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on

Establishing Water Release Rules for Koka Reservoir for Wet Seasons.

By
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A Thesis Submitted in Partial Fulfillment for the Degree of Master of Science in Civil Engineering

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ABSTRACT

Koka is the only single Reservoir found in the Upper Awash River Basin being utilised both for Hydropower and Irrigation uses. The current maximum capacity of the reservoir is estimated at 1,000 million m^3. Up to the inlet to the Reservoir, the Upper Awash River Basin drains a catchment area close to 11,300 Km^2. A number of important schemes found at the flood prone areas, downstream of the reservoir, are in need of a workable operation rule for Koka which can give them some protection against flooding during wet seasons.

In this research an operation rule curve is established. The operation rule curve is established in such a way that all important variables for the hydropower operation and flood control are optimised subject to the constraints. Linear programming in MS-FORTRAN language is used to develop the optimisation model for Koka reservoir operation. The dry season water requirements of downstream irrigation schemes are implicitly guaranteed since the optimised rule curve will ultimately lead to a full storage at the end of every wet season which is desirable by those schemes.

Emphasis was also given on the flexibility of the model to allow adaptive operation rules responding to the changing constraints and boundary conditions which are quite noticeable in the Koka reservoir system. The Real-Time operation procedure is formulated to be used in conjunction with the established rule-curve to enable the reservoir operators to make decisions, regarding releases for various purposes, in a considerably shorter period of time (e.g.,daily) using the current hydrometeorological information.

The most important component in Real-Time reservoir operation is the Rainfall-Runoff model. These models enables, for a known rainfall, to predict the expected runoff which is an important information for the Real-Time operation of the reservoir and for assessing the possibility of flood hazards well in advance of its catastrophic consequences. It is, however, unfortunate that most models are data intensive that the scarcity of data is the major bottle-neck in using them. In this research, a rainfall-runoff model is established employing the SCS

unit hydrograph and Curve Number concepts. While developing both the Rainfall-Runoff and the Optimisation models much emphasis was given to enable the user to run the models with presently available and easily obtainable types of hydrometeorological data in Ethiopia.

By relative terms the structure of the models is simple with only few parameters to determine and their performance is, however, satisfactory.

1. INTRODUCTION

1.1 General

Ethiopia is endowed with abundant water resources. There are 12 river basins which carry quite a big amount of water annually. Most of the river basins, being very steep at the upstream and gentle in the down stream, are quite suitable both for hydropower development and Irrigated agriculture. Yet, the country has not made the best out of the so called ''***white coal*** ''.

All the area lying between the catchment of the Wabi-shebele river to the south, the catchment of the Blue Nile to the west, the inland depression of the Danakil desert to the north and the border of Somalia to the east, is designated as the basin of Awash, extending to some 120,000 Km^2 (figure 1.1). The average normal annual rainfall and runoff for the whole of the Awash basin are estimated at 710 mm and 4900 million m^3 respectively. The highest sources of the Awash lie in a mountain range lying near the southern edge of the 'high plateau' of Ethiopia some 150 Km west of the capital, Addis Ababa, at an altitude of about 3000 m above sea level (a.s.l). After flowing to the south-east for about 250 Km, the river enters the great rift valley, which it follows for the rest of the course, to where it ends in lake Abe on the border with Djibouti, at an altitude of about 250 m a.s.l (Halcrow, 1989).

The Awash River Basin has been the most intensively studied river basin in Ethiopia and because of its strategic location, good access facilities, available land and water resources, is currently the most developed part of Ethiopia in terms of its irrigated agriculture. The first comprehensive study of the Awash River Basin was carried out by FAO/Sogreah team during the period 1961 to 1964 (Halcrow, 1989).

Based on the agro-climatological classification, the Awash basin has been traditionally divided into four distinct zones. These are Upper basin, Upper valley, Middle valley and Lower plains. The Upper, Middle and Lower valleys are those areas of the basin below Koka reservoir and the river course in these part used to be an intermittent river carrying large flows in the rainy season. The Awash river used to dry up in the low season due not only to the irregular incidence of the rainfall, but also to the heavy losses caused by spill and evaporation in flood plains, swamps and lakes, and by seepage in permeable ground, and, perhaps, faults.

The Upper Awash Basin (Fig. 1.2) with drainage area of 11,300 Km^2 contains most important establishments as Koka Hydropower plant. The typical normal annual rainfall for the upper Awash is about 1000 mm. The mean annual runoff in to Koka reservoir amounts to some 1660 million m^3. About 90% of this runoff occurs in the period July to October. Since the construction of the reservoir and hydroelectric station at Koka in the upper basin, the flows downstream of the reservoir have been largely controlled. On the average the power plant can discharge about 40 m^3/s. As a result, further downstream the areas flooded are reduced, and the loss by evaporation correspondingly diminished. It is clear that the Koka reservoir has significantly modified the hydrological regime of the river (Halcrow, 1989).

There are three hydropower plants in a series along the Upper Awash River named as Koka, Awash II and Awash III. The combined installed capacity of the three power plants is 107 MW. The first power plant in the series is Awash I or Koka Hydropower plant with an installed capacity of 43.2 MW. This plant draws water directly from the Koka Reservoir which has an original storage capacity of 1,850 Million m^3. Sedimentation of Koka reservoir has been assessed by hydrographic surveys in 1959, 1973, 1981 and as part of the Halcrow's study in 1989. Although there are anomalies between the results of these various surveys it is calculated that the average rate of sedimentation in the reservoir is in the order of 25 million m^3 per annum (Halcrow, 1989).

Fig. 1.1 Ethiopia and Location of The Awash Basin

The other two power plants are downstream with no storage reservoir except a temporary storage pond known as forebay. Down stream of the power plants there are about 69,000 hectares of irrigated land and some important plants like Wonji Sugar plantation and Factory.

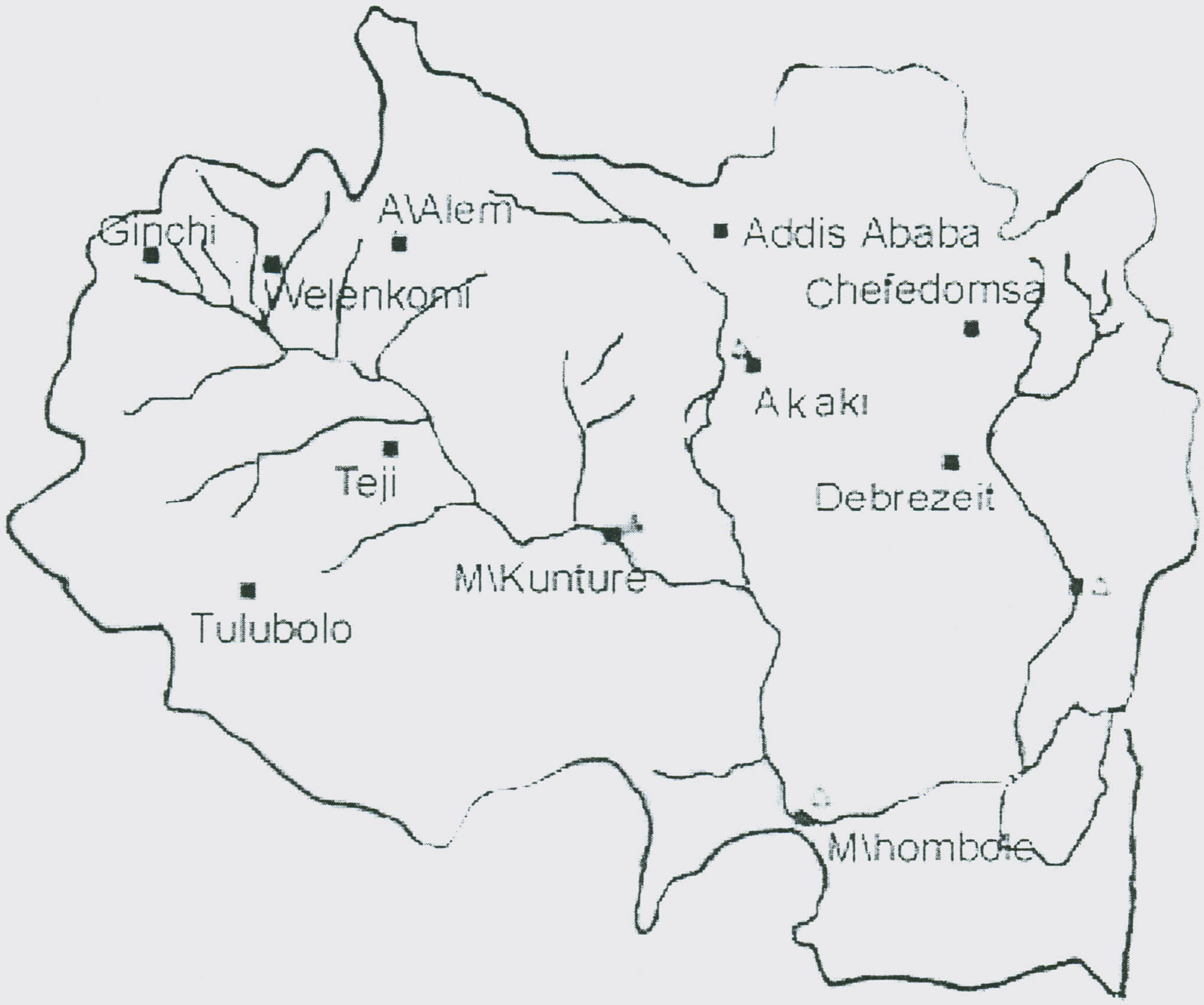

Fig. 1.2 The Upper Awash Basin

1.2 Statement of the Problem

During wet seasons flooding has become more of a common phenomena than an exception affecting the local people and some important development works along the Awash river. This happens usually as a result of a sudden and large amount of water released from Koka reservoir. In order to safeguard the Koka hydroplant, a sudden release is sometimes done when unnecessarily high amount of water is kept in storage in the early days of the wet season without making appropriate provisions for the expected potential flood-inflow in to the reservoir. For instance, the July 1996 catastrophic flooding, which resulted mainly due to cases mentioned above, has incurred heavy property damage and loss of life in the downstream areas.

The other problem associated with the absence of appropriate water release is the critical shortage of water. The problem arises when unnecessarily high amount of water is released from the reservoir expecting that enough water will inflow in to the reservoir in the future. In some cases it may not happen as expected and may result in a critical shortage of water in the coming dry season both for power generation and irrigation purposes. Undesirable and unpopular decisions like power-rationing and low water releases for irrigation, often accompany this shortage.

These problems are as old as the construction of the dam itself and perhaps may continue to exist until an appropriate water release rule for Koka reservoir is established and well implemented.

1.3 Previous Studies

In the past, various studies have been conducted on the Awash basin development schemes. Sogreah/FAO (1965), Acres (1980) and Halcrow (1989) are some of consultants who have made such studies. Some of the previous studies were master plans for the development of water resources in the Awash Basin. These studies had put forward some important proposals for Awash Basin water resource development.

With respect to flood mitigation rules the required level of protection downstream of Koka reservoir has been recommended (Acres, 1980 and 1981) at 500 m^3/s. This figure was confirmed by Nedeco (1982) and is accepted by Halcrow's study as appropriate (Halcrow, 1989). The current level of protection at Wonji sugar estate is stated to be in the order of 300 m^3/s so that it remains desirable to maintain outflows at or below this level. Such a policy was adopted by Nedeco in their flood simulation studies which invoked maximum flood releases from Koka of 300 m^3/s until a flood level of 1.5 m below the maximum flood storage level of 1592.2 m is reached (dam crest is at 1593.2). The flood release is then increased to 500 m^3/s until the maximum flood storage level is reached at which time the gates are fully opened to release the flood capacity which is 1400 m^3/s at level 1593.2. (Refer to page 74 for more details about the Koka dam and reservoir)

The recent study is the "Master Plan for the Development of Surface Water Resources in the Awash Basin" made by Halcrow (1989). This study presented a detailed analysis on different topics like climate and hydrology, dams and hydropower, river basin modeling e.t.c. and put forward some important proposals. One of the outputs of that study was the operation rule curve for Koka reservoir shown in Figure 1.3 (Halcrow, 1989).

The Ethiopian Electric Power Corporation (EEPCO) has been using those proposals for operation of the Koka reservoir. However, the strict implementation of those proposals was very difficult for different reasons. One major problem could be getting the hydrometeorological data in the required quantity and quality in time. This was one of the major limitation to use those models. This has lead to a problem in the appropriate implementation of operation rule for Koka reservoir. It is usually the case that the data intensive nature of models developed by those studies could limit their application under Ethiopian conditions in general and the Awash basin in particular.

The other limitation of the past studies, when one wants to apply them directly, is the changes on the state of the reservoir which have taken place since then. Sedimentation has already clogged the bottom outlet and rendered it useless for future utilization. Excessive sedimentation has reduced the volume of the reservoir considerably. Even the most recent rule curves established by Halcrow in 1989 could be obsolete for the present state of Koka reservoir. As the reservoir capacity is continuously decreasing, its operation meeting the power generation, irrigation and flood protection demands is even becoming more and more complex and competitive.

The need for revisions to operational procedures to accord with any changes in a particular set of conditions is also emphasized in Halcrow's report (1989). Some of the reasons for revising operational procedures mentioned in that report are;

- changes in reservoir storage capacities
- change in hydrological conditions
- changes in policy priorities
- variations in other demand patterns

The rule curves therefore need to be sufficiently flexible to permit the operating authority to react to such changes and to simply adapt one or more of the relevant parameters, either on a temporary or permanent basis. The effects of applying the rule curves should be closely monitored by the responsible authority and the results reviewed from time to time. Such intervals should not exceed five years. (Halcrow, 1989).

All these and other reasons call for a more closer attention for establishing appropriate models for Koka reservoir operation and improving them when more recent data on hydrometeorological, reservoir characteristic, e.t.c are made available.

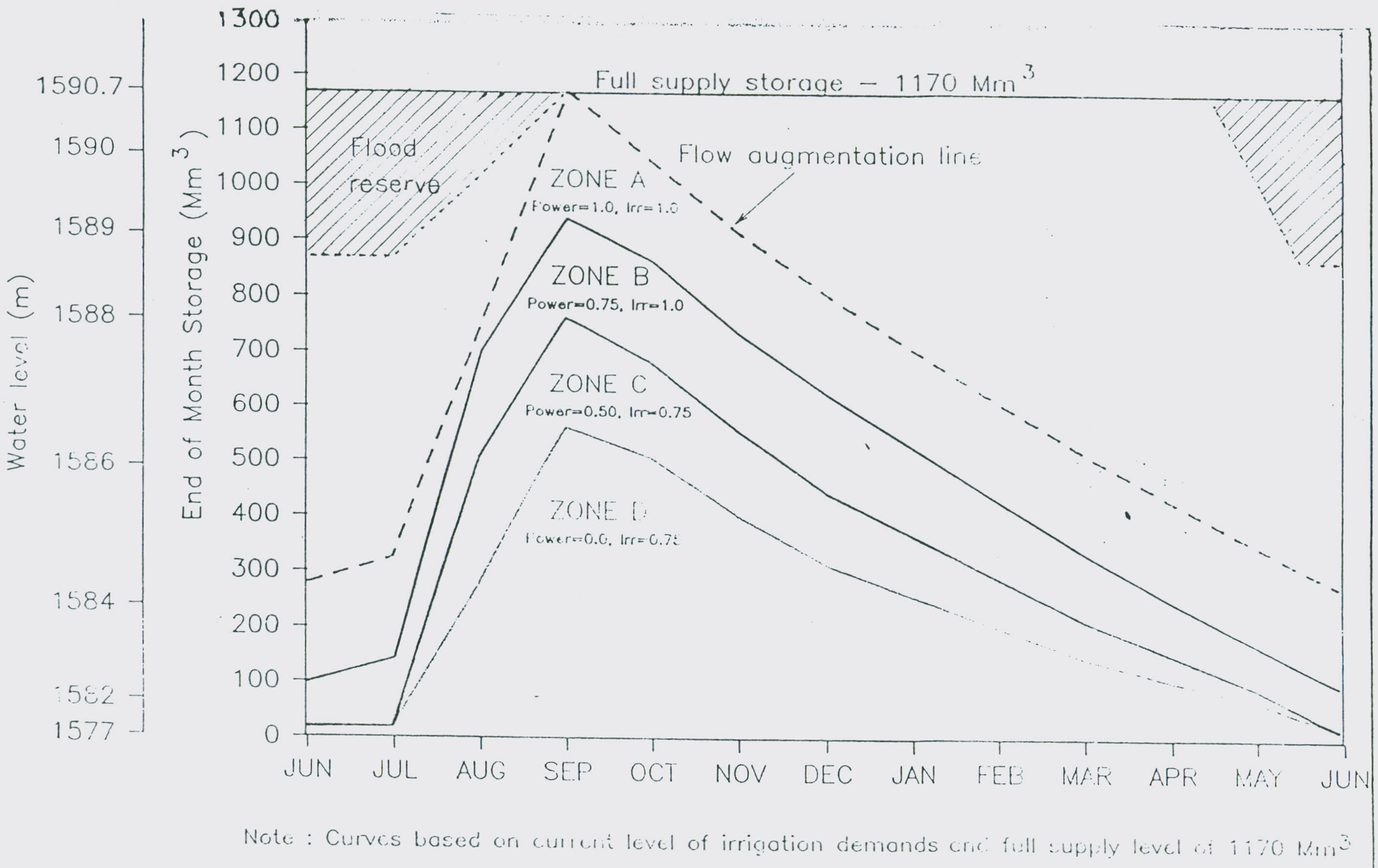

Fig. 13: Release Rule Curves for Koka Reservoir – 1988 (Halcrow)

1.4 Existing Koka Reservoir Operation Practices

EEPCO is a governmental organization which runs Koka hydropower station including the reservoir. Operational rules adopted by EEPCO require spillway discharge at reservoir levels above 1588.4 m if the reservoir level is increasing at a rate of 30 cm/day or more. In principle, if the extrapolated reservoir rise is not completed by the end of the 'spill period' (18 August to 18 September), spill is released to achieve that end. The purpose is largely to avoid flooding problems at Wonji which begin at river discharges over 300 m^3/s and become extensive at flow exceeding 500 m^3/s (Halcrow, 1989).

Following the study of the Awash Basin by Halcrow (1989), EEPCO employes the rule curve established from that study shown in figure 1.3.

As part of this research work, the operation procedure of Koka reservoir has been studied in detail and a lot of discussions have been made with the Koka power-plant operation work-force to have a closer look at the Koka reservoir. The historical releases and the reservoir levels as a function of time were examined to see the effectiveness of those operations.

Some of the historic water level of Koka reservoir which show how the reservoir was operated for those years is shown in Figure 1.4. One can observe that, for some years the reservoir was not full at the end of the wet season which may result in shortage of water both for hydropower and irrigation purposes. This problem may arise as a result of excess amount of water released through the spillway. The historic releases through the spillway for some years of wet seasons is shown on Table 1.1. If water had been saved from spillage then it would have been probable that the reservoir would be full at the end of wet season. But this assertion is not necessarily true because for 'a low wet season' enough water may not inflow in to the reservoir which can fill the reservoir at the end of wet season.

In general, it is very difficult to say the operation of Koka reservoir makes use of the state-of-the-art reservoir operation techniques employed by some countries. These techniques require a good quality historical and real time hydrometeorological data, suitable simulation and optimization models and qualified personnel to run and interpret them. The implementation of this system on all single and multipurpose reservoirs in Ethiopia is highly important to effectively use water resources while protected against flood.

Since the 1996 flooding, a committee under the title "Koka Technical Committee" has been established comprising professionals from the Ministry of Water Resources (MWR), EEPCO and other concerned parties and good initiatives have been taken to operate the reservoir in a more scientific way. The awareness and the steps being taken are encouraging but it can generally be said that a lot remains to be done and the Koka reservoir operation procedure is still far from the more effective technique most countries follow.

Table 1.1 Historic water release through the spillway for wet seasons.
(Source: EEPCO, 1997)

Year	Spillage, Mm^3
1988	91.3
1989	144.0
1990	164.3
1991	129.1
1992	0
1993	761.8
1994	20.3
1995	0
1996	>1500

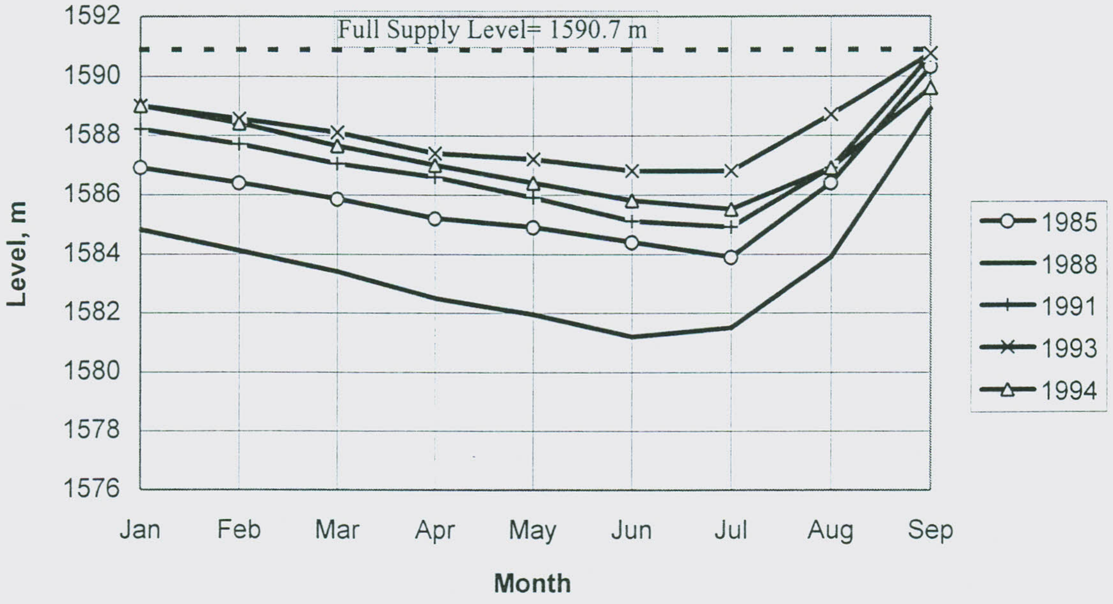

Fig.1.4 Koka Reservoir historic water level
(Source: EEPCO, 1997)

1.5 Objective of the Study

Awash being, by relative terms, the most extensively utilized basin, quite a lot of studies have been made on it and important proposals, as a 'by-product' of the Awash Master Plan, have been made regarding the operation of Koka reservoir. However, it is a fact that the changing demand and physical conditions (power, reservoir capacity, e.t.c) make some of the past proposals obsolete.

The main objective of this research is to establish an operation rule for Koka reservoir using the presently available hydrometeorological information. It is required that this operation rule should maximize the overall benefit we get from the system. As much as possible and practicable, it has been tried to simplify the number and type of input data required to run the models so that the user will be encouraged to use them. For instance, the rainfall-runoff model developed by this research uses only the daily total rainfall amount from the existing rainfall stations to predict the runoff. Also, the basic concept used to develop the optimization model is simple to understand that the user can make his own modifications, if desired, to include or remove certain variables in the constraints, water balance or the objective function. For instance the model can be re-used (re-run) any time when the reservoir capacity changes, if turbine is added or removed from the system e.t.c.

2 Physical Characteristics of Upper Awash Basin

2.1 Topography and Climate

The upper Awash catchment is found in the highlands of central Ethiopia with all lands above 1500 m above see level (a.s.l). It comprises that section of the catchment from the headwaters to Koka reservoir. The altitude approximates the boundary between the settled mixed agriculture above and the nomadic livestock areas below generally with annual rainfall above 800 mm and daily temperature below 20 degrees Celsius.

The climate of the upper Awash basin, in general, comes under the influence of the Inter Tropical Convergence Zone (ITCZ). This zone of low pressure marks the convergence of dry tropical easterlies and moist equatorial westerlies. The explanation of the seasonal rainfall distribution within the basin lies in the annual migration of the ITCZ across the basin. The ITCZ starts its advance across the basin from the south in March, bringing the small or spring rains. In June and July the ITCZ reaches its most northerly location beyond the basin which then experiences the heavy or summer rains throughout. The ITCZ returns southwards during August, September and October, restoring a drier, easterly air-streams which prevails until the ITCZ resumes its northward migration in March. Sample of the average daily rainfall distribution over the wet season in the upper Awash Basin is shown in Figure 2.1.

2.2 Land use and Soil type

The land use condition in the upper Awash catchment includes mainly of cultivated agricultural land, grass land, forest land, rural and towns settlements. It is estimated that 67% is intensively cultivated, 25% is moderately cultivated, 5% is bush land or shrub land or wooded grass land, and 3% is urban area and alpine vegetation. Strictly speaking, even the land use with in the upper Awash is diverse. In the upper most part where there is high rainfall, land use is complete in May with barley and teff. Steeper slopes are heavily wooded with natural acacia and much eucalyptus. On the lower most part, however, rainfall is too unreliable and the sparse dry acacia scrub gives way to wide stretches of bare ground with clumps of coarse grass and occasional thickets of acacia. The mode of land use of the area downstream of Koka dam is different from that of the upstream in that the considerable proportion of the land is used for irrigation agriculture. It is dominated by sugarcane and citrus, 74 and 8 percent respectively of which the highest proportion is for state farms (Halcrow, 1989).

The soil type in the upper Awash basin is diverse. The most common soil types are Clay, Sand, loam, Clay-Loam, silty-Clay-Loam, Sandy-Clay, Silty-Clay.

2.3 Awash River upstream of Koka Reservoir

The Upper Awash River covers the river section from its source up to Koka Reservoir. The highest sources of the Awash river lie in a mountain range lying near the southern edge of the 'high plateau' of Ethiopia some 150 Km west of Addis Ababa. Awash river first flows east, draining the Becho plains and is joined by several small tributaries before entering Koka Reservoir. The upper Awash river drains a catchment area close to 11,300 Km^2 and the length of the River up to Koka is around 220 Km. After being released through Koka Dam, which came under operation in 1960, it descends in to the rift valley. The fall of the river in this reach is exploited by hydropower installations at Koka and at a series of run-of-river schemes designated Awash II and III (Halcrow, 1989).

The major tributaries to the upper Awash are Akaki and Mojo rivers (Figure 2.2). Akaki river starts from the mountainous areas of the northern part of Addis Ababa and passes through a small impoundment called Akaki lake (Akaki Hayk) south of Akaki town having an area of some 12 Km^2 and joins the main Awash river between Melka-kunture and Melka-hombole gauging stations.

Mojo river, the other main tributary to Awash, originates from the high lands north-east of Addis Ababa. It drains a catchment area close to 1,900 Km^2 and travels a total length of about 105 Km before joining Awash

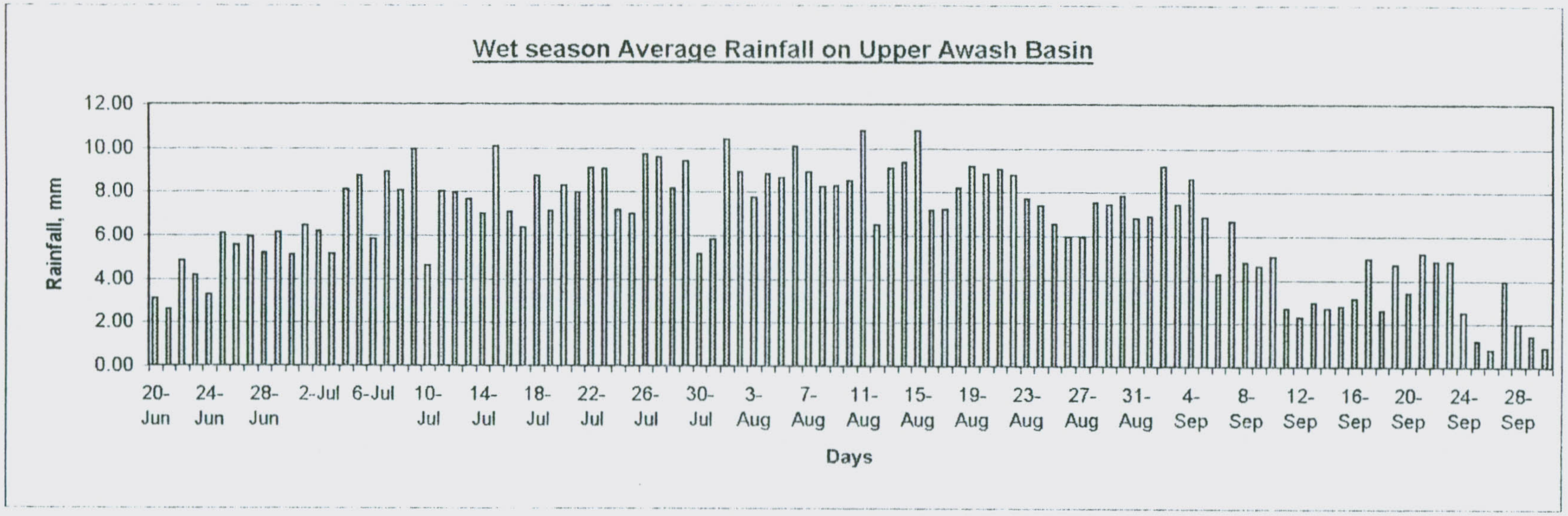

Figure 2.1 Average daily rainfall variation during a wet season over the Upper Awash Basin (1989-1996)

2.4 Description Subcatchments of the Upper Awash basin

A watershed is usually a complex and heterogeneous system. Its characteristics vary in space. Hydrologic processes vary both in space and time. One way to account, at least partly, for spatial variability of governing hydrologic factors is to divide the watershed in to nearly homogeneous subcatchments. Differences in soil, vegetation, land-use, or topography that significantly affect streamflow must be considered to demarcate sub-basins. Of course, the larger the number of subcatchments, the greater the accuracy as well as the cost of simulation. There is, thus a tradeoff between accuracy of simulated stream flow and cost of simulation that must be determined. Once the watershed is decomposed, computations proceed from the most remote upstream subcatchment in a downstream direction. (Singh, 1989)

Therefore, for the purpose of this research the Upper Awash catchment is conveniently sub-divided into three major Sub-catchments and one local catchment using the topographic map of 1:50,000. These major subcatchments are named as Subcatchment-A, Subcatchment-B, Subcatchment-C. Figure 2.2 shows these subcatchments and some details of each subcatchment are described below.

Subcatchment-A:

The boundary of Subcatchment-A starts from the northern-western end of the head water of the Upper Awash basin, where the river begins, and covers the area up to the Melka-kunture gauging station. Small towns like Addis Alem and Welenkomi are found in this subcatchment. It has a drainage area of about 4,541 Km^2, a stream length of 118 Km and an average land slope of 0.64%. This Subcatchment contains places which get the highest amount of annual rainfall in the Awash basin as a whole and consequently it has the major influence on the flood hydrograph of the Upper Awash river. There is a runoff gauging station at Melka-kunture.

Subcatchment-B:

Subcatchment-B covers the drainage area downstream of Melka-kunture and covers all the areas up to Koka inlet. Cities like Addis Ababa, Akaki and Debre ziet are found in this subcatchment. It has a drainage area of about 4,200 Km^2, a stream length of 158 Km and an average land slope of 0.53%. One of the main tributaries to this subcatchment is Akaki river. Part of the runoff from this subcatchment is gauged at Melka-hombole gauging station which is, unfortunately, found some 42 km upstream of Koka inlet (end of Subcatchment-B). Consequently, the total Awash inflow in to Koka is not explicitly known as there is no any observation gage at the subcatchment's outlet (Koka inlet).

Subcatchment-C:

Subcatchment-C, or alternatively Mojo subcatchment, starts from the highlands east of Addis Ababa and goes further down the lowlands of Mojo area and finally joins Koka reservoir. Mojo town is found in this subcatchment. This subcatchment includes areas of low annual rainfall amount. It has a drainage area of about 1,900 Km^2, a stream length of 105 Km and an average land slope of 0.80%. Like the case of Subcatchment-B, the gauging station of the Mojo subcatchment is found near Mojo town, well upstream of its confluence with Koka reservoir, as a result the total flow contribution of Subcatchment-C is not explicitly known.

Local catchment:

The local catchment is found on the southern and south-eastern part of Koka reservoir. The land is covered with open wood land and Savannah and has an average slope of about 2% and has no defined stream. It covers an area of nearly 475 Km^2 which is relatively much smaller than the other Subcatchments. Moreover, the catchment gets the lowest rainfall amount and its contribution to the total runoff (or total inflow) is very small. As a result it is even more convenient to implicitly include it to one of the Subcatchments (preferably to Subcatchment-C) and make some provisions during the runoff calculation.

Table 2.1 Summary of subcatchments physical characteristics

Subcatchment	Catchment area, Km^2	Stream length, Km	Average slope, %
1. A	4,541	118	0.64
2. B	4,200	158	0.53
3. C	1,900	105	0.80
4. Local	475	Nil	2.00

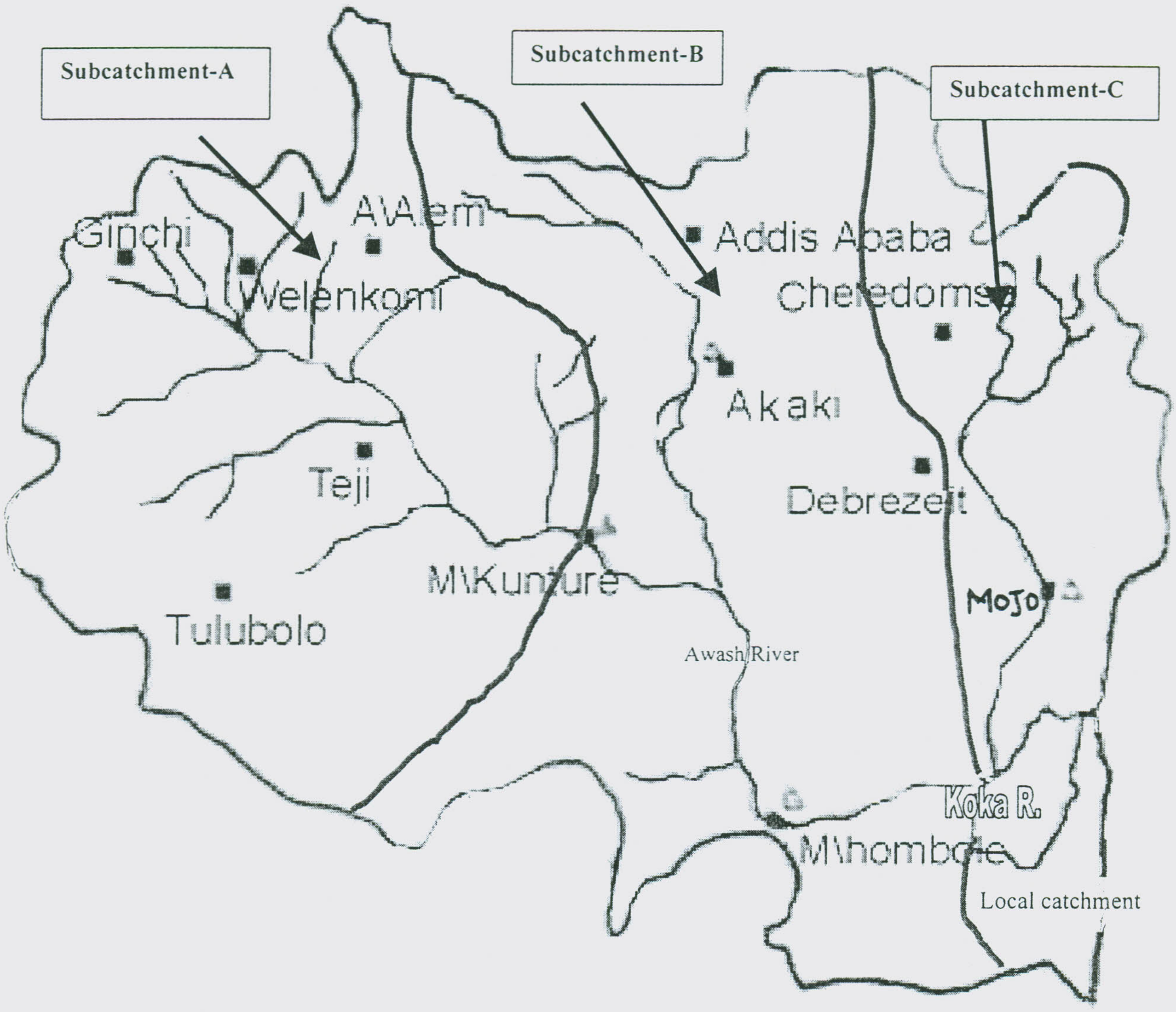

Fig. 2.2 Subcatchments of the Upper Awash Basin

3 Hydro-Meteorological Data

3.1 Meteorological data

The other most important data for this research are the meteorological data which include the daily and continuous rainfall records. There are a number of rainfall recording stations distributed over the upper Awash basin. Some of the daily rainfall data are, however, incomplete and in some areas like Mojo subcatchment there are small number of rainfall gauging stations. Out of all the available rainfall recording stations about 15 daily rainfall recording stations were systematically selected to be used for this research. The location of this stations is shown on Table 3.1. A total of 12 years of daily rainfall records were collected for each stations from the national Meteorological Service Agency (NMSA). In selecting the stations, the main criteria were quality and long-term completeness of rainfall data. Moreover, it is required that as much as possible, the location of the stations be such that they are fairly distributed over the catchment so that the stations can be good representatives when calculating the Areal rainfall. These rainfall stations are shown in figure 3.1.

In the upper Awash basin only Addis Ababa, Akaki and Debre-zeit have continuous rainfall recording stations. Even these stations do not have complete record that it is very difficult to use them for hydrological calculation related to rainfall-runoff relation except that they only give some idea about the nature of rainfall processes in those places.

Some rainfall stations have very doubtful daily records due to a number of reasons. For instance, the Koka rainfall recording station for some years, shows a very high rainfall amount which doesn't reconcile with all the neighboring stations, like Mojo and Debre-zeit, which show a lower rainfall amount for the same period. These unexpected very high or very low rainfall records became a big obstacle when trying to develop the Rainfall-Runoff model. As a result, for stations where these errors are more pronounced and clearly identified, the rainfall record of that station is rejected and the Rainfall-Runoff relationship is established using the remaining rainfall stations. It is worth mentioning that information from previous researches about both rainfall and runoff records was helpful in the screening process of the meteorological data.

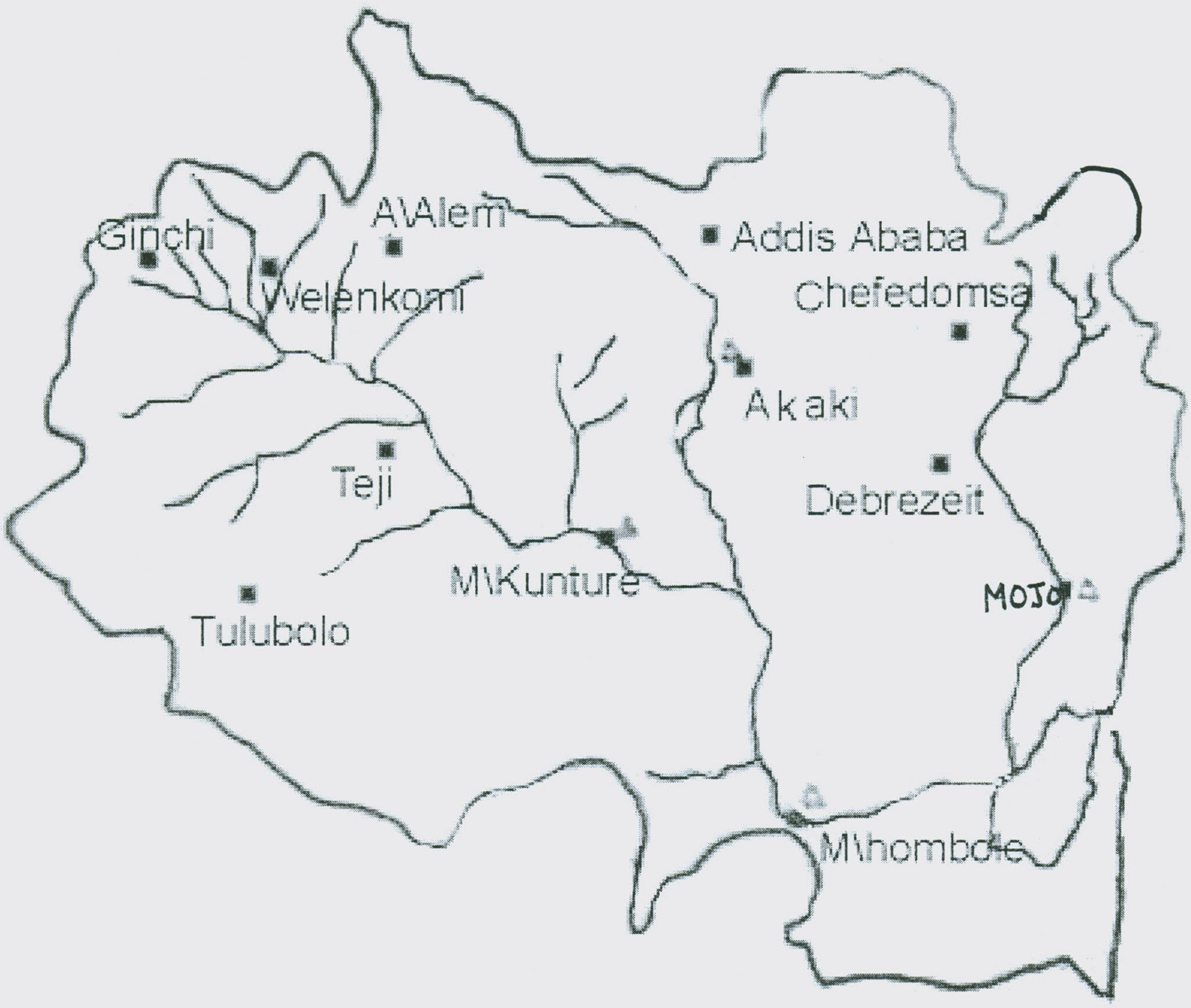

Fig. 3.1 Location of Hydro-meteorological stations in the Upper Awash Basin

3.2 Hydrological data

In the Upper Awash basin there are recording type gauging stations. Out of these the Melka-kunture, Melka-hombole and Mojo are the most important ones for this research.

The daily flow records from 1980-1996 of Melka-kunture, Melka-hombole and Mojo were obtained, in hard copy, from the Hydrology department. In addition, the Koka reservoir level readings were collected from EEPCO.

It is usually the case that the daily discharge records of most rivers in Ethiopia (including the Awash river) are taken once a day. This could usually over-estimate or under-estimate the actual runoff specially during the wet season where there could be a high variability of flow with in a day. Therefore the representativeness of a discharge data taken in this manner becomes questionable specially when one tries to compare the observed discharge (specially peak discharges) with the simulated flood runoffs during verification of rainfall-runoff models.

In addition to the above mentioned problems, one sometimes observes an unexpected very high or very low flow records. For instance in the 1989 Halcrow study of the Awash basin it was reported that the Mojo flow record is less good. The study also revealed several cases where sudden high or low figures in the data record could not be reconciled with adjacent figures or records at adjacent stations. (Halcrow, 1989). Another instance for this is the 1996 flood discharge measurement at Melka-hombole. During the 1996 wet season, measuring the flood runoff was a major problem as the discharge gages were over flooded. For this reason, the actual discharge measurements were not taken for a considerable time (for example, from August 7 to 22). As a result extrapolation was employed to approximate the flood discharge which makes these data questionable.

4 A Review of Some Rainfall-Runoff Models

4.1 Rainfall and Runoff Phenomena

4.1.1 Rainfall

The term precipitation as used in hydrology includes all forms of water deposited on the earth's surface and derived from atmospheric vapor. The principal forms are rain, snow, hail, sleet and mist. Unless otherwise specified, the terms precipitation and rainfall are often used indiscriminately to apply to any or all of the forms included in this group. Rainfall may be classified in accordance with the conditions that produce a rising column of unsaturated air, of which there are three: convectional, orographic and cyclonic. (Wisler et.al, 1985) Convectional rainfall is most common type of rainfall in Tropics including Ethiopia.

Rainfall is extremely variable both in time and space. The variation is brought about by differences in the type and scale of development of precipitation-producing processes, and is also strongly influenced by local and regional factors, such as topography and wind direction at the time of rainfall. It is, however, assumed that each individual rain-gauge is representative of a very considerable area around it. This assumption is not correct. Because of the very considerable spatial variation of precipitation depth and intensity, particularly for short durations and for sever convectional storms as is the case in most parts of Ethiopia. There is no guarantee that point rainfalls will in any way provide a reliable guide to the rainfall of immediate surrounding areas.

One particular type of analysis of the spatial character of rainfall involves the derivation of the areal rainfall from a number of point rainfall data. For many hydrological and engineering studies, such as reservoir and water resource development or in flood studies, a detailed picture of rainfall patterns or movement across the area concerned is not required. What is actually required is some measure of the total amount of water falling on that area.

The simplest and most obvious initial approach to the derivation of areal rainfall is to calculate an arithmetic mean of the values from all point rain gauges. However, this is the crudest approximation among the rest of Areal rainfall calculation methods. The other better approximation methods include: the Isohytal method and Thiessen method. Both techniques, however, demand initial laborious construction of a geometric matrix and involve the derivation of areas which can again be time-consuming.

4.1.2 Runoff

Runoff is that part of the rainfall, as well as any other flow contributions, which appears in surface streams of either in perennial or intermittent form. This is the flow collected from a drainage basin or watershed, and it appears at an outlet of the basin. According to the source from which the flow is derived, runoff may consist of surface runoff, subsurface runoff and ground water runoff (Buras, 1972).

For the practical purpose of runoff analysis, total runoff in stream channels is generally classified as base flow and direct flow which consists of all other types of flows. The direct runoff is that part of runoff which enters the stream promptly after the rainfall. It occurs only when the rainfall rate is greater than the infiltration rate. The base flow is defined as the sustained runoff composed of ground water runoff and delayed subsurface runoff. (Buras, 1972)

From the hydrologic point of view, the runoff from a drainage basin may be considered as a product in the hydrologic cycle, which is influenced by two major group of factors: climatic factors and physiographic factors. The influence of the first group depend mainly on types of precipitation, rainfall intensity, duration of rainfall, distribution of rainfall on basin, antecedent moisture condition(AMC) and direction of storm movement. The influence of the second group depend mainly on land use, type of soil and topography (Buras, 1972).

During a runoff-producing storm, the total precipitation may be considered to consist of precipitation excess and abstractions. The precipitation excess is that part of the total precipitation that contributes directly to the surface runoff. When the precipitation is rainfall the precipitation excess is known as rainfall excess and it must be noted that in most tropical countries, including Ethiopia, almost all part of the precipitation is in the form of rainfall. The abstractions are the remaining parts which do not eventually become surface runoff, such as infiltration, interception, transpiration, evaporation and depression storage.

4.1.3 Separation of Total Runoff

The baseflow is normally defined as that portion of streamflow derived from groundwater storage or other delayed sources. It represents withdrawal of water from ground water storage (Singh, 1989).

The separation of direct runoff and baseflow is made in an entirely arbitrary manner. Nevertheless, the separation is made to avoid the very long tails that must otherwise be associated with the part of the hydrograph due to each rainfall storm (Singh, 1988). There are, however, a number of alternatives which help to separate the two important components. Only some of them are discussed below (Maidment, 1993).

1) The separation of baseflow from surface runoff is achieved by drawing tangents to the recession curves at the points of start and finish of direct runoff and assuming a shape for the baseflow hydrograph between these tangent points.

2) A straight line separation is often quite acceptable where the maximum baseflow discharge is well below 10 percent of the maximum discharge.

3) A more realistic separation may be obtained by continuing the average baseflow recession forward up to the a point approximately below the peak of the total hydrograph, and joining this extended baseflow recession up to the finish of direct runoff by a smooth curve. The latest possible occurrence of the peak of baseflow (to give a smooth curve) agrees with the slower travel of this component.

4) The other alternative states that to obtain a total hydrograph a constant baseflow can be assumed. An average baseflow for the basin should be used if possible, or if this is not known a regional value may be used.

Most of the base flow separation theories are good when we want to analyze only single runoff hydrograph. In the case of Upper Awash basin, where we have multiples of hydrographs during the wet season, some of the above alternatives look too theoretical to strictly follow. However, in this research, the above concepts are combined with the actual observed conditions, during the simulation, in order to estimate the baseflow. Moreover, it can be noted that from the hydrometeorological conditions of Upper Awash Basin during the wet season, where the base flow contribution is very small, adopting one of the most convenient alternatives will not have a major influence on the overall simulation of the rainfall-runoff process.

4.2. A brief Description of Rainfall-Runoff Models

In order to apply optimum control policies for the actual operation of reservoirs during floods, it is necessary to know the input information for reservoirs in advance. This requires forecasting of the flood hydrograph for each reservoir in the watershed at the beginning of a storm. One way of achieving this objective is to use rainfall-runoff models. Rainfall-runoff models provide information needed to solve a wide variety of water resource problems including flood forecasting and control, stream flow regulation encompassing reservoir operation and other efforts to satisfy such instream flow needs as Hydroelectricity, navigation and recreation (Singh, 1982).

Hydrologic phenomena are extremely complex, and difficult both to measure and understand in full detail. In the absence of perfect knowledge, however, they may be represented in a simplified way by means of the system concept which is a set of connected parts that forms a whole. The hydrologic system may be treated as a system whose components are precipitation, evaporation, infiltration, runoff and other processes in the hydrologic cycle. The different components can each be grouped together into subsystems or broken down into subprocesses, depending on the level of detail in the analysis and the purpose of the analysis. For most practical applications, only a few processes of the hydrologic cycle are considered at a time, and only for a small part of the earth's surface, usually in a catchment.

A hydrological system can be defined as a structure or volume in space, surrounded by a boundary that accepts water and other inputs, operates on them internally and produces an output. The objective of hydrological system analysis is to the system operation and predict its internal states and output. A hydrological system model is an approximation of the actual system. Its inputs and outputs are measurable hydrological variables and the model's structure is a set of equations linking input to output. Central to the model structure is the concept of system transformation. The input and output can be expressed as a functions of time *I(t)* and *O(t)* respectively. A system performs a transformation of the input into the output represented by a transformation operator or equation.

A watershed can be looked upon as an operator transforming the moisture input *(I(t):* precipitation) into outputs (*O(t):* runoff, evaporation, transpiration). A simplified representation (a model) of this transformation operator can be written by mathematical equations and a logical statements, describing the structure for a rainfall-runoff model for the catchment. Figure 4.1 shows a watershed seen as a hydrological transformation operator.

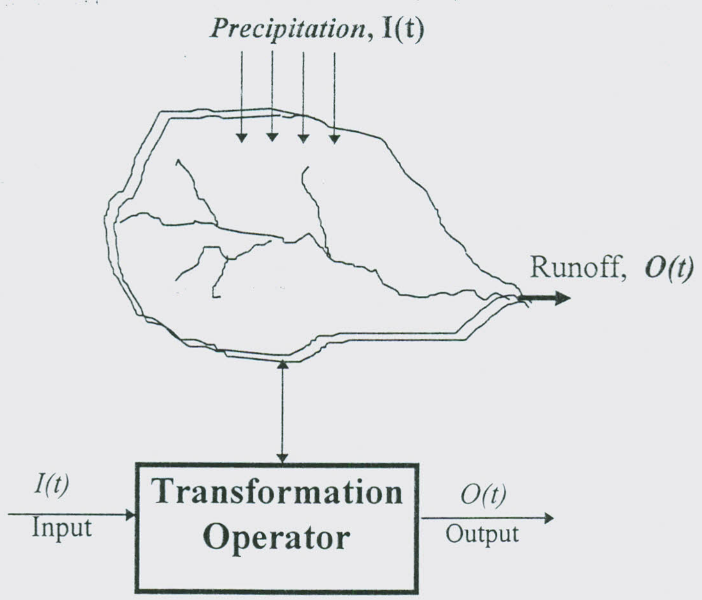

Fig. 4.1 A watershed seen as a hydrological transformation operator

Because of the great complications, it is not possible to describe all the physical processes within the watershed with exact physical laws. Using the system concept the effort is instead directed to the construction of a model representing the most important processes, and their interaction within the total system. A conceptual knowledge of the physical system will still be valuable to determine the main processes, and to develop a simplified but useful model. The term conceptual modeling is used for this type of analysis. (Killingtveit, 1993)

4.2.1 Classification of A Rainfall-Runoff Models

Several system of classification of hydrologic models have been used. In one system of classification, the models are classified according to three main criteria.

1. Randomness (deterministic or stochastic)
2. Spatial variation (lumped or distributed)
3. Time variability (time-dependent or time-independent)

In the other system of classification, hydrological models are divided into two main categories: physical models and abstract models. Physical models include scale models such as hydraulic models of a spillway, and analog models which use another physical system having properties similar to those of the real system. Abstract models represent the system in mathematical form. The system operation is described by a set of equations and logical statements (Killingtveit, 1993).

In total a number of different model classes are classified in this system. The simplest type of model will be a deterministic lumped time-independent model. The most complex type of model would be a stochastic model with space variation in three dimensions and with time variation (Killingtveit, 1993).

The rational formula is, for instance, one of the simplest and oldest deterministic models in hydrology, more popular in the design of drainage systems. Although this formula is based on a number of simplifying assumptions which can not be readily satisfied under actual circumstances, its simplicity has won it popularity (Chow, 1988).

As a final comment on the classification of models, many short-term hydrologic models could be classified as event-simulation models as contrasted with sequential or continuous models. An example of the former is the Corps of Engineers single-event model, HEC-1, and an example of the later is the Stanford watershed model.

There are quite a number of hydrological models. Some available models cover many possible applications (with varying degree of reliability) while others concentrate on one or two. Some make approximate estimates from limited information, and others require a great deal of descriptive data and use a large amount of computer time and detailed computations.

Hydrological models of various types have been extensively used for hydropower planning and single and multi-purposes reservoir operation. In the following the most important types of deterministic models are described.

HEC-1 flood hydrograph model

It was developed by the Hydrologic Engineering Center (1981) to simulate the direct runoff hydrograph due to precipitation by representing the watershed with interconnected hydrologic and hydraulic components. In addition, the model has options for multiplan-multiflood analysis, dam-break simulation, economic assessment of flood damage, and optimal sizing of flood control systems. Many elements of simulation are modeled using several options.

Infiltration is estimated using four options :

(a) initial and uniform loss rate,

(b) exponential loss rate,

(c) SCS curve-number method, and

(d) Holtan's infiltration equation.

The direct runoff hydrograph (DRH) is estimated using the unit hydroraph method (with clark, synder, and SCS Dimensionless unit hydrograph as options) and the kinematics wave method. The conic method, normal-depth storage and outflow, and modified plus method are used for storage flow routing, whereas the lag and route and Muskingum methods are used for channel routing. A univariate search technique is employed to determine optimal model parameters. The model is one of the most commonly used models in the United States and can be used for hydrologic analysis under a wide variety of conditions(Singh, 1989).

The Streamflow Synthesis and Reservoir Regulation Model (SSARR)

The SSARR model is another widely used continuous streamflow simulation model designed for large basins. It was developed primarily for streamflow and flood forecasting and for reservoir design and operation studies.

Applications of the model begin with a subdivision of the drainage basin into homogeneous hydrologic units of a size and character consistent with subdivides, channel confluences, reservoir sites, diversion points, soil types, and other distinguishing features. The stream flows are computed for all significant points throughout the river system.

Rainfall data can be input at any number of stations in the basin. The part that will run off is divided into the base flow subsurface or interflow, and surface runoff. The division is based on indices and on the intensity of the direct runoff. Each component is simply delayed according to different processes, and all are then combined to produce the final sub-basin outflow hydrograph. This sub-area runoff is then routed through stream channels and reservoirs to be combined with other sub-area hydrographs, all of which become part of the output (Viessman et.al, 1989).

The HBV Model

The HBV model was developed at the Swedish meteorological and hydrological institute. It is a conceptual precipitation-runoff model which is used to simulate the runoff process in a catchment based on data for precipitation, air temperature and potential evapotranspiration. It is a mathematical model of the hydrological processes in a catchment. It is to some extent a linear model, meaning that most of the mathematical expressions in the model are linear. Some parts of the model (e.g. in the soil moisture routine) are non-linear, but the runoff generating response function is based on linear reservoirs with some modifications

The HBV model is basically a lumped model, meaning that the catchment is treated as one unit without any considerations to the spatial variation within the catchment. It is also a deterministic model, meaning that two equal sets of inputs will always yield the same output, if it is run through the model from identical start conditions and with identical model parameters (Killingtveit, 1993).

SCS TR-20 model

The SCS TR-20 model was developed by the Soil Conservation Service (SCS, 1973) for inclusion of hydrologic processes in project formulation. The primary objective was to improve the quality of watershed projects and, at the same time, reduce overall costs by providing a means of analyzing alternative systems of structural measures. The model uses the SCS Dimensionless hydrograph method to estimate surface runoff resulting from any synthetic or natural rainfall, which is then routed through stream channels using convex method and through reservoirs using storage indication method. The model has the flexibility to accommodate other aspects of watershed planning, provision of input data and use of engineering judgment.

Watershed Hydrology Simulation Model (WASH)

The WASH model, was developed by Singh (1983,1987), is designed for the prediction of the direct runoff hydrograph for a specified rainfall event from an ungaged watershed. Rainfall hyetograph, observed at one or more points, constitutes input to the model. In addition, soil-vegetation-land use and geomorphic characteristics are needed to estimate model parameters. The model is a two-parameter linear model, the watershed unit hydrograph is determined using geomorphologic concepts involving one parameter - the watershed lag estimated simply from watershed area. The direct runoff amount is obtained from the SCS curve-number method.

5 Development of Rainfall - Runoff Model for Upper Awash Basin

5.1 Introduction

Fundamental to rainfall-runoff simulation is the determination of the amount of effective rainfall or the volume of direct runoff that a given rainfall event will produce on each portion of the watershed. Determination of this quantity is most difficult and is a major cause of error. Methods for obtaining this quantity is, for example, the SCS curve-number method.

The SCS method is developed by the Soil Conservation Service (1964, 1973) for estimating direct runoff from storm rainfall on small ungaged watersheds. Because of its simplicity and relative accuracy, it is one of the most frequently utilized methods for runoff estimation. It employs rainfall and watershed data that are ordinarily available or easily obtainable. The model can be applied to large watersheds with multiple land uses. For instance Williams and LaSeur (1976) developed a model, using the SCS curve-number technique and the soil-moisture accounting procedure, for prediction of runoff from agricultural watersheds of areas up to 2590 Km^2 (Singh, 1989).

From the above discussions it is clear that the SCS method is quite applicable in Ethiopian condition where most of the watersheds (or part of the watersheds) are not gauged and the hydrometeorological data are generally scarce. Consequently, in this research, this method is used as a major tool to develop the rainfall-runoff model for Upper Awash Basin.

The rainfall-runoff model for upper Awash is made up of two modules each module representing the most important hydrological processes. Both modules are written in the *Micro soft-Excel* spread sheet and they are quite convenient and simple to use. The modules must be run successively as the output of the first module is the input for the second module.

The first module is used to calculate the excess rainfall amount or runoff from a number of observed point rainfall readings spread over the subcatchments. One of the most important parameters which are needed to run this module are classification of the Antecedent

Moisture Condition (AMC) and the Curve Number (CN). These parameters are already determined for all subcatchments and put in the Excel spread sheet.

The second module contains the unit hydrograph ordinates of the Subcatchments with its convolution programs. It also contains the 'normal' or the routed unit hydrograph ordinates of a subcatchment where necessary. The input data to this module are the excess rainfall determined in first module. The module employs a convolution technique to determine the runoff hydrograph at desired locations The determination of the unit hydrograph ordinates for each subcatchment is fully described in article 5.7.

5.2 Estimation of Areal rainfall using Thiessen polygon method

Determination of the average amount of rain that falls a watershed during a given storm is fundamental requirement for many hydrologic studies. Many factors affect spatial variation of rain falling on the ground and generating runoff. Rain gage networks are designed to sample this distribution optimally. A method of estimating mean areal rainfall must be able to represent this distribution. Although there are several methods to estimate areal rainfall no method accurately represents this distribution. Singh and Chowdhury (1986) studied the various methods for calculating areal average precipitation, including the ones described here, and concluded that all the methods give comparable results, especially when the time period is long (Chow, 1988).

A number of techniques for estimating mean Areal rainfall have been developed. Some of these techniques are simple, well-tried and as old as modern hydrology. (Singh, 1989) The arithmetic-mean method is the simplest method for determining areal average rainfall It involves averaging the rainfall depths record at a number of gages. This method is satisfactory if the individual gauges are uniformly distributed over the area and the individual gauge measurements do not vary greatly about the mean. The other methods include Isohytal, Thiessen and Reciprocal-distance-squared method (Chow, 1988).

In this research, Thiessen polygon method is used for the purpose of estimating the areal rainfall on each sub-catchment.

In this method polygons are constructed so that surrounding each gauge is a polygon representing the area for which the point rainfall data is taken to be representative. Each polygon side is constructed along the perpendicular bisector of lines joining each pair of adjacent gauges. The area weighting coefficients are then calculated as area of each polygon divided by total area of the catchment for which the areal rainfall is to be found. Each rainfall station, therefore, will have its own weighting coefficient. Obviously the sum of all weighting coefficients in each subcatchment should be unity. Once the observed point rainfall amounts for each station are known the weighting coefficients can be used to estimate the areal rainfall on each sub-catchment. Weighting coefficients for all rainfall stations in the upper Awash are determined for all of the three subcatchments and shown in Table 5.1, 5.2 and 5.3.

Table 5.1 Weighting coefficients for rainfall stations in Subcatchment-A

Rainfall stations	Weighting Coefficients
1. Addis Alem	0.2502
2. Welenkomi	0.1039
3. Ginchi	0.1457
4. Teji	0.1796
5. Tulubolo	0.1904
6. Melka-kunture	0.1302
Sum = 1.0000	

Table 5.2 Weighting coefficients for rainfall stations in Subcatchment-B

Rainfall stations	Weighting Coefficients
1. Addis Ababa at T/Haimanot	0.1302
2. Addis Ababa at Bole	0.1302
3. Akaki	0.1448
4. Debere Zeit at IAR	0.1156
5. Melka-kunture	0.1813
6. Melka-hombole	0.2979
Sum = 1.0000	

Table 5.3 Weighting coefficients for rainfall stations in Subcatchment-C

Rainfall stations	Weighting Coefficients
1. Mojo	0.3824
2. Chefa Donsa	0.4411
3. Debere Zeit at IAR	0.1765
Sum = 1.0000	

5.3 Excess rainfall determination by SCS method (SCS, 1972)

5.3.1 General

The SCS method of estimating direct runoff (excess rainfall) from total rainfall is based on methods developed by the SCS hydrologists. The principal application of the method is in estimating quantities of excess rainfall in flood hydrographs or in relation to flood peak rates. The hydrologic principles of the method are not new, but they are put to new uses. Because most SCS work is with ungauged watersheds the method was made usable with rainfall and watershed data that are ordinarily available or easily obtainable for such watersheds.

In flood hydrology it is customary to deal separately with base flow and to combine all other types in to direct runoff, which consists of surface runoff, subsurface runoff and channel runoff in unknown proportions. The SCS method estimates direct runoff, but the proportions of surface runoff and subsurface flow are ignored, can be better appraised by means of the runoff curve number(CN), which is another indicator of the probability of flow types: the larger the CN the more likely that the estimate is of surface runoff. The most generally available rainfall data in most part of the world are the amounts measured at a non-recording rain gages, and it was for the use of such data or their equivalent the rainfall-runoff relation was developed. The data are totals for one or more storms occurring in a calendar day, and nothing is known about the time distributions. The relation therefore excludes time as an explicit variable; this means that rainfall intensity is ignored. If everything but storm duration or intensity is the same for two storms, the estimate of the runoff is the same for both storms.

The SCS method can, therefore, be useful for Ethiopian watersheds in general where there is scarcity of hydrometeorological data.

5.3.2 Development of the Curve Number (CN) equations

A combination of a hydrologic soil group (soil) and a land use and treatment class (cover) is a hydrologic soil-cover complex. The Curve Number(CN) of a given drainage basin indicates the runoff potential of a soil complex or the excess rainfall for a given amount of total rain; the higher the CN the higher the potential (SCS, 1972).

If records of natural rainfall and runoff for a large storm over a small area are used, a plotting of accumulated runoff versus accumulated rainfall will show that runoff starts after some rain accumulate (there is an 'initial abstraction' of rainfall) and that the double -mass line curves, becoming asymptotic to a straight line. The relation between rainfall and runoff can be related from this plotting, but a better understanding of the relation is found by first studying a storm in which rainfall and runoff begin simultaneously (the initial abstraction does not occur). For the simpler storm the relation between rainfall, runoff, and retention (the rain not converted to runoff) at any point on the mass curve can be expressed as (SCS, 1972).

$$\frac{Fa}{S'} = \frac{Pe}{P} \tag{5.1}$$

where;

Fa = actual cumulative abstraction

S' = potential maximum retention (S'>F)

P = cumulative total rain

Pe = actual excess rainfall (direct runoff) ($P \geq Pe$)

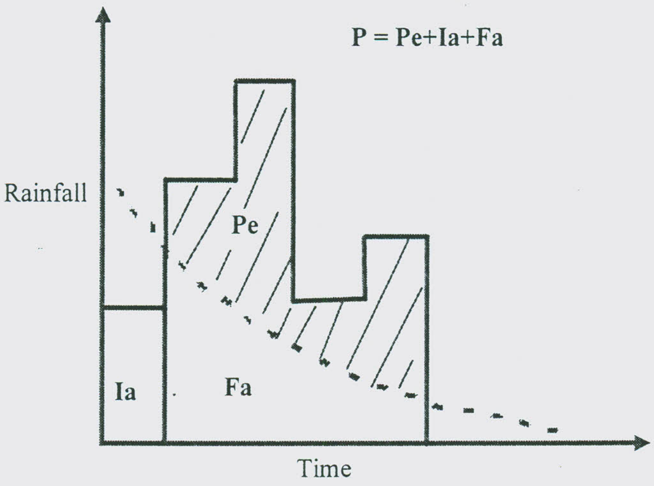

Fig. 5.1 Variables in the SCS method of rainfall abstractions.(Source: Chow, 1988)

(Ia = initial abstraction, Pe = excess rainfall, Fa = cummulative abstraction, P = total rainfall)

Note that direct runoff and excess rainfall are in effect similar when they are expressed in linear units (cm or mm).

Equation 5.1 applies to on-site runoff; for large watersheds there is a lag in the appearance of the runoff at a stream gauge, and the double-mass curve produces a different relation. But if rainfall totals for P and Pe are used equation 5.1 does apply even for large water shades because the effects of the lag are removed.

The parameter S' in equation 5.1 does not contain the initial abstraction and differs from the parameter S to be used later. The abstraction S' is a constant for a particular storm because it is the maximum that can occur under the existing conditions if the storm continues with out limit. The actual cumulative abstraction F varies because it is the difference between P and Q at any point on the mass curve, or

$$Fa = P - Pe \tag{5.2}$$

Equation 5.1 can therefore be written as:

$$\frac{P - Pe}{S} = \frac{Pe}{P} \qquad (5.3)$$

Solving for Pe produces the equation:

$$Pe = \frac{P^2}{P + S} \qquad (5.4)$$

which is a rainfall-runoff relation in which the initial abstraction is ignored.

The initial abstraction is brought in to the relation by subtracting it from the rainfall. The equivalent of equation 5.1 becomes:

$$\frac{Fa}{S} = \frac{Pe}{P - Ia} \qquad (5.5)$$

where Ia is the initial abstraction, $F \leq S$ and $Pe \leq (P - Ia)$. The parameter S includes Ia; that is, $S = S' + Ia$.

Equation 5.2 becomes:

$$Fa = (P - Ia) - Pe \qquad (5.6)$$

Equation 5.3 becomes:

$$\frac{(P - Ia) - Pe}{S} = \frac{Pe}{(P - Ia)} \qquad (5.7)$$

Equation 5.4 becomes:

$$Pe = \frac{(P - Ia)^2}{((P - Ia) + S)} \qquad (5.8)$$

Equation 5.8 is the rainfall-runoff relation with the initial abstraction taken in to account.

The initial abstraction consists mainly of interception, infiltration, and surface storage, all of which occur before runoff begins. To remove the necessity of estimating some variables in equation 5.8, the relation between Ia and S (which includes Ia) was developed by means of rainfall and runoff data from experimental small watersheds. The empirical relationship is.

$$Ia = 0.2 \times S \qquad (5.9)$$

Substituting 5.9 in 5.8 gives:

$$Pe = \frac{(P - 0.2 \times S)^2}{(P + 0.8 \times S)} \qquad (5.10)$$

Equation 5.10 is the total rainfall-runoff (excess rainfall) relation used in the SCS method of estimating direct runoff from rainfall.

Equations 5.6 and 5.10 can be combined to get a relation for the actual cumulative abstraction (Fa) as:

$$Fa = \frac{S(P - Ia)}{(P - Ia + S)} \qquad (5.11)$$

5.3.3 Retention Parameters and Antecedent Moisture Conditions (AMC)

Equation 5.9 states that an average of 20 percent of the potential maximum retention S is the initial abstraction Ia, which is the interception, infiltration, and surface storage occurring before the runoff begins. The remaining 80 percent is mainly the infiltration occurring after runoff begins. This later infiltration is controlled by the rate of infiltration at the soil surface or by the rate of transmission in the soil profile, or by the water-storage capacity of the soil profile, whichever is the limiting factor. A succession of storms, such as one a day for a week, reduces the magnitude of S each day because the limiting factor does not have the opportunity to completely recover its rate or capacity through weathering, evapotranspiration, or drainage. But there is enough recovery, depending on the soil-cover complex, to limit the reduction. During such a rainfall period the magnitude of S remains virtually the same after the second or third day even if the rain are large so that there is, from a practical view point, a lower limit to S for a given soil-cover complex. Similarly there is a practical upper limit to S, again depending on the soil-cover complex, beyond which the recovery cannot take S unless the complex is altered.

In the SCS method the change in S (actually in CN) is based on an antecedent moisture condition (AMC) determined by the total rainfall in the 5-day period preceding a storm. Three levels of AMC are used: AMC-I is the lower limit of moisture or the upper limit of S, AMC-II is the average moisture condition for which the CN values are tabulated, and AMC-III is the upper limit of moisture or the lower limit of S. The CN for high and low moistures levels were empirically related to the average moisture levels(AMC-II) as shown below.

$$CN(I) = \frac{(4.2 \times CN(II))}{(10 - 0.058 \times CN(II))} \qquad (5.12)$$

$$CN(III) = \frac{(23 \times CN(II))}{(10 + 0.13 \times CN(II))} \qquad (5.13)$$

The parameter CN (runoff curve number or the hydrologic soil-cover complex number) is a transformation of S, and it is used to make interpolating, averaging, and weighing operations more nearly linear. The transformation (where S is in linear mm) is :

$$S = \frac{25400}{CN} - 254 \qquad (5.14)$$

5.3.4 Classification of Antecedent Moisture Conditions

The amount of runoff expected from a given total rainfall depends, among other factors, on the antecedent moisture condition (AMC) of the soil. SCS has set some values for the classification of AMC based on the total 5-day antecedent rainfall. For instance when the total 5-day antecedent rainfall is below 36 mm, from 36 mm to 53 mm and above 53 mm, the AMC group is taken to be Dry (AMCI), Normal (AMCII) and Wet (AMCIII) respectively.

Classification of antecedent moisture classes for the SCS method of abstractions is shown in Table 5.4. (Chow, 1988)

Table 5.4 SCS classification for antecedent moisture conditions

Total 5-day antecedent rainfall (mm)		
AMC group	Dormant season	Growing season
I (Dry)	below 12.7	below 35.6
II (Normal)	12.7 - 27.9	35.6 - 53.3
III (Wet)	above 27.9	above 53.3

The above classification values are, however, only guide-lines and therefore can not be directly adopted for our purpose except that they only give some idea about the classification procedures. It is usually advisable to set the final values of classification after making some trials with different classification values and choosing the ones which give the best result in simulating the rainfall-runoff processes; which is in effect a calibration exercise. But it must also be put in mind that these values can not be arbitrarily chosen rather the values must be in the order of the SCS recommended values shown in Table 5.4.

During the rainfall-runoff simulation of the upper Awash Basin, it was observed that Subcatchments-A&B can not have the same classification of AMC as that of subcatchment-C. This is due to the fact that AMC classification depends heavily on the hydrological soil cover complex which includes land use, soil type, etc. It can also be observed that the AMC classification depends also on the season of the year. Since our purpose here is to simulate the rainfall-runoff process during wet season, when a lot of land is covered with crops, this process needs to be taken in to consideration. On the 'normal wet season', rainfall on the upper Awash Basin starts near the end of June and lasts until mid of September. The AMC classification, therefore, takes in to consideration the average rainfall condition in each month of the wet season as shown in Table 5.5 and 5.6.

Table 5.5 **Classification of AMC for Subcatchments-A & B**

Total 5-day antecedent rainfall (mm)		
	Dates of the wet season	
AMC group	July 11 - August 9	August 10 - End of wet season
I (Dry)	below 41	below 20
II (Normal)	41 - 75	20 - 75
III (Wet)	above 75	above 75

Also note that from beginning of wet season (June 20) up to July 10, AMC is Dry.

Table 5.6 Classification of AMC for Subcatchment-C

Total 5-day antecedent rainfall (mm)	
AMC group	
I (Dry)	below 24
II (Normal)	24 - 42
III (Wet)	above 42

Also note that from beginning of wet season (June 20) up to July 10, AMC is Dry.

5.3.5 Estimation of CN for The Upper Awash Basin

Basis for the estimation of the CN is found from the soil and land-use maps of the Upper Awash Basin.

A considerable volume of data has been generated by soil and land classification surveys in the Awash Basin since the first formal studies carried out by Sogreah in 1965. The results of these classifications are seldom easily comparable and occasionally conflict. Some of the divergence in the land classification results obtained in these surveys relates to the adoption of different standard methodologies (Halcrow, 1989).

The most recent study conducted on the Awash Basin is that of the 1989 Halcrow study on the Basin's water resource development. However, Halcrow's Soil and Land Resource study was concerned with the collection and analysis of the considerable number of previous soil and land classification surveys in order to assess the total potential irrigable area in the Awash Basin and it has listed a number of previous soil surveys in the Awash valley. Generally, the report doesn't include any recently made land use and soil map for the Upper Awash Basin

Concerning the soil surveys, Halcrow has listed those made on Becho plains by the Water Resources Development Authority in 1983 and it has commented that this soil map is not available; and the other survey is the one made on the whole Awash Basin by FAO/Sogreah in 1965 (Halcrow, 1989). And quite a number of other surveys are also listed which were conducted on the Upper, Middle, and lower Valleys which are found out side the research area of this thesis.

Therefore, in the absence of a recently made soil and land-use map on the Upper Awash Basin, it was necessary to estimate the average CN based on the land-use and soil map Survey made by FAO/Sogreah to a scale of 1:1,000,000 and reported in "Survey of the Awash Basin,Vol.II, FAO,1965.'' Obviously so many changes have taken place, specially, on the land use of the Awash basin since 1965. But there are no other comprehensive land-use studies recently made on the basin. It was, therefore, necessary to use the available data as a guide to approximate the CN. The approximated CN are further tuned during the rainfall-runoff simulation exercise to improve the CN values. By so doing the CN values which were approximated from the 1965 land-use survey are expected to improve according to, specially, the land-use changes.

From the 1965 Survey of the Awash River Basin, the soil complexes and land-uses of Upper Awash Basin can be approximated as shown below.

i)The soil condition in the upper Awash is 50% sandy-clay-loam (C), and 50% clay-loam, silty-clay-loam, sandy-clay, silty-clay or clay(D).

ii)The land cover condition is assumed to be (during winter)

* cultivated agricultural land with:

1/3 smallgrain for average hydrological condition, and

1/3 fallow (crop residue cover) on average

*other agricultural land with:

1/6 pasture and grass land with fair condition

1/6 wood and grass combination with good condition

With the above assumptions, the Normal condition Curve number, CN(II) is found to be 83. However, during the simulation of the rainfall-runoff process, the CN(II) for Subcatchment-C was readjusted to 75.

Therefore, the dry and wet condition curve numbers are calculated using the equations 5.9, 5.12, 5.13, and 5.14.and the Cumulative abstraction (Fa) and excess rainfall (Pe) are calculated from equations 5.6 and 5.10.

Subcatchments-A and -B

For Dry antecedent condition, AMC-I

CN(I)=67.2

S=124.0

Ia=24.8

Fa=124(P-24.8)/(P+99.2)

Pe=P-24.8-Fa

For Normal antecedent condition, AMC-II:

CN(II)=83

S=52.02

Ia=10.4

Fa=52.02(P-10.4)/(P+41.6)

Pe=P-10.4-Fa

For Wet antecedent condition, AMC-III

CN(III)=91.82

S=22.63

Ia=4.53

Fa=22.63(P-4.53)/(P+18.1)

Pe=P-4.53-Fa

Subcatchment-C

For Dry antecedent condition, AMC-I

CN(I)=56

S=199.6

Ia=39.9

Fa=199.6(P-39.9)/(P+159.9)

Pe=P-39.9-Fa

For Normal antecedent condition, AMC-II:

CN(II)=75

S=84.7

Ia=16.9

Fa=84.7(P-16.9)/(P+67.8)

Pe=P-16.9-Fa

For Wet antecedent condition, AMC-III

CN(III)=87

S=38

Ia=7.6

Fa=38(P-7.6)/(P+30.4)

Pe=P-7.6-Fa

5.4 Hydrographs

Hydrographs, or some elements of them such as peak rates, are used in planning and design of water control structures. They are also used to show the hydrologic effects of the existing or proposed watershed projects.

Runoff occurring on the uplands flows downstream in various patterns of flow which are affected by many factors such as spatial and temporal distribution of rainfall, hydraulics of streams, watershed and channel storage, and others that are difficult to define. The graph of flow (rate versus time) at a stream section is a hydrograph, of which no two are exactly alike. There is no satisfactory mathematical analysis of flood hydrographs, and the empirical relations have been developed, starting with the "Rational Method" and progressing to the Unit Hydrograph method in the 1930's, and to more recent use of Dimensionless or Index hydrographs. The empirical relations are simple elements from which as complex a hydrograph may be made as needed.

There are different types of hydrographs suitable for watershed work. Some of them are natural hydrographs, obtained directly from the flow records of a gauged stream; synthetic hydrographs, obtained by using watershed parameters and storm characteristics to simulate a natural hydrograph; unit hydrographs, which are natural or synthetic hydrographs for a unit of excess rain (direct runoff); and Dimensionless hydrographs, made to represent many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units.

5.5 Unit Hydrographs

A major step forward in hydrological analysis was the concept of the unit hydrograph introduced by an American engineer Sherman in 1932. He defined the unit hydrograph as the hydrograph of surface runoff resulting from effective rainfall falling in the unit time such as 1 hour or 1 day and produced uniformly in space and time over the total catchment area. (Shaw, 1983). The unit hydrograph procedure assumes that discharge at any time is proportional to the volume of runoff and that time factors affecting hydrograph shape are constant.

Both field data and laboratory tests have shown that the assumption of a linear relationship between watershed components is not strictly true. The non-linear relationships have not been investigated sufficiently to ascertain their effects on a synthetic hydrograph.

The fundamental principles of invariance and superposition make the unit hydrograph an extremely flexible tool for developing synthetic hydrographs: 1) the hydrograph of surface runoff from a watershed due to a given pattern of rainfall is invariable, and 2) the hydrograph resulting a given pattern of rainfall excess can be built up by superimposing the unit hydrograph due to the separate amounts of rainfall excess occurring in each unit period. This includes the principle of proportionality by which the ordinates of the hydrograph are proportional to the volume of rainfall excess.

5.5.1 Dimensionless Unit hydrograph

The Dimensionless unit hydrograph was derived from a large number of natural unit hydrographs from watersheds varying widely in size and geographical locations. This Dimensionless curvilinear hydrograph, shown in Figure 5.2, has its ordinate values expressed in a Dimensionless ratio q/Qp and its abscissa values as t/Tp. This unit hydrograph has a point of inflection approximately 1.70 times the time-to-peak (Tp) and the time-to-peak 0.2 of the time-of-base (Tb). The lag time (Tl) is defined as the time from the center of mass of excess rainfall (Tr) to the time to peak (Tp) of a unit hydrograph.

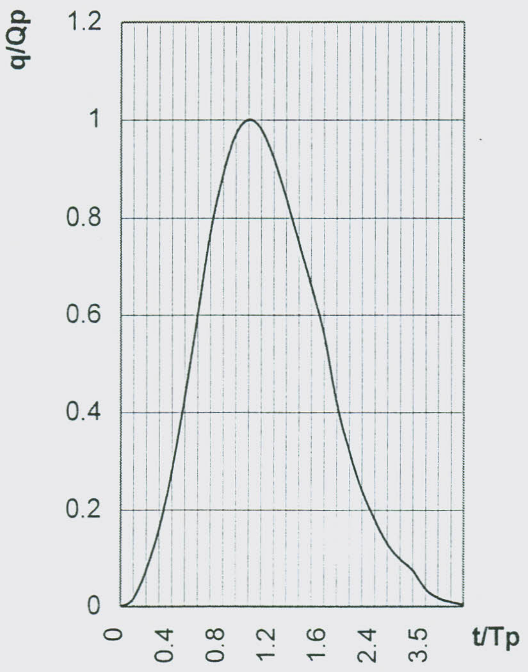

Fig. 5.2 Dimensionless unit hydrograph. (Source:, W. Viessman, 1989)

Table 5.7 Coordinates of SCS Dimensionless unit hydrograph. (Source:, Viessman, 1989)

t/t_p	q/q_p	t/t_p	q/q_p
0	0	1.4	0.75
0.1	0.015	1.5	0.66
0.2	0.075	1.6	0.56
0.3	0.16	1.8	0.42
0.4	0.28	2.0	0.32
0.5	0.43	2.2	0.24
0.6	0.60	2.4	0.18
0.7	0.77	2.6	0.13
0.8	0.89	2.8	0.098
0.9	0.97	3.0	0.075
1.0	1.00	3.5	0.036
1.1	0.98	4.0	0.018
1.2	0.92	4.5	0.009
1.3	0.84	5.0	0.004

5.6 Duration of Unit Hydrographs

The unit time or 'unit hydrograph duration' is the optimum duration for occurrence of precipitation excess._ It has been stated by some investigators that the unit hydrograph duration should not be more than the period of rise or time of concentration. It is a good rule to select a period of 12 hours as a unit hydrograph duration for catchments over 2500 Km2, 12, 8, or 6 hours for catchements between 2500 to 250 Km^2. Where rainfall records for less than daily period are not available, and the area is greater than 1250 Km^2, even one day (24 hours) may be adopted as a unit hydrograph duration (Garg, 1989).

Generally speaking ,the upper Awash Basin has no any continuos or lesser duration rainfall record except the daily one. Moreover each of the three Subcatchments has a catchment area greater than 1250 Km^2. Consequently, the unit hydrograph duration for the upper Awash Subcatchments is chosen to be 1-day (24 hours) based on the recommendations made above. In fact, at present, deriving unit hydrographs of lesser duration would be of no importance for runoff calculation as there are very few rain-gauge stations which take a record of less than one day.

5.7 Establishing SCS Unit Hydrographs for Subcatchments

The SCS (Soil Conservation Service) method is one of the widely used method in deriving rainfall-runoff models. Because most SCS work is with ungaged watersheds and its consideration of the physical characteristics, the method is quite applicable for watersheds with limited hydrometeorological data, like the Awash. Consequently this method is followed in developing the rainfall-runoff model. The SCS unit hydrograph can be constructed for the upper Awash basin once the values of Qp and Tp are determined. In this method, Qp and Tp are empirically expressed as a function of the average slope of the subcatchment, stream length, and normal condition curve number (CNII). (Haan, 1996)

$$S = \frac{1000}{CN(II)} - 10 \qquad (5.15)$$

$$Tl = \frac{L^{0.8} \times (S+1)^{0.7}}{1900 \times (Y)^{0.5}} \qquad (5.16)$$

$$Qp = \frac{2.08 \times A}{Tl} \qquad (5.17)$$

$$Tb\,\text{max.} = 5 \times Tp \qquad (5.18)$$

where;

CN(II) = normal condition curve number

L = stream length in feet.

A = area of the subcatchment in Km^2.

Y = the average slope of the subcatchment in percent.

S = the potential maximum retention in Inch.

Tl = Lag time in hours.

Tp = time-to-peak ((Tr/2)+Tl) in hours.

Tr = duration of rainfall (here fixed to 24 hours)

Tbmax. = the maximum possible base time of the unit hydrograph (could be lesser).

Qp = peak flow in m^3/s.

5.7.1 SCS Hydrograph for Subcatchment-A at Melka-kunture station

Area of the subcatchment, A=4541 Km^2

Stream length, L =118 Km = 387139 feet.

Average slope of the catchment, Y =0.64%

Duration of rainfall Tr =24 hours.

Curve number, CNII =83

Using the formulas (5.15), (5.16), (5.17) and (5.18);

Potential maximum retention, S=2.05

Lag time, Tl=42.4 hrs.

Time-to-peak, Tp=54.4 hrs.

Maximum base time, Tbmax.=272hrs.

Peak flow, Qp=173.6cumcs per cm.

Now the ordinates of the unit hydrograph can be calculated using the ordinates of the Dimensionless unit hydrograph with Tp and Qp shown in Table 5.7.

Table 5.8 Unit hydrograph Ordinates for Subcatchment-A

t/Tp	q/Qp	t(days)	$q(m^3/s$ per cm)
0	0	0	0
0.441	0.341	1	59.20
0.882	0.956	2	165.89
1.323	0.819	3	142.23
1.726	0.447	4	77.53
2.203	0.239	5	41.51
2.643	0.123	6	21.37
3.084	0.068	7	11.88
3.524	0.035	8	5.97

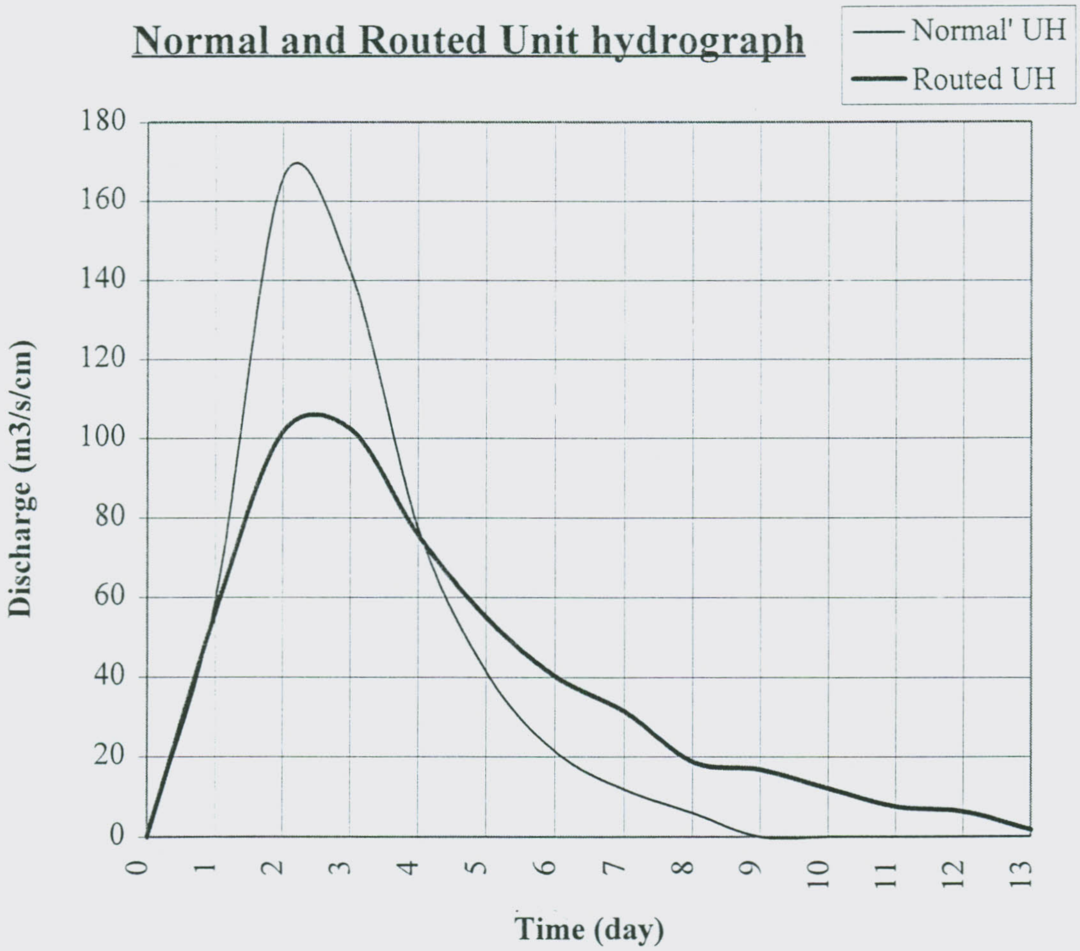

Fig. 5.3 'Normal' and Routed Unit hydrograph for Subcatchment-A

5.7.2 SCS Hydrograph for Subcatchment-B at Koka inlet

Area of the subcatchment, $A=4200\ Km^2$

Stream length, L =158 Km = 518373 feet.

Average slope of the catchment, Y =0.53%

Duration of rainfall Tr =24 hours.

Curve number, CNII =83

Using the formulas (5.15), (5.16), (5.17) and (5.18);

Potential maximum retention, S=2.05

Lag time, Tl=58.9 hrs.

Time-to-peak, Tp=70.9 hrs.

Maximum base time, Tbmax.=354.5hrs.=14.8days.

Peak flow, Qp=123.2 m^3/s per cm.

Now the ordinates of the unit hydrograph can be calculated using the ordinates of the Dimensionless unit hydrograph with Tp and Qp shown in Table 5.7

Table 5.9 Unit hydrograph Ordinates for Subcatchment-B

t/Tp	q/Qp	t(days)	q(m^3/s per cm)
0	0	0	0
0.3385	0.2062	1	25.40
0.6770	0.7309	2	90.05
1.0156	0.9969	3	122.82
1.3541	0.7913	4	97.49
1.6926	0.4952	5	61.01
2.0311	0.3076	6	37.89
2.3697	0.1891	7	23.30
2.7082	0.1127	8	13.88
3.0467	0.0714	9	8.77
3.3852	0.0450	10	5.51

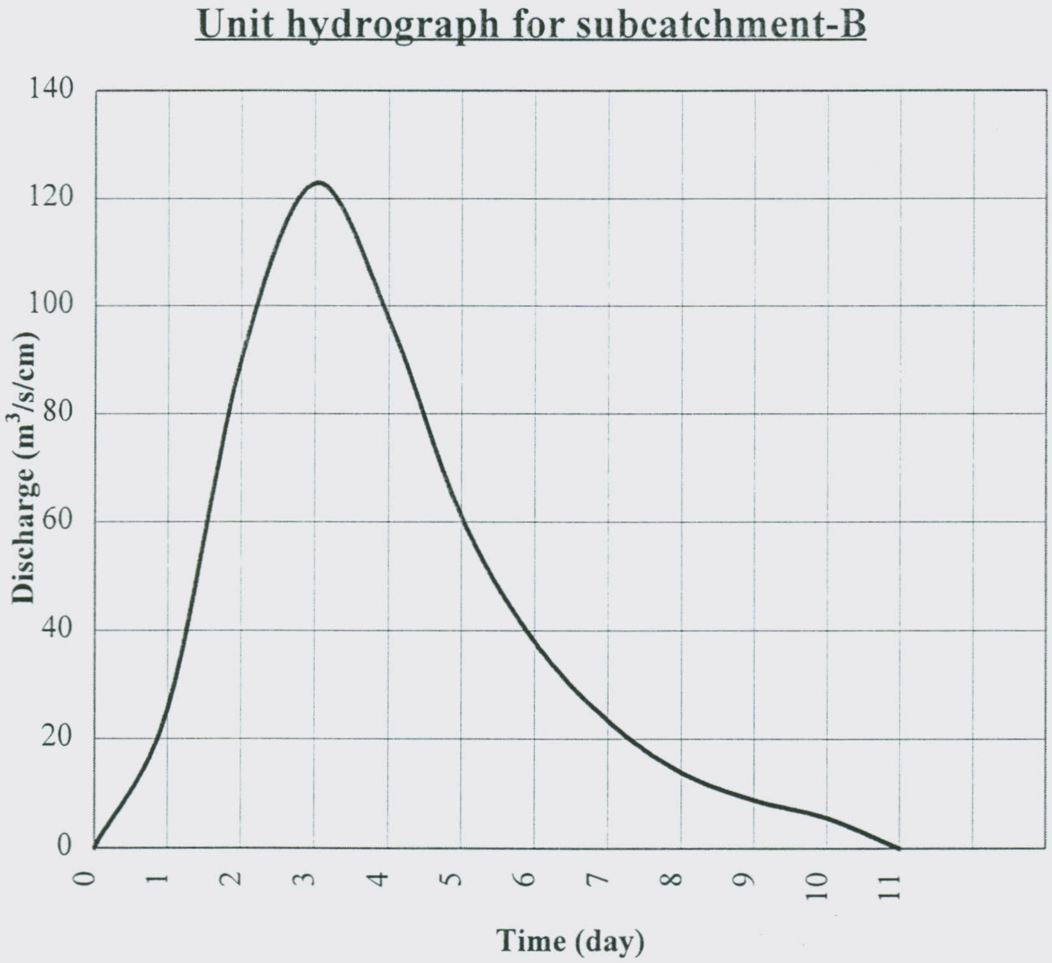

Fig. 5.4 Unit hydrograph for Subcatchment-B at koka Inlet

5.7.3 SCS Hydrograph for Subcatchments-(A+B) at Koka inlet

Note that Subcatchment-(A+B) is the sum of Subcatchment-A and Subcatchment-B taken as one catchment (drainage system).

Area of the subcatchment, A=8741 Km^2

Stream length, L =212 Km = 695538 feet.

Average slope of the catchment, Y =0.48%

Duration of rainfall Tr =24 hours.

Curve number, CNII =83

Using the formulas (5.15), (5.16), (5.17) and (5.18);

Potential maximum retention,, S=2.05

Lag time, Tl=78.25 hrs.

Time-to-peak, Tp=90.2 hrs.

Maximum base time, Tbmax.=451hrs.=18.8days

Peak flow, Qp=201.6 m^3/s per cm.

Now the ordinates of the unit hydrograph can be calculated using the ordinates of the Dimensionless unit hydrograph with Tp and Qp shown in Table 5.7.

Table 5.10 Unit hydrograph Ordinates for Subcatchment-(A+B)

t/Tp	q/Qp	t(days)	q(m^3/s per cm)
0	0	0	0
0.2661	0.1312	1	26.98
0.5322	0.4847	2	97.72
0.7983	0.8880	3	179.01
1.0644	0.9871	4	199.00
1.3305	0.8126	5	163.81
1.5966	0.5634	6	113.58
1.8627	0.3887	7	78.35
2.1288	0.2685	8	54.13
2.3949	0.1995	9	40.23
2.6610	0.1202	10	24.24
2.9271	0.0834	11	16.81
3.1932	0.0599	12	12.08
3.4593	0.0392	13	6.28

Therefore, the unit hydrograph ordinates (m^3/s/cm) are;

U1=26.98, U2=97.72, U3=179.01, U4=199.00 ,U5=163.81, U6=113.58, U7=78.35, U8=54.13, U9=40.23, U10=24.24, U11=16.81, U12=12.08, U13=6.28

5.7.4 SCS Hydrograph for Subcatchment-C at Koka inlet

Area of the subcatchment, A=1900 Km^2.

Stream length, L =105 Km = 344488 feet.

Average slope of the catchment, Y =0.80%

Duration of rainfall Tr =24 hours.

Curve number, CNII =75

Using the formulas (5.15), (5.16), (5.17) and (5.18);

Potential maximum retention, S=3.30

Lag time, Tl=50.9 hrs.

Time-to-peak, Tp=62.9 hrs.

Maximum base time, Tbmax.=314.5hrs.=13days.

Peak flow, Qp=62.8 m^3/s per cm.

Now the ordinates of the unit hydrograph can be calculated using the ordinates of the Dimensionless unit hydrograph with Tp and Qp shown in Table 5.7.

From the observed runoff data it was observed that the hydrograph of Subcatchment-C (Mojo subcatchment) is smoother than that of the calculated. As a result the calculated unit hydrograph ordinates were adjusted keeping the sum of the ordinates the same (1 cm) during the calibration process to improve the simulation as follows:

Therefore, the unit hydrograph ordinates ($m^3/s/cm$) are;

U1=10.2, U2=27.00, U3=40.50, U4=35.50, U5=25.00, U6=20.59, U7=18.50, U8=16.00, U9=10.50, U10=8.00, U11=5.20, U12=3.00

Table 5.11 Unit hydrograph Ordinates for Subcatchment-C

t/Tp	q/Qp	t(days)	q(m^3/s per cm)
0	0	0	0
0.3816	0.2062	1	12.10
0.7631	0.7309	2	43.00
1.1447	0.9969	3	61.50
1.5262	0.7913	4	59.00
1.9078	0.4952	5	23.20
2.2893	0.3076	6	14.40
2.6709	0.1891	7	7.50
3.0525	0.1127	8	4.70

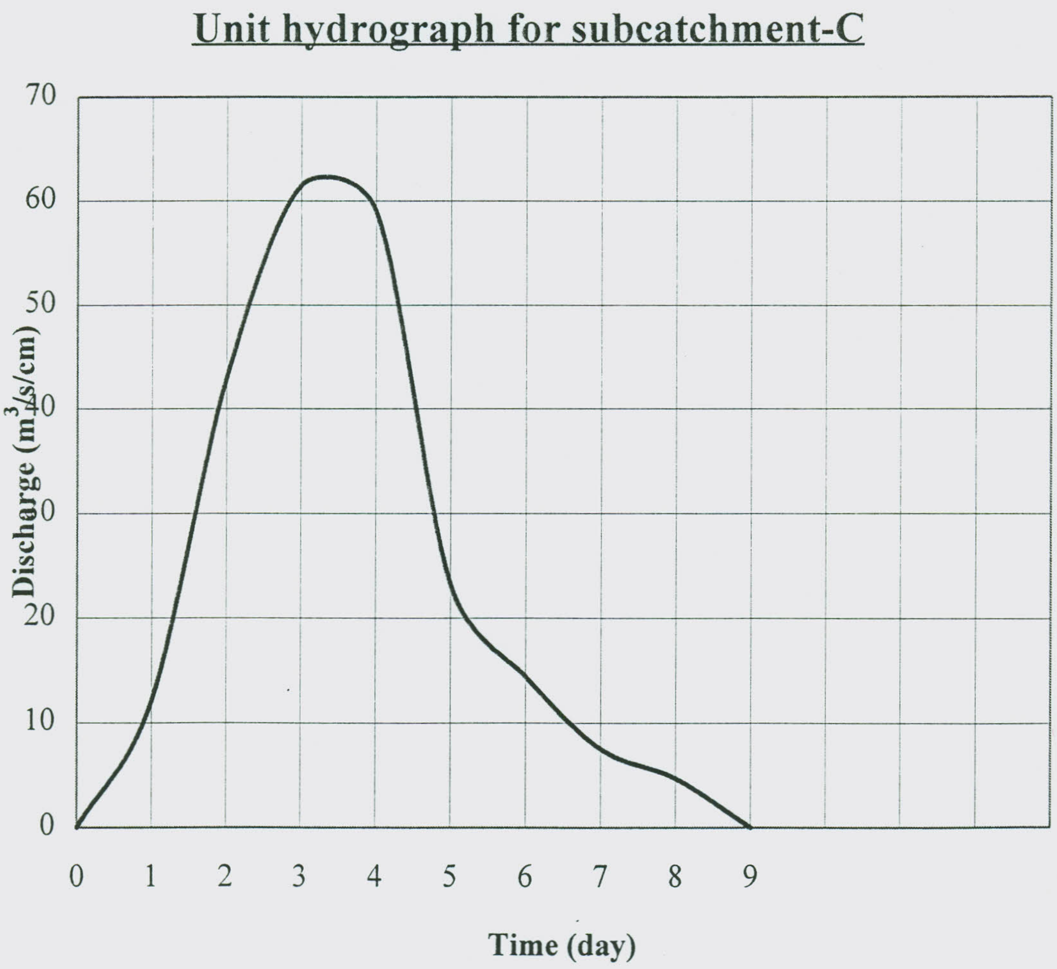

Fig. 5.5 Unit hydrograph for Subcatchment-C at koka Inlet

5.7.5 SCS Hydrograph for Local catchment

The local catchment is found on the south-eastern side of Koka reservoir. There is no defined stream which drains this local catchment rather it directly flows in to Koka reservoir. The peak flow calculated in a similar way to the other Subcatchments is about 133.5 m^3/s. But the time base of this catchment is only 19.8 hours (less than one day) ; therefore there is no need for unit hydrograph rather this can simply be taken as an inflow when ever there is excess rainfall in the area. More conveniently, the inflow contribution of the local catchment could be included in subcatchment-C by area proportion to simplify the calculation of the total inflow in to the reservoir.

5.8 Total Simulated Runoff

The Computation of the total simulated runoff (total inflow in to Koka reservoir) is done by proceeding from the most remote upstream subcatchment in a downstream direction. This requires, depending up on its location, each of the Subcatchments to be routed during its journey to the catchment outlet. The total simulated inflow is then found using the principle of superposition all at the same outlet (Koka inlet).

Based on the division made on the upper Awash sub-catchements, both subcatchment - B and C have the same outlet at Koka reservoir unlike subcatchment-A whose outlet is at Melka-kunture gauging station which is well upstream of Koka. Hence, the unit hydrograph subcatchment-A should be routed through the Awash river channel from Melka-kunture gauging station up to Koka inlet. Considering the local catchment, there is no defined stream which drains this local catchment rather it directly flows in to Koka reservoir. The peak flow calculated in a similar way to the other Subcatchments is about 133.5 m^3/s. But the time base of this catchment is only 19.8 hours (less than one day) ; therefore there is no need to establish a unit hydrograph. The obvious alternative is to calculate the daily inflow whenever there is excess rainfall in the area. But for ease of calculation, the contribution of the local catchment is included in Subcatchment-C and the a factor is used to account for this additional area.

The total inflow is, therefore, found by superposing the routed runoff of Subcatchment-A, Subcatchments-B, the factored runoff of Subcatchment-C and the base flow.

5.8.1 Flow Routing

Theoretically, flow routing can be effected by using a number of routing methods. Suppose that the Muskingum method is chosen for flow routing through river channels. Then a number of parameters have to be found. Data like channel cross-sections and simultaneous runoff observations at the two ends of the river; in our case at Melka-kunture and Koka inlet, are required in order to determine Muskingum parameters which are hardly available. Therefore, the applications of the Muskingum and other flow routing methods are limited by lack of needed data.

It was, therefore, necessary to look for other alternatives which can enable us to know the routed (attenuated and delayed) hydrograph of subcatchment-A when it gets at Koka inlet. The most applicable proposal is to subtract the unit hydrograph of subcatchment-B at Koka inlet from unit hydrograph of Subcatchments-(A+B) at Koka inlet. This procedure will automatically give us the unit hydrograph of subcatchment-A at Koka inlet as is normally found by other methods of river routing. Finally, the routed unit hydrograph ordinates, found using the above technique, are 'fine-tuned' to improve the rainfall-runoff simulation result. The Unit hydrograph found in this way can be refereed to as ***the routed unit hydrograph of subcatchment-A at Koka inlet'***.

This type of approach is fully discussed in the 1982 Water Resource Publication under the title Modeling Component of Hydrologic Cycle: "A Study of Mathematical Rainfall-Runoff Modeling", edited by V.P. Singh.

Routed-unit hydrograph of subcatchment-A at Koka inlet (in $m^3/s/cm$) are

U1=1.58, U2=7.67, U3=56.19,U4=101.51, U5=102.28, U6=75.69, U7=55.05,
U8=40.25, U9=31.46, U10=18.73, U11=16.81, U12=12.08, U13=6.28

The above ordinates are adjusted keeping the sum of the ordinates the same (1 cm) during the calibration process to improve the simulation as follows:

$U1=56.19$, $U2=101.51$, $U3=102.28$, $U4=75.69$, $U5=55.05$, $U6=40.25$, $U7=31.46$, $U8=18.75$, $U9=16.81$, $U10=12.08$, $U11=7.57$, $U12=6.28$, $U13=1.68$

5.9 How to Use the Rainfall-Runoff model

In the previous articles it was shown how all important parameters for the rainfall-runoff model have been determined for each subcatchment. In this article a brief description is given how to enter the input data and run the model in Excel spread sheet. The format is self explanatory and simple to use. As is always the case, it is required that before taking the output of the model the user's judgment is always important in interpreting the results and making some corrections if necessary.

This model consists of four Excel files. The first three are used to simulate the direct runoff contribution of each sub-catchment for any causative rainfall. It is important to note that if there is no rainfall in a particular day(s) the rainfall values must be entered as zero. It must be bore in mind that all the simulated direct runoff are at the Koka inlet. For instance, the excel file which simulates subcatchment-A is not at the outlet to this subcatchment it is rather the routed runoff at Koka inlet. Hence care must be exercised when using the model outputs. The logic behind this technique is already described in article 5.8.1.

The last excel file is used to sum up all the simulated runoffs of each subcatchment and the baseflow to find the total inflow in to Koka reservoir. Note that while working in these Excel files the original format and all settings must always be retained. Moreover. it is advisable to open an independent file for each of the four excel files

Some important steps are described as follows:

The flow chart shown on figure 5.6 outlines important steps to be followed when using the rainfall-runoff model.

Step1) The first step is to enter the point rainfall data of all rainfall stations in the assigned place in the excel files. When the first step is completed, the file automatically calculates the Areal rainfall and 5-days antecedent moisture condition indicator (AMC-indicator) are automatically calculated.

Step2) The second step needs some attention because the user has to see the calculated AMC-indicator to determine whether the AMC of the subcatchment at each day is dry, normal or wet. The AMC can be determined by referring to the already established AMC classifications in article 5.3.4. These classifications are found at top of each Excel file for the respective Subcatchments.

Step3) Once the AMC is decided, then the user has to select the appropriate formula assigned for each AMC to calculate the excess rainfall amount. This is because each AMC has a different excess rainfall calculation formula as formulated in article 5.3.5. Note that all the alternative formula pertaining to the AMC conditions are found in these files. When this step is completed the excess rainfall matrix will be calculated and made ready for the next step.

Step4) The fourth step is copying the excess rainfall matrix and pasting them in the field consisting of the unit hydrograph matrix. This field uses the principle of convolution and converts the excess rainfall in to a time series of direct runoff. Note that the same step is followed for all of the three Excel work sheets.

Step5) In the final step, we work with the fourth Excel file which calculates the total runoff. One needs to copy the simulated direct runoff from each Excel sheet and paste them in this file at the already assigned place. The time series of total runoff is calculated by adding the base flow and the simulated runoff. The base flow is already determined itteratively during the development stage of the model from the observed discharges of Melka-hombole and Mojo.

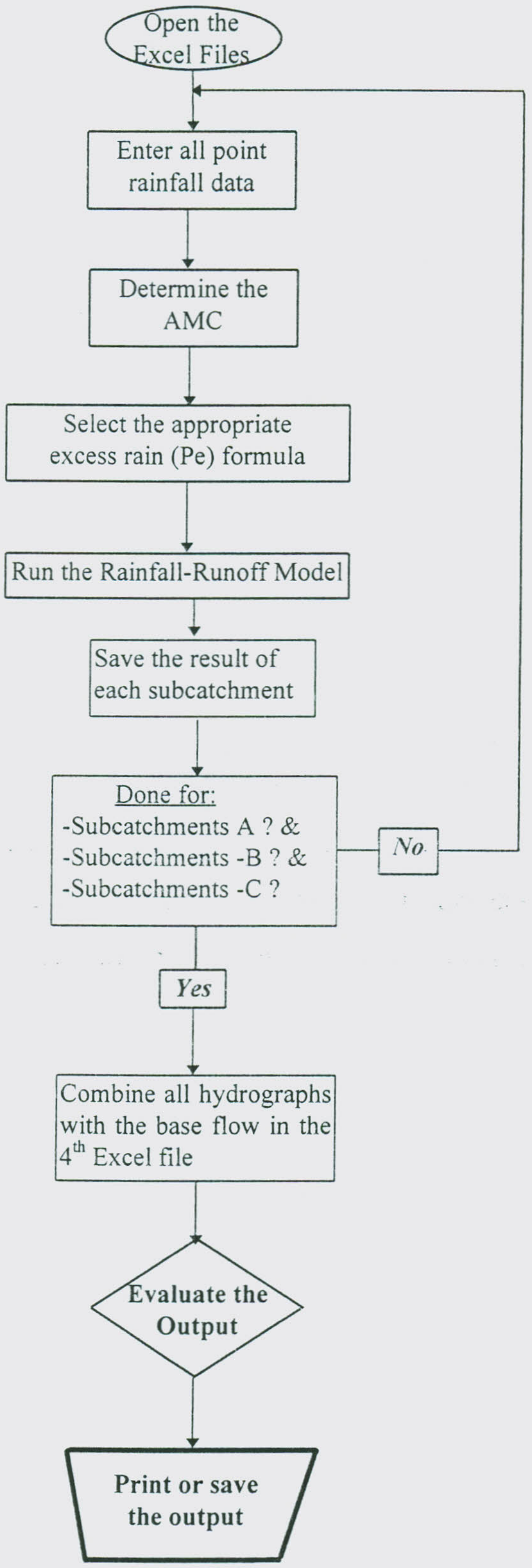

Fig. 5.6 Flow chart for the Rainfall-Runoff Model

5.10 Discussion on Results of the Rainfall-Runoff Model

The main objective of developing the rainfall-runoff model in this research has been to estimate the total daily inflow in to Koka reservoir during the wet season for real-time operation of the reservoir. Hence, the comparison of the simulated and observed daily total inflows into Koka has been taken as the major criterion to see if the model works good or not. While developing the model the basic initial parameters were determined using the concepts of SCS. However, some adjustment or 'calibration' were made to some of these parameters to suite the actual condition and to improve the simulation. In 'calibrating' some of the catchment parameters, four years of (1993-96) observed hydrometeorological data were used and 'verification' of the model was done with another four years of (1989-1992) observed data.

The problem faced during the model 'calibration' and 'verification' ranges from the limitation of the model itself up to the low quality hydrometeorological records. Therefore, the limitation of the model and the assumptions made in developing the model must be clearly understood while using it.

A number of methods are available to verify the rainfall-runoff model. The most straight forward method is a subjective one where one can judge by simply looking at the graph of the simulated and observed values plotted to the same scale. The other widely known method is an objective method like R^2 (Correlation coefficient) which can be used to statistically determine the quality of the simulation. In this work both the subjective and objective methods are used to see how good the simulation is. In this article two sample simulation years of 1989 and 1994, shown in Figure 5.7 and 5.8 are selected for discussion and interpretation of the simulation results.

Considering the 1994 simulation year: the simulated runoff is seen to be well above the observed runoff specially for the part of the wet season (July23-August16). One possible reason for this could be that the model couldn't simulate the process well. But this is no always the only reason. If we take the days from July23-August13, the simulated runoff is higher than the observed because the rainfall records showed that the rainfall amount was high on these days. Hence the model, being a deterministic one, simply used these high rainfall amounts to calculate the simulated runoff: at best, the simulation can be as good as the observed rainfall data. And in most cases it is very difficult to identify whether the rainfall or runoff or the model has gone wrong. The simulation, however, improves for the other days of the wet season specially beginning of the wet season and after end of August. But, the R-square value was already influenced more by the simulation of (July23-August16) than the other days of better simulations. This is so because runoff values for those days were very large that they increased the errors (simulated-observed) and brought the R-square value down to 0.44. When such type of problems arise during simulation (for instance in or real-time operation of the reservoir), the judgment of the user is required in correcting the simulated values based on the actual conditions (checking and counter-checking rainfall data observation of the actual runoff, land cover, soil moisture, e.t.c.).

However, the 1989 simulation year which gave an R-square of 0.81, is considered to be good simulation. It has followed the trend of the observed hydrograph well. But it is also worth mentioning that the simulation has missed some of the observed peak discharges. But, this is not a critical problem when considering the purpose this model is intended for. The overall trend of the simulated hydrograph which represents the sustained inflow into the reservoir is much more important because there is already storage reservoir (Koka) up stream of the important schemes we want to protect and the reservoir can attenuate some of these missed peak discharges. Simulating the peaks would have been more important for flood plains with no storage reservoir in the near-by upstream. All graphs of the observed and simulated runoff are shown in Appendix (A.1.5-A.1.12). The average R-square value for all eight years of simulation is 0.63 which is considered acceptable according to the requirements from rainfall-runoff models

The rainfall-runoff model was also tested to see how well it has simulated the total volume of runoff during the wet season for the upper Awash Basin. Table 5.12 Shows the total volume of the observed and simulated runoff and the wet season observed total areal rainfall over the Upper Awash Basin. From this table we can see that for years-1989 and 1995, the simulated runoff volume is almost equal to the observed volume.

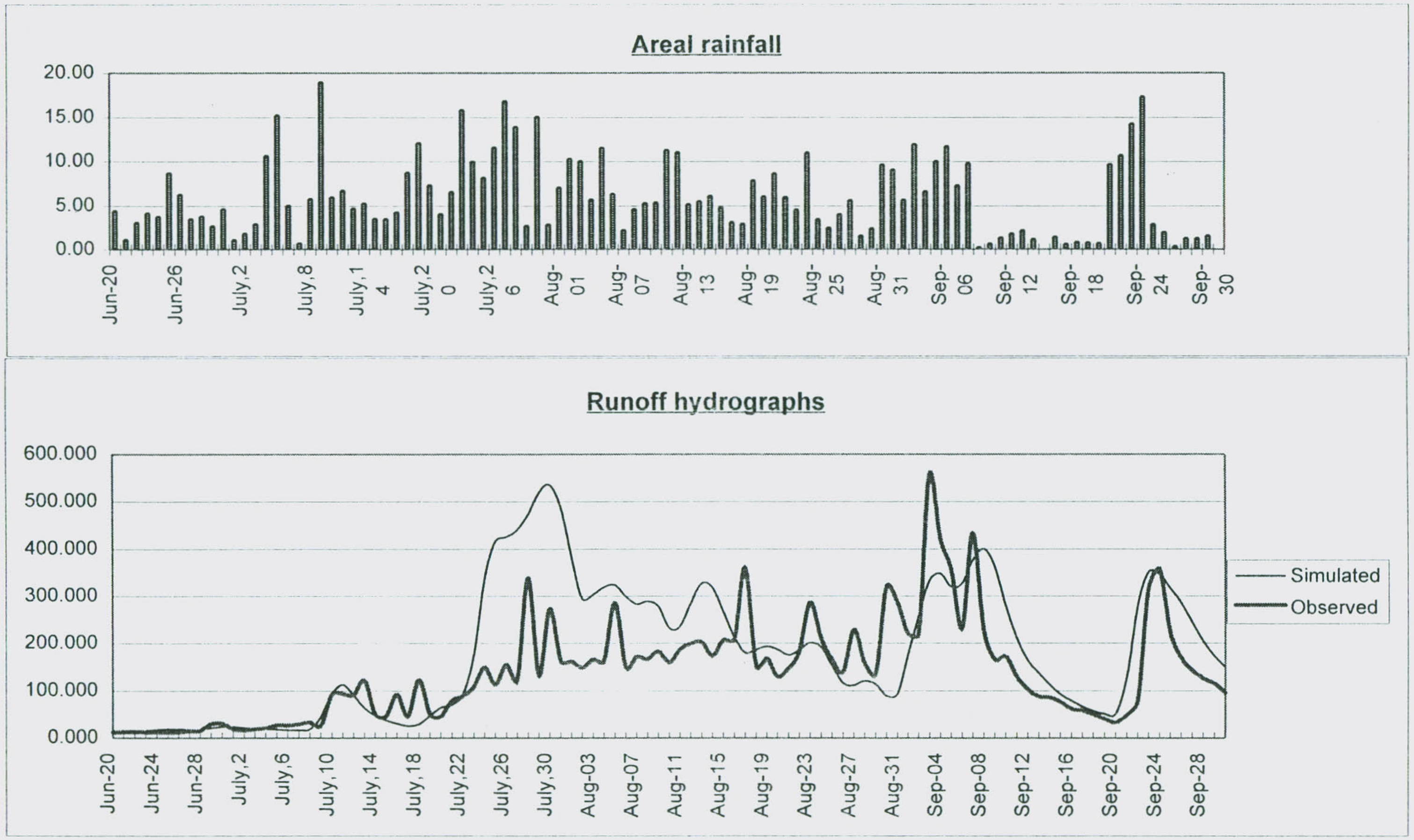

Figure 5.7: Observed and Simulated runoff for simulation year- 1994

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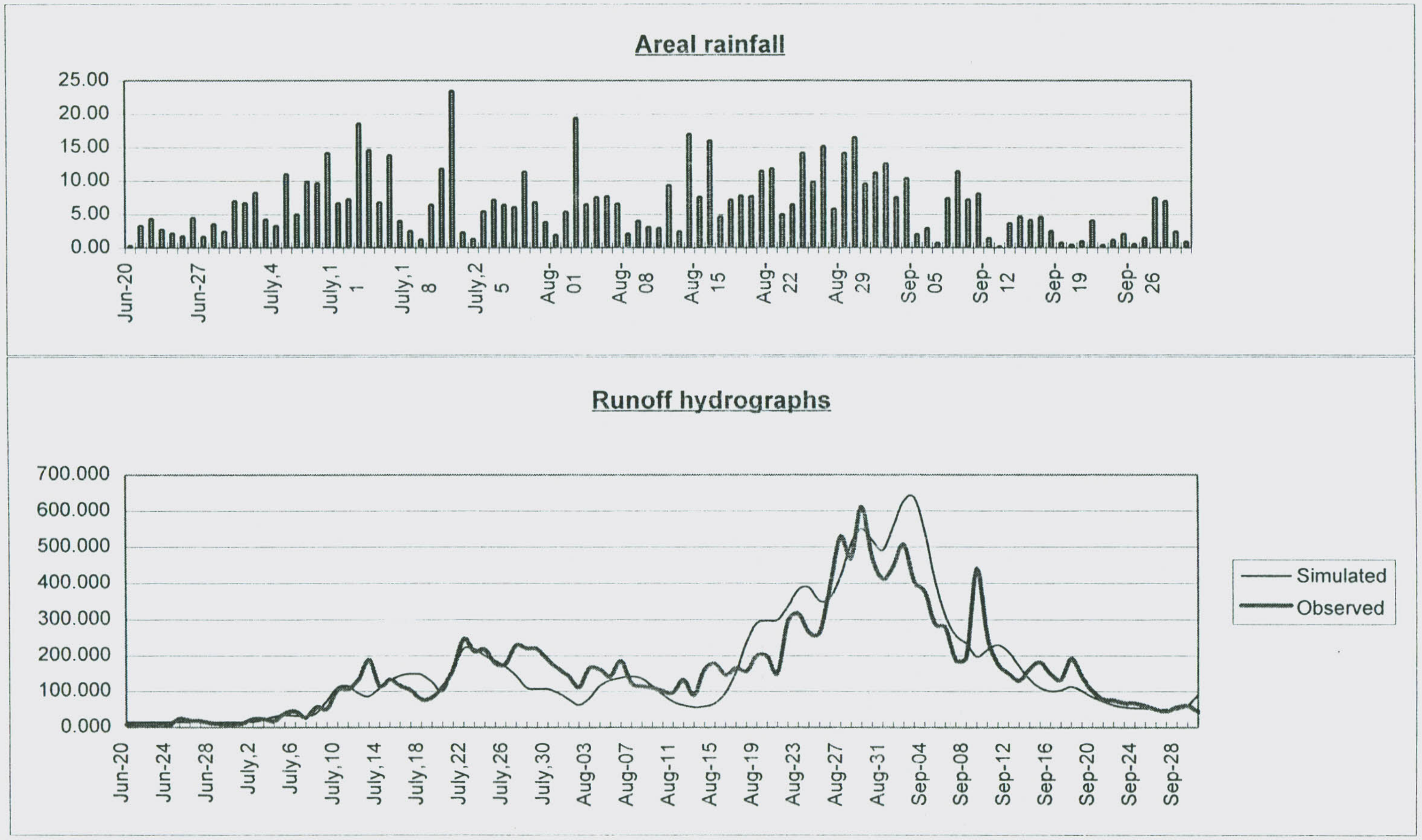

Figure 5.8: Observed and Simulated runoff for simulation year- 1989

The simulation of total runoff volume for the other years is also good except year-1993 which shows that the simulated runoff volume is well more than the observed one. The reason for this higher value of simulated runoff volume can be explained as follows. As shown on the table, year-1993 had the highest total areal rainfall amount which is even much higher than year-1996 which caused the highest flood recorded on upper Awash river. However, the observed runoff volume of 1993 is much lower than that of the 1996. In the case of the 1993 simulation, the runoff volume is higher than that of 1996 which is consistent with the total areal rainfall amount (for a "high" rainfall season the simulated runoff volume is high and viseversa). But the observed runoff volume shows some inconsistency. Quite logically, under normal condition, two rainfall seasons having a pronounced difference in total rainfall amount must also demonstrate a difference in total runoff volume proportionally. It is, therefore, suspected that some of the rainfall data have observation errors.

However, it can generally be concluded that simulation result of the total runoff volume using the rainfall-runoff model is good and simulation result could be further improved if a good quality hydro-meteorological data are used. And it is believed that the model could serve the purpose it is intended for, if used in conjunction with sound judgment in interpretation of input and output data.

Table 5.12 Comparison of Observed and Simulated runoff volume

Simulation Year	Total areal rainfall (June - September), (mm)	Simulated runoff volume, (Mm3)	Observed runoff volume, (Mm3)
1989	688.5	1421.96	1414.04
1990	620.2	1320.79	1427.08
1991	736.7	1865.27	1751.07
1992	713.4	1730.57	1373.26
1993	918.4	3167.64	1963.37
1994	646.8	1604.78	1235.77
1995	555.4	1054.05	1047.43
1996	708.3	2226.85	2509.66
Average	**698.5**	**1798.98**	**1590.21**

6 Establishing Operation Rules for Koka Reservoir

6.1 Introduction

6.1.1. Purposes of Reservoirs

The purpose of reservoir regulation is to smoothen out the peak flows and low flows of a river so as to obtain greater beneficial use of water resources. Water is stored at times of excess flow either for the purpose of reducing downstream flood damage or to conserve water for later use at times of low flow. In those reservoirs built with flood control as a major purpose it is essential that the reservoir capacity reserved for storage of flood water be emptied as soon as practicable after a flood. In some cases, because of a definite seasonal pattern of floods, stored flood water may also be retained, at least partially, for later conservation uses. In those reservoirs built for conservation uses, stream flows in excess of current requirements may be stored in the storage zones reserved for those purposes and not released until needed later. The release of stored water may be for a variety of uses; power generation, irrigation, industrial and public water supply, recreation, etc (Chow, 1988).

Reservoirs and their operations are generally classified according to whether they are for a single purpose or for more than one purpose. Single purpose reservoirs for conservation uses, such as power, irrigation, public water, public water supply and downstream wildlife prevention, require a gated outlet to control the release of water and reservoir capacity to store excess flow in sufficient amounts to supply the demand during periods of deficient flow. In the case of single purpose power project, the turbine gates control the outflow when the water level is below the spillway. The storage capacity is governed by water supply available in the stream or by physical features of the site. Any number of uses compatible with the physical limitation of the reservoir site and the available water supply may be combined in a multiple-purpose reservoir (Chow, 1988).

6.1.2 Operation of Reservoirs

The optimal development of water resources is conditional on the establishment of appropriate operation policies. By operating policy we understand a time schedule of releases from reservoirs, of pumpages from aquifers and/or reservoirs. It is clear that not all water resource systems will necessitate the three kind of operations. At any rate, the establishment of such schedules, which indicate quantities of water to be affected through the action of the manager at defined points in time, is an important problem in water resource engineering. The problem is, of course, the selection of the operation procedure that will best achieve the stated objective(s) of the development scheme.

It was customary for a long time to establish operating rules on the basis of personal judgment alone. No alternative procedures were tested. The rules were, generally simple: (1) store all inflow unless needed to meet a target output; (2) when available, release water from storage to fulfill immediate needs; (3) study all damaging floods on record in the flood control analysis. As water resource systems became more complex, however, it became apparent that operating procedures need to consist of three (and possibly four) kinds of decisions.. Storages and release of water must be apportioned among (1) reservoirs, (2) purposes, (3) time periods, and possibly (4) depth layers from a reservoir to provide water of required quality. Furthermore, it was recognized that operating procedures are sequential decision problems and have to be treated as such. These problems take account of the fact that a decision is likely to have consequences that extend over a considerable period of time. A decision regarding the release of water from a reservoir, for example, is of this nature. The analysis of operating procedures is performed with the help of certain methods of applied mathematics. such as Linear Programming, Dynamic Programming, etc.

It would be impossible to discuss operational problems without mentioning flexibility. Once an operating rule has been established and adopted, some degree of flexibility must be allowed. We should remember that a policy is a series of decisions, and that these decisions are made in face of uncertainty: hydrological uncertainty, meteorological uncertainty etc. Attempts are made to provide confidence limits for the operating of storage facilities. The active storage in a reservoir is divided in to three regions: the upper region, within which water has to be abstracted from the reservoir at the full capacity of the abstracting facilities in order to minimize the likelihood of loss of water through spills from this reservoir; the middle region, within which water can be pumped from the reservoir in the most economic manner for the satisfaction of current needs; the lower region, within which pumping has to be discontinued to minimize the likelihood of shortages of water. The lines dividing these regions represent given probabilities of spills or of shortages.

In the case mentioned above the criteria of the establishment of operating rules were defined in relatively simple terms (minimize likelihood of spills, or of shortages), a more complex situation arises when reservoirs have to be operated in connection with heavily industrialized regions. There, it is necessary to take into account the water quality downstream from the point of storage, as affected by the industrial wastes discharged in to the stream. In such cases, storage reservoirs are also operated for what is known as ''low flow augmentation.''

The most complex operating rules are those connected with multipurpose reservoirs. In simple terms, we can state that the objective of operating such systems is to manipulate supply so as to reduce competition among demands and to increase complementarity among them. The principal demands are electric power production, flood prevention, irrigation, water supply, navigation, waste carrying and recreation (Buras, 1972).

Today there are a number of reservoir simulation and operation models which include HEC-3, HEC-5, SSARR, ENMAG etc. Most of these models are fully described in Water Resource Publications (WRP) and some other text books. Generally speaking the underlying assumptions made during the development of these models are not very different from each other except the emphasis they give for the different hydrological processes.

The 1982 WRP has made a thorough discussion on different approaches on this topic under the title 'Experience in operation of Hydrosystems'. In this publication a number of proposals have been put forward based on different experiences and researches on operation of reservoirs. In one instance a hierarchical model was proposed for real-time operation based on the experience gained from the operation of a three large hydroelectric system in Quebec, system in Quebec, Canada. Considering that the total operational management of a hydro system encompasses a whole spectrum from long-term planning to real-time operation, a model was developed consisting of vertically arranged and hierarchically ordered multiple levels of optimization (Unny et.al, 1982)

Another model which was developed at NTH/SINTEF is the ENMAG model. The assumptions and basic concepts of this model is clearly discussed at length in a text under the title "Hydropower development - Hydrology". ENMAG model is a hydropower simulation model developed in the mid 1980's. This model is based on a detailed description of the hydrological conditions and production systems, while consumer systems and operation strategy are described with a simple model. Basic equations to simulate filling and release of water from the reservoir in a simulation model are fairly simple and are based on water balance considerations for the reservoir. Inflow to the reservoir is based on observed hydrological data, or with scaling in order to obtain correct value. Permitted maximum and minimum reservoir contents will normally be identical to lowest regulated water level (LRWL) and highest regulated water level (HRWL) respectively, but may also be different from these if there are restrictions on the reservoir filling. ENMAG uses the rule curves (guide curves) as a major operation strategy which informs the operator how to decide the optimal release from the reservoir. But this operation strategy (rule curve) is not the output of the ENMAG model. This means that this model is best suited for development planning and less so for optimizing the operation of a hydropower plant (Killingtveit, 1993).

6.1.3 The Koka Hydropower System

Koka Reservoir is located about 100 Km south of the capital Addis Ababa. This reservoir was created as a result of the construction of Koka dam in 1960. It is the only multi-purpose reservoir found in the whole of the Awash basin being utilized for both Hydropower generation and Irrigation purposes. The surface area of the reservoir at full level is about 177 Km^2. The minimum and maximum water level in the reservoir are 1580.7 m and 1590.7 m a.s.l. respectively. As the reservoir borders settlement and agricultural areas, it is quite common to see, during wet seasons, large inundated areas posing great hazard to the local people and irrigation schemes.

Based on the surveys made in 1959, 1981 and 1988, the reservoir Elevation-Capacity and Elevation-Area curves have been established. As there is no any recently made survey on the reservoir, the previous data are extrapolated in order to predict the present status of the reservoir. According to the extrapolation of the previous curves, the reservoir has lost a considerable amount of its volume due to siltation. The bottom outlet is also completely clogged to the extent that it is no more usable. At present it is estimated that the capacity of the reservoir has dropped from an original volume of 1,850 million m^3 to only 1,000 million m^3.

Koka dam is a concrete gravity dam. It was commissioned in 1960 after three years of construction. Its crest elevation is at 1593.2 m a.s.l with a maximum height of 23.8 m and a crest length of 458 m. The major constructions in the dam are an Ogee spillway, a bottom outlet and an intake tunnel. The 5.5 m diameter intake has a length of 71.5 m concrete lined tunnel and a 145.37 m concrete pipe covered with 70 cm earth cover. The spillway has four radial gates with a crest level at 1585 m and top of the gate level at 1590.7 m. It has a total maximum discharge of 1000 m^3/s at an elevation of 1590.7 m.

The water from the dam passes through the intake tunnel and the three penstocks and gets at the Koka power house and finally joins Awash river through a tail race canal. At Koka power house, there are three vertical Francis turbines with a rated head and total discharge of 42 m and 123.5 m^3/s respectively. The installed and firm capacity of Koka power house is 43.2 MW and 34.5 MW respectively (EEPCO, 1997).

6.2 Construction of Operation Rule Curve

6.2.1 General

Reservoir operation involves determining decision policies which will optimize certain objectives subject to various constraints. Consequently, determining optimum reservoir operating policies has been viewed as an ideal application for system analysis techniques. There is no single type of reservoir operation problem but rather a multitude of decision problems and situations. The objective functions, decision variables and constraints vary for different types of reservoir operation problems. Mathematical simulation and optimization models are used for various purposes in various situations. Reservoir operation is based on conflicting objectives of maximizing the amount of water available for conservation purposes and maximizing the amount of empty space available for storing flood waters to reduce downstream damages. Conservation purposes include hydroelectric power generation, agricultural, municipal and water supply, maintenance of stream flows, recreation, etc. A number of models have been developed to address a single conservation purpose. For example, optimizing hydroelectric power generation from a system of reservoirs has often been a primary objective. Other models have been used to determine flood control operation policies.

6.2.2 The Optimization Model

6.2.2.1 General

Optimization is an act of obtaining the best result under given circumstances. Optimization in its broadest sense can be applied to so many engineering problems. One of the typical applications is for design of water resource systems for maximum benefit. (Rao, 1996)

Reservoir optimization models allow the modeler to produce operating or planning scenarios that in some measurable sense are considered optimal. Optimization techniques can be divided into three distinct categories: (1) linear programming, (2) dynamic programming, and (3) non-linear programming. Each of three categories has certain distinguishing features that separate them from the others. One of the categories is no better than any of the others in a general sense. Each method of the programming has a certain type of problem for which it is best designed, and the modeler must be aware of the virtues of each model type in order to effectively solve this problem (Wurbs et.al, 1985).

Each of the methods has some general characteristics which they share with each other. All of the model types require an objective function. They each have decision variables and some of the constraints on the solution space.

The set of decision variables is analogues to the operating policy of the reservoir. These set of decision variables will define how the system is to be operated. It will define how much water is to be released and when. The decision variable set defines how much water will be allowed to flow through the outlet structures, and it will define how much water will be kept in storage. This variable set is the desired output of the optimization model.

The constraints on the system force the model to obey the physical laws that govern the system. The maximum volume of water that can be released in a time period and/or the maximum amount of storage in the reservoir are suitable physical constraints. There are also institutional and contractual constraints. The minimum amount of water released in a time period in order to meet water quality restrictions or previously assigned water rights are examples of this type of constraint.

The objective function is the heart of the optimization model. The solution to complex water resources problems require, in order to apply formal mathematical algorithms, a way to measure the level of service provided by a specific changes in the decision variables. Many benefits that accrue from a water resource project, related to hydroelectric power, irrigation, water supply, flood protection, fish and wildlife enhancement are not amenable to comparison.

The generally accepted method for determining a system's performance has been one of economic impact. Either the total cost of the system is minimized or the total economic benefit of the system is maximized. With this type of analysis a large measure of objectivity is introduced. Some of the benefits, such as those achieved by increased hydroelectric power production, reduction in flood damages, or increased irrigation water availability can be reasonably converted to an economic benefit. It must be bore in mind though that the this conversion might not be suitable for some optimization models. For instance non-linear objective functions and/or constraints are difficult to manipulate in linear programming techniques which we are going to use for Koka reservoir operation.

Though there are some limitations to the use of linear programming, there are some important applications in the area of water resources, along with some additional techniques which extend and amplify the usefulness of linear programming (Wurbs et.al, 1985).

6.2.2.2 Linear Programming

Linear programming is an optimization model applicable for the solution of problems in which the objective function and the constraints appear as linear functions of the decision variables. Linear programming is considered a revolutionary development that permits us to make optimal decisions in complex situations. At least four Nobel Prizes were awarded for contributions related to linear programming (Rao, 1996).

In this research Linear Programming (Simplex Method) using *MS-FORTRAN* language is used to develop an optimization model for establishing operation rule for Koka reservoir. this model generally deals with the allocation of limited resources to a number of competing activities to obtain the maximum return measured by some utility function. This allocation is then the 'Optimal' allocation and this solution is the optimal solution according to the criteria given to the model to make such a decision.

In order to develop rule curves using the optimization model, a number of input data are required. The type and amount of data required for an optimization model vary according to the objective and even the limitation of the model. In our case, where we employ linear programming as our optimization model, the major input data are historical total inflows in to the reservoir, user-defined requirements, and the physical characteristics of the reservoir. The listing of the optimization model in *MS-FORTRAN* language is shown in Appendix.

6.2.2.3 Objective Function, Constraints and Water balance equation

The benefit gained from the reservoir system which include hydropower, irrigation, and flood protection benefits can be expressed in terms of the objective function.

On trying to choose an objective function for optimization, the first and straight forward choice could be to optimize the power generation. As explained in the previous articles the objective function in linear programming can not be a non-linear function. And Hydropower which is a function of discharge, head, and specific gravity is a non-linear function and consequently is not suitable as an objective function in linear programming.

Therefore, taking spillage as an objective function is a better alternative in order to go around the non-linearity problems and at the same time to achieve the task of optimizing the reservoir operation using linear programming. The underlying logic for taking spillage as an objective function is that by minimizing the amount of water released (spilled) at each time step in the wet season, water could be saved for power generation and/or for irrigation purpose and in effect it is optimization of the over all benefit.

The constraints in the optimization model are mathematical expressions which enable us to force the optimized variables fulfill certain requirements. For our case this constraints are formulated, for example, in such a way that they give maximum protection against flood, avoid zero power production at any time and meet the physical conditions.

The water balance equation is a mathematical expression which continuously accounts for both the inflow in to the reservoir and outflow from the reservoir at a user defined time-step (eg.,one week). It is a necessary condition that the water balance equation be satisfied at each time step.

The Objective function, the water balance equation, and the constraints are formulated as shown below:

Objective function (F) is expressed as the sum of spillage at each time step:

$$F=Vs(1)+Vs(2)+\dots\dots\dots+Vs(t-1)+Vs(t) \qquad (6.1a)$$

Note that since the linear programming is written to maximize the given function, the objective function has to be rewritten as the negative of equation (4.1) in order to minimize it as shown.

F=-Vs(1)-Vs(2)-................................-Vs(t-1)-Vs(t) (6.1b)

The water balance equation is expressed as:

S(t+1)=S(t)+I(t+1)-Vh(t+1)-Vs(t+1) (6.2)

The constraints are expressed as;

1. Maximum storage at any time shall be less or equal to the capacity of the reservoir (i.e 1000 Mm^3) (EEPCO, 1997).

$$S(t) \leq 1000 \tag{6.3}$$

2. Maximum weekly water demand for hydropower generation shall be less than or equal to 55 Mm^3 (90 m^3/s) (EEPCO, 1998).

$$h(t) \leq 55 \tag{6.4}$$

3. The sum of weekly water release through turbine and spillway shall be less or equal to 242 Mm^3. (assuming a maximum allowable release to avoid downstream flooding to be 400 m^3/s) (Halcrow, 1989).

$$h(t) + Vs(t) \leq 242 \tag{6.5}$$

4. Minimum reservoir storage at any time shall be greater than or equal to 150 Mm^3.

$S(t) \geq 150$ (The absolute minimum storage is 130 Mm^3) (EEPCO, 1998) (6.6)

5. Minimum weekly water demand for hydropower generation shall be greater than or equal to 12 Mm^3 (20 m^3/s) (EEPCO, 1998).

$$h(t) \geq 12 \tag{6.7}$$

6. Minimum weekly spillage shall be greater or equal to zero Mm^3.

$$s(t) \geq 0 \tag{6.8}$$

where;

t is the week number; in our case it runs from the first to the thirteenth week which are respectively the beginning and end of the wet season.

S(t) is the reservoir storage at the end of the t-th week,

Vh(t) is volume of water (in million m^3) required for power generation at the t-th week,

Vs(t) is the volume of water (in million m^3) spilled in the t-th week,

I(t) is the net inflow (in million m^3) into the reservoir during t-th week considering the total inflow, rainfall on the reservoir, the evaporation loss from the reservoir and leakage through the reservoir bed.

6.2.2.4 Calculation of Input data for the Optimization Model

The basic assumption in rule curve establishment is that the historical (observed) inflows used to develop the rule curves are assumed to be statistically similar (not identical) to the future inflows. Therefore, if one develops an optimized rule curve based on the historical inflows, these rule curves can effectively be used as a guide line for operation of reservoirs in the future.

In optimizing the Koka reservoir operation, 12 years of daily historical total inflows were used as an input to the model. The total inflow into Koka reservoir is not directly Known from gauging stations. This is because the nearest gauging stations to Koka are at Melka-hombole and at Mojo which are approximately 42 km and 35 km respectively upstream of the reservoir inlet. Therefore, when the total inflow in to Koka is calculated from the observed flows at Melka-hombole and Mojo, scaling has to be used in order to account for the additional area below the gauging stations.

One of the major input to the optimization model is the net inflow which considers, in addition to the total inflow, the rainfall on the reservoir, evaporation from the reservoir and leakage through the reservoir bed.

In the past studies have been made to estimate the evaporation and leakage from the reservoir. In the Awash Master Plan report (Halcrow,1989) a water balance study to estimate the leakage from koka reservoir was made over the period from 1968 to 1987. Inflow up to the inlet was calculated using the rainfall/ area/ runoff regression equation and inflow values to the lake are derived as:

Monthly Inflow = 1.065*Melka hombloe + 1.18*Mojo

Based on the water balance study Halcrow recommended for the purpose of reservoir simulation that monthly loss which is the sum of leakage and evaporation loss is estimated by:

Monthly loss = Ki * monthly Evaporation loss*Area

where:

Ki is a constant determined to be 1.557

Area is the water surface area of Koka reservoir. (The variation in the Koka water surface is considered at each time step).

In this research a time step of one week is chosen to calculate the inflow input data for the optimization model. But in the Halcrow study it was not mentioned whether it is still possible to use the above formulas for lesser time steps (e.g. weekly time step) other than a monthly one. And these was not discussed in other studies. In the future, a better solution to this problem is to install a gauge at the inlet to the reservoir in order to explicitly determine the inflow amount. For the purpose of the optimization model, however, the above formula is sufficient.

Components to consider in calculating the net inflow in to Koka are:

I) The observed flow at Melka-hombole

II) The observed flow at Mojo

III) Evaporation and leakage losses

IV) Rainfall on the reservoir

The net inflows are then converted in to volumes at a weekly time-step to be used as final input in to the optimization model. The optimization time-step can be decreased or increased as required by the user. Though the rule curve is based on weekly flows, however, it can as well be prepared based on daily time-step to get a more realistic result. By choosing a weekly time step it is implicitely assumed, with in a week, the flows are linearly varying. In fact one week time step is quite reasonable from the current operational practices. In this research a time step of one week is chosen and in the upper Awash basin where the normal wet season spans from around end of June to around end of September, we will have about thirteen wet weeks. However, it must also be borne in mind that, as far as the optimization model is concerned, there is no restriction on the number of weeks (optimization horizon). The input data format can be reformatted if additional or lesser time steps are required as the same format cannot be used for the different time-steps.

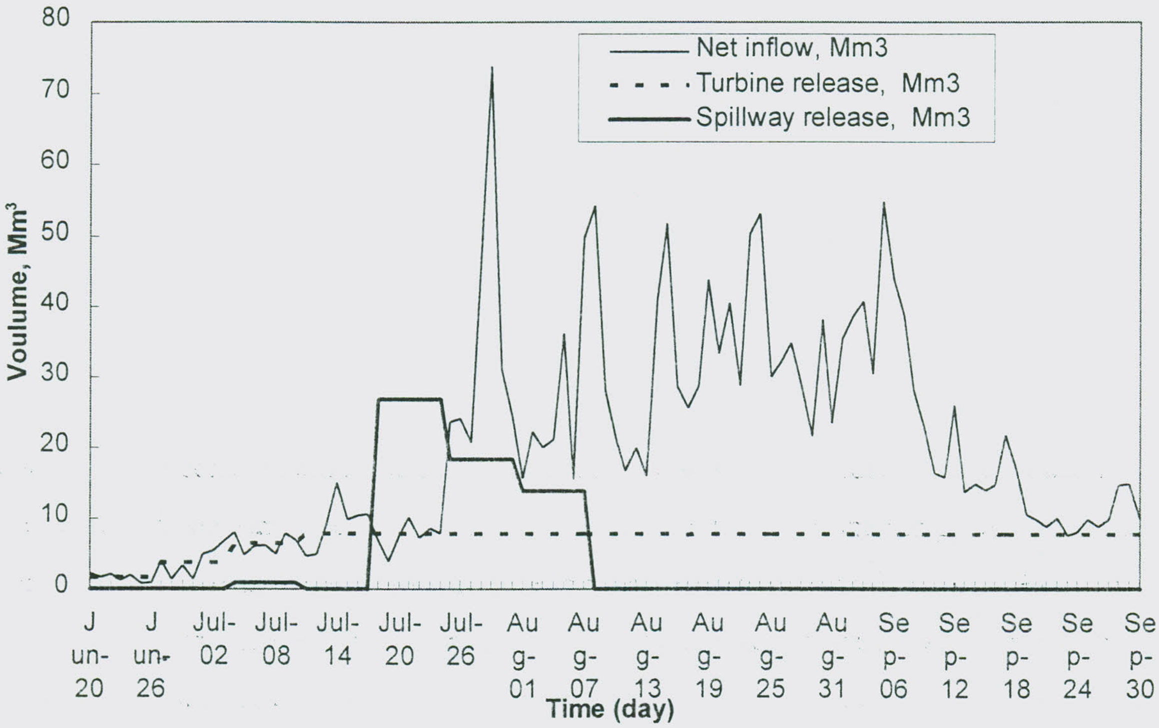

Fig. 6.1 Net inflow and the Optimized Turbine & Spillway releases for year 1993 (sample).

6.2.3 The Rule Curve for Koka Reservoir

Rule curves are guide lines which propose the status of the reservoir as a function of time for a period of one year.

As already shown in the previous articles, Optimization models using the linear programming (Simplex Method) has been employed as a key tools for deriving the rule curves. Once the water releases from the reservoir are optimized for some years of historical (observed) inflows, the rule curves can be established using the output of the optimization model.

Figure 6.2 shows 12 curves plotted on a storage versus time axis. These 12 curves are found using the optimization model with an input data of the 12 years of historical inflows and all other important parameters required by the model as described in the previous article. Each curve can be viewed as the optimum rule curve for that particular year's hydrometeorological and reservoir condition (reservoir capacity). As can be seen from Figure 6.2, the thickness of the band varies as we go through the different weeks of the wet season. The band is usually thick around the middle of the wet season and it becomes thinner near the beginning and end of the wet season. The former happens as a result of the high variability of inflows from one wet year to the other. For obvious reasons, for a generally light-wet season, the optimization model forces the curve to follow the upper side of the band and viseversa.

Generally, out of these bands of curves, a single rule curve can be established by taking the average of all optimized rule curves. But, during reservoir operation it is not practical to expect the reservoir operators to follow the single rule curve strictly. There should be a range (rule curve-envelope), upper and lower bounds, within which the reservoir storage is expected to fall. This range can usually be decided from experience. One proposal is to take the upper and the lower boundary of the curves. Figure 6.3 shows the envelop produced by taking the upper and the lower boundary of the optimized rule curves from Figure 6.2.

From a long-term view point the reservoir operation should follow as far as possible the space between the upper and lower bound. A deviation is possible, for example, because of the extremes caused by low inflows or high inflows e.t.c. If the deviation occurs, then the reservoir operation in the subsequent period or periods should tend to bring the reservoir state back to that defined by the rule curves.

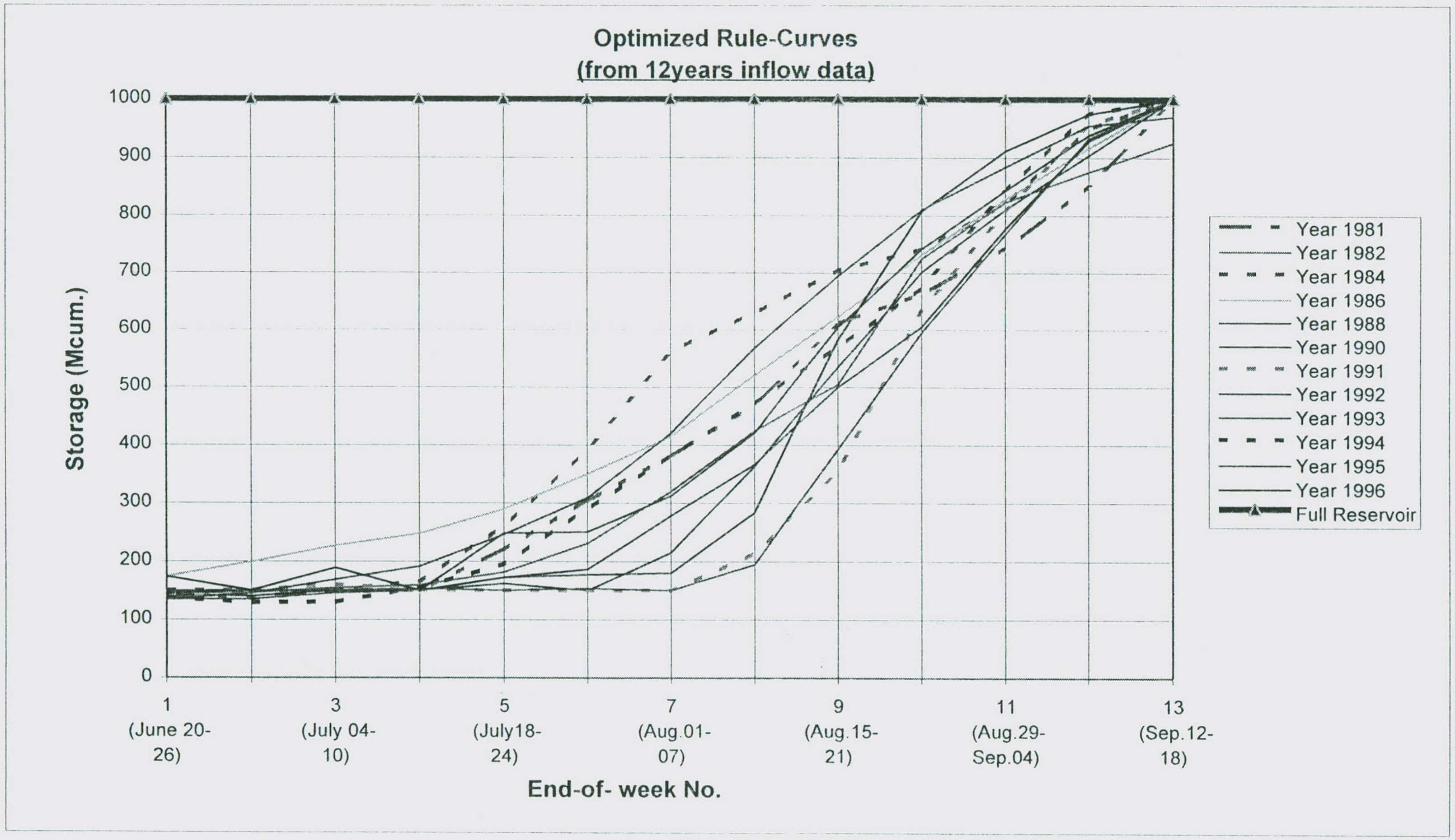

Fig. 6.2 Optimized rule curves for 12- years for Observed inflow data

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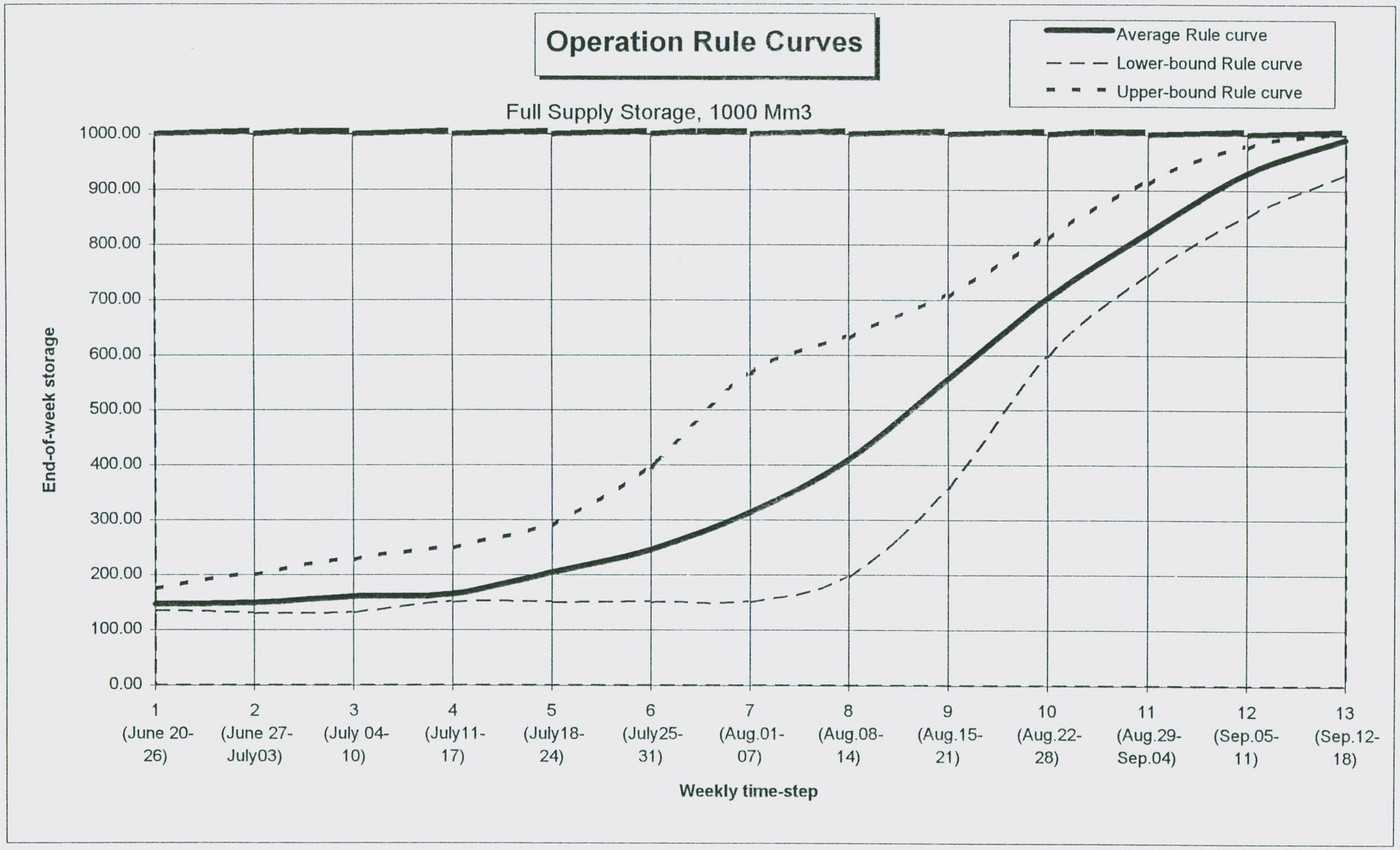

Figure 6.3 Optimized Release Rule Curve for Koka Reservoir

6.3 Real-Time Operation (R.T.O) of Koka Reservoir

Real-time reservoir operation is concerned with the optimal operation of an existing reservoir system, and decisions regarding releases for various purposes have to be made in a considerably shorter time period (Yeh, 1985). Alternatively it can be defined as a release decisions made periodically (e.g.,daily) by operators using current and some imprecise future information (Ginn et.al,1989). The basic current information could be rainfall, soil moisture, state of the reservoir. Once the basic data are acquired, the rainfall-runoff model can be used to forecast the runoff amount expected to inflow in to the reservoir. Soil moisture could be estimated from the antecedent rainfall, the soil type and land use conditions.

In real-time reservoir operation the release decisions at the beginning of the current period are determined using the water balance equation, the rainfall-runoff model and the optimized rule curve with all important hydrometeorological information at hand. The flow chart shown in Figure 6.4 outlines the general procedures to be followed for the real time operation (R.T.O).

1. The first task in real time operation is to choose the operating time-step. In this research a daily time-step is conveniently chosen.

2. Collect the daily rainfall data from all stations (or preferably, if possible, the forecasted rainfall one day ahead).

3. Use the rainfall-runoff model to calculate the runoff contribution from each subcatchment and superimpose them to get the total inflow in to Koka,

4. Calculate the net inflow by considering the total inflow, evaporation, leakage and the rainfall on the reservoir,

5. Use the net inflow in conjunction with the rule curve and the water balance equation to determine the required combined release through the turbine and spillway,

6. Out of the combined release, assign the maximum possible amount of water for the turbine release and the rest, if any, for the spillway release.

The same procedure is repeated every day (in real-time) if a one day time-step of operation decision is required.

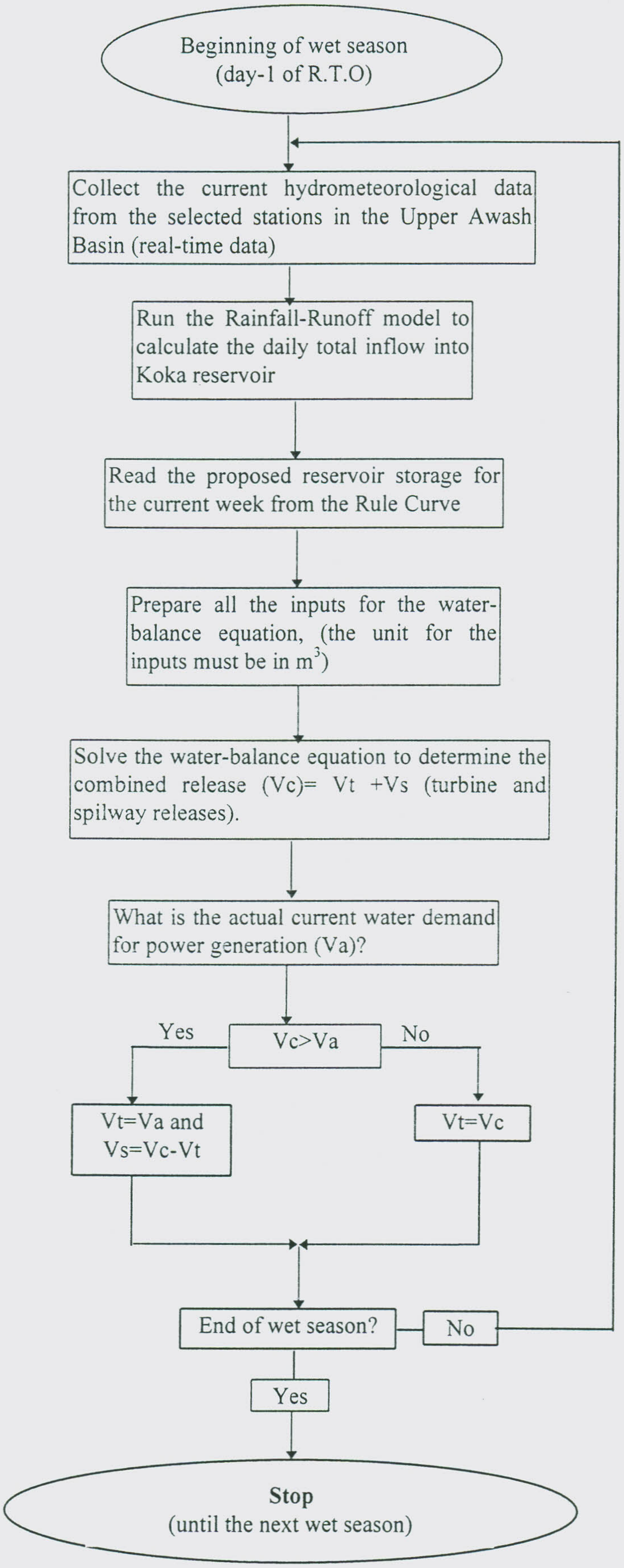

Fig. 6.4 flow chart for the real-time operation of Koka reservoir

6.4 Discussion on the Results of the Optimization Model

The linear programming used as an optimization model has obviously its own limitations. One major limitation is that it can't handle non-linear objective functions. As a result it was not possible to take the most straight forward parameter (power generation) as an objective function. The Linear programming was, however, effectively used by taking a linear function, spillage, as an 'alternative' objective function and the optimization model has worked well under the given set of variables for Koka reservoir.

As already discussed previously, currently it is estimated that the minimum and maximum reservoir storages are 150 Mm^3 and 1000 Mm^3 respectively. But, while using the optimization model with the given inflow (historical or synthetic), the storage may, sometimes, fall short of the above constraints as a result of a very low inflow wet season for instance. Under this condition it is essential to see how big this deviation is. According to EEPCO's operation practice a small deviation is usually allowed. For instance, a minimum storage as low as 130 Mm^3 (beginning of wet season) is usually acceptable as it is expected that the coming wet season will bring the reservoir back to normal.

The major outputs of the optimization model are hydropower releases, spillage and storage at a weekly time step. Storage which is one of the most important output of the optimization model is plotted on storage versus time graph to get a group (band) of curves. Each curve can be interpreted as the most effective operation rule curve for that particular year of hydrometeorological condition. This information will help to give an idea on how the reservoir should have been operated in the past and will give some information on how the reservoir operation should be in the future.

Presently EEPCO operates the reservoir using the Halcrow's rule curve established 10 years ago. This curve is not recommendable as the reservoir conditions have already changed from the past assumptions on which the Halcrow's rule curve was based. However, the optimization model developed by this research can effectively be used to update operation rule curves when ever conditions (operational and reservoir) change. One instance is if the proposal to add more turbine (s) in Koka power system is implemented the model can still be used for establishing new rule curves.

It is learnt that EEPCO uses the meteorological forecast of the so-called 'analogues year'. This means, for example, the NMSA informs EEPCO that the meteorological conditions of the current year will be similar to one of the past years. Then EEPCO uses that information to operate the reservoir. This information could be exploited more effectively by using the optimization model. This means that by using the historical inflows of the analogues year with the present reservoir and operational condition, a tentative optimized rule curve can automatically be established. This rule curve can be used for operation and it can be improved as more and more reliable data are made available in real time.

7. Conclusion and Recommendation

Water resource problems are quite complex that they are not amenable to a pure mathematical formulation. At best, the involved variables in every model can only be as complex as the modeler can visualize them. In reservoir operation, for instance, release decisions are made in face of uncertainty which include hydrological uncertainty and meteorological uncertainty. Water resource models are used to minimize these uncertainties, as much as possible.

In this research, attempt has been made to develop a rainfall-runoff and reservoir operation models based on the underlying fundamental hydro-meteorological processes. The rainfall-runoff model and the operational rule can be used so as to facilitate the decision making based on the real time hydrometeorological conditions. However, models can play the major role if they are supplemented with sound engineering judgment in the analysis of problems and interpretation of results.

In this research the following results have been found:

1. A rainfall-runoff model is developed using the already available and/or easily obtainable hydrometeorological data. The model structure is relatively simple both to understand and apply. The model has been verified with actually observed data and its performance is satisfactory with an average R^2 =0.63. The model also simulated the wet season total volume of runoff quite well.

2. The Linear optimization model has been developed which performed well under the given set of objective function and constraints. The output of this model was used to establish the rule curve. This rule curve can effectively be used in conjunction with the rainfall-runoff model for the real-time operation of koka reservoir.

3. Both the rainfall-runoff and optimization model are quite flexible to be applied with changing conditions like land-use, reservoir capacity, demand e.t.c, when ever such changes

do occure. The models, accompanied by sound engineering judgment will serve as a major tool for an efficient operation of Koka reservoir.

Finally it is recommended that:

1. The rainfall-runoff model and the rule curves be implemented as soon as possible for operation of Koka reservoir. The implementation could be accomplished by EEPCO in conjunction with MWR and other concerned professionals and the models can be improved whenever necessary.

2. The real-time operation of Koka reservoir requires a quick transfer of hydrometeorological information to the reservoir operators. It is, therefore, essential to maintain and strengthen the existing hydrometeorological network stations and improve the existing data transfer system. More over, continuos rainfall recording stations need to be setup at representative locations in the Upper Awash basin like Ginchi, Tulu Bolo, Addis Alem, Akaki, Melka Kunture, Melka Hombole, Chefe Donsa, Mojo And Koka.

3. The actual inflow in to koka reservoir is not explicitly known. This is because the existing gauging stations of Melka-hombole and Mojo are well upstream of the inlet to Koka reservoir. This is obviously the major bottle-neck in the reservoir and river simulation exercise. Up-grading of the existing hydrometric stations at Melka Kunture, Melka Hombole and Mojo and installation of new station at Koka inlet need to be done.

4. The rule curve is established using the present reservoir and requirements conditions. This rule curve is good as long as those conditions don't change considerably. However, it is clear that the Koka reservoir capacity is changing continuously. It is, therefore, very important to make a periodic survey of Koka reservoir to revise the existing Area /Capacity/Elevation curves. This will also enable to improve the accuracy of estimates of the reservoir life. The Optimization model can still be used with the new requirements and reservoir capacity conditions. The conditions in the downstream power houses (Awash II & III) can aslo be

included in the optimization model to establish the rule curves. Using this model new rule curves can be established to suit the new boundary conditions.

5. The rule curve is established using limited number of observed inflows in to Koka reservoir. The results could further be improved if many years of hydro-meteorological data are used. Since many years of inflow data is not available at present, generating and using synthetic inflow series is an alternative in the future.

6. It is usually the case that a lot of water is spilled during wet seasons. Structural measures such as raising the height of Koka dam to enable additional water storage, and/or means to reduce the sediment inflow need to be pursued.

7. SCS method looks more promising for the Study of some water resource problems under Ethiopian conditions where scarcity of hydrometeorological data is the major constraint. In this study it is observed that the rainfall-runoff model developed based on the SCS concept has performed well both in the estimation of the discharge hydrograph and the total runoff volume for the Upper Awash Basin. It is, therefore, advisable to do more research on the wide applicability of this method until a better alternative is found.

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Subcatchment-A

Appendix A.1.1
Simulation Year: (Sample)

Date	Rainfall Stations						Thiessen Areal rainfall (mm)	cummlative Total rain P(mm)	AMC INDICATOR	Cummulative Excess rain	Hyetograph Excess rain(cm)	**Simulated Direct Runoff at Koka M^3/s**
	M/kunture	Addis Alem	Tulubolo	Welenkomi	Teji	Ginchi						
Jun-20	0.00	0.00	0.00	0.00	2.30	2.30	0.75	0.75	27.82	0.00	0.00	0.000
Jun-21	0.00	5.30	20.00	0.00	2.70	5.00	6.35	7.09	25.73	0.00	0.00	0.000
Jun-22	0.00	5.30	28.00	8.60	3.40	4.50	8.82	15.91	24.52	0.00	0.00	0.000
Jun-23	0.00	0.00	27.00	3.80	0.00	0.00	5.53	21.45	21.45	0.00	0.00	0.000
Jun-24	9.40	3.10	0.00	1.30	0.00	2.50	2.50	23.94	23.94	0.00	0.00	0.000
Jun-25	0.00	3.30	0.00	4.90	3.40	0.50	2.02	2.02	25.21	0.00	0.00	0.000
Jun-26	0.00	9.50	10.60	4.70	0.00	14.40	6.98	9.00	25.85	0.00	0.00	0.000
Jun-27	0.00	3.20	10.00	0.00	3.30	1.90	3.57	12.57	20.61	0.00	0.00	0.000
Jun-28	0.00	0.70	1.00	2.30	4.60	0.00	1.43	14.00	16.50	0.00	0.00	0.000
Jun-29	0.00	0.10	20.00	0.00	1.50	11.30	5.75	19.75	19.75	0.00	0.00	0.000
Jun-30	0.00	8.80	10.00	5.00	0.40	0.00	4.70	4.70	22.43	0.00	0.00	0.000
July,1	4.60	10.00	20.00	3.40	0.00	8.40	8.49	13.18	23.93	0.00	0.00	0.000
July,2	24.10	14.80	0.00	11.80	31.10	13.20	15.58	28.76	35.94	0.12	0.01	0.688
July,3	10.20	3.50	0.00	16.30	11.60	7.60	7.09	35.85	41.60	0.90	0.08	5.632
July,4	7.50	4.60	10.00	6.90	1.60	1.30	5.22	41.07	41.07	1.89	0.10	14.711
July,5	4.60	12.30	15.00	8.60	8.30	12.10	10.68	10.68	47.05	0.00	0.00	18.904
July,6	3.10	3.40	25.00	19.50	0.40	31.50	12.70	23.38	51.27	0.00	0.00	16.650
July,7	4.30	7.10	20.40	7.40	7.40	3.10	8.77	32.15	44.46	0.41	0.04	14.552
July,8	9.40	6.70	8.00	26.60	7.70	13.90	10.60	42.75	47.97	2.27	0.19	23.561
July,9	15.30	6.80	10.00	14.60	7.80	13.50	10.48	53.23	53.23	5.30	0.30	46.758
July,10	28.40	0.70	0.00	0.00	0.00	0.00	3.87	3.87	46.42	0.00	0.00	57.673

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Up to September 30

Subcatchment-B

Appendix A.1.2

Simulation Year: (Sample)

Date	Rainfall Stations						Thiessen Areal rainfall (mm)	Cummlative Total rain P(mm)	AMC INDICATOR	cummulative Excess rain	Hyetograph Excess rain(cm)	**Simulated Direct Runoff at Koka M^3/s**
	Hombole	A/Ababa T/H	A/Ababa bole	Akaki	D/zeit IAR	M/kunture						
Jun-20	NA	0.00	0.00	0.00	0.00	0.00	0.00	0.00	2.23	0.00	0.00	0.000
Jun-21	NA	0.00	1.60	9.80	0.00	0.00	2.32	2.32	2.66	0.00	0.00	0.000
Jun-22	NA	10.70	6.20	0.00	0.00	0.00	3.13	5.45	5.53	0.00	0.00	0.000
Jun-23	NA	0.40	0.00	0.00	0.90	0.00	0.22	5.67	5.67	0.00	0.00	0.000
Jun-24	NA	3.00	2.00	0.00	0.20	9.40	3.39	9.06	9.06	0.00	0.00	0.000
Jun-25	NA	0.00	0.00	2.90	7.90	0.00	1.90	1.90	10.96	0.00	0.00	0.000
Jun-26	NA	15.30	0.00	7.80	0.00	0.00	4.45	6.35	13.09	0.00	0.00	0.000
Jun-27	NA	3.30	0.00	0.00	0.00	0.00	0.61	6.96	10.57	0.00	0.00	0.000
Jun-28	NA	18.00	19.00	0.00	7.00	0.00	8.01	14.97	18.36	0.00	0.00	0.000
Jun-29	NA	0.70	7.10	0.00	0.00	0.00	1.45	16.42	16.42	0.00	0.00	0.000
Jun-30	NA	7.60	7.50	0.30	0.00	0.00	2.86	2.86	17.38	0.00	0.00	0.000
July,1	0.00	9.50	8.50	8.90	4.00	4.60	4.93	7.79	17.86	0.00	0.00	0.000
July,2	0.00	10.00	5.50	9.60	1.90	24.10	8.00	15.79	25.25	0.00	0.00	0.000
July,3	0.00	5.20	3.80	8.60	0.80	10.20	4.36	20.15	21.59	0.00	0.00	0.000
July,4	0.00	1.20	8.30	5.50	0.00	7.50	3.39	23.54	23.54	0.00	0.00	0.000
July,5	23.40	9.80	13.40	4.00	6.00	4.60	12.10	12.10	32.77	0.00	0.00	0.000
July,6	0.00	3.20	1.80	0.00	0.90	3.10	1.32	13.42	29.16	0.00	0.00	0.000
July,7	4.20	17.00	19.80	11.50	0.00	4.30	8.49	21.90	29.65	0.00	0.00	0.000
July,8	0.00	40.80	9.60	6.20	4.00	9.40	9.63	31.53	34.92	0.35	0.03	0.880
July,9	39.50	10.70	9.50	0.00	3.40	15.30	17.56	49.09	49.09	3.98	0.36	12.348
July,10	0.00	0.00	0.00	33.20	21.00	28.40	12.38	12.38	49.38	0.00	0.00	36.974

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Up to September 30

Subcatchment-C

Appendix A.1.3

Simulation Year: (Sample)

Date	Rainfall Stations			Thiessen Areal rain fall (mm)	cummlative Total rain P(mm)	AMC INDICATOR	cummulative Excess rain	Hyetograph Excess rain(cm)	**Simulated Direct Runoff at Koka M^3/s**
	Mojo	Cheffe-donsa	D/zeit IAR						
Jun-20	0.00	0.40	0.00	0.18	0.18	3.44	0.00	0.00	0.000
Jun-21	0.00	2.60	0.00	1.15	1.32	3.44	0.00	0.00	0.000
Jun-22	0.00	2.40	0.00	1.06	2.38	2.47	0.00	0.00	0.000
Jun-23	0.00	5.20	0.90	2.45	4.83	4.83	0.00	0.00	0.000
Jun-24	0.00	1.10	0.20	0.52	5.36	5.36	0.00	0.00	0.000
Jun-25	0.00	0.00	7.90	1.39	1.39	6.57	0.00	0.00	0.000
Jun-26	5.20	0.00	0.00	1.99	3.38	7.41	0.00	0.00	0.000
Jun-27	0.00	1.90	0.00	0.84	4.22	7.19	0.00	0.00	0.000
Jun-28	0.00	0.00	7.00	1.24	5.46	5.98	0.00	0.00	0.000
Jun-29	0.00	0.00	0.00	0.00	5.46	5.46	0.00	0.00	0.000
Jun-30	16.30	16.60	0.00	13.56	13.56	17.62	0.00	0.00	0.000
July,1	0.00	13.70	4.00	6.75	20.30	22.38	0.00	0.00	0.000
July,2	0.60	1.40	1.90	1.18	21.49	22.72	0.00	0.00	0.000
July,3	0.20	2.40	0.80	1.28	22.76	22.76	0.00	0.00	0.000
July,4	0.00	2.70	0.00	1.19	23.95	23.95	0.00	0.00	0.000
July,5	13.80	8.90	6.00	10.26	10.26	20.66	0.00	0.00	0.000
July,6	0.00	1.80	0.90	0.95	11.21	14.86	0.00	0.00	0.000
July,7	21.20	9.70	0.00	12.39	23.60	26.07	0.00	0.00	0.000
July,8	14.20	5.80	4.00	8.69	32.29	33.49	0.00	0.00	0.000
July,9	19.30	14.60	3.40	14.42	46.72	46.72	0.23	0.02	0.230
July,10	0.00	0.00	21.00	3.71	3.71	40.16	0.00	0.00	0.608

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Up to September 30

Total Runoff

Appendix A.1.4

Simulation Year: (Sample)

Date	Sim DRO subcatA	Sim DRO subcatB	Sim DRO subcatc	Observed at mojo	Observed at hombole	**Observed Total Runoff, M³/s**	**Simulated Total Runoff, M³/s**
Jun-20	0.00	0.00	0.00	0.12	7.41	8.48	15.00
Jun-21	0.00	0.00	0.00	0.11	7.26	8.29	15.00
Jun-22	0.00	0.00	0.00	0.09	6.13	6.99	15.00
Jun-23	0.00	0.00	0.00	0.09	6.13	6.99	15.00
Jun-24	0.00	0.00	0.00	0.09	5.60	6.41	15.00
Jun-25	0.00	0.00	0.00	0.15	22.58	25.51	15.00
Jun-26	0.00	0.00	0.00	0.11	17.44	19.70	15.00
Jun-27	0.00	0.00	0.00	0.06	15.92	17.92	15.00
Jun-28	0.00	0.00	0.00	0.06	10.47	11.81	15.00
Jun-29	0.00	0.00	0.00	0.05	9.03	10.18	15.00
Jun-30	0.00	0.00	0.00	0.04	8.69	9.80	15.00
July,1	0.00	0.00	0.00	0.08	10.47	11.85	15.00
July,2	0.69	0.00	0.00	0.04	21.37	23.99	15.69
July,3	5.63	0.00	0.00	0.04	21.67	24.32	20.63
July,4	14.71	0.00	0.00	0.06	16.42	18.48	29.71
July,5	18.90	0.00	0.00	0.15	35.96	40.50	33.90
July,6	16.65	0.00	0.00	0.16	41.18	46.37	31.65
July,7	14.55	0.00	0.00	0.22	25.48	28.86	29.55
July,8	23.56	0.88	0.00	0.70	51.97	59.25	39.44
July,9	46.76	12.35	0.23	0.60	47.36	53.95	74.34
July,10	57.67	36.97	0.61	0.65	96.97	109.58	110.25

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Up to September 30

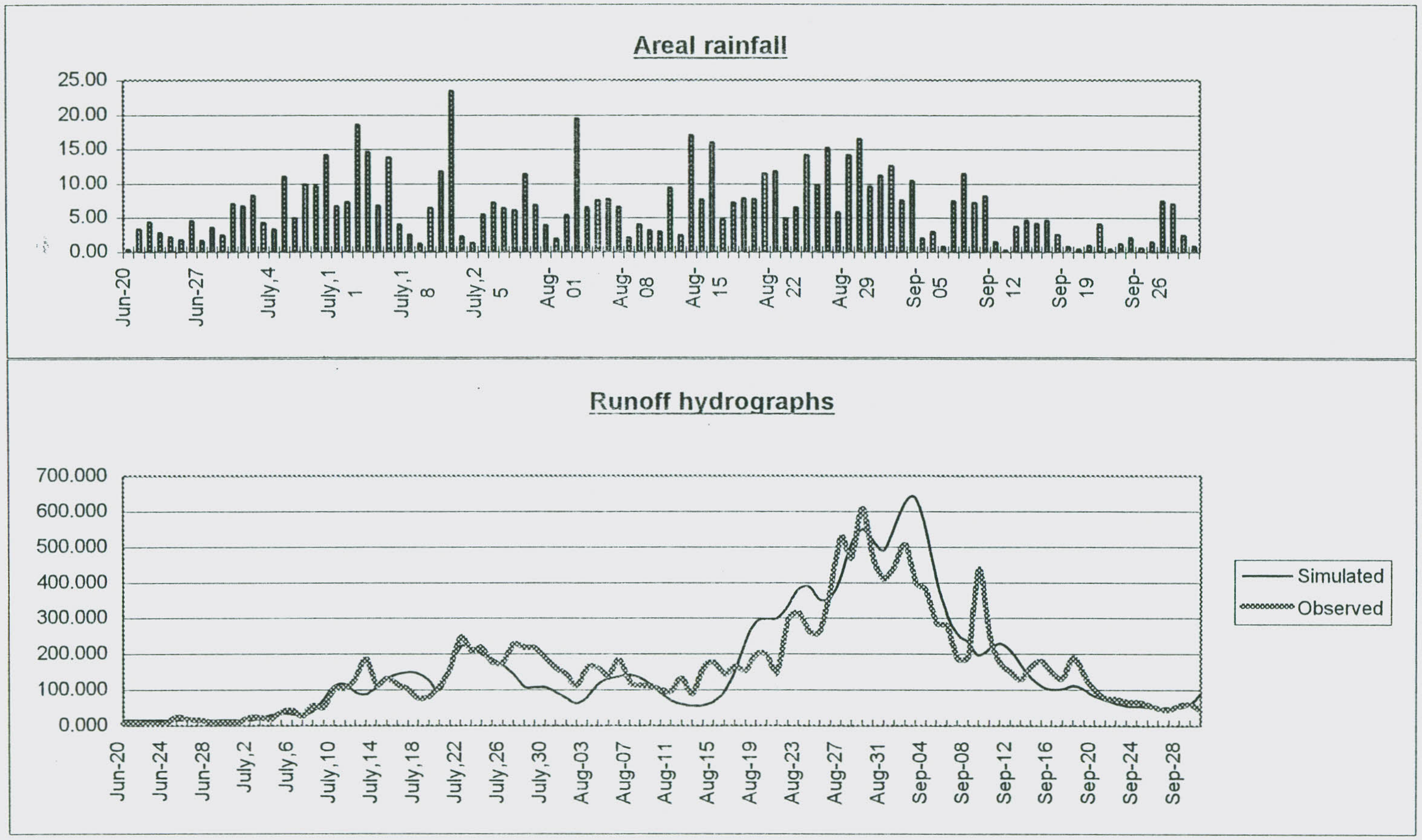


Fig.1: Observed and Simulated runoff for simulation year- 1989

A.1.5

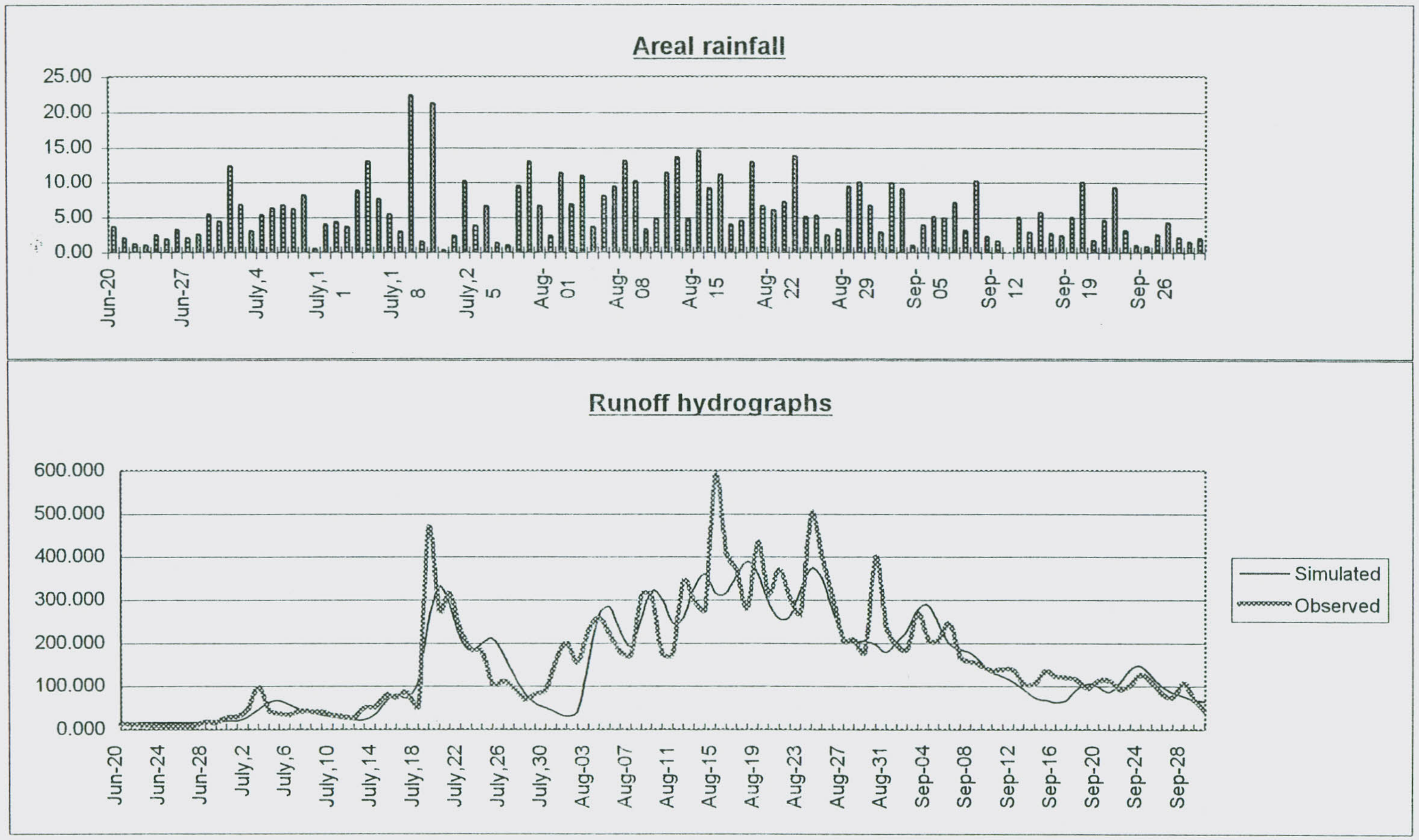


Fig. 2: Observed and Simulated runoff for simulation year- 1990

A.1.6

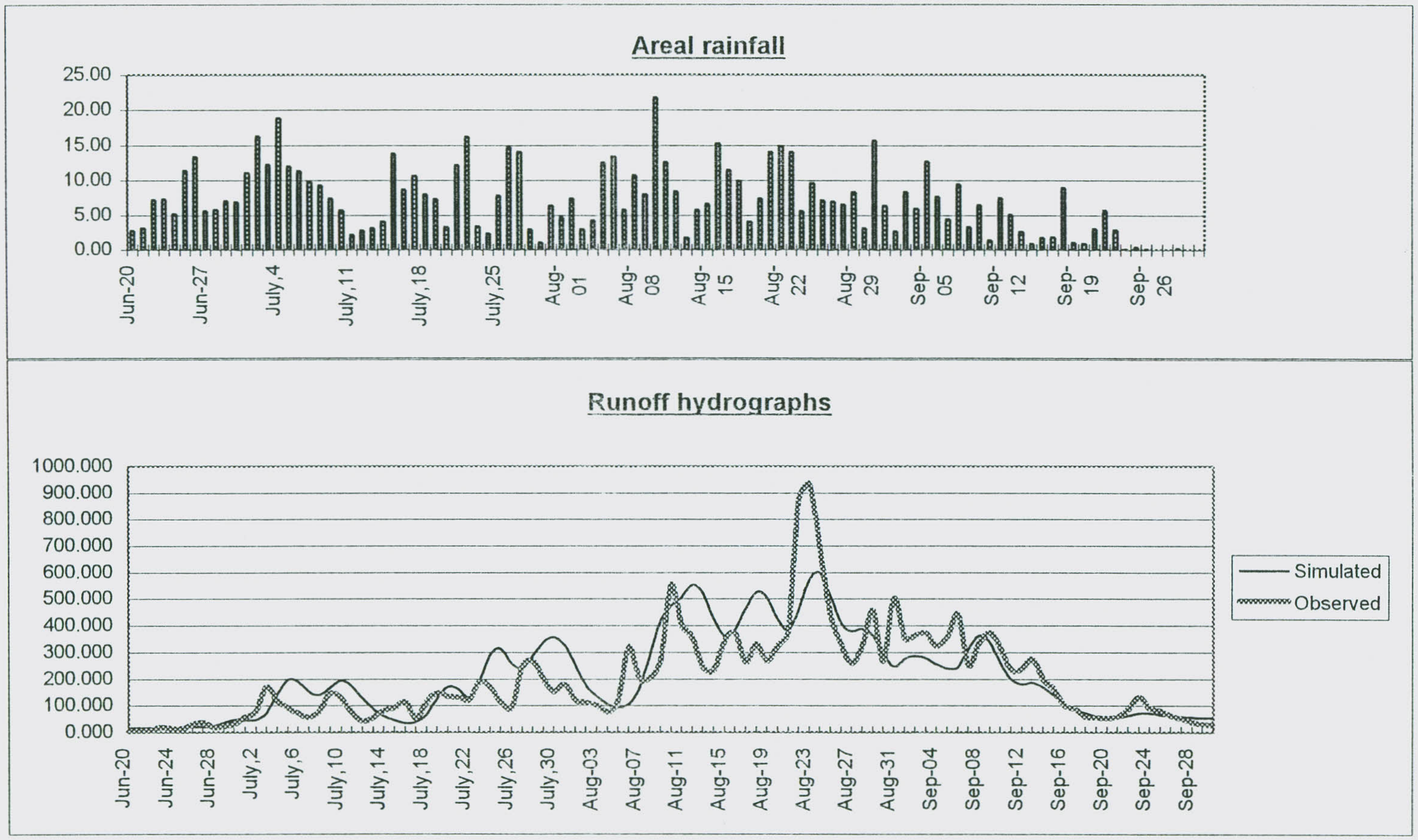
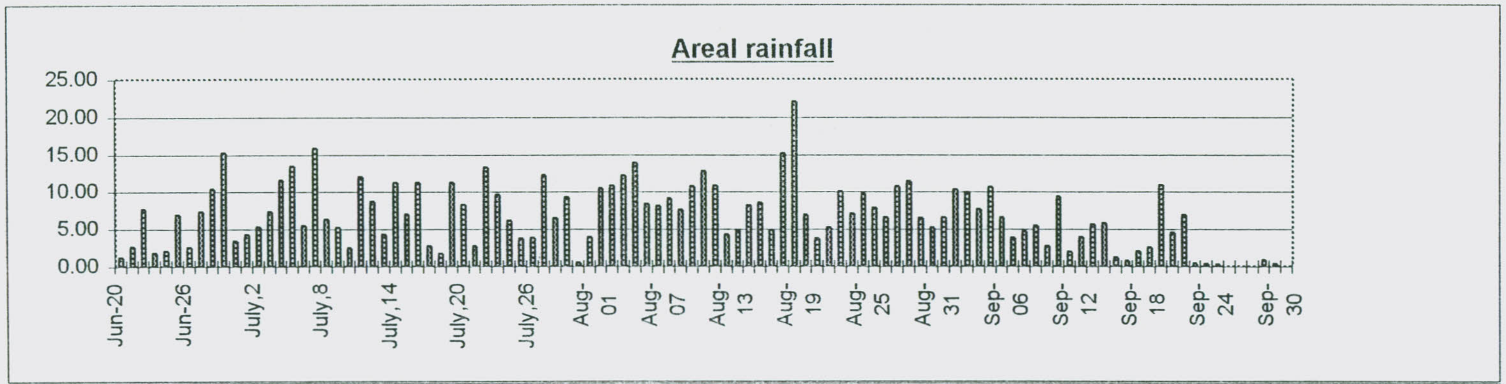


Fig. 3: Observed and Simulated runoff for simulation year- 1991

A.1.7



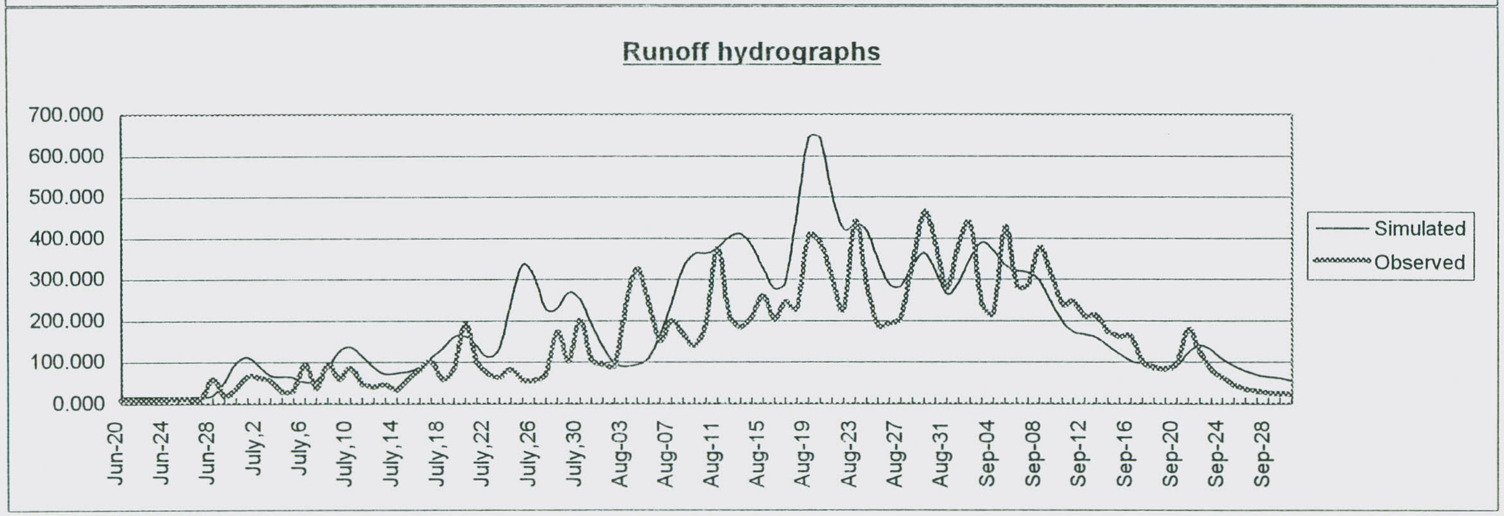


Fig.4: Observed and Simulated runoff for simulation year- 1992

A.1.8

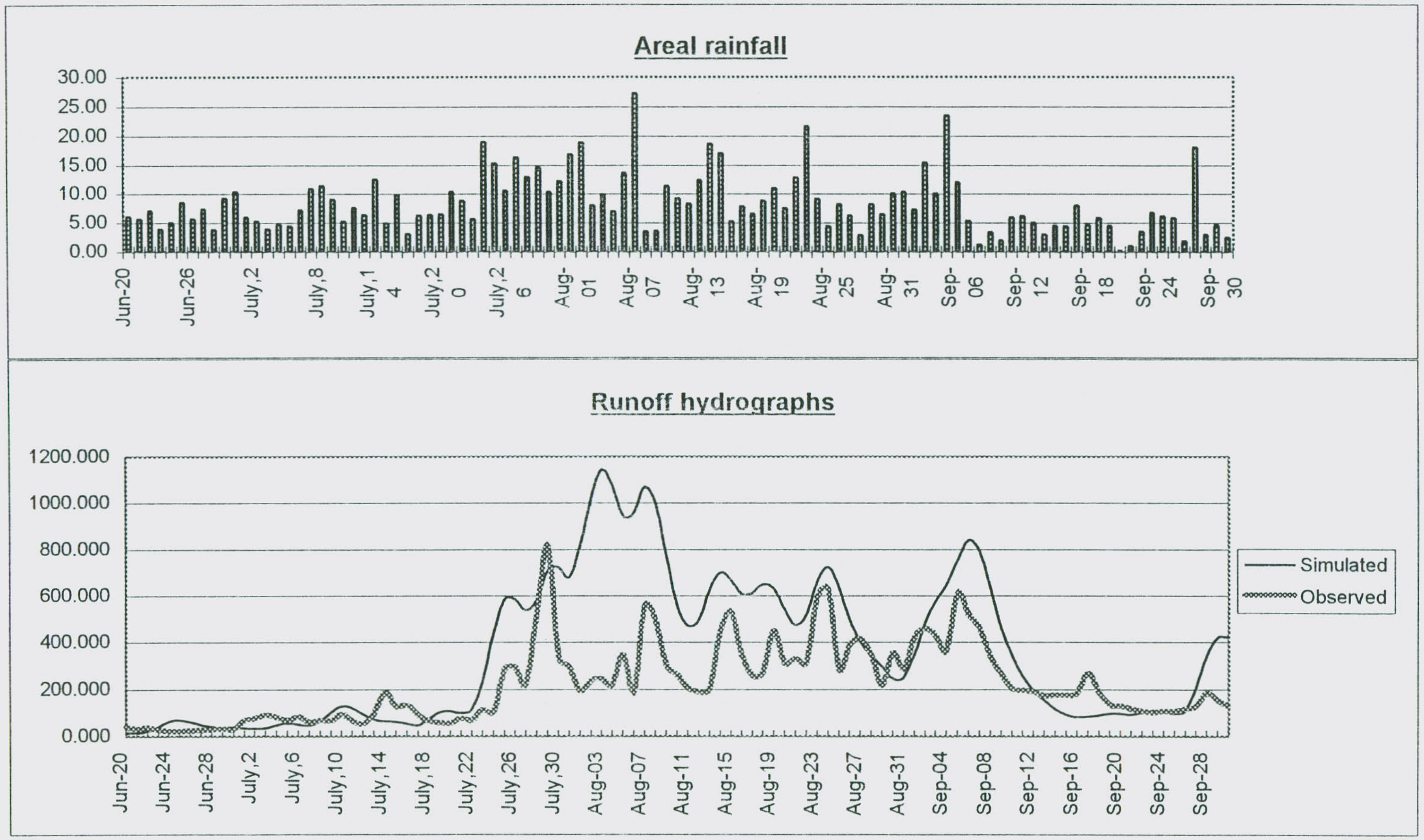


Fig.5 Observed and Simulated runoff for simulation year- 1993

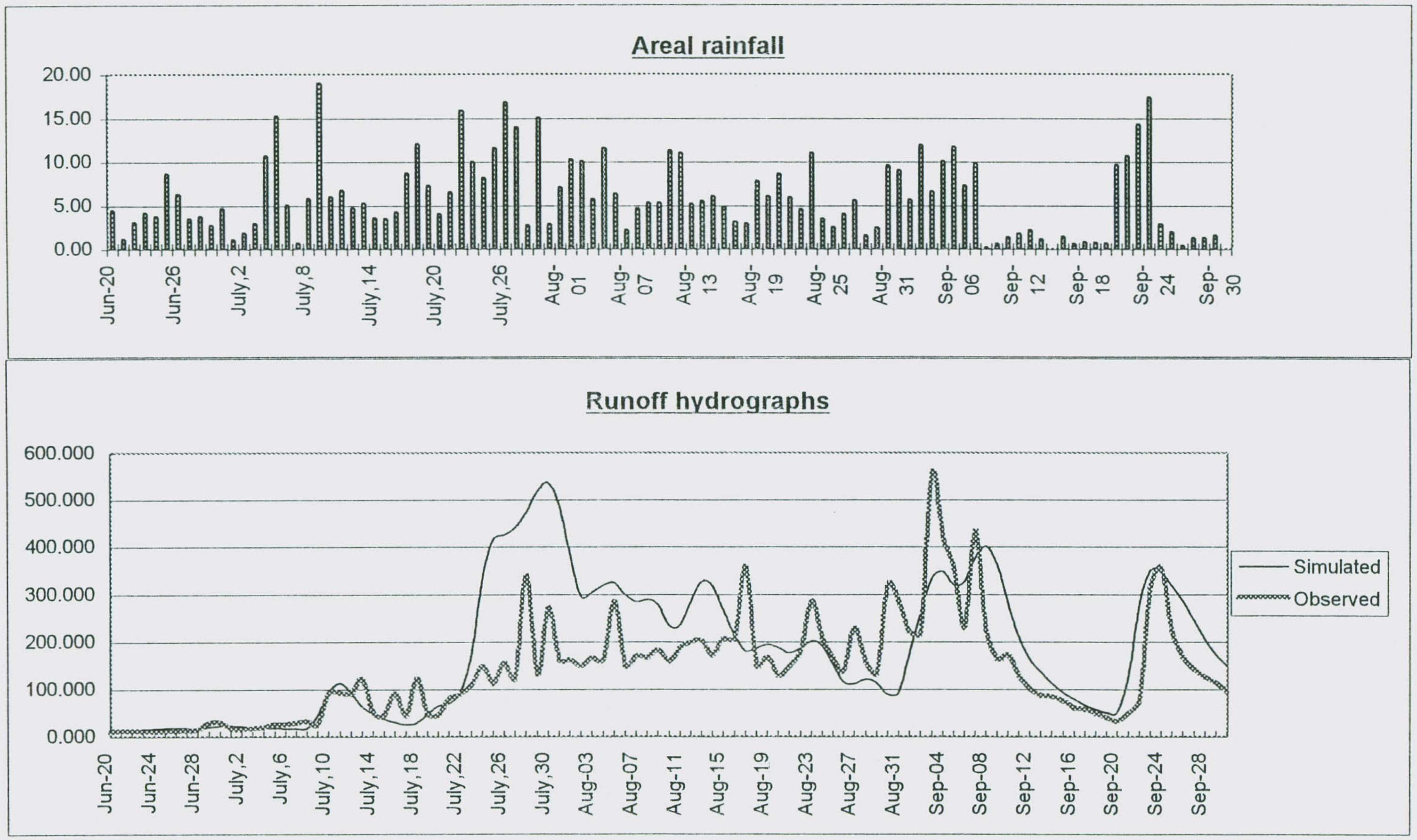


Fig.6: Observed and Simulated runoff for simulation year- 1994

A.1.10

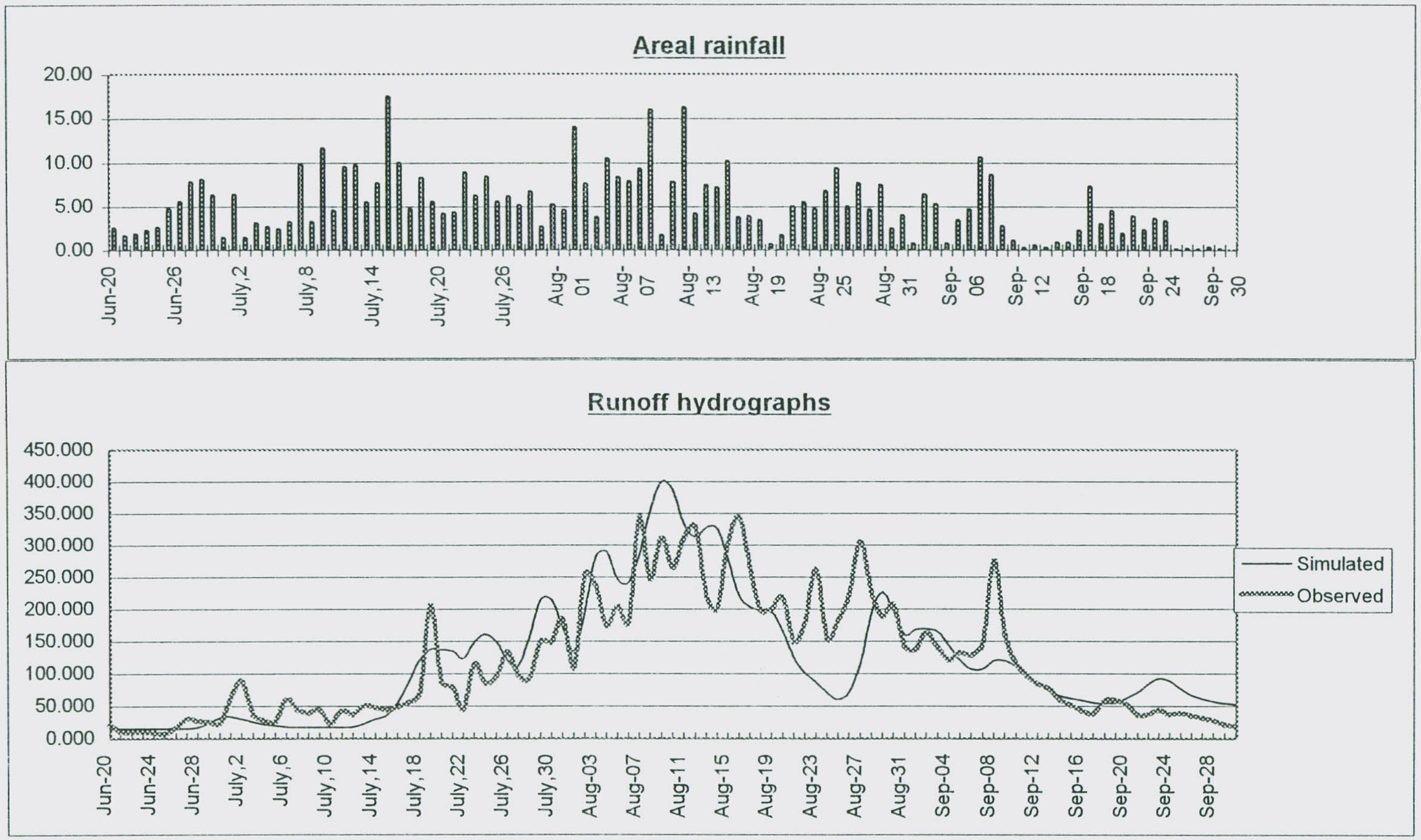


Fig.7 Observed and Simulated runoff for simulation year- 1995

A.1.11

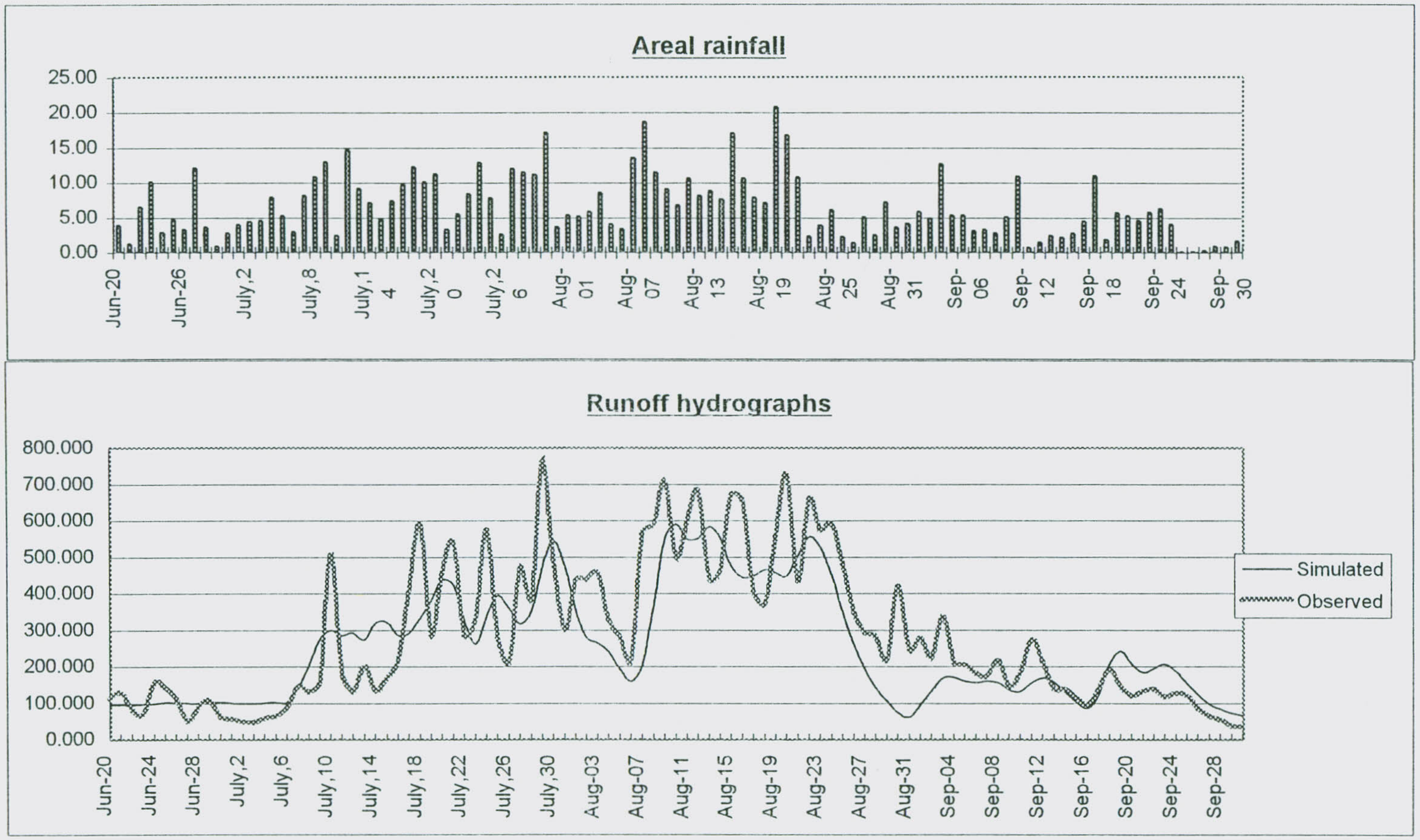


Fig.8 Observed and Simulated runoff for simulation year- 1996

A.1.12

A.2. 13: Sample format for Input-data-entry for the Optimization Model

(including the Objective function, constraints and Operational requirements)

INPUT DATA INCLUDING OBJECTIVE FUNCTION BELOW:

0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,-1,-1,-1,-1,-1,-1,-1,-1,-1,-1,-1,-1,0,0,0,0,0,0,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0
1000.0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0
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55,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
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55,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
55,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0
242,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0
135,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0
135,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0
145,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0

(continued)

150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0
150,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1
12,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
12,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0
147.36,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0
13.600,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0,0,0,0,0,0,0
22.180,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0,0,0,0,0,0
35.080,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0,0,0,0,0
152.90,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0,0,0,0
57.250,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0,0,0
116.61,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0,0
163.54,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0,0
238.76,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0,0
192.06,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0,0
145.01,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1,0
107.80,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,0,-1,0,0,0,0,0,0,0,0,0,0,0,1,-1
926.73,0,0,0,0,0,0,0,0,0,0,0,0,1,0,0,0,0,0,0,0,0,0,0,0,0,1,0,0,0,0,0,0,0,0,0,0,0,-1

Declaration

I, the undersigned, declare that this thesis is my work and all sources of materials used for this thesis have been duly acknowledged.

Name: **Paulos Shemeles**

Signature:

Place: **Addis Ababa**

Date of submission: **December, 1998**