on annual basis for minimum level of spillway gate operation and is indicated in Tables 6.11 and 6.12. The reservoir water balance results of both inflow options show that the gain volumes are in excess of the loss volumes for the ten years simulation.

Year	Initial storage(m^3)	In-flow(m^3)	Evaporation loss(m^3)	Out-flow(m^3)	Gain(m^3)	Loss(m^3)
1992	1.42E+09	1.41E+10	3.95E+08	1.23E+10	**1.55E+10**	**1.27E+10**
1993	1.71E+09	2.41E+09	3.95E+08	1.95E+09	**4.12E+09**	**2.34E+09**
1994	1.82E+09	2.90E+09	3.95E+08	2.67E+09	**4.72E+09**	**3.06E+09**
1995	1.72E+09	2.19E+09	3.95E+08	1.91E+09	**3.91E+09**	**2.31E+09**
1996	1.67E+09	2.06E+09	3.95E+08	1.92E+09	**3.73E+09**	**2.31E+09**
1997	1.50E+09	2.09E+09	3.95E+08	1.91E+09	**3.59E+09**	**2.31E+09**
1998	1.40E+09	4.38E+09	3.95E+08	3.59E+09	**5.78E+09**	**3.99E+09**
1999	1.84E+09	3.58E+09	3.95E+08	3.22E+09	**5.42E+09**	**3.62E+09**
2000	1.86E+09	1.85E+09	3.95E+08	1.92E+09	**3.71E+09**	**2.31E+09**
2001	1.47E+09	1.89E+09	3.95E+08	1.91E+09	**3.35E+09**	**2.31E+09**

Table 6.11. Reservoir water balance for upstream inflow option.

Year	Initial storage(m^3)	In-flow(m^3)	Evaporation loss(m^3)	Out-flow(m^3)	Gain(m^3)	Loss(m^3)
1992	1.71E+09	1.29E+10	3.95E+08	1.20E+10	**1.47E+10**	**1.24E+10**
1993	1.86E+09	2.90E+09	3.95E+08	2.54E+09	**4.76E+09**	**2.94E+09**
1994	1.86E+09	2.28E+09	3.95E+08	2.00E+09	**4.14E+09**	**2.40E+09**
1995	1.80E+09	2.03E+09	3.95E+08	1.91E+09	**3.84E+09**	**2.31E+09**
1996	1.59E+09	1.42E+09	3.95E+08	1.92E+09	**3.00E+09**	**2.31E+09**
1997	8.12E+08	1.61E+09	3.95E+08	1.38E+09	**2.43E+09**	**1.77E+09**
1998	8.41E+08	3.19E+09	3.95E+08	2.16E+09	**4.03E+09**	**2.56E+09**
1999	1.60E+09	2.36E+09	3.95E+08	1.91E+09	**3.97E+09**	**2.31E+09**
2000	1.74E+09	2.15E+09	3.95E+08	1.97E+09	**3.89E+09**	**2.37E+09**
2001	1.59E+09	2.22E+09	3.95E+08	1.98E+09	**3.81E+09**	**2.37E+09**

Table 6.12. Reservoir water balance for Tendaho inflow option.

6.6. Tendaho reservoir wet season operation rules

At the very beginning of this study, it was stated that the main objective of this research was developing wet season release rule for Tendaho reservoir on a daily basis operations. The wet season in the case of Tendaho reservoir was considered to be the duration spanning from 10th of June to 20th of November. This duration is the time gap in which the huge floods arrive at Tendaho. Thus, based up on the ten years simulation result obtained from HEC-ResSim model, the wet season reservoir operation rule was established for the minimum spillway gate operation alternatives of both inflow source options considered for comparison. The ten years wet season pool levels as well as the Lower, Upper and Average operation rule curves to be followed for operations of the irrigation outlet and the side spillway have been developed on excel spreadsheet taking the ten years wet season operations simulated by HEC-ResSim. The illustration of the ten years pool level and operation rule curves developed for the upstream and Tendaho inflow options are presented in Figures 6.18 and 6.19 and Figures 6.20 and 21 respectively.

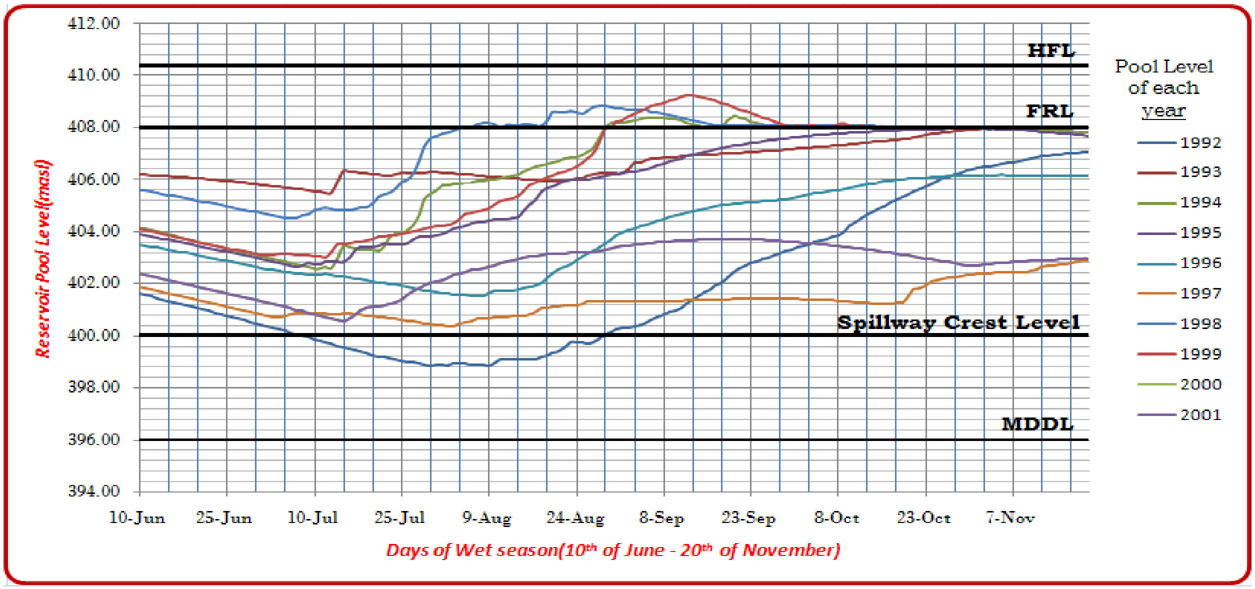

Figure 6.18. Reservoir pool level of wet season of each year for upstream inflow option.

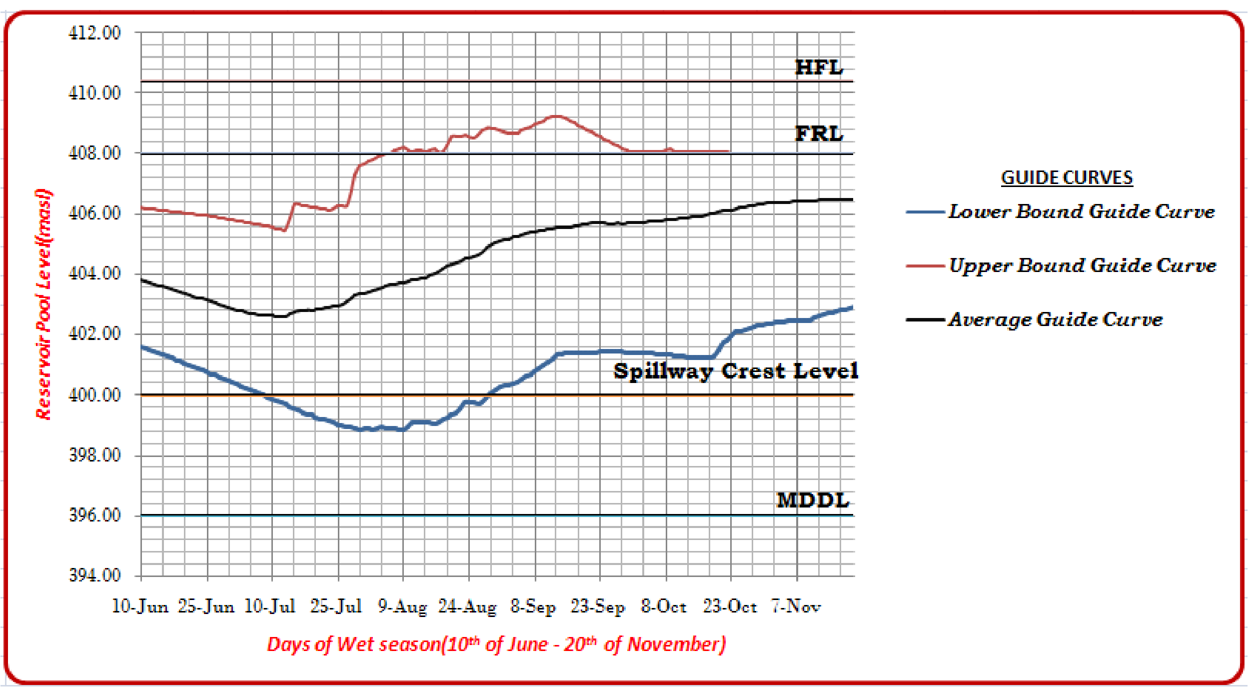

Figure 6.19. Wet Season Reservoir Operation Guide Curves for upstream inflow option.

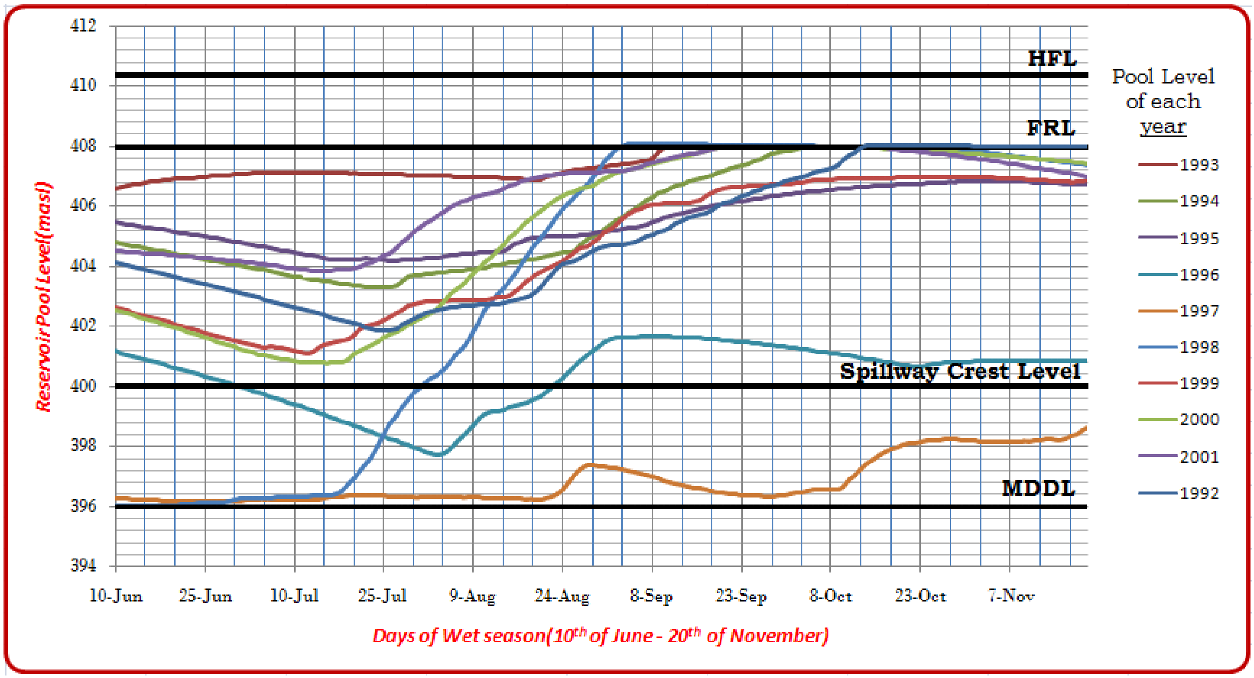

Figure 6.20. Reservoir pool level of wet season of each year for Tendaho inflow option.

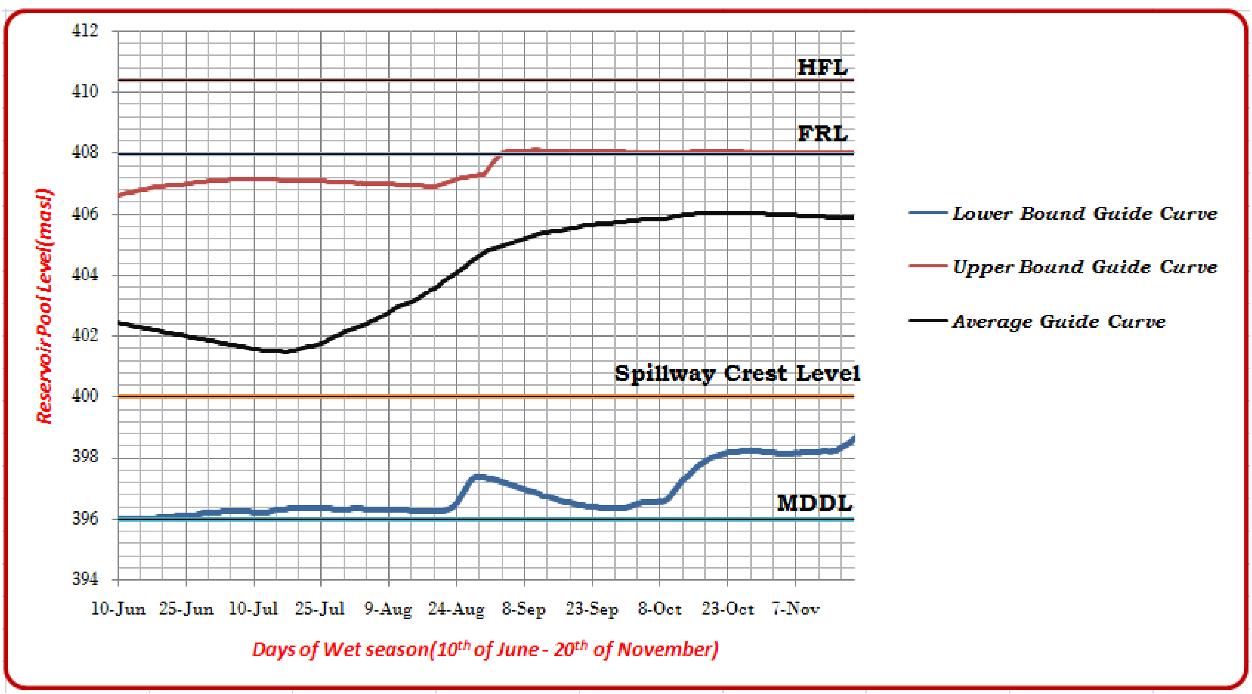

Figure 6.21. Wet Season Reservoir Operation Guide Curves for Tendaho inflow option.

Operations

In either of the inflow options considered, any of the three curves can be followed to operate the irrigation outlet and side spillway or the irrigation outlet alone for effective utilization of the reservoir impoundment. If the Lower Bound Guide Curve is to be followed, the spillway gates are closed and only the irrigation outlet is open to release the minimum irrigation and ecological flow requirements throughout the season. As the pool level is with in the Conservation Storage Zone, the same operation procedure is followed if the Average Guide Curve is to be applied. In these cases, the intake gate is adjusted at the beginning of each month based on the monthly irrigation and environmental requirements.

If the Upper Bound Guide Curve is to be followed, the irrigation outlet is open to release the minimum irrigation and ecological demands until the pool level reaches the reservoir's FRL (408.0 masl). For the pool level above the FRL, the irrigation outlet is operated to release a flow amounting to its maximum capacity (78m^3/s) and the three side spillway gates are opened to a height of 1.0m to withdraw the floods in excess of the irrigation outlet capacity.

Downstream flow conditions

As the minimum level gate setting alternative has been chosen to develop the final wet season operation guide curves, the wet season releases from the reservoir and contribution of Logiya river to the downstream river reach need consideration. The computation results of simulations made for the two inflow source options show that the maximum flow over the spillway is 282.35m^3/s for the upstream inflow option and 259.82 m^3/s for the Tendaho inflow. The corresponding release through the irrigation outlet is 78m^3/s, out of which 5.13m^3/s of flow is diverted to the river. From the hydrological data obtained, the flow recorded at Logiya gauging station on similar day was 6.82m^3/s for Tendaho inflow option and non for upstream inflow option. Thus, for the simulation period and operation rule curves selected the maximum flows that could occur at downstream of Logiya-Awash confluence of the river are some 287.48m^3/s for upstream inflow option and 271.77 m^3/s for Tendaho inflow option.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Conclusions

In this research work, attempts have been made to establish an operation guide rules that would enable operation of Tendaho dam using HEC-ResSim software. The model simulated the reservoir operations based on the ten years inflow data of two alternative inflow sources (upstream inflow sources and Tendaho gauging station), monthly irrigation and ecological requirements, monthly evapotranspiration rate and stream reach data of the selected tributaries.

In the upstream inflow option considered in this study the two major sources of inflow to Tendaho reservoir are Mille and Awash rivers. The daily flows of these two rivers recorded at Mille bridge and Adaitu have been routed to the reservoir inlet. At downstream of these gauging stations there are five ungauged catchments of different catchment areas contributing floods during wet seasons of the year. To account for these floods, SCS method was applied.

Calibration of SCS-CN method was made against the flows transposed from the Logiya catchment. The calibration statistical results show that the regression model used in all the tests does not have strong correlation with the transposed flow and it also has high MAE. Moreover, the SCS-CN method under predicts runoff in all the tests made. Generally, the statistical parameters used in calibration have not given sound results and this could be mainly attributed to poor performance of the flow transposition method applied and the SCS-CN method itself.

The computation result of the ten years simulation shows that there is no water scarcity for the irrigation and ecological requirements for the whole simulation period in case of upstream inflow option. However, for Tendaho inflow option there was high reservoir drown and hence water stress for irrigation during the years 1997 and 1998 as there were low inflows during the wet seasons of the respective years.

Reservoir operation guide curves for the wet season (10^{th} of June to 20^{th} of November) have been developed using excel spread sheet based on the ten years model computation results of both inflow options. The operation rules developed for the ten years data considered three side spillway gate setting alternatives: Opening to a height of 1.0m (Minimum Level Operation), Opening to a height of 4.0m (Average Level Operation) and Opening to a height of 6.9m (Maximum Level Operation).

The alternative that satisfies the purpose for which the reservoir has been planned is the minimum level operation in which the spillway gates are opened to maximum height of 1.0m to spill a minimal flow to downstream river reach keeping the irrigation outlet releasing to its maximum capacity of 78m^3/s. For low level gate setting, the spillway releases a maximum flood of 282.35m^3/s for upstream inflow option and 259.82m^3/s. The wet season reservoir Operation Guide Curves have been also developed for the best alternative selected. The three limiting operation guide curves are the Upper Guide Curve, the Lower Guide Curve and the Average Guide Curve.

Application of whichever the Operation Guide Curve developed depends on the in-flow condition. For wettest years, using the Upper Guide Curve is of advantageous and for driest years it is wise applying the Lower Guide Curve. For moderate years applying the Average Guide Curve is advantageous. In following any of the Guide Rules developed, the purposes for which the dam has been constructed are entirely satisfied. The only difference is the amount of floods stored in the flood control zone of the reservoir for the Upper Guide Curve.

The maximum and minimum releases through the irrigation outlet for the upstream inflow option are 41.74m^3/s and 78m^3/s respectively. These values correspond to the minimum monthly irrigation plus ecological requirements and the maximum outlet capacity respectively. For the months May and June the irrigation outlet discharges to its maximum capacity, flows equivalent to the irrigation demand of theses months. Thus, there is no flow to be diverted to the river for ecological purpose. Therefore, the lake complexes are replenished by Logiya flow for these two months.

The maximum and minimum releases through the irrigation outlet for the Tendaho inflow option are 1.86m^3/s and 78m^3/s respectively. The minimum release computed is by far less than the minimum irrigation plus environmental flow demands of any month.

For the side spillway gate setting alternative selected, the spillway releases which have caused the maximum flood at the downstream river reach for the upstream and Tendaho inflow options are 282.35m^3/s and 259.82m^3/s respectively. The corresponding release through the irrigation outlet is 78m^3/s, out of which only 5.13m^3/s is diverted to the river. The respective floods of

Logiya River joining Awash downstream of Tendaho dam on the same days are some 0.0m^3/s and 6.82m^3/s. Thus, for the simulation period and operation rule curves selected the maximum floods that could occur at downstream of Logiya-Awash confluence of the river are some 287.48m^3/s for upstream inflow option and 271.77m^3/s for Tendaho inflow option.

The reservoir Operation Guide Curves can be developed for even a lesser side spillway gate opening option as the flood control zone of the reservoir is large. Furthermore, partial gate opening option could also be considered to simulate the reservoir operations. However, the gate setting option considered in this research has been adopted because its application is simple and any one can operate the spillway gates with no complication.

Finally, the operation rule curves developed for Tendaho reservoir can be used as a guide to determine the amount of releases through the irrigation outlet and over the side spillway on daily basis operations. The Guide Curves developed are entirely dependent on the inflows. The most important inflow sources of Tendaho reservoir are Awash and Mille rivers. Thus, with the knowledge of the magnitudes of floods of these rivers at the key stations at the beginning of the wet season enables the reservoir operator to decide which of the curves to choose.

7.3. Recommendations

As Tendaho dam is a newly constructed structure, the operation modeling was developed with the hypothesis that the water resources problems (water scarcity during the dry months and flooding problems during the wet seasons) reported at various times will continue even after the establishment of the dam. Thus, the wet season reservoir operation rule developed in this research work can be used as a simple guide to make decisions on how much water to be released on each day of a wet season.

The Reservoir Operation Guide Curves developed can be deviated depending on the inflow conditions to return the reservoir pool level back to the Guide Curves.

The quality and continuity of the flow recordings at Adaitu and Mille gauging stations have to be seriously maintained so that whether the Upper Guide Curve or the Lower Guide Curve has to be chosen can be decided accordingly.

As the applicability of the developed operation rule entirely depends on the functionality of the irrigation outlet and the side spillway gates, there need to be a timely follow up of the proper functioning of these structures.

The spillway gate setting can be adjusted to open the gates to still a minimum height to hold back the floods in to the flood control zone of the reservoir without causing overtopping of the dam. However, it takes a long time trial and error procedure to arrive at the best alternative.

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APPENDICES

Appendix-1. Double mass curve analysis result of the selected stations

Figure 1.1. Mille Mean Annual RF before double mass analysis.

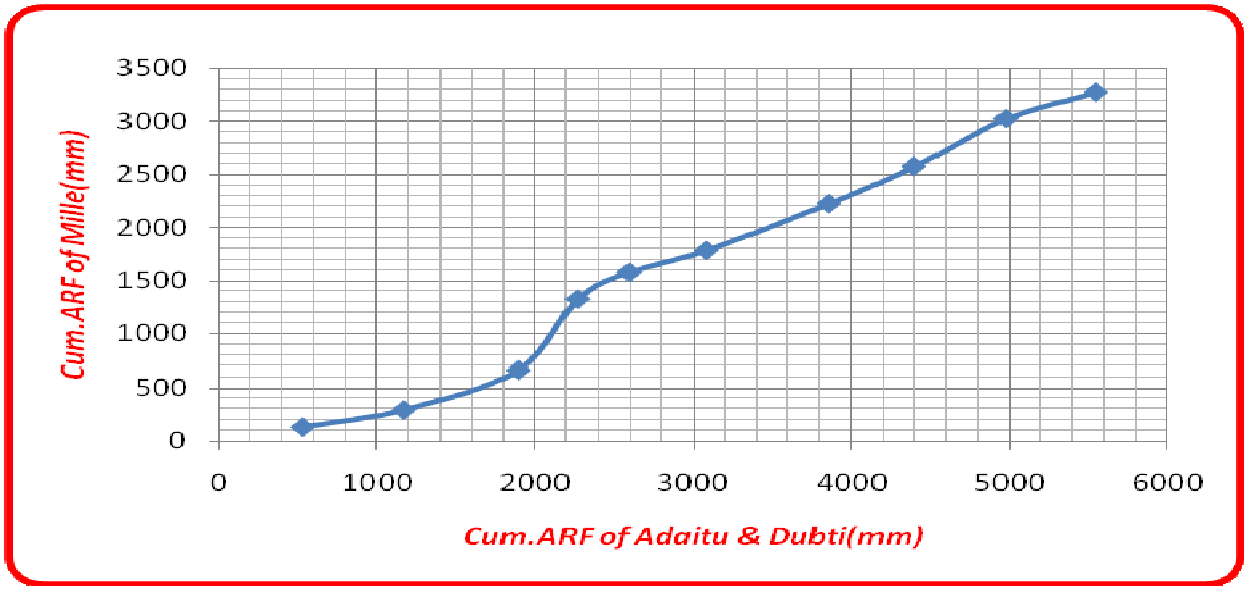


Figure 1.2. Mille Mean Annual RF after double mass analysis.

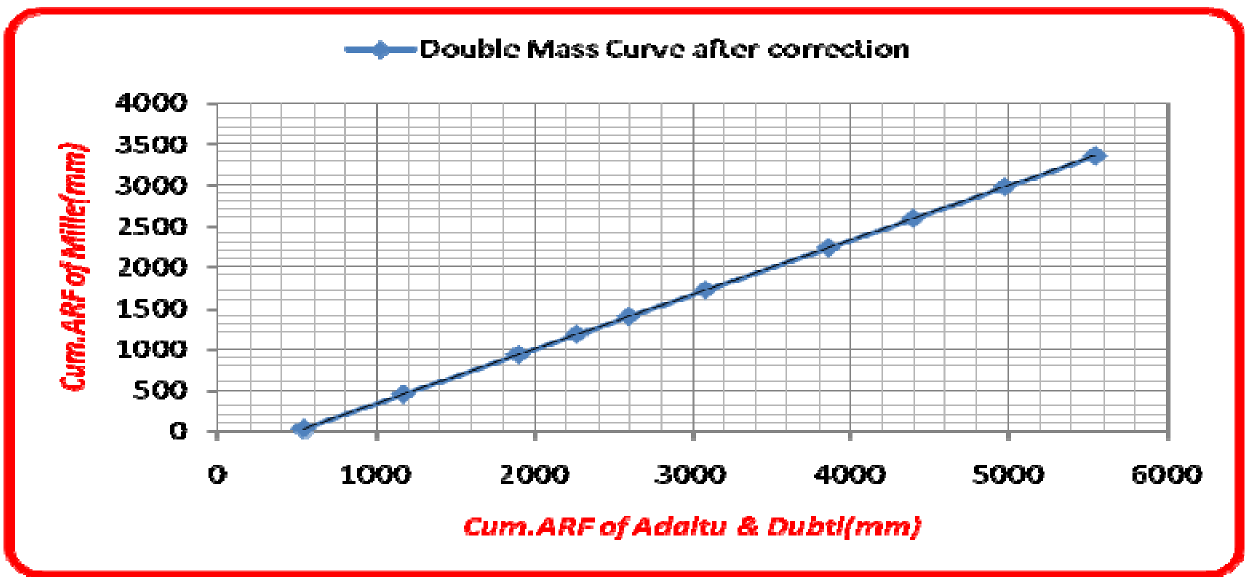


Figure 1.3. Dubti Mean Annual RF before double mass analysis (No need of correction).

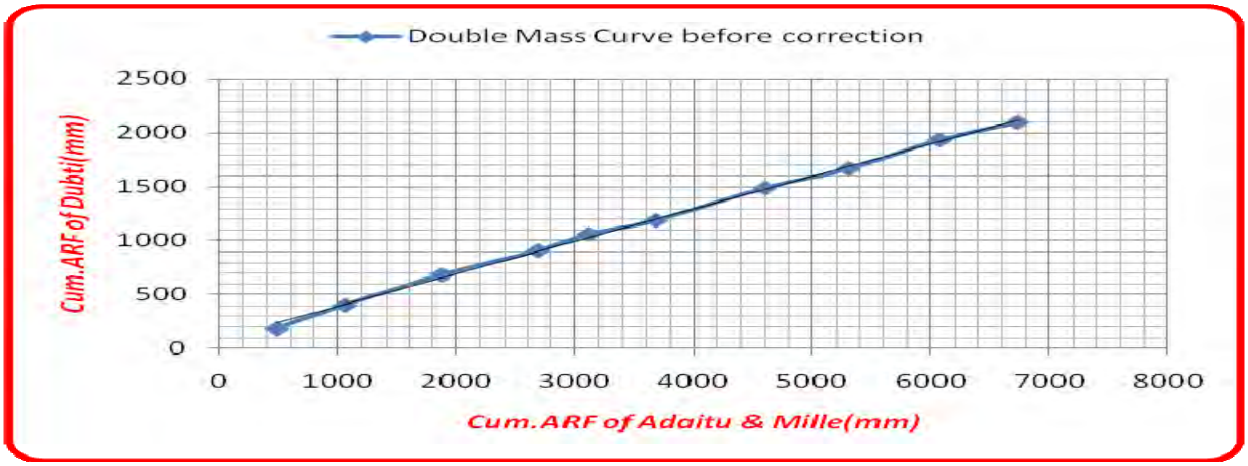


Figure 1.4. Bati Mean Annual RF before double mass analysis.

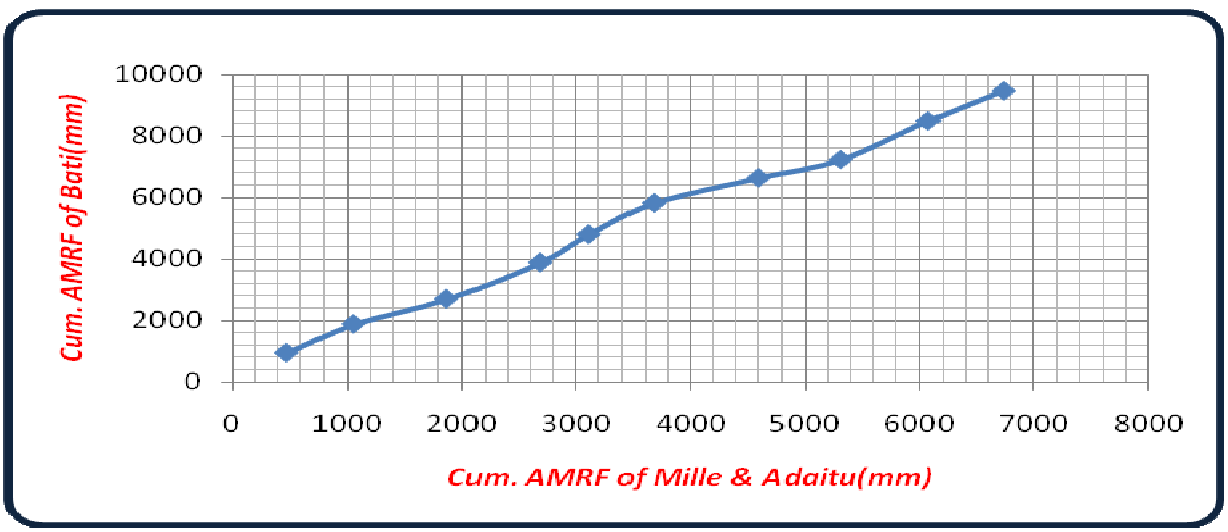
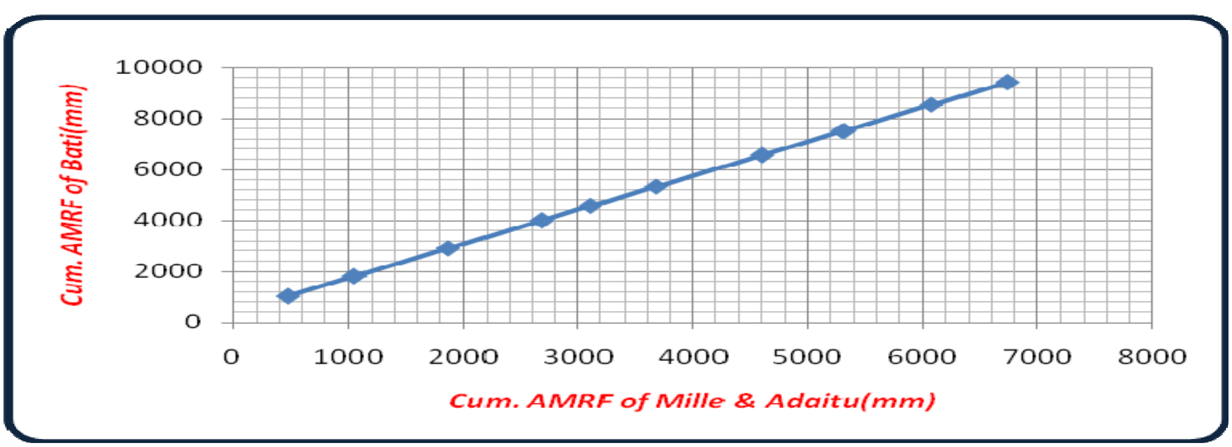


Figure 1.5. Bati Mean Annual RF after double mass analysis.



Appendix-2. Mean monthly rainfall of the selected stations after double mass analysis.

Table 2.1. Mean monthly rainfall (mm) of Adaitu station.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	0.00	0.00	0.00	35.31	0.00	0.00	89.93	238.49	35.31	11.77	0.00	0.00
1993	50.97	20.81	0.00	42.86	28.92	5.20	26.42	31.21	7.80	2.60	0.00	0.00
1994	0.00	42.43	42.43	16.97	0.00	0.00	136.62	75.01	54.31	0.00	6.52	2.38
1995	0.00	0.00	0.00	193.21	15.43	0.00	102.88	198.90	0.00	0.00	0.00	0.00
1996	0.00	0.00	68.02	39.62	44.90	11.48	33.68	27.07	0.00	7.26	0.00	0.00
1997	20.82	0.00	0.00	16.74	0.00	6.97	74.57	50.16	0.00	2.95	20.12	0.00
1998	4.10	16.21	101.34	2.77	14.98	0.00	118.49	117.16	29.93	21.00	0.00	0.00
1999	0.00	0.00	41.97	3.90	0.00	4.24	103.52	86.14	53.67	11.02	0.00	0.00
2000	0.00	0.00	35.31	5.00	3.40	5.36	140.59	144.38	28.25	28.77	19.62	0.00
2001	0.00	0.00	65.19	1.16	6.98	8.15	80.03	53.37	22.35	0.00	0.00	0.00

Table 2.2. Mean monthly rainfall (mm) of Mille station.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	10.09	2.22	0.00	0.00	0.00	0.00	0.15	14.54	0.17	1.37	0.00	0.00
1993	6.05	22.38	0.00	30.71	35.54	1.05	170.06	34.22	101.09	2.37	0.00	0.00
1994	0.00	0.00	82.67	11.37	14.60	0.00	222.06	62.01	78.28	0.00	4.52	6.46
1995	0.00	14.70	61.21	31.62	25.73	11.18	49.24	40.92	11.29	0.00	0.00	2.55
1996	0.69	0.00	48.97	0.60	23.28	13.36	41.90	59.41	2.16	0.00	22.33	0.00
1997	0.00	0.00	15.92	27.16	3.28	15.14	75.40	113.49	0.31	70.87	1.25	0.00
1998	5.73	10.87	69.18	2.80	14.84	0.00	161.97	149.82	54.46	45.58	0.00	0.00
1999	0.00	0.00	22.83	17.10	5.53	0.00	66.44	169.43	17.61	58.97	0.00	0.00
2000	0.00	0.00	0.00	39.39	0.00	0.00	84.85	154.29	37.33	58.57	11.56	0.00
2001	1.22	0.00	59.37	69.57	0.00	0.00	42.78	122.85	33.49	0.00	0.00	49.93

Table 2.3. Mean monthly rainfall (mm) of Dubti station.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	0.00	0.00	0.00	0.00	0.00	0.00	78.20	75.40	32.70	0.00	0.00	0.00
1993	19.60	27.10	0.00	56.00	25.00	0.00	52.53	27.30	9.20	0.40	0.00	0.00
1994	0.00	0.00	56.20	6.10	7.50	0.00	103.26	68.40	33.00	0.00	7.50	0.00
1995	0.00	17.30	4.90	11.90	6.10	16.70	58.04	99.30	11.05	0.00	0.00	0.00
1996	2.24	0.00	46.90	0.00	7.70	0.00	29.20	62.50	2.00	0.00	0.50	0.50
1997	39.10	0.00	9.70	0.00	12.30	4.80	5.00	21.80	13.20	21.80	0.00	0.00
1998	2.40	16.90	104.90	2.30	12.60	0.00	71.70	77.40	9.60	2.10	0.00	0.00
1999	0.00	0.00	41.00	6.30	0.00	11.40	31.50	54.00	1.90	33.80	0.00	0.00
2000	0.00	0.00	0.00	31.20	2.40	0.00	27.80	112.60	22.50	44.40	25.20	0.00
2001	0.00	0.00	30.20	0.50	22.90	0.00	56.60	35.20	17.40	0.70	0.00	0.00

Table 2.4. Mean monthly rainfall (mm) of Bati station.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	99.76	51.98	23.56	32.41	15.04	10.95	161.03	348.94	169.11	41.92	0.44	79.96
1993	74.48	85.01	0.00	201.33	95.20	4.39	168.60	26.93	55.93	63.80	0.00	0.00
1994	0.00	3.09	86.68	38.78	24.29	20.40	443.49	274.81	142.64	0.00	57.16	7.51
1995	0.00	42.86	93.27	125.43	80.46	28.29	248.75	324.41	101.93	13.46	0.00	32.26
1996	61.75	0.00	90.99	46.69	78.25	9.54	117.03	96.57	42.48	0.00	30.87	1.07
1997	24.86	0.00	104.33	27.91	11.83	38.92	161.93	101.80	28.06	103.59	157.10	0.00
1998	65.99	123.49	131.07	58.25	48.39	0.00	433.11	255.32	104.22	13.50	0.00	0.00
1999	57.79	0.00	16.69	38.51	12.36	0.00	57.79	371.81	201.39	191.80	1.05	4.01
2000	0.00	0.00	7.75	29.10	162.71	26.34	216.95	224.88	73.46	67.73	118.37	103.48
2001	5.31	11.43	207.20	21.95	70.80	34.91	253.43	154.92	96.17	5.40	0.00	22.02

Appendix-3.Curve Numbers Selection

Table 3.1. SCS Runoff Curve Numbers for Urban areas

Cover description

Cover type and hydrologic condition	Average % Impervious Area*	Curve number for hydrologic soil group			
		A	B	C	D
Fully developed urban areas (vegetation established):					
Open space (lawns, parks, golf courses, cemeteries, Etc.)+					
Poor condition (grass cover < 50%)		68	79	86	89
Fair conditon (grass cover 50 to 75%)		49	69	79	84
Good conditon (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding Right-of-way		98	98	98	98
Paved; open ditches (including right-of way)		83	89	92	93
Gravel (including right-of –way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)		63	77	85	88

Artificial desert landscaping (impervious weed
Barrier, desert shrub with 1-to 2-in sand Or gravel mulch and
basin borders

* The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in food hydrologic condition. CNs for other combinations of conditions may be computed.

CNs shown are equivalent to those of pasture. Composite CNs may be computed for other combination of open space cover type.

Composite CNs for natural desert landscaping should be computed based on the impervious are percentage (CN=98) and the pervious area CN. The pervious area CNs are assumed equivalent to desert shrub in poor hydrologic condition.

Composite CNs to use for the design of temporary measures during grading and construction should be based on the degree of development (impervious area percentage) and the CNs for the newly graded pervious areas.

Urban districts:		96	96	96	96
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas:					
Newly graded areas (previous areas only,					
No vegetation		77	86	91	94

Table 3.2. SCS Runoff Curve Numbers for cultivated agricultural areas.

Cover description			Curve numbers for Hydrologic soil group			
Cover type	**Treatment***	**Hydrologic Condition**	**A**	**B**	**C**	**D**
Fallow	Bare soil	-	77	86	91	94

	Crop residue cover(CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	poor	72	81	88	91
		Good	67	78	85	89
	SR+CR	poor	71	80	87	90

* Crop residue cover applies only if residue is on at least 5 percent of the surface throughout the year. Hydrologic condition is based on combination of factors that affect infiltration and runoff, including (i) density and canopy of vegetative areas, (2) amount of year-round cover, (3) amount of grass or close-seeded legumes in rotations, (4) percent of residue cover on the land surface (good≥ 20%), *and* (5) degree of surface roughness.

Poor; Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

	Good	64	75	82	86
Contoured(C)	Poor	70	79	84	88
	Good	65	75	82	86
C+CR	poor	69	78	83	87
	Good	64	74	81	85
Contoured and terraced(C&T)	poor	66	74	80	82
	Good	62	71	78	81
C&T+CR	poor	65	73	79	81
	Good	61	70	77	80
Small grain SR	poor	65	76	84	88
	Good	63	75	83	87
SR+CR	poor	64	75	83	86
	Good	60	72	80	84
C	poor	63	74	82	85
	Good	61	73	81	84
C+CR	poor	62	73	81	84
	Good	60	72	80	83
C&T	poor	61	72	79	82
	Good	59	70	78	81
C&T+CR	poor	60	71	78	81
	Good	58	69	77	80
Close seeded SR	poor	66	77	85	89
Or broadcast	Good	58	72	81	85
Legumes or C	poor	64	75	83	85
Rotation	Good	55	69	78	83
Meadow C&T	poor	63	73	80	83
	Good	51	67	76	80

Table 3.3. SCS Runoff Curve Numbers for other agricultural areas.

Cover description	Hydrologic Condition	Curve numbers for hydrologic soil group A	B	C	D
Pasture, grassland, or range- continuous forage	poor	68	79	86	89
For grazing*	Fair	49	69	79	84
	Good	39	61	74	80
Meadow – continuous grass, protected from Grazing and generally mowed for hay	_	30	58	71	78
Brush __ brush-weed-grass mixture with brush The major element	poor	48	67	77	83
	Fair	35	56	70	77
	Good	30	48	65	73
Woods-grass combination (orchard or tree farm)	poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
	Poor	45	66	77	83
Wood	Fair	36	60	73	79
	Good	30	55	70	77
Farmsteads __ buildings, lanes, driveways, and Surrounding lots	__	59	74	82	86

*poor :< 50% ground cover or heavily grazed with no mulch.

Fair: 50 to 75% ground cover and not heavily grazed.

Good :> 75% ground cover and lightly or only occasionally grazed.

Poor :< 50% ground cover.

Fair: 50 to 75% ground cover.

Good :> 75% ground cover.

CNs shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CNs for woods and pasture. Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning. Fair: Woods are grazed but not burned, and some forest litter covers the soil. Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 3.4. SCS Runoff Curve Numbers for Arid and semiarid range areas.

Cover description		Curve numbers for Hydrologic soil group			
Cover type	Hydrologic Condition*	A	B	C	D
Herbaceous __ mixture of grass, weeds, and	poor	80	87	93	
Low-growing brush, with brush the min	Fair	71	81	89	
Element	Good	62	74	83	
Oak-aspen – mountain brush mixture of oak	poor		66	74	79
Brush, aspen, mountain mahogany, bitter brush,	Fair		48	57	63
maple, and other brush	Good		30	41	48
Pinion-juniper __pinion, juniper, or both: grass	poor		75	85	89
understory	Fair		58	73	80
	Good		41	61	71
	Poor		67	80	83
Sagebrush with grass understory	Fair		51	63	70
	Good		35	47	55
Desert shrub__major plants include saltbush,	Poor	63	77	85	88
Greasewood, creosote bush, backrush, bursage,	Fair	55	72	81	86
Paid verde, mesquite, and cactus	Good	49	68	79	84

* poor:< 30% ground cover(litter, grass, and brush overstory).

Fair 30 to 70% ground cover.

Good:> 70% ground cover.

Curve numbers for group A have been developed only for desert shrub.

Appendix-4. SCS-CN method runoff generation.

Table 4.1. Standardized CN for each land use type of the ungauged catchments.

<u>Catchment</u>: ***Ledi***

Land use description	Catchment area(km^2)	Hydrologic soil group	CN for AMC(II)
Forest and bush	355.1	B	77
Bare land	33.2	B	86
Shrub land	423.1	B	85
Total	811.4		

<u>Catchment</u>: ***Geram***

Land use description	Catchment area(km^2)	Hydrologic soil group	CN for AMC(II)
Bare soil	229.00	B	86
Shrub land	49.00	B	85
Total	278.00		

<u>Catchment</u>: ***Weranso***

Land use description	Catchment area(km^2)	Hydrologic soil group	CN for AMC(II)
Forest and bush	211.3	B	77
Bare land	791.7	B	86
Shrub land	60.0	B	85
Total	*1063.0*		

<u>Catchment</u>: ***Gafura-Magalala***

Land use description	Catchment area(km^2)	Hydrologic soil group	CN for AMC(II)
Forest and bush	184.1	B	77
Bare land	496.9	B	86
Shrub land	402.0	B	85
Total	*1083.0*		

<u>Catchment</u>: ***Intervening catchment***

Land use description	Catchment area(km^2)	Cover condition	Hydrologic soil group	CN for AMC(II)
Bare soil	106	Poor	B	86
Dessert shrubs	80	poor	B	85
Total	***186***			

Table 4.2. Procedures for calculating daily runoff generated from the ungauged catchments

1	2	3	4	5			6	7	8	9
Days	*Daily Rainfall, R(mm)*	5days AM	AMC condition	CN(II) values for each land use type			CNavg	S(mm)	Q(mm)	$Q(m^3/s)$
				Forest and bush(1)	Bare land(2)	Shrub land(3)				
				76	85	84				
1-Jan-92	0.00	0.00	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
2-Jan-92	0.00	0.00	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
3-Jan-92	0.57	0.00	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
4-Jan-92	0.27	0.00	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
5-Jan-92	0.20	1.03	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
6-Jan-92	0.13	1.17	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
7-Jan-92	0.17	1.33	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
8-Jan-92	0.00	0.77	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
9-Jan-92	0.27	0.77	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
10-Jan-92	0.00	0.57	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
11-Jan-92	0.80	1.23	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
12-Jan-92	0.67	1.73	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
13-Jan-92	0.07	1.80	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
14-Jan-92	0.00	1.53	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
15-Jan-92	0.00	1.53	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
16-Jan-92	0.00	0.73	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
17-Jan-92	0.27	0.33	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
18-Jan-92	1.33	1.60	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
19-Jan-92	0.30	1.90	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
20-Jan-92	0.90	2.80	I	57.1	70.4	68.8	63.7	144.51	0.00	0.00
21-Jan-92	10.60	13.40	II	87.9	92.9	92.4	90.4	26.86	0.85	8.00
22-Jan-92	0.07	13.20	II	76.0	85.0	84.0	80.5	61.37	0.00	0.00
23-Jan-92	2.30	14.17	II	87.9	92.9	92.4	90.4	26.86	0.00	0.00
24-Jan-92	3.40	17.27	II	87.9	92.9	92.4	90.4	26.86	0.00	0.00
25-Jan-92	0.27	16.63	II	87.9	92.9	92.4	90.4	26.86	0.00	0.00
26-Jan-92	0.00	6.03	I	79.0	86.0	70.0	74.6	86.51	0.00	0.00
27-Jan-92	6.60	12.57	I	79.0	86.0	70.0	74.6	86.51	0.00	0.00
28-Jan-92	0.07	10.33	I	79.0	86.0	70.0	74.6	86.51	0.00	0.00
29-Jan-92	0.00	6.93	I	79.0	86.0	70.0	74.6	86.51	0.00	0.00
30-Jan-92	0.00	6.67	I	79.0	86.0	70.0	74.6	86.51	0.00	0.00
31-Jan-92	4.20	10.86	I	79.0	86.0	70.0	74.6	86.51	0.00	0.00

Steps followed in calculating runoff:

1. Column-1 Year of record and daily calendar in ascending order (1992, Jan-01 to 2001,Dec-31),
2. Column-2 Daily rainfall in (mm),
3. Column-3 the sum total of the previous five days consecutive rainfall values,
4. Column-4 Antecedent moisture condition according to the definition of SCS for the growing season referring the values in column 3 of the same day,
5. Column-5 Estimated curve number for each land use for each antecedent moisture condition based on curve number CN(II) by the formula applied to each conditions stated by SCS, $CN(I) = \left(\frac{4.2CN(II)}{10 - 0.058\,CN(II)} \right)$ and $CN(III) = \left(\frac{23CN(II)}{10 + 0.13CN(II)} \right)$
6. Column-6 The weighted curve number values for each antecedent moisture condition in column 5,

$$CN(I)_{avg} = \frac{CN(I)_1 * A_1 + CN(I)_2 * A_2 + ...CN(I)_n * A_n}{A_T})$$

Where:

CN $(I)_n$ is the curve number value estimated for each land use in column 5 for each antecedent moisture condition I,

A_n is the area of particular land use coverage

A_T is the total catchment area

7. Column-7 Values of S computed using the formula $S(mm) = 254 * \left(\frac{100}{CN} - 1 \right)$
8. Column-8 Values of runoff, $Q(mm)$ computed by using the formula

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

9. Column-9 Runoff values $Q(m^3/se)$ computed by using the formula

$$Q(m^3/s) = \frac{(Q(mm) * A_i(m^2)}{1000 * days(\sec)}$$

Where:

Q_i = The daily runoff values for each ungauged catchment (m^3/s),

A_i = The catchment area of each ungauged catchment (Km^2).

Appendix-5. Monthly flow generated from the ungauged catchments

Table 5.1. Monthly flow data of Geram catchment

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	0.01	0.00	0.14	0.00	0.00	0.00	0.42	1.71	0.20	0.00	0.00	0.03
1993	0.27	0.07	0.00	0.86	0.00	0.00	4.70	0.00	1.38	0.22	0.00	0.00
1994	0.00	0.07	1.34	0.00	0.00	0.00	6.46	0.74	1.51	0.00	0.00	0.00
1995	0.00	0.00	1.45	0.15	0.00	0.00	6.70	1.60	0.01	0.00	0.00	0.00
1996	0.00	0.00	1.20	0.00	0.48	0.05	0.77	0.01	0.03	0.00	0.09	0.00
1997	0.00	0.00	0.05	0.04	0.00	0.00	0.79	0.29	0.00	0.88	2.51	0.00
1998	0.00	1.10	0.71	0.00	0.00	0.00	2.77	0.83	0.10	0.10	0.00	0.00
1999	0.00	0.00	0.00	0.00	0.00	0.06	1.06	2.72	0.02	0.61	0.00	0.00
2000	0.00	0.00	0.00	0.00	3.75	0.00	2.41	2.90	0.00	0.26	1.41	0.31
2001	0.00	0.00	0.33	0.38	0.20	0.00	2.23	0.50	0.11	0.00	0.00	0.09

Table 5.2. Monthly flow data of Weranso catchment

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	0.01	0.00	0.42	0.00	0.00	0.00	1.27	5.67	0.63	0.00	0.00	0.08
1993	0.92	0.18	0.00	2.83	0.00	0.00	16.91	0.00	4.97	0.69	0.00	0.00
1994	0.00	0.18	4.66	0.00	0.00	0.00	23.04	2.38	5.33	0.00	0.00	0.00
1995	0.00	0.00	4.92	0.46	0.00	0.00	23.83	5.38	0.01	0.00	0.00	0.00
1996	0.00	0.00	4.07	0.00	1.59	0.12	2.64	0.01	0.08	0.00	0.24	0.00
1997	0.00	0.00	0.13	0.09	0.00	0.00	2.61	0.93	0.00	2.92	8.92	0.00
1998	0.00	3.81	2.33	0.00	0.00	0.00	9.41	2.76	0.29	0.30	0.00	0.00
1999	0.00	0.00	0.00	0.00	0.00	0.16	3.39	9.27	0.04	2.11	0.00	0.00
2000	0.00	0.00	0.00	0.00	13.84	0.00	8.12	10.04	0.00	0.83	4.98	0.97
2001	0.00	0.00	1.04	1.24	0.62	0.00	7.81	1.61	0.31	0.00	0.00	0.26

Table 5.3. Monthly flow data of Gafura-Magalala catchment

	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001
Jan	0.00	0.00	0.00	0.00	0.00	0.24	0.00	0.00	0.00	0.00
Feb	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Mar	0.00	0.00	12.29	0.52	4.51	0.00	16.59	0.00	0.00	0.02
Apr	0.00	3.18	0.00	0.00	0.00	0.00	0.00	0.00	0.09	6.76
May	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00
Jun	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Jul	0.17	22.66	19.20	2.06	0.00	0.01	6.70	0.00	0.00	0.25
Aug	0.08	0.00	1.09	0.37	0.01	5.90	5.86	6.74	14.07	4.19
Sep	0.00	7.98	8.58	0.00	0.00	0.00	0.29	0.00	0.15	0.00
Oct	0.00	0.00	0.00	0.00	0.00	6.70	0.74	4.45	8.76	0.00
Nov	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.15	0.00
Dec	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.79

Table 5.4. Monthly flow data of the intervening catchment

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1992	0.00	0.00	0.00	0.00	0.00	0.00	0.07	0.04	0.00	0.00	0.00	0.00
1993	0.00	0.00	0.00	0.62	0.00	0.00	4.06	0.00	1.46	0.00	0.00	0.00
1994	0.00	0.00	2.26	0.00	0.00	0.00	3.59	0.27	1.57	0.00	0.00	0.00
1995	0.00	0.01	0.15	0.00	0.00	0.00	0.40	0.13	0.00	0.00	0.00	0.00
1996	0.00	0.00	0.91	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00
1997	0.08	0.00	0.00	0.01	0.00	0.00	0.01	1.09	0.00	1.25	0.00	0.00
1998	0.00	0.00	3.02	0.00	0.02	0.00	1.35	1.13	0.10	0.20	0.00	0.00
1999	0.00	0.00	0.01	0.00	0.00	0.00	0.00	1.35	0.00	0.84	0.00	0.00
2000	0.00	0.00	0.00	0.04	0.00	0.00	0.00	2.67	0.06	1.59	0.06	0.00
2001	0.00	0.00	0.02	1.24	0.00	0.00	0.10	0.84	0.00	0.00	0.00	0.21

Figure 5.1. Monthly flow hydrograph of flow from Geram catchment

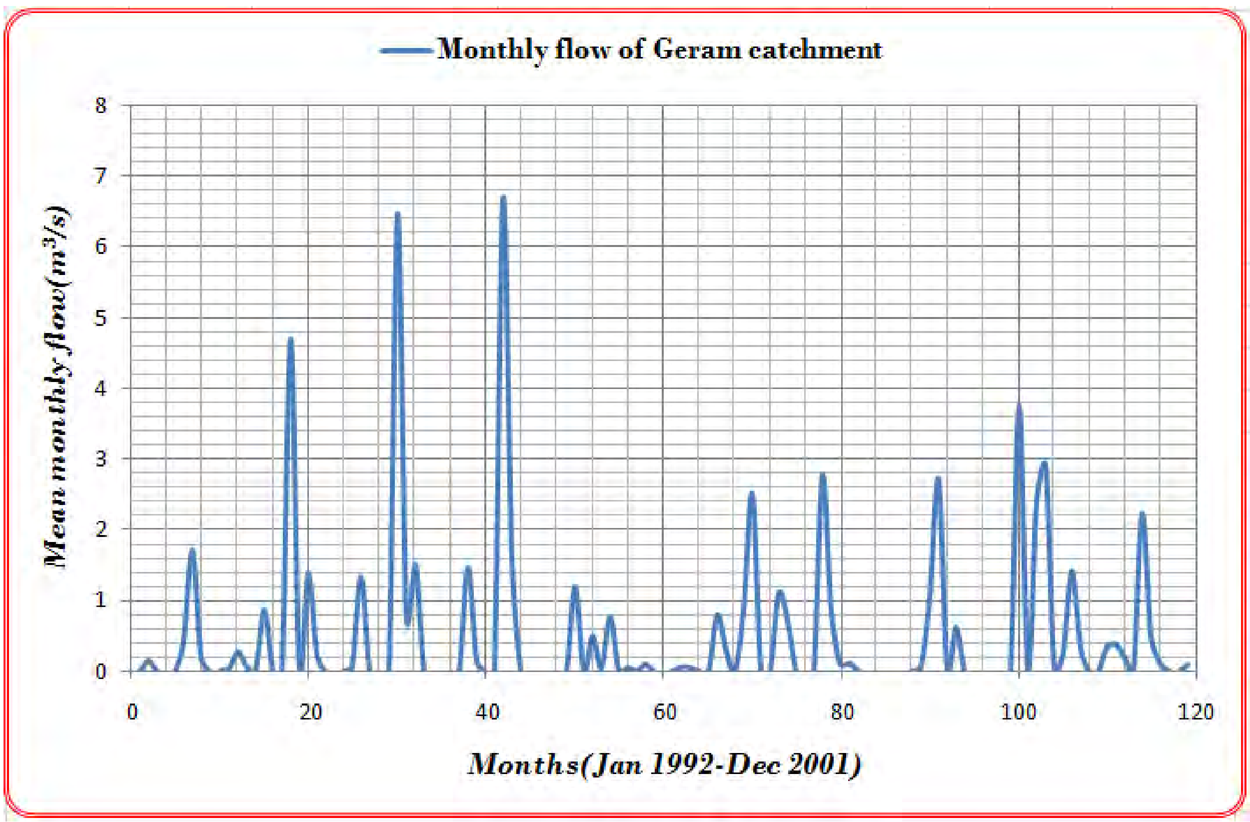


Figure 5.2. Monthly flow hydrograph of flow from Weranso catchment

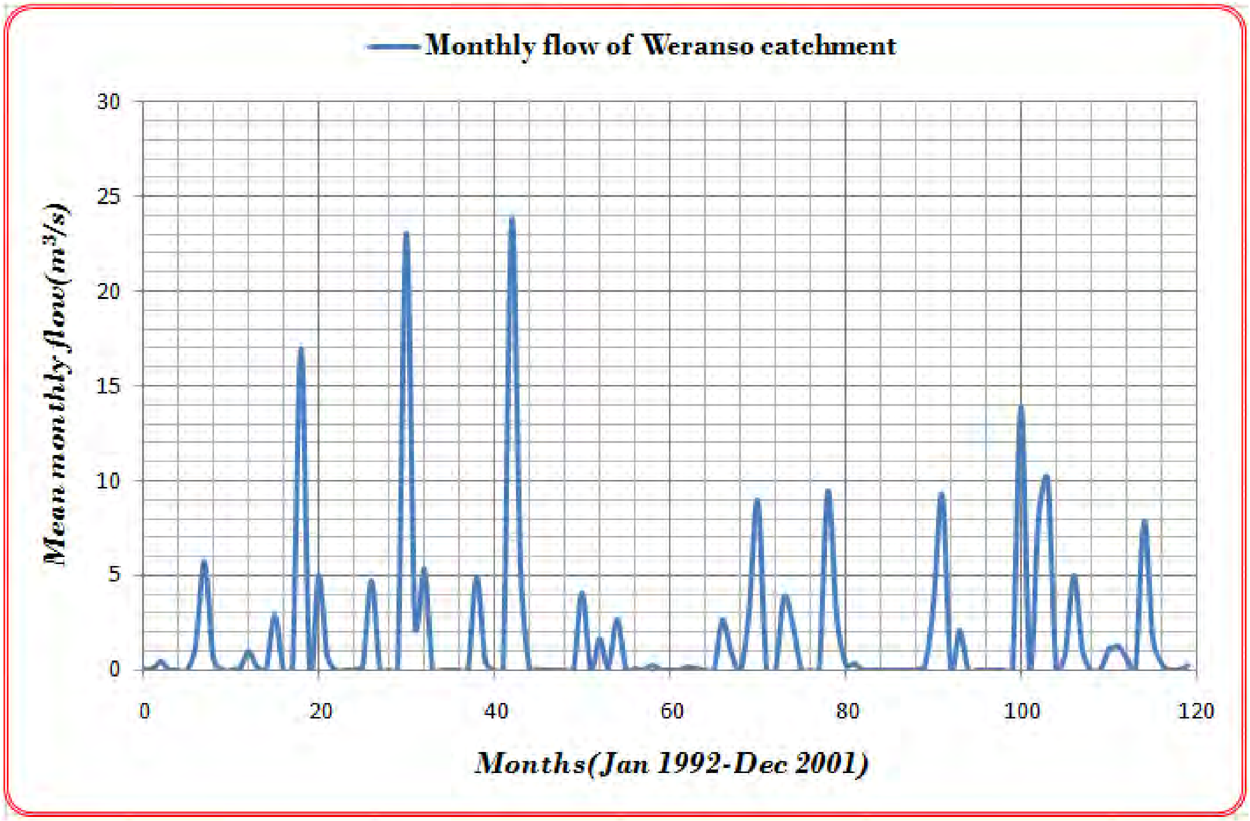


Figure 5.3. Monthly flow hydrograph of flow from Gafura-Magalala catchment

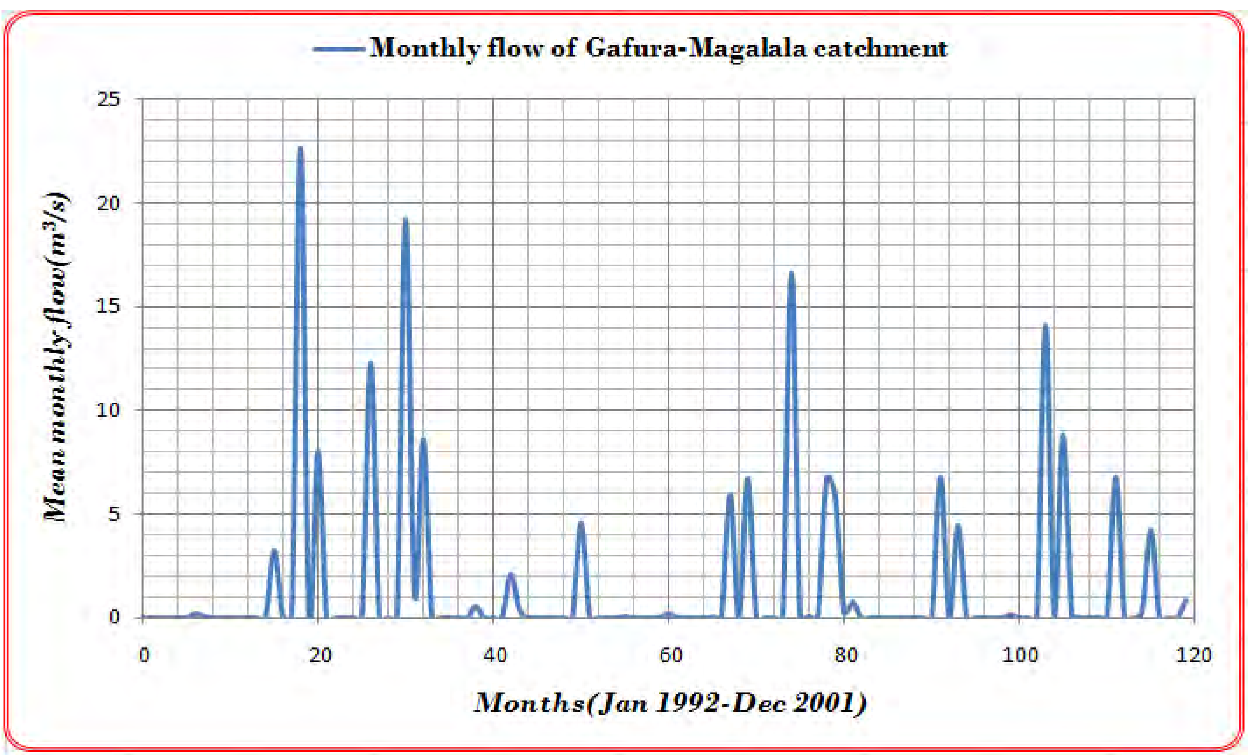
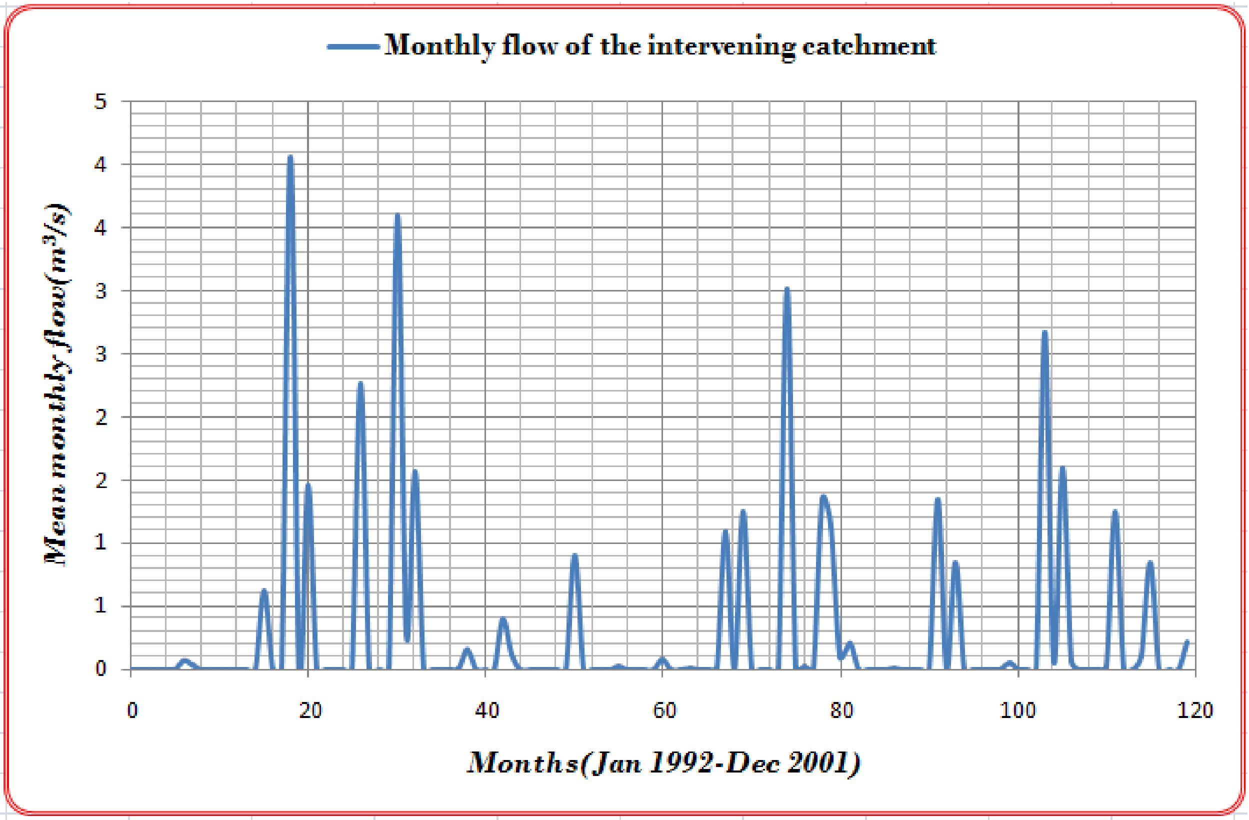


Figure 5.3. Monthly flow hydrograph of flow from the intervening catchment



Appendix-6. Physical and Operational parameters of Tendaho dam.

Figure 6.1. Physical data of Tendaho reservoir.

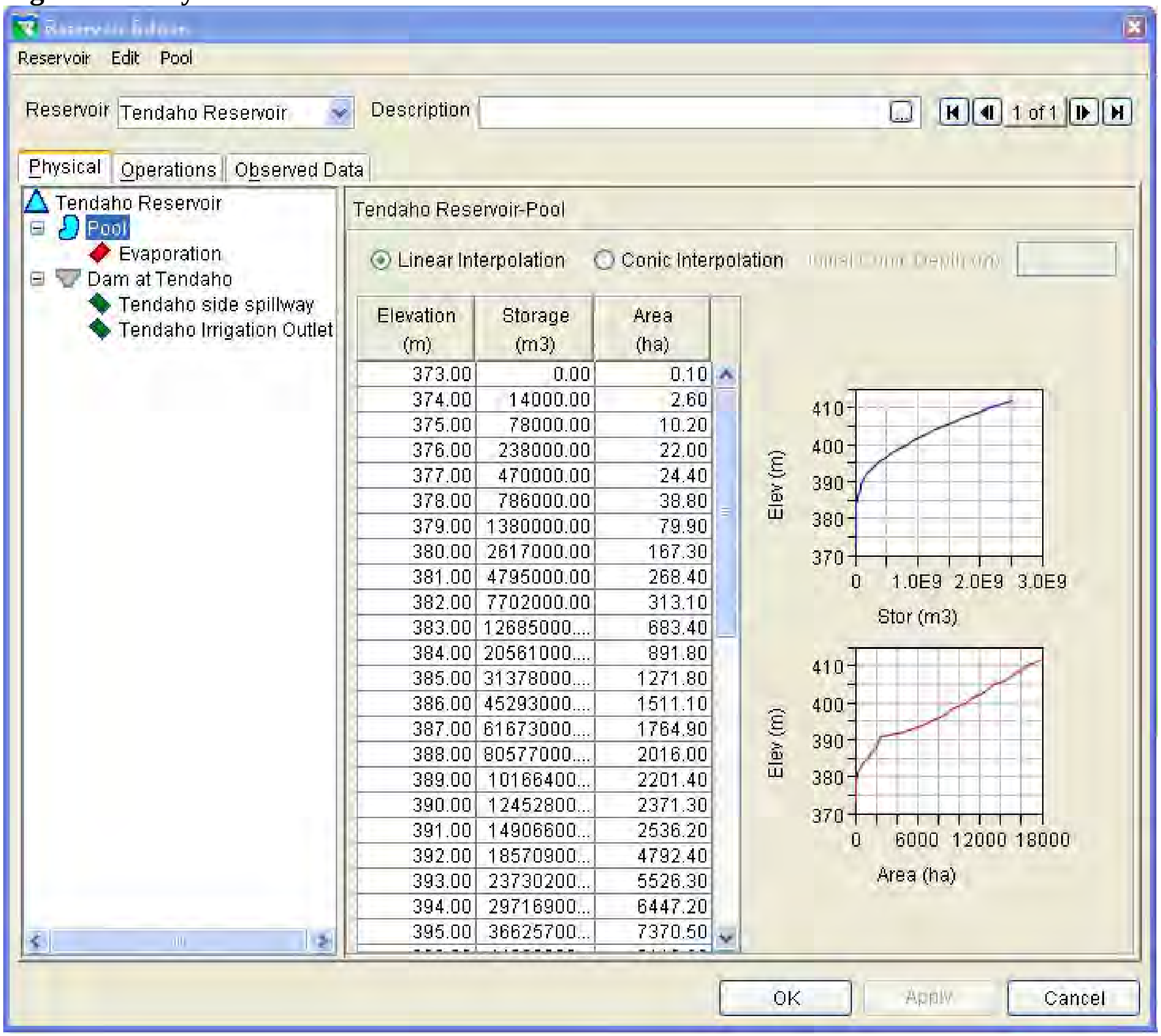


Figure 6.2. Evaporation data of Tendaho reservoir.

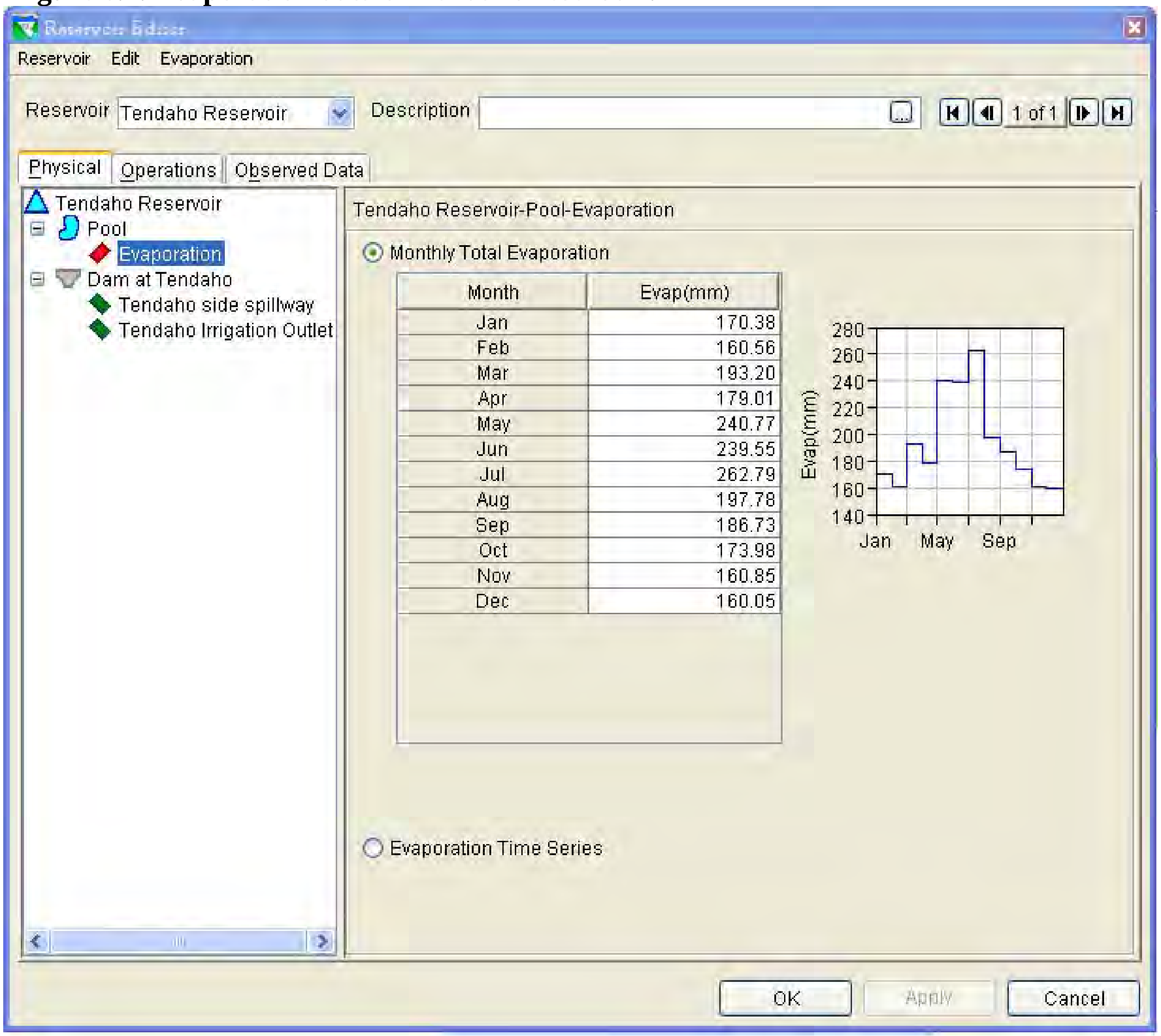
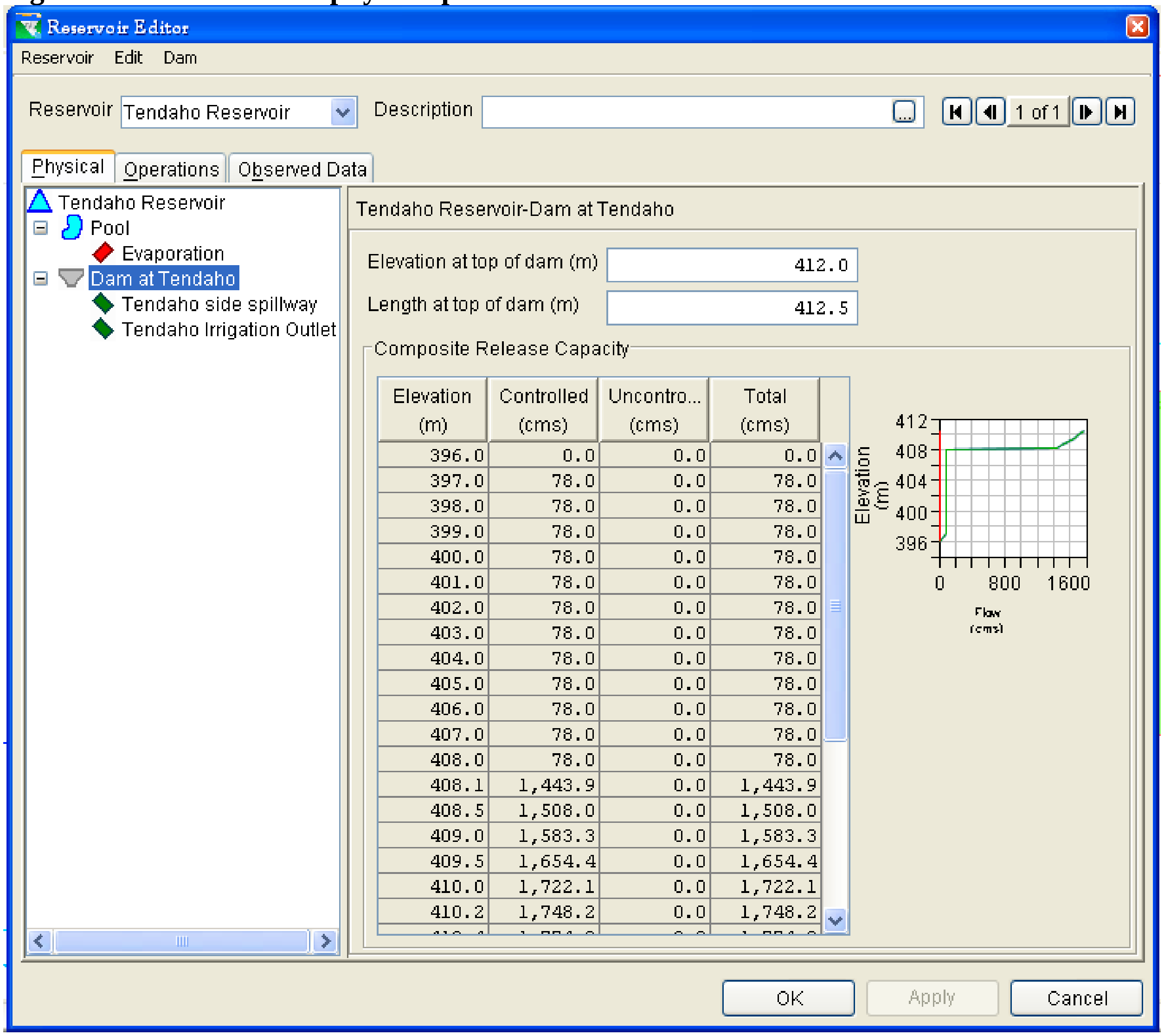


Figure 6.3. Tendaho dam physical parameters.



Elevation (m)	Controlled (cms)	Uncontro... (cms)	Total (cms)
396.0	0.0	0.0	0.0
397.0	78.0	0.0	78.0
398.0	78.0	0.0	78.0
399.0	78.0	0.0	78.0
400.0	78.0	0.0	78.0
401.0	78.0	0.0	78.0
402.0	78.0	0.0	78.0
403.0	78.0	0.0	78.0
404.0	78.0	0.0	78.0
405.0	78.0	0.0	78.0
406.0	78.0	0.0	78.0
407.0	78.0	0.0	78.0
408.0	78.0	0.0	78.0
408.1	1,443.9	0.0	1,443.9
408.5	1,508.0	0.0	1,508.0
409.0	1,583.3	0.0	1,583.3
409.5	1,654.4	0.0	1,654.4
410.0	1,722.1	0.0	1,722.1
410.2	1,748.2	0.0	1,748.2

Figure 6.4. Tendaho side spillway rating curve.

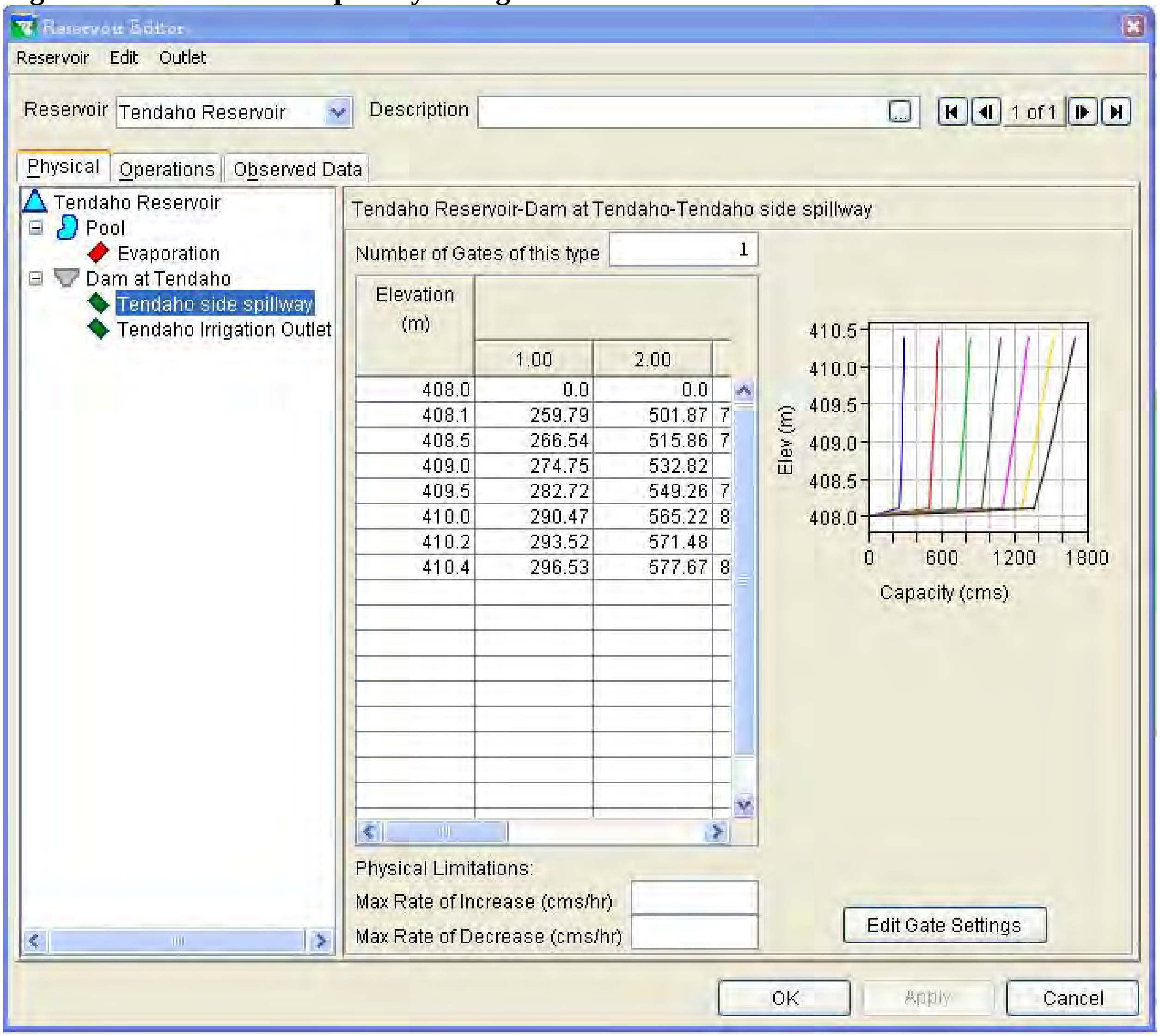


Figure 6.5. Tendaho irrigation outlet Stage-Discharge curve.

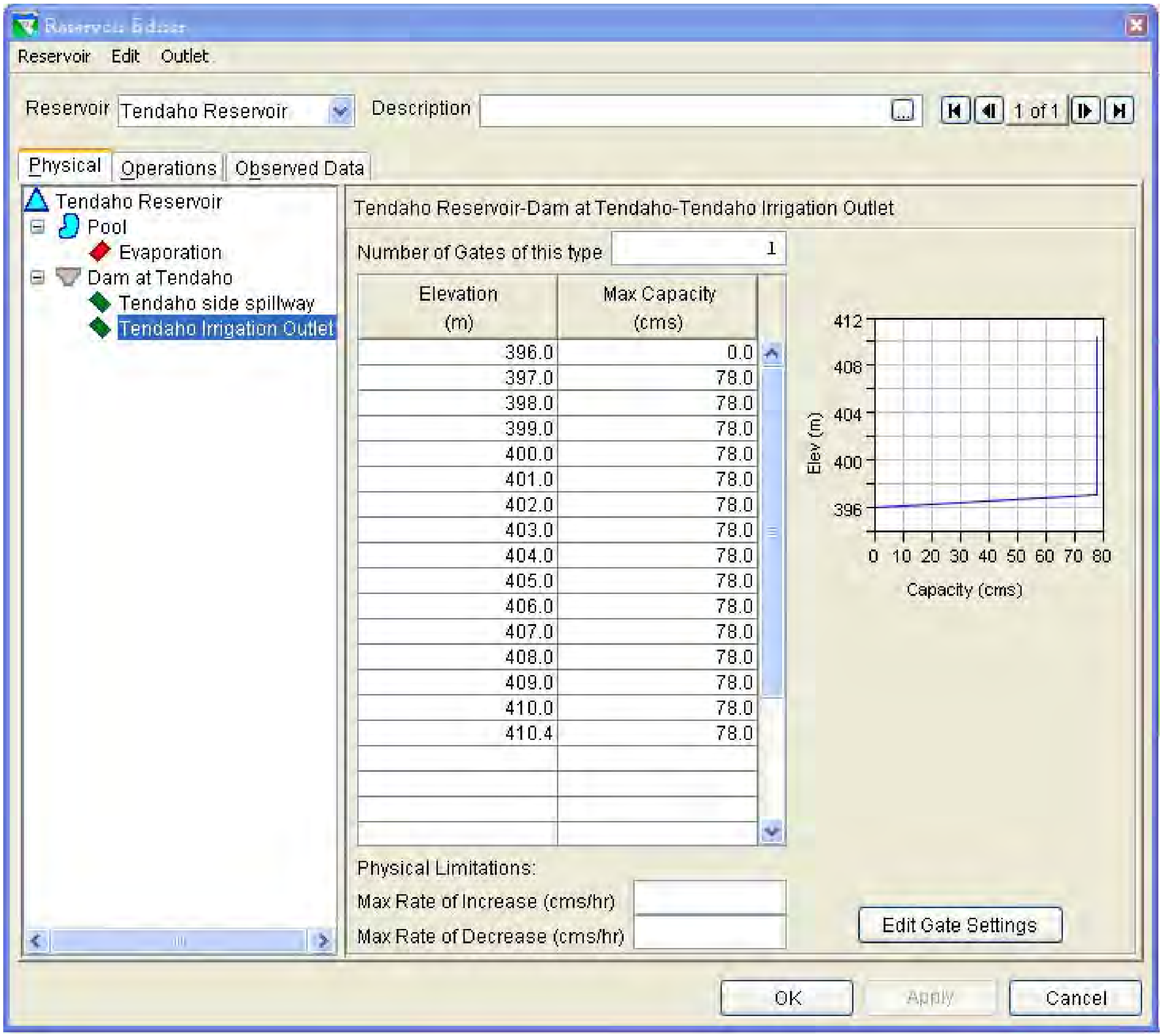


Figure 6.6. Tendaho irrigation outlet monthly diversion data.

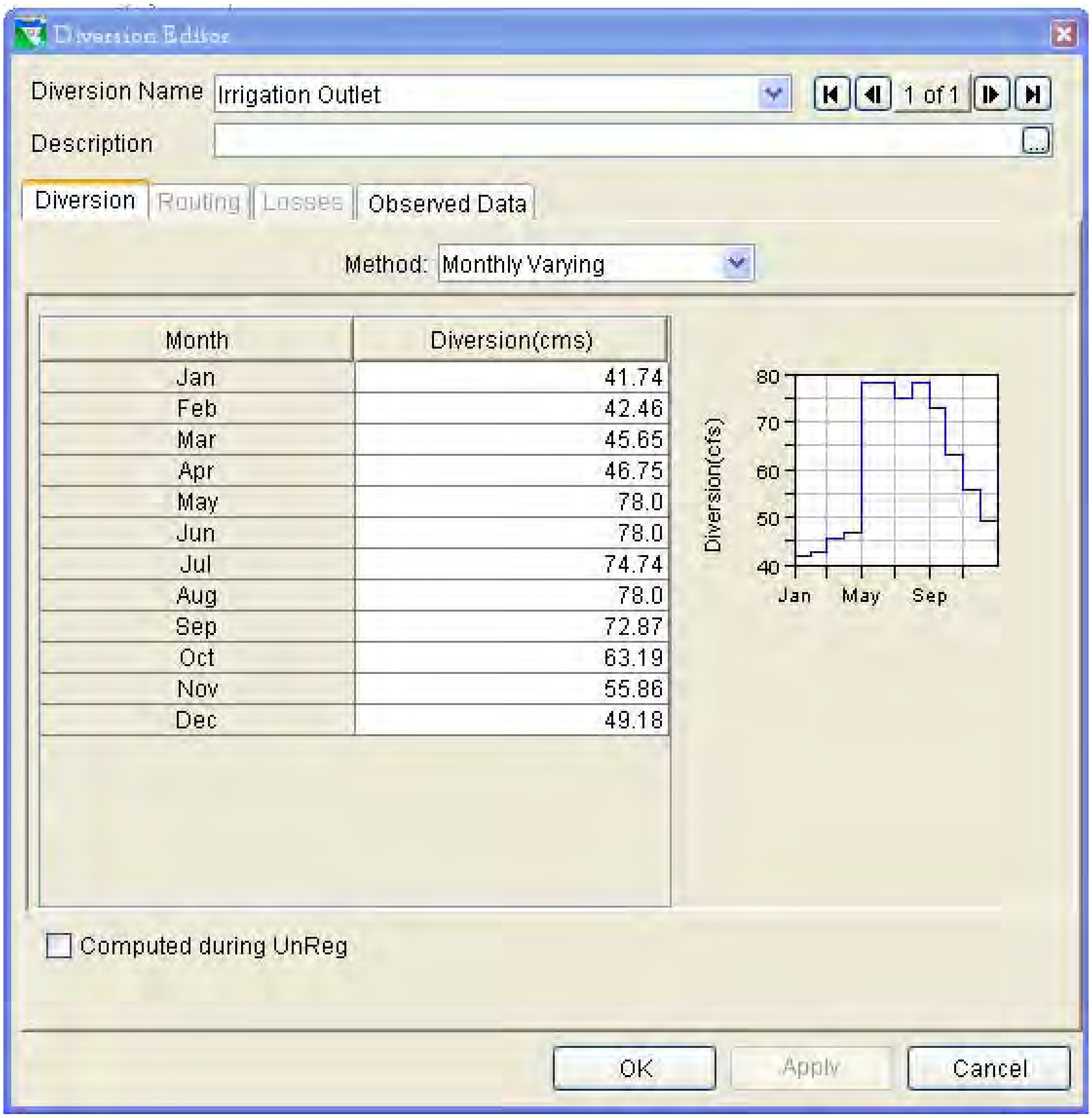


Figure 6.7. Operation rule data for flood control zone and the irrigation outlet.

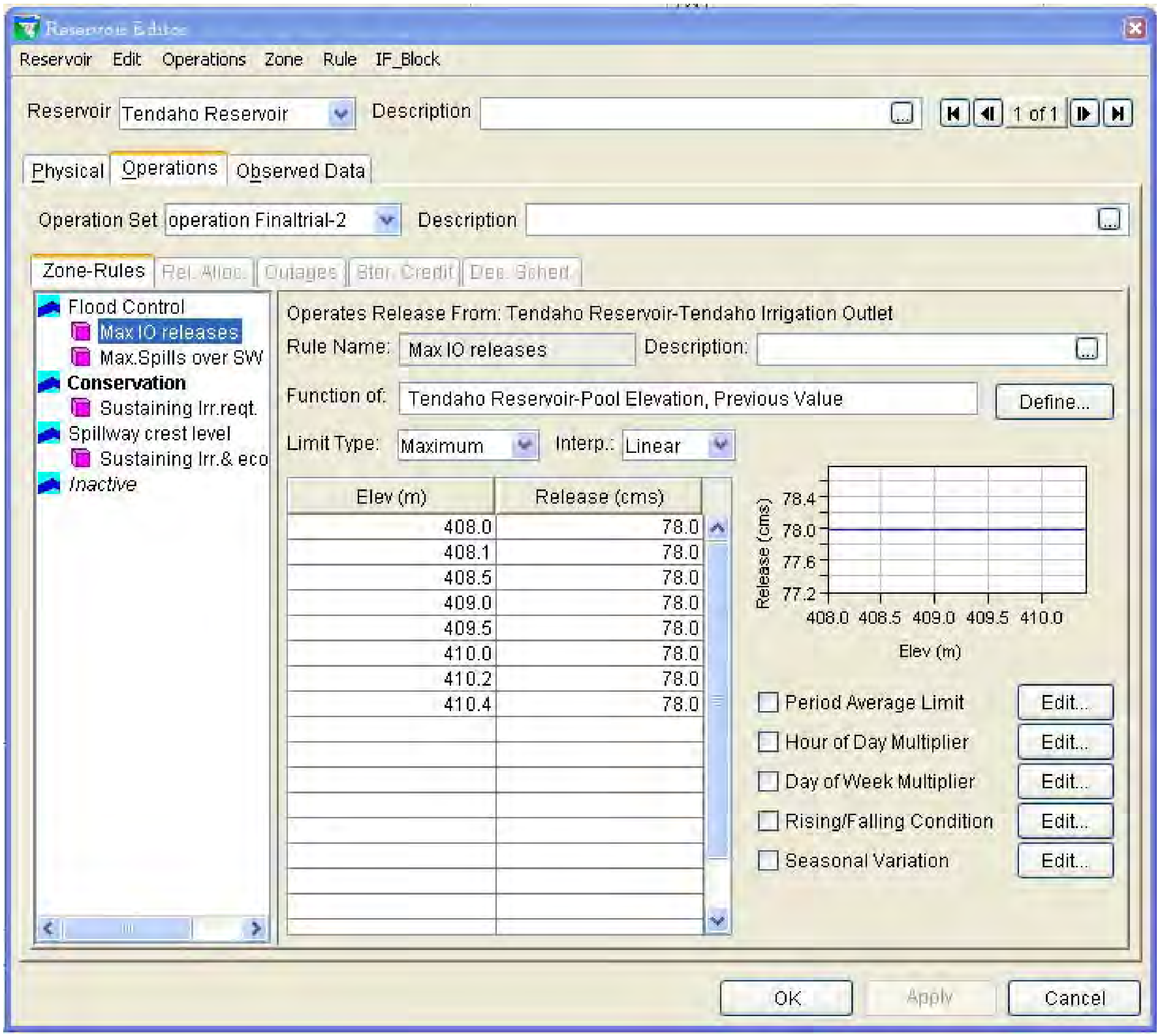


Figure 6.8. Operation rule data for flood control zone and the side spillway.

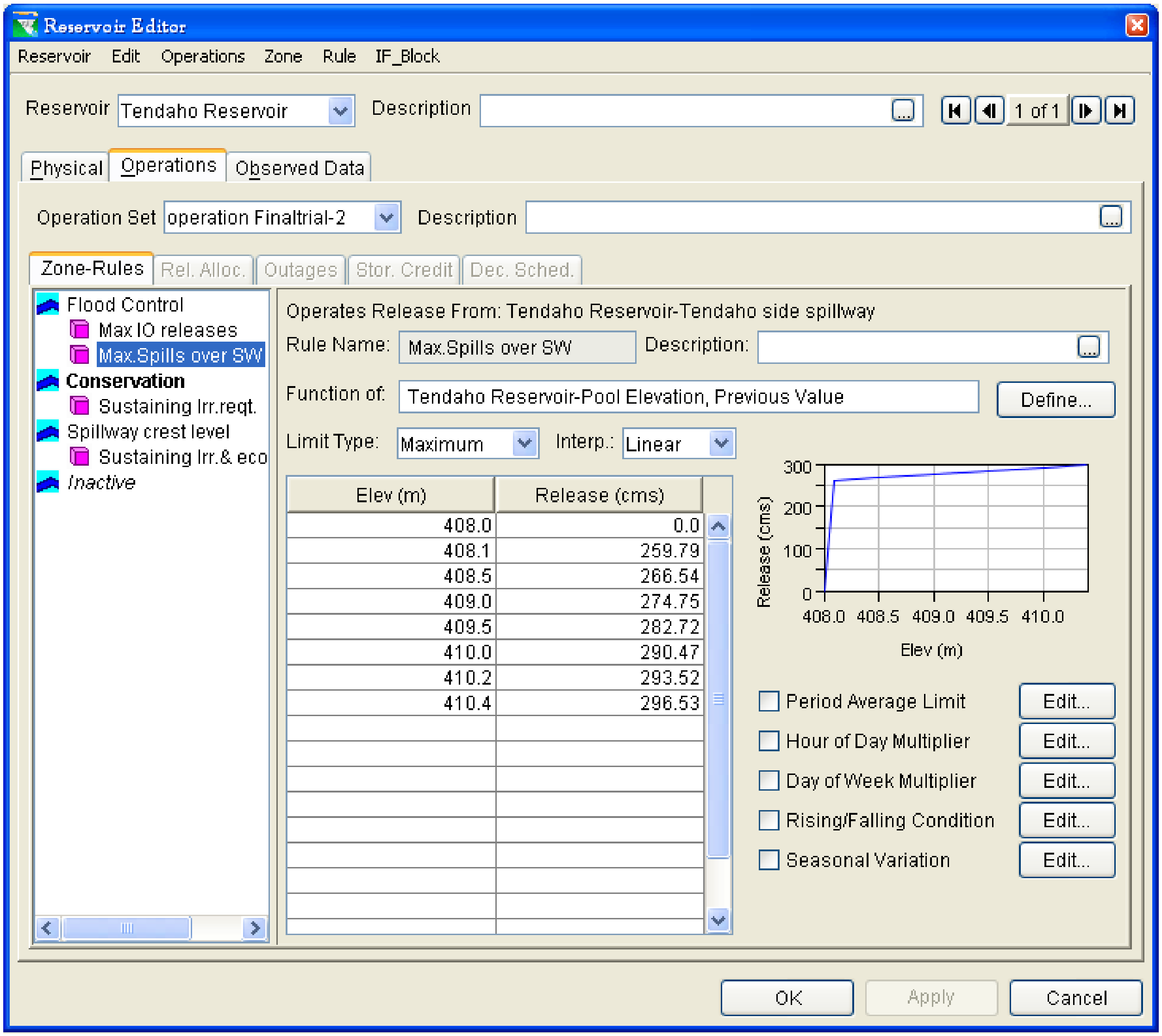


Figure 6.9. Operation rule for the conservation zone and irrigation outlet.