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Assessment of micro-dam irrigation projects and runoff predictions for ungauged catchments in Northern Ethiopia

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Assessment of micro-dam irrigation projects and runoff predictions for ungauged catchments in Northern Ethiopia.

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Abstract

Tigray is located in the northernn part of Ethiopia and is solely dependent upon rainfall for agriculture and other water needs. Often here the practice is cultivating one crop per year and the crop harvests are insufficient due to less rainfall. The region is one of the most degraded and drought prone regions of Ethiopia. In the past decade many micro-dam irrigation projects have been implemented by governmental and non-governmental organizations. These micro-dams and ponds are providing supplementary irrigation and supply water for both livestock and people.

Unfortunately, some of the irrigation projects are either functioning at low capacity or not functioning at all. In 2007 during the first field campaign of this study a survey was made on 40 dams to identify the major problems attributing for this. The survey concluded that the most prominent problems are insufficient inflow towards the reservoir (water harvest failure), excessive seepage from reservoirs, reservoir early sedimentation, poor irrigation water application and management, structural and dam stability and, social and institutional related problems. The stated problems are wide and multi-disciplinary in their nature. Few studies have been made to address some of the cited problems by different researchers. The water harvest problem which was not yet covered in any of the researches made in the region is studied in-depth based on the data collected from three monitoring stations established for this study.

The watersheds of all micro-dam irrigation projects found in the region are considered as ungauged catchments since there is no flow record required for sizing the reservoir. In this study several approaches have been explored and new findings are presented that would give a basis for estimating runoff for ungauged catchments in the Northern part of Ethiopia. The research is the first of its kind in the region and most of the data were collected by the author for this particular study. A number of software packages have been utilized to compile, analyze and produce useful results from the data.

About 20 rainfall-runoff events were calibrated and optimized to derive the most optimum parameter sets for the three watersheds. Regionalization of model parameters was successfully done by relating them with watershed characteristics that can be generated from maps, field surveying, laboratory testing and GIS processing. Similarly the runoff coefficients used in the existing design process were

evaluated and the drawbacks were outlined. Accordingly a new set of runoff coefficients in the form of graphs and maps were produced that can be used for planning and assessment of runoff from a given watershed.

Acknowledgment

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Abbreviations

AMC	Antecedent Moisture Condition
CDF	Cumulative Distribution Function
CoSAERT	Commission for Sustainable Agriculture and Environmental Rehabilitation in Tigray
CN	Curve Number
CSA	Central Statistical Agency
CV	Coefficient of Variation
DEM	Digital Elevation Model
DIN	Deutsches Institut für Normungen
EM	Engineer Manual
EMA	Ethiopian Mapping Agency
ER	Engineering Regulation
ESRDF	Ethiopian Social Rehabilitation Fund
FAO	Food and Agricultural Organisation
FDRE	Federal Democratic Republic of Ethiopia
PM	Penman Monteith
F-DIST	F-Distribution
FEMA	Federal Emergency Monitoring Agency
GDP	Goss Domestic Product
GIS	Geographic Information System
GLUE	Generalized Likelihood Uncertainty Estimate
GPS	Global Position System
HEC-GeoHMS	Hydraulic Engineering Center Geospatial Hydrologic Modeling System
HEC-HMS	Hydraulic Engineering Center Hydrologic Modeling System
HSG	Hydrologic Soil Group
ICOLD	International Commission On Large Dams
IAHS	International Association of Hydrological Sciences
ILWIS	Integrated Land and Water Information System
IWMI	International Water Management Institute
MCS	Monte Carlo Simulation
NEH	National Engineering Hydrology
NGO	Non Governmental Organisations
NMSA	National Meteorological Service Agency
NRCS	Natural Resources Conservation Service
NS	Nash Sutcliffe
PEP	Percentage Error in Peak
PEV	Percentage Error in Volume
REST	Relief Society of Tigray
RMSE	Root Mean Square Error
SCS	Soil Conservation Service
S_e	Standard Error of estimate
SOF	Saturation Overland Flow
SSR	Sum Squared residuals
TBAaRD	Tigray Bureau of Agriculture and Rural Development
TBWREM	Tigray Bureau of Water Resources Energy and Mines
USDA	United States Department of Agriculture
USACE	United States Army Corps of Engineers
WB	World Bank

1. INTRODUCTION

1.1 Statement of the problem

The expenditure incurred on the implementation of micro-dam irrigation projects is significant and the feasibility of the project depends on returns and sustainability. This could be achieved through proper planning, design, construction and operation of such projects. Generally, a micro-dam for irrigation projects and most of the components of small scale projects are constructed using locally available earth, and is prone to failure unless constructed under the strictest possible supervision and quality control. Once the project is commissioned, the operation, management and periodic maintenance are equally important for attaining the intended purpose and a sustainable project.

According to annual reports of Commission for Sustainable Agriculture and Environmental Rehabilitation in Tigray (CoSAERT) and Tigray Bureau of Water Resources Energy and Mines (TBWREM) some of the projects constructed have not met the intended objectives. Moreover, Leul (2003) reported that the performance of the micro-dam irrigation projects was diminished due to seepage and failure to harvest the designed runoff. Hence, it requires critical study and analysis of the possible causes of failure attributed to each project. Awulachew et al. (2005); Bashar et al. (2005); Yohannes (2004); Nigussie et al. (2005) made assessment of small scale irrigation, water harvesting and micro irrigation projects in Ethiopia, and identified a number of related key problems and constraints.

The problems encountered in the micro-dam irrigation projects in Tigray region can be grouped broadly into agriculture & agronomical, geological, hydrological, hydraulic, and geotechnical. The magnitude of the problems differs from one project to another. Meanwhile, the prominent technical problems observed in many of irrigation projects are insufficient inflow towards reservoirs, excessive seepage from reservoirs, reservoir early sedimentation, poor irrigation water management application, structural and dam stability, and social and institutional related problems.

By its nature, the study, design and problem-assessment of micro-dam irrigation project entails a multi-disciplinary approach and it is impossible to cover in a single PhD study. Furthermore previous works made by (Mintesinot, 2002; Mintesinot et al., 2005; Fasil (n.d.); Eyasu, 2005 and Girmay et al., 2000) discussed issues related to poor irrigation water management and its ill effect on the downstream command area. Also, social and institutional problems are highlighted in Girmay et al. (2000); Eyasu (2005); Teshome (2003) and several unpublished reports. Tamene et al. (2006); Nigussie et al. (2005, 2006) also illustrated the early reservoir sedimentation taking some micro-dam irrigation projects as case studies.

One of major problems that have not been covered yet by the aforementioned studies is the water harvesting problem. Thus, this PhD study is outlined to highlight the problems of the micro-dam irrigation projects and addressed in depth the harvest problem and its possible measures.

There is no any measured flow data at the watershed of any of the micro-dam irrigation projects designed and constructed in the region. Thus the water resources potential of all the dams is predicted with a lumped empirical model which often leads to wrong incoming inflow estimation. According to reservoir level and water volume records collected in the years 2001-2005 by CoSAERT, more than 50% of the surveyed dams can only store less than 50% of their storage capacity. Thus it is self evident that there is a need to carefully look at the hydrological process, study the relationship between rainfall and runoff for ungauged catchments which is the principal component of estimating the volume of water entering the reservoirs.

Unless a thorough investigation is carried out on the prominent problems of micro-dam irrigation projects mentioned above, and the drawbacks are understood, then the very purpose of these projects will not be served. Moreover these failures will be incorporated in future planning, design and construction.

1.2 Objectives and methodology

The objectives of the research are:

- assess short-comings of the micro-dam irrigation projects in Tigray region related to technical, institutional and management problems,

- explain in detail possible causes of the short-comings related to water harvesting problems by undertaking field measurement, sampling, testing, analyzing and mapping,
- undertake hydrological studies and modeling that suits the existing situations,
- develop rainfall-runoff relationships that can be transferred to ungauged catchments, and
- develop a new set of runoff coefficients that represent the watersheds of the region.

To undertake this study the following methods and study approaches are employed:

- documentation review of existing projects and collecting secondary data,
- review of similar studies, cases, experiences in different parts of the world,
- pilot study on all dams and selecting focus study areas that meet the study objectives,
- field survey involving a survey on the extent of problems based on prepared questionnaire or inspection formats,
- installation of equipment or instruments required for data collection and monitoring,
- field data collection, measurement and laboratory testing: this approach includes determining engineering properties of soils in the irrigation project areas, hydro-metrological, agronomical and hydraulic parameters. Testing of collected samples in the laboratory to determine the physical and engineering properties,
- topographic surveys of reservoirs, and reservoir water level measurements,
- data analysis, interpretation and develop different maps using standard software,
- hydrological model development: generation of a hydrological model for selected catchments using the available software programs that correlate the rainfall and runoff of selected catchments having different land use, slope and topography,
- test and calibrate hydrological models,
- sensitivity and uncertainty analysis,
- test and validate the outputs of the modeling and calibration results, and
- develop recommendation of rainfall-runoff relationships and runoff coefficients that can be adapted in similar designs in the future

1.3 Organization of the Thesis

The statement of the problem discusses the motivation of this PhD study. The objectives of the research were outlined combining applied and basic researches. The methods and study approaches starting from data collection, analysis and presentation listed above will give insight to the study process followed to arrive at the stated objectives. The next part of the thesis is presented into four chapters.

Chapter 2 gives the background information of Ethiopia in general and the study area in particular. Situation assessment of micro-dam irrigation projects in the region were discussed in details giving insight to the problem and constraints attributed to low projects performance.

The reservoirs of all micro-dam irrigation projects were sized with limited data inputs since gauging stations were not available in their watersheds from where the water drains to the reservoirs. Chapter 3 presents runoff prediction for ungauged catchments based on the information gathered from three monitoring stations. This study is new to the study area and most of the inputs were gathered by the author. To compile, analyse and produce useful results from the data a number of software packages have been utilized. The input data were used for hydrological modeling and calibration. In this chapter calibration and optimization of 20 rainfall-runoff events were done to arrive at optimum model parameters. Uncertainty analysis on runoff and evapotranspiration estimation done with Monte Carlo simulation is presented. Sensitivity analysis and model validation are also incorporated in chapter 3. The use of bootstrapping techniques for generating synthetic data for model validation for ungauged catchments where historical flow information is highly limited are also explained. Regionalization of the rainfall-runoff process of ungauged catchments is a top research priority in the fields of applied hydrology as many of the watersheds in the world are not gauged or with limited data availability. Availability of data is more serious problem in developing countries like Ethiopia. To this end three different approaches namely derivation of equations through regression, regionalization of hydrologic model parameters and monthly water balance approach are presented based on the rainfall-runoff events gathered from the selected monitoring stations. Chapter 3 will give basis for understanding the rainfall-runoff process in Northern

Ethiopia and estimate runoff for ungauged catchments with similar catchment features to the monitoring stations.

Chapter 4 deals with the runoff coefficient determination and development which is one of the basic inputs required for reservoir sizing during planning and design of all micro-dam irrigation projects in the region. In this chapter different method of analysis like event, decadal and seasonal methods were evaluated and finally new runoff coefficient data sets in the form of diagrams and tables are provided.

Summary and conclusion of this PhD thesis is presented in chapter 5, and the list of reference materials used for this study are reported in chapter 6.

2. ASSESSMENT OF MICRO-DAM IRRIGATION PROJECTS IN TIGRAY REGION

2.1 Background

Ethiopia is located in East Africa, with geographical location between 3°25' and 14°48' North latitudes and 32°42' and 47°59'East longitudes. More than 85% of the population lives in rural areas which depend on farming for their livelihood. Agriculture accounts over half of the country Gross Domestic Product (GDP) Block (1999). The water used for agricultural production predominantly comes from rainfall. But, rainfall is so unevenly distributed, with good rainfall in the southwest of the country to scanty rainfall in the north and south eastern parts of the country, causing frequent droughts. In countries like Ethiopia where widespread poverty, poor health, low farm productivity and degraded natural resources are major problems, expansion of irrigated agriculture is vital. The importance of introducing irrigated agriculture into the economy of developing countries is based on the fact that rain-fed agriculture is not capable of supplying the desired amount of production to feed the increasing population. Irrigation is not only required to supplement the deficit from annual crop demand, but also to adjust seasonal variations or erratic nature of rainfall distribution. This inadequacy of moisture will inevitably lead to considerable yield reduction. The struggle to secure food in the country can be greatly assisted by increasing production using irrigation water from small-scale, medium or large-scale irrigation schemes.

2.1.1 Water resources and irrigation development in Ethiopia

According to FAO(2005) survey (base year 2002), Ethiopia withdraws only 5.558 billion m^3/a which is about 5% of the total surface flow, with 6% for domestic sector, 0.34 % in industry and 93.6% allocated for agriculture. The total potential irrigable land in Ethiopia is estimated to be around 3.7 Million ha. In contrast the actual irrigated land is 250,000 ha (Awulachew et al., 2005). It is self evident that the potential and the current use are quite incomparable. Despite to this fact there are many traditional irrigation schemes that have been under practice for so many years. Modern irrigation schemes have been implemented notably in Awash Valley and in the regional government states (Figure 2.1) through government as well as non-governmental institutions (NGOs). Irrigation development is viewed as an integral part of the economic development. Currently, the MoWR has identified about 560 irrigation potential sites on the major river basins and details with respect to the river basin is discussed in Awulachew et al.(2007).

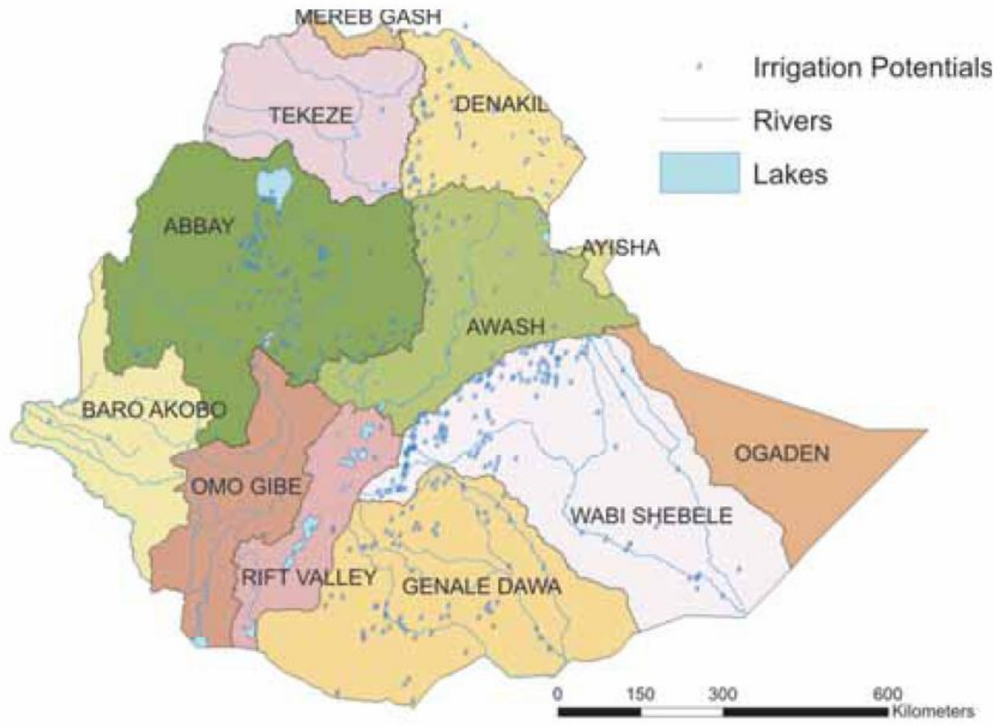

Figure 2-1: Major river basins of Ethiopia and irrigation potentials (Awulachew et al., 2007)

2.2 Description of the study area

2.2.1General

Tigray region is located in the Northern Ethiopia and its geographical location is 12°15'-14°58' North and 36°22'- 40°00'East with total land area of 50,078.64 km^2 (Figure 2-2). The region capital Mekelle is about 770 km north of Addis Ababa. It is divided into five zonal administrations namely, North western, Western, Central, Eastern, Southern and Mekelle city, a special zone (Figure 2-2).According to Central Statistical Agency (CSA), (CSA, 2008) the population of the region is slightly above 4.3 Million and about 80.5 % of the population lives in rural area. The average population density of the region is 79 person/km^2 and high concentrations in the Eastern, Southern and Central zones (CSA, 2008).

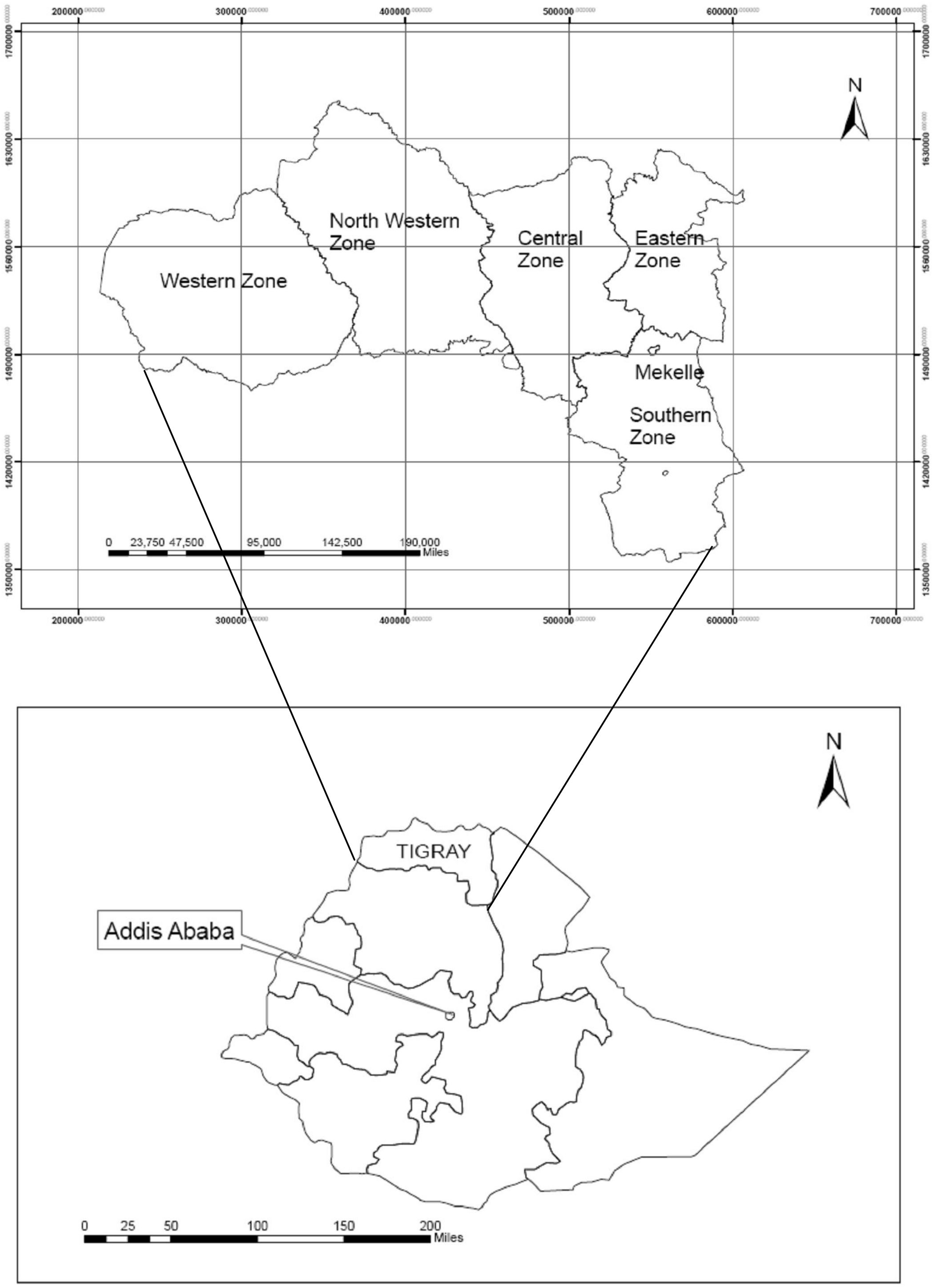

Figure 2-2: Tigray region location map

2.2.2 Irrigation development

The rainfall distribution of the region is not uniform and often not sufficient for crop production. Thus recurrent drought and famine has been observed in the region frequently. As a result, repeated crop failure and scarcity of food have forced the farmers to join famine relief aids in the form of food for work. Farmers have been trying for centuries to supplement the rainfed agriculture with traditional irrigation.

Reports indicated that irrigation has been practiced traditionally since many years back in different parts of the region (Gebremedhin and Kiflom, 1997; Solomon and Yoshinobu, 2006). Traditional river flow diversion was made possible by constructing a barrier wall with existing local material (stone, mud, woods, etc…) across the river channel and conveys water through a series of canal network system. This system is common where there is perennial flow throughout the year. In low lands like in the southern region of Tigray, “Raya” farmers tend to divert the seasonal flows coming from nearby highlands by constructing temporary barriers and canal networks to farmer plots.

The current government considers irrigation to be the means to improve the socio-economic situation of the farmers and also improve the environment. Establishing CoSAERT in 1994 was part of the regional government development intervention and endeavour. The commission was mandated to design and construct small scale micro-dam irrigation projects. Other governmental and non-governmental organizations have been also involved in the development of irrigation in the region. Modern diversion schemes, spate irrigation, ponds, shallow wells and lift irrigation have been also considered as strategic means to irrigate more land. In some unpublished reports the total irrigated area is stated as 6500 ha, which is about 0.6 % of the total arable land (Leul, 2003). Figure 2-3 shows the location of micro-dam irrigation projects in Tigray region.

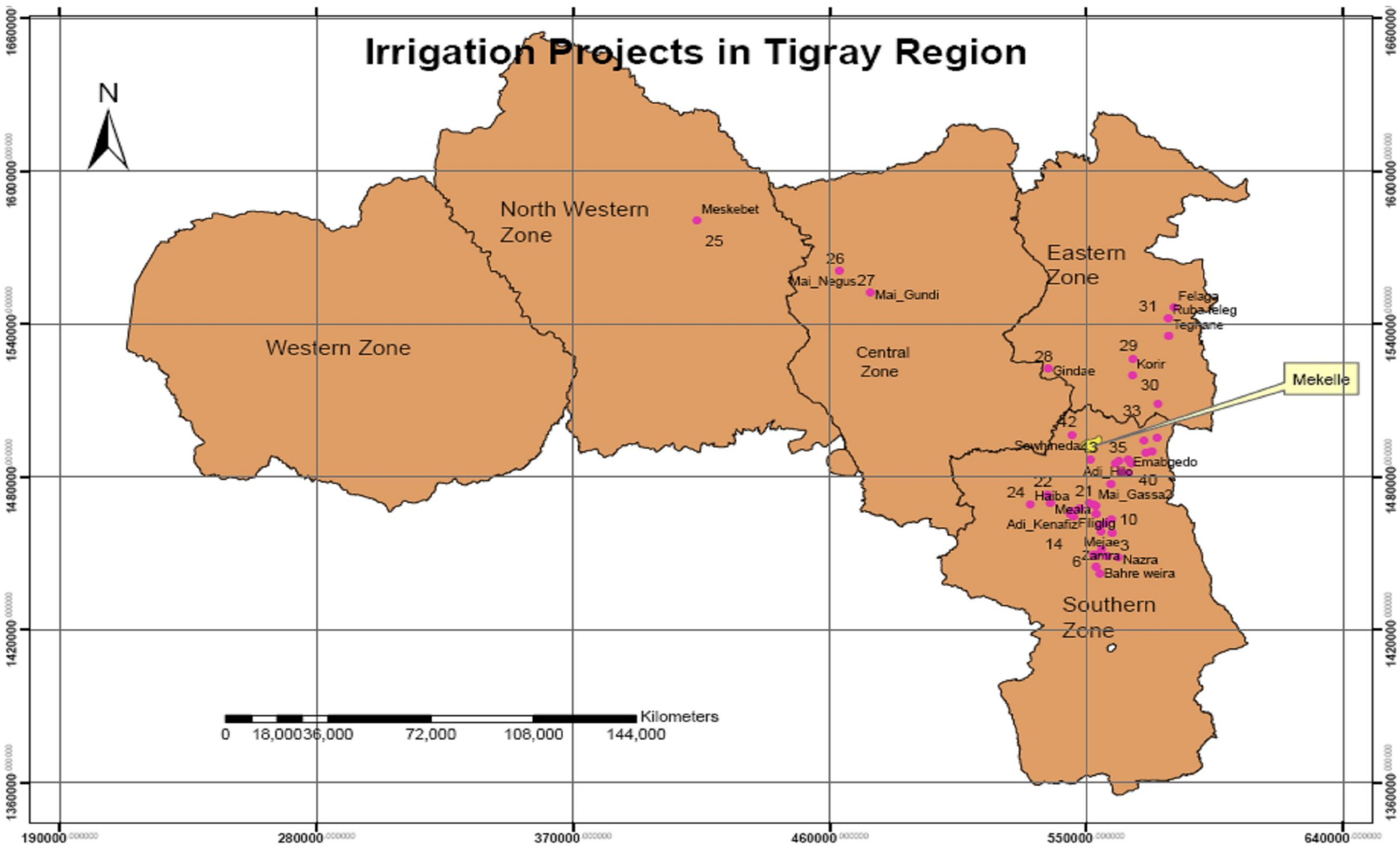

Figure 2-3 Location of irrigation projects in Tigray region (Source TBWREM)

2.3. Situation assessment of micro-dam irrigation projects

When CoSAERT was initially launched the target was to construct 500 projects and irrigate about 50,000ha within 10 years. The project formulation document stressed that with the cultivation of 50,000ha, it is possible to attain food security in the region. It was also indicated that CoSAERT will also work towards the rehabilitation of the region's environment. The first three years of the implementation period was targeted to build institutional capacity, human resources and implementation capacity in line with the design and construction of 47 projects. In the next seven years the remaining projects should have been completed to attain the final target. However, the project implementation was not as intended.

In the period 1994 to 2002, significant achievements were made on the development of agriculture through irrigation by employing seasonally harvested runoff using earth dams (Eyasu, 2005; Nigussie et al., 2006). However it was also noted that the implemented irrigation projects are not meeting the intended purpose due to several constraints and also due to mistakes made during pre-planning or after the completion of the projects.

Most of the reservoirs are failing their design target due to insufficient inflow, excessive seepage and sediment deposition (Leul, 1994; Nigussie et .al., 2006). A survey made on 40 dams in 2007 by the author indicated that there are a number of technical and non-technical problems which are affecting the performance of many irrigation projects. The survey was mainly focused on technical problems. The assessment was made based on the United States Federal Emergency Management Agency (FEMA) dam-safety guidelines with minor modifications to address the irrigation networks (Appendix 2.1). The observed problems were categorized as: insufficient inflow; excessive seepage; early sedimentation; poor irrigation management and water application; structural and dam stability issues; agriculture and extension; institutional; and social and related problems. These are explained below.

2.3.1 Insufficient inflow

At the end of each rainy season CoSAERT/ TBWREM measures the reservoir water level and estimates the volume of water that entered the reservoirs. The measurements were started in 2001. The existing records were not consistent over time and between dams, and so a preliminary selection was made for this thesis to identify dams having records over the analysis period (2001-2005). A comparison of 29 dams was made between the measured volumes of water and the volumes expected in the planning phase. The categorized results are shown in Table 2-1.

Table 2-1: Percentage of dams annual water harvest compared to the designed estimate

Year	Percentage of dams harvest volume of water				SUM
	Cat. 1< 25% of the designed estimate [%]	Cat. 2 25% to 50% of the designed estimate [%]	Cat. 3 >50% to 75% of the designed estimate [%]	Cat. 4> 75% of the designed estimate [%]	[%]
2001	3	0	31	66	100
2002	31	28	20	21	100
2003	38	31	28	3	100
2004	55	35	0	10	100
2005	41	31	14	14	100

The volume of water that entered the reservoirs in 2004 was very low compared to others and this could be attributed to low rainfall throughout the region. In 2001 the rainfall was above average and many dams stored more water than in other years.

In order to estimate the water resources potential of given watersheds it is necessary to have good sets of climate and flow data. But all dams lack such data within their watersheds and thus the hydrologic response was evaluated based on a lumped rainfall abstraction model by assuming a runoff coefficient for given land uses, soil types and topography.

$$\dot{V} = 1000 \times C \times \dot{h}_P \times A_C \qquad (2.1)$$

Where

$\dot{V}$ = volume of inflow (m^3/a)

C = runoff coefficient (1)

$\dot{h}_P$ = precipitation (mm/a)

A_C =catchment area (km^2)

Nigussie et al.(2006) described qualitatively the reasons behind insufficient inflow. However, quantitative assessment is more important and is useful for addressing the most important input parameters that significantly influence the current design estimation procedure. To address this question 10 watersheds (Figure 2-4) have been selected for analysis. Care has been taken to avoid dams which stored more than 90% of the design estimate during the analysis period. This kind of assumption will help to avoid uncertainty in reservoir inflow estimation, due to spillway overflow.

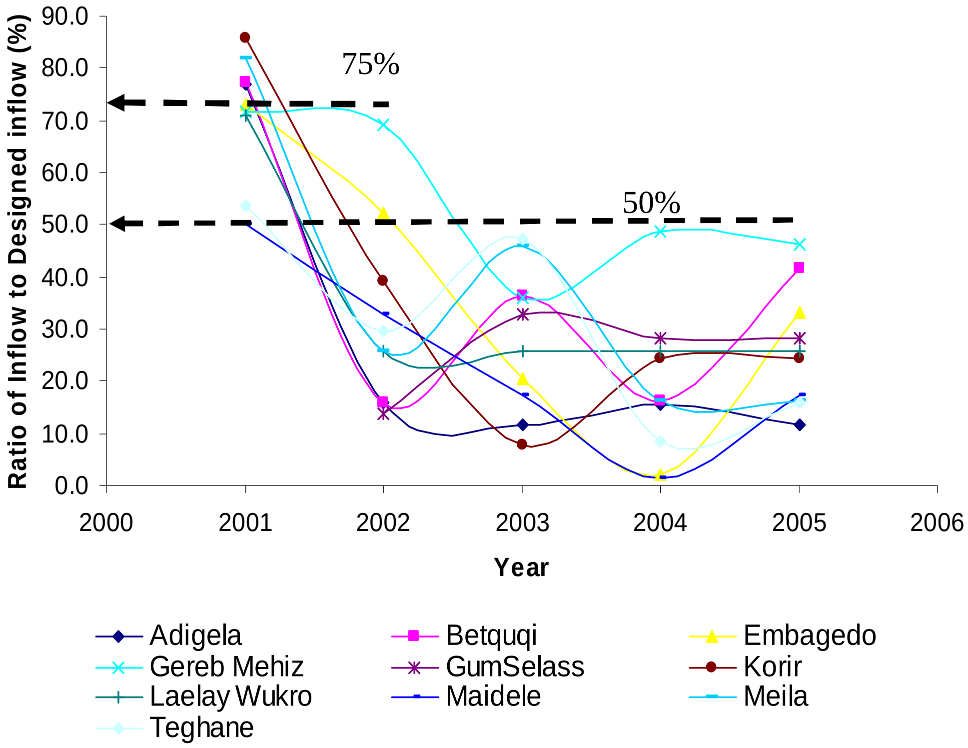

Figure 2-4: Ratio of measured inflow to estimated inflow during planning phase for the selected watersheds during analysis period

All dams store less than 50% from 2003 to 2005 and, except for two dams, the remaining dams stored less than 50% in 2002 of their respective design estimate. The input parameters for equation 2.1 are discussed hereunder separately.

Catchment area

Area delineation made on the 10 watersheds showed that there is no significant discrepancy between the designed estimation and delineation made by this study. But there are two exceptional dams, with one being overestimated and the other underestimated. For this study purpose underestimation is not considered significant.

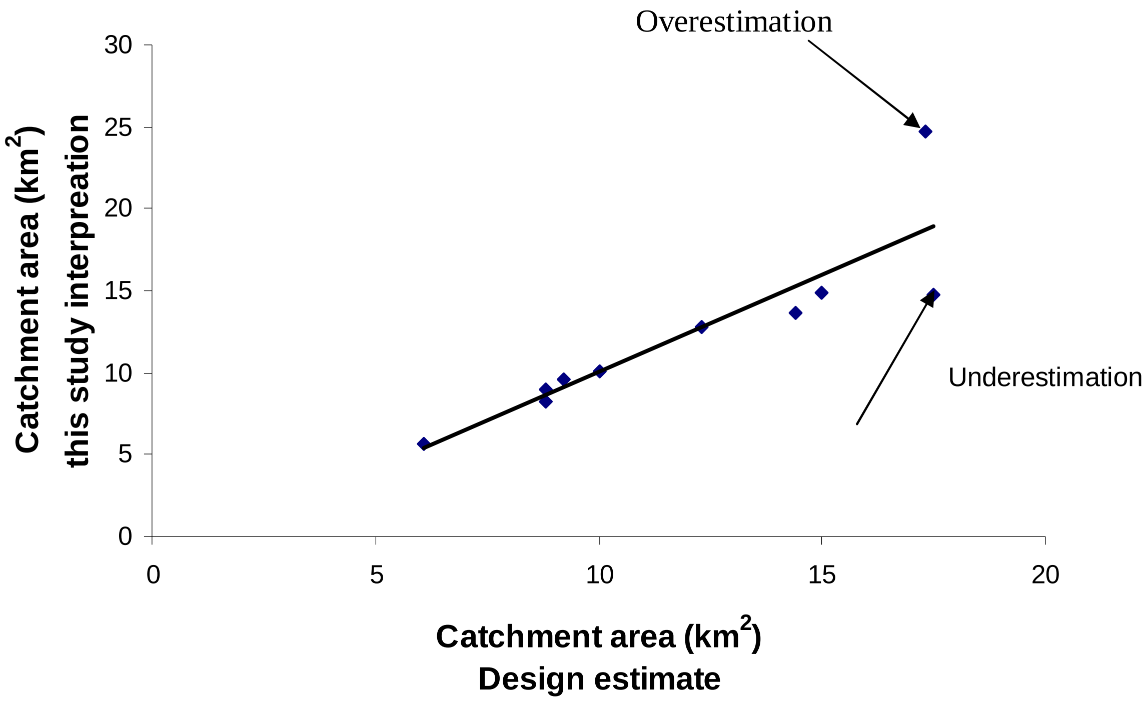

Figure 2-5: Comparison of catchment delineated by this study and taken from design reports (Source CoSAERT)

Rainfall

There is no rain gauge station at any dam site. During the planning phase the designers either used rainfall records available from nearby stations or data transfer with scientifically accepted procedures. As shown in Table 2-2, in 2001 and 2003 excess rainfall was observed on all watersheds, whereas there was a deficit in 2002 and 2004 in GumSelassa, Gereb Mehiz and Maidelle watersheds in comparison with the assumptions made during planning.

Table 2-2: Comparison of measured rainfall and the rainfall depth adapted during design

Watershed	Distance from rain gauge (km)	Design rainfall (mm)	Analysis years									
			2001		2002		2003		2004		2005	
			P_r (mm)	R (%)	P_r (mm)	R (%)	P_r (mm)	R (%)	P_r (mm)	R (%)	P_r (mm)	R (%)
Adigela	12.0	367	na		na		na		na		na	
Betqua	2.4	362	786	117	542	50	568	57	508	40	731	102
Embagedo	11.0	350	na		535	53	533	52	390	11	598	71
Gereb Mehiz	10.0	329	624	90	288	-12	417	27	241	-27	342	4
GumSelassa	5.8	365	624	71	288	-21	417	14	241	-34	342	-6
Korrir	6.0	350	1112	218	588	68	471	35	445	27	495	41
Laelay Wukro	3.6	314	1112	254	588	87	471	50	445	42	495	58
Maidelle	7.0	350	424	21	288	-18	417	19	241	-31	342	-2
Meila	3.9	362	786	112	542	50	568	57	508	40	731	102
Teghane	2.0	410	na		na		na		na		na	

Note: P_r=recorded precipitation for each year; R=excess or deficit per year (percentage); na=no record

Runoff coefficient

The runoff coefficient indicates the percentage of the total rainfall that is changed into runoff. The runoff coefficients were selected from literature, based on watershed land use, soil and topography. The comparative assessment made in this study showed that the synthesized runoff coefficient (from rainfall and runoff records) is very low compared to the value adapted during design.

Table 2-3: Runoff coefficients determined in this study and the values used during design for respective dam

Watershed	Analysis year									
	2001		2002		2003		2004		2005	
	$C_{analysis}$	C_{design}	$C_{analysis}$	C_{design}	$C_{analysis}$	C_{design}	$C_{analysis}$	C_{design}	$C_{analysis}$	C_{design}
Adigela	na		na		na		na		na	
Betqua	0.12	0.30	0.04	0.30	0.08	0.30	0.04	0.30	0.07	0.30
Embagedo	0.16	Na	0.14	na	0.05	na	0.01	na	0.08	na
Gereb Mehiz	0.10	0.28	0.21	0.28	0.08	0.28	0.18	0.28	0.12	0.28
GumSelassa	0.12	0.30	0.04	0.3	0.06	0.30	0.09	0.3	0.06	0.30
Korrir	0.08	Na	0.07	na	0.02	na	0.06	na	0.05	na
Laelay Wukro	0.06	0.33	0.04	0.33	0.05	0.33	0.05	0.33	0.05	0.33
Maidelle	0.14	0.30	0.14	0.3	0.05	0.30	0.01	0.30	0.06	0.30
Meila	0.13	0.27	0.06	0.27	0.10	0.27	0.04	0.27	0.03	0.27
Teghane	na	0.30	na	0.3	na	0.30	na	0.3	na	0.30

Note: $C_{anlysis}$ = runoff coefficient determined by analysis: C_{design}= the value adapted during planning

In summary, it can be stated that it is possible to minimize the error resulting from wrong area estimation, one precondition being the availability of larger scale topographic maps (1:50,000 or larger) and the second being the undertaking of detailed field works. In order to achieve a better estimation during planning, the rain gauge network should be increased. This may take many years and is a continuous process. But it is urgent and should be started as soon as possible. Adapting runoff coefficients which are not calibrated for the conditions of the study area is a problem and thus has to be backed by a research.

2.3.2 Excessive seepage

Excessive seepage has been observed either at the downstream toe of the dam or through the abutments. In most cases the seeping water was clean, but there were dams where the water had a darkish colour and contained some soil materials. Excessive seepage, on the one hand, is a loss of productive water and, on the other hand, is a threat to the safety of the dam leading to failure. The seepage quantity estimated by CoSAERT based on records made in successive months in 2001 varied significantly among reservoirs with the lowest being 0.27 l/s and the highest 81 l/s. The reported seepage quantity is based on the records made during the end of the rainy season and will decrease as the level of the reservoir water subsides.

Except in 2001, there was no documented seepage water measurement made by CoSAERT/TBWREM. The current practice is to assume 25% of the stored water as a seepage and reservoir evaporation loss during annual budgeting and planning. About four dams are almost abandoned, because the stored water will last only for a few months. In 1999 TBWREM investigated the causes of problems and prepared typical rehabilitation designs for three selected dams. But none of the rehabilitation designs has been implemented until now.

Some of the reasons for the excessive seepage mentioned in different studies (Gebremedhin, 1999; CoSAERT, 1999; Nigussie et al., 2006) are:

- lack of detailed site investigation,
- poor data interpretation,
- the lack of professional experience,
- the geological formations on which the dams are built,
- excessive excavation of reservoirs blanketing soil, and
- provision of shallow cut-off depths.

Sometimes some experts argue that the seepage water has some positive impact in the downstream environment, which can be considered as an indirect benefit. It is common practice to see farmers divert the seepage water somewhere downstream of the dams either by constructing temporary weirs or using pumps to lift the water to their farm plots. Also micro-dams are playing an important role in recharging the groundwater of the nearby aquifer. Yet there is no concrete study on the extent and amount of water recharging the groundwater. However, the yield of springs and streams found downstream of dams has been increased since the construction of dams. In addition the boreholes drilled downstream of the dams are continuously supplying water without any problem.

Measuring the quantity of seepage, undertaking a study and preparing a rehabilitation plan, are tasks for the TBWREM. However, investigation of the seepage mechanisms within the dam body through foundation and abutments and its possible impact on the dam safety, is a research question that needs to be addressed. Understanding the seepage paths or lines within the dam will enable the undertaking of timely remedial measures and, perhaps, avoid potential failure. Seepage accounts for 25% of the causes of partial or complete failures of earth dams (Middelbrooks, 1953 in Singh, 1996). The various seepage and piping failure modes that can develop can generally be categorized into three main types, namely, 1) internal erosion through embankments, 2) internal erosion through foundations, and 3) internal erosion of embankments into/at the foundations (FEMA, 2003).

Most dams constructed in the region have a dam height less than 15 m, and in most cases the dams are located in a valley where there is no significant settlement (*Dam safety report*, Ethiopian Social Rehabilitation Fund (ESRDF), ESRDF, 2005). So the impact and the associated risk might not be significant, although no clear working guideline is available in the country for dam safety assessment**.** But the fact that the seepage water observed on the downstream side of one dam (e.g. Rubafeleg dam) is not clear water, the dam is located upstream of some settlements and the dam is full every year, there is a great concern about the safety of this particular dam. In 1997 longitudinal and transverse cracking had been observed during the initial filling of the dam. To this end, after undertaking some study, Mohammed (1998) reported the possible causes of the dam cracking. Dam cracking, internal erosion and piping are highly related. Thus, considering the past history of the dam and the existing situation it is necessary to identify the seepage lines either within the dam body or the foundation and thereby asses its impact on the safety of the dam.

2.3.3 Reservoir sedimentation

Tigray region is one of the regions where the environment is highly degraded. Due to population pressure in rural areas, steep slopes are still cultivated that exacerbate soil loss. In many parts of Tigray, soil erosion has made cultivation of old farms impossible, thus farmers have been forced to constantly cultivate new and more marginal areas (Esser et al., 2002). Agricultural land is the dominant land use in almost all watersheds of the micro-dam irrigation projects, which are often susceptible to erosion. Many researches indicated that the soil mass eroded from these cultivated lands ended up at the reservoirs.

During the planning phase, all dams are designed with the assumption that the part of the reservoir storage allocated for sediment will be filled up at the end of the estimated service life, often 25 years. Meanwhile the live storage (i.e. gross storage less dead storage) capacity of many dams is reducing quickly and in some cases the dams are completely silted up to dead storage level. According to the survey made in 2007, the author noticed that the bottom outlets of many dams had been blocked by sediments after being operational for few years. Nigussie et al. (2005); Tamene et al. (2006) and Eyasu (2005) tried to evaluate the existing design procedures and estimate the sediment yield by taking sample watersheds in the region. Nigussie et al. (2005) and Tamene et al. (2006) concluded that most of the reservoirs found in Tigray will be filled with sediment within a period less than their life expectancy.

The problems associated with early reservoir sedimentation can be attributed mainly to the use of poor data sets. All authors reported that the sediment yield varies considerably across the sampled watersheds and also the sample standard deviation is high. So it might not be possible to generalize from a single value. However, the techniques adapted are simple and can be replicated. Thus, with the inclusion of more dams and perhaps developing relationship between model parameter values and physical watershed parameters, it is possible to attain workable estimates for sediment inflow in the region.

None of the studies made an attempt to measure stream flow and correlate the sediment yield with the inflow discharge. Thus research in this regard will give a full picture of the sediment yield in the region and the experience can be transferred to other parts of the country.

2.3.4 Poor irrigation management and water application

Insufficient inflow is one problem, as discussed earlier, but the already stored water is not managed well and is not applied to the farm lands properly. Irrigation is necessary to alleviate the moisture stress; over irrigation or uncontrolled irrigation and defective canal irrigation network systems will also lead to undesirable negative impacts.

Some of the problems discussed in CoSAERT (1999) related to poor irrigation were absence of flow-measuring facilities, poor irrigation infrastructure layout, seepage from canals and canal structures, and lack of periodic maintenance of the irrigation infrastructure system. Mintesinot (2002) also reported that low irrigation efficiency and rapid deterioration of the physical infrastructure are manifestations of some of the schemes.

Mintesinot et al. (2005) made a performance assessment on three selected irrigation projects using International Water Management Institute (IWMI) comparative indicators. The conveyance efficiency reported in this study was at best 74.5% and at worst 53.2%, which indicated considerable losses from the canal system. Also output per unit water consumed for the three dams was found to be 0.115514USD/m^3, 0.13189USD/m^3 and 0.1546USD/m^3 for Haiba, Meila and Mainugus irrigation projects respectively.

Field observations and personal communication with professionals involved in the development of irrigated agriculture showed that unattended irrigation is resulting in water-logging of farm land near by the canals and at the low lying farm lands. Water logged areas are not suitable for farming practices and are liable to salinity. Eyasu (2005) and Fasil (n.d.) reported that salinity is becoming a treat to the development of irrigation in the region; in fact some lands started to be abandoned due to salinity.

2.3.5 Agriculture and extension

The performance of the irrigation projects can be also evaluated in terms of the proposed cropped area during planning and actual irrigated land. For this study purpose, data for actual irrigated land of 14 dams (2002-2006) were extracted from records of the Regional Government of Tigray Bureau of Agriculture and Rural Development (TBAaRD). Analysis made for the 14 dams having records from 2002 to 2006, showed that the ratio of actual irrigated land to the proposed irrigated land during planning is very low, as shown in Table 2-4.

Table 2-4: Comparison of the actual irrigated land to proposed area during planning for 14 dams (2002-2006)

Year	Proposed Irrigated area (ha)	Actual Irrigated area (ha)	Ratio irrigated to proposed (%)	Min Ratio (%)	Max. Ratio (%)	Median (%)
2002	1376.0	872.6	63.4	14.2	91.4	67.4
2003	1376.0	581.7	42.3	8.3	93.6	38.7
2004	1376.0	621.9	45.2	5.8	100.0	36.2
2005	1376.0	419.5	30.5	2.8	100.0	24.8
2006	1376.0	609.9	44.3	5.6	100.0	49.3

Note= Min./ Mix. refers to the minimum/maximum ratio among the assessed dams

The Min and Max ratio in Table 2-4 indicates from the total surveyed dams having minimum and maximum ratio of the actual irrigated to the proposed irrigated area during planning. The low ratio is mainly attributed to less inflow towards the reservoir. Poor irrigation extension and farmers knowledge in irrigated agriculture, lack of periodic maintenance of the irrigation infrastructure system and poor infrastructure design also contributed to the poor performance.

In most cases the type of crops grown are different from the proposed cropping pattern during the planning phase. This problem is partly driven from the interest of the farmers who prefer to grow crops like maize, because the forage can be used for animal feed. It is also worth mentioning that the extension support given to the farmers often fails to address such gaps.

2.3.6 Structural and dam stability issues

The major structural components of micro-dam irrigation systems are the dam (earth dam), the inlet and outlet (intake, buried pipe within the dam and outlet) and the spillway (approach channel, control section, conveyance channel, terminal structures, exit channel, retaining wall, etc…).

Slope stability design is an important element in the embankment dam designs procedure. Before the establishment of the Geotechnical Laboratory in the region, designs were carried out based on Terzaghi's slope recommendation. On looking at the annual reports of the CoSAERT/TBWREM, and from personal discussions with the engineers, there was no significant report on slope failure either upstream or downstream of the embankment. This, perhaps, could be because many of the dams are low in height and are not filling to the maximum level every year. But most of dams designed in recent years are large dams, both in terms of dam height and storage capacity. Thus the designs should be critically reviewed by a panel of experienced and qualified experts before implementation.

Another concern in the inlet and outlet works was the layout and the foundation where the pipes are placed. Pipes placed on heavy clay soil were susceptible to differential settlement resulting in the dislocation of pipe joints. Water leaking through these joints eroded the embankment material leaving the pipe without sufficient support. Such problems had been reported on two dams where immediate repair was made before total dam failure.

Many of the dams do not fill every year and thus spillway may not be functional periodically. The spillway arrangement of many dams is a chute spillway with a discharge carrier (chute) lined with masonry and stilling basin at the end. During field visit the author noticed that almost in all dams the chute floors are deteriorating with time because no maintenance was carried out. The most significant problems, and which need immediate attention, were the terminal structure (stilling basin and exit channel) of the spillway and those spillways where cascade drops are installed in the discharge carrier chute. The hydraulic and structural design of spillways of nine irrigation projects (namely, Rubafeleg, Mainugus, GumSelassa, Hizati Wedicheber, Meskebet, Gereb Mehiz, Adikenafiz and Maiserakit) should be checked and, where necessary, design modification should be carried out. In fact the terminal structure of Rubafeleg is already in danger and the vertical drops of Adikenafiz spillway is more likely to fail soon.

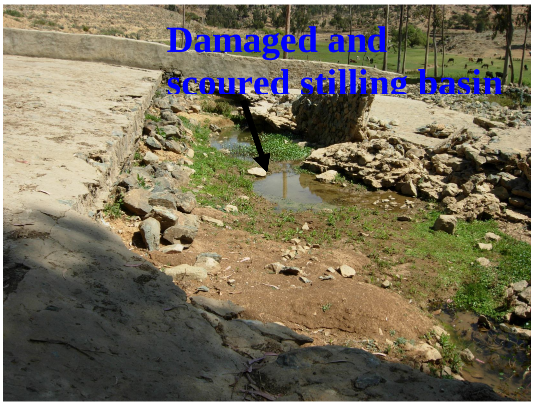

Figure 2-6: Terminal structure failures in Rubafeleg dam

Figure 2-7: Vertical drop spillway failures

2.3.7 Design and construction guidelines

The hydraulic and structural design of dam and associated appurtenance structures should meet standard design guidelines or codes of practice set by respective agencies or institutions of a given country. Unfortunately, in Ethiopia there is no standard guideline or code of practice that would be adapted to design such structures. ESRDF (1994) formulated some guidelines. However the documents were not institutionalized and were not accepted as working guidelines for design and construction of similar projects throughout the country. Often designers were following the standard design procedures available in literature, which designers frequently criticized for not following the same approach among their designs.

A dam safety review made by ESRDF (2005), indicated that, except for one dam (Gereb bati), all dams are categorized as low risk and are not a threat to the safety of human beings. CoSAERT/TBWREM considered all dams in the region as micro-dams. Different countries or international institutions classify dams based on dam height, storage capacity and associated risk to human life etc… (the United States Army Corps of Engineers, USACE (1979), ER1110-2-106); International Commission on Large Dams (ICOLD, 1987, Bulletin 59); World Bank (2001), (WB OP 4.37); European Standard (variable for respective countries); German standard (DIN 19700-11,2004-07). In Ethiopia there are no stated criteria for classifying dams; more or less the World Bank classification is adapted typically in CoSAERT/TBWREM. As more projects are under design and construction every year the Ministry of Water Resources, Regional Bureaus, universities and governmental or non-governmental organizations working in the areas of water resources should work towards setting a working guidelines and codes of practice for design, construction, operation and dam safety appraisal procedures.

2.3.8 Institutional issues

Few modern irrigation schemes have been designed and constructed by the Tigray Regional Bureau of Natural Resources and Environmental Protection in the period from 1991 to 1994. Later the duties and responsibilities, including the manpower, were officially transferred to CoSAERT when it was established in 1994. CoSAERT was mandated to study, design and construct micro-dam irrigation projects and to maintain major components of the irrigation schemes. Likewise Relief Society of Tigray (REST) was also involved in the design and construction of irrigation projects. Watershed management, irrigation and agronomy, extension services, and establishing and assisting the water users association, were the responsibility of the Bureau of Agriculture. Often the institutions fail to work in a coordinated manner.

Human resource turnover was very high in CoSAERT, despite many efforts made by the government to upgrade the professional capacity of all experts in the commission. It was almost impossible to retain the experienced professionals and recruit new staff, who are qualified and experienced, due to the competitive market. As a result many of the designs were carried by experts having few years of experience. In 1995 the numbers of design teams were six, with three engineers in each team. But in 2008, the numbers of design teams in TBWREM were only two, having engineers who are nearly beginners. Thus human resource turnover is significant and had a detrimental effect on the yearly plan and the quality of expected output.

2.3.9 Market and infrastructure

Availability of sufficient market facilities is a basic requirement for the expansion of irrigated agriculture and increased income for farmers. Similarly, availability of a road network is equally important for bringing the agricultural produce on time to the market. It has been observed that irrigation schemes nearby cities are relatively better compared to others located away from cities and road networks. However, production of perishable vegetables at the same time was a serious problem for the farmers. During my field work in May 2008 in Wukro, I noticed the cost of tomato was about 0.028USD/kg (1Birr= 9.0USD) which is roughly about 12% of the price of tomatoes two months before. In such desperate situations storage facilities and agro-processing industries might solve the problem, which needs government intervention and involvement of private sector.

2.3.10 Social, health and environmental

Due to implementation of micro-dam irrigation projects some lands will obviously be inundated by the reservoir. In some cases also, due to the spillway route, some houses were re-located. Such issues do create social resistance during design, construction and operation as well. The community reaction was more serious in those areas where the performance of dams is low. In some areas irrigation is new and farmers were hesitant to adapt the new technology. Sometimes the farmers were also suspicious perhaps the government might take their land in the long run or make them pay for the water they use for irrigation.

Downstream of many of the dams is marshy and the seepage resulting from the canal system creates favourable condition for mosquito breeding. Overall incidence of malaria for the villages close to dams was seven-fold compared with those living further away (Ghebreyesus et al., 1999). Theses statistics are significant and the most affected were children.

Summary

The observed problems in the micro-dam irrigation projects are wider in scope and demands immediate attention by the responsible institutions. Some of the studies made related to the stated problems should be transferred into practical applications. Micro-dam irrigation projects design, construction and operation experience in the country should be compiled and communicated across the different governmental and non-governmental institutions at the Federal and Regional Administration regions. This will pave a way towards preparation of working guide lines and standards that need to be followed starting from planning through implementation and later during operation to ensure project sustainability and minimal environmental impact.

The hydrological design is the most critical component of the overall planning and design of those projects. As all micro-dam irrigation projects are located in ungauged catchments, runoff prediction in ungauged catchments is the main research gap where no study has been done in previous works in the region. Thus the next chapters of this PhD thesis will focus on “Runoff predictions for ungauged catchments in northern Ethiopia”

3. RUNOFF PREDICTIONS FOR UNGAUGED CATCHMENTS IN NORTHERN ETHIOPIA

One of the most important inputs for determining the size of the water harvesting structure is the volume of water that will be expected to be generated from the catchment. Data analysis made on 29 dams from the reservoir level records made by CoSAERT in the years 2001-2005 showed that about 60%, 70%, 90% and 72% of dams harvest less than 50% of their design capacities in the years 2002, 2003, 2004 and 2005 respectively, This could be caused by errors in rainfall estimation and rainfall-runoff transformations. The focus of this study is on the later component, i.e. rainfall-runoff transformation.

All the built dams were in ungauged catchments at the time of design undertaking, and so hydrologic models were used to generate design runoff given historical records of precipitation measured by rain gauges. The problem of runoff prediction in ungauged basins is common to many countries (particularly developing countries) across the world, and it has always been an issue of great practical significance in engineering hydrology. Recognizing this problem, the International Association of Hydrological Sciences (IAHS) has embraced 'Prediction in ungauged basin' through a decade-long (2003-2012) initiative aimed at exploring ways of improving runoff prediction accuracy (Sivapalan et. al., 2003).

The over all objective of this study is to shed light on performance of runoff prediction in Northern Ethiopia. To meet this objective, we selected three CoSAERT dams in Ethiopia and examined the rainfall-runoff relationship in each watershed using data and hydrologic simulations. The data were obtained through a network of rain gauges, reservoir level observations installed by us and existing facilities installed by TBWREM and other necessary input data for model simulations. To the best of our knowledge, this study is the first of its kind in the study area in that it attempts to developed a better runoff prediction approaches through regionalization and parameterization of rainfall-runoff events that represent the existing condition of the study area. The proposed approaches can be also used for similar watershed features in other parts of the country.

3.1. Data preparation and data analysis results

3.1.1 Selection of watersheds

Pilot studies were carried out on 40 irrigation projects found in Tigray region. The pilot study was aimed at assessing the overall problem of the projects and select specific projects that will assist in attaining the stated objectives.

The factors considered during selection of representative watersheds were:

- Watershed landuse: most of the micro-dam projects watershed were dominated with cultivated land, thus the selected watershed would be preferred
- Landuse and soil uniformity: as the measurement would be taken at the outlet, more or less uniformity in soil texture and landuse should prevail
- Topography: flat and hill slope topography should be represented
- Availability of previous records: in order to make use of previous records made by the TBWREM, watersheds with long records and data are preferred
- Proximity: for installation and periodic monitoring proximity to the main road or accessible route was considered as one factor.

Based on the selection criteria, the following watersheds GumSelassa, Haiba and Laelay Wukro were selected. Table 3-1 show how the selected watersheds measured up to the section criteria.

Table 3-1: Selected watersheds and respective evaluation criteria

Watershed	Criteria				
	Landuse	Uniformity of soil texture	Topography	Previous record	Proximity
GumSelassa	90% cultivated land	clay and silt clay	Mean slope = 3.2%	No previous record	3.0-7km
Haiba	73% cultivated land	clay, silt clay and silt clay loam	Mean slope = 5.6%	2001 record is available	2.5km
Laelay Wukro	40.4% cultivated land and 56% bush, grass land (area enclosure)	Dominantly loam soils	Mean slope = 25.7%	2001 record is available	2.5-4km

Note: Proximity walking distances from the main road to the established monitoring station facilities.

Table 3-2 provides the salient features of the selected watersheds. The dams on these three watersheds were constructed between 1994 and 1997.

Table 3-2: Important features of the selected irrigation projects

Project name	Location				Catchment area (km^2)	Reservoir capacity 10^6(m^3)	Designed command area (ha)	Dam height (m)
	Zone	Wereda	Geographical location					
			North	East				
GumSelassa	Southern	Adigudom	13°11' – 13°16.1'	39°32.6' – 39° 35.4'	24.6	1.9	110	16
Haiba	Southern	Samre - Seharti	13°19.4' – 13°16.4	39°16.7' – 39°20.8'	24.4	3.1	250	16
Laelay Wukro	Eastern	Wukro	13°46.8' – 13°47.6'	39°32.6' – 39°35.4'	9.6	0.85	45	14.3

Note: the important features are taken from respective design reports

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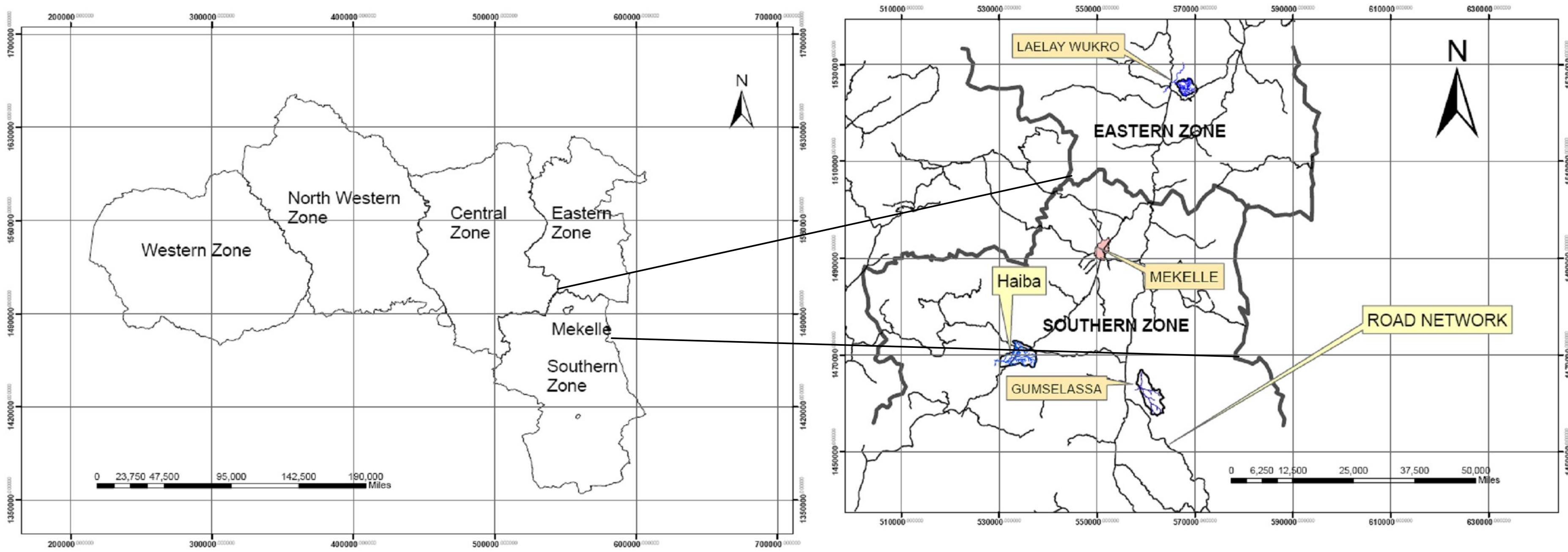

Figure 3-1: Location map of selected irrigation projects

3.1.2 Spatially distributed data

Geographic Information System (GIS)

With the availability of digital and remote sensing data, GIS is increasingly used in hydrology. GIS has got several applications in Hydrology such as input/output data handling, derivation of flow direction and slope maps (SKOPE and Loaiciga, 1998). In this study data were collected, organized and synthesized in GIS environment for hydrological modeling. Many of the input data preparations for hydrologic simulation are referred here as watershed processing and are discussed in the following sub topics. The basic software packages used in this study are Arcview GIS 3.3, Arc GIS 9.1, ILWIS 3.3 Academic and the Hydrologic Engineering Center Geospatial Hydrologic Modeling Extension (HEC-GeoHMS), which is an Arcview extension.

3.1.2.2 Watershed processing

The hydrological modeling process starts with identifying the drainage network, delineating the watershed and sub-basin boundaries, followed by analyzing and preparing the input catchment parameters for simulation. The total watershed area was initially delineated on topographic maps and areal photos of 1:50,000 scale. Later the drainage area was verified on ground and corrected accordingly. The sub-watershed area delineation and extraction were done based on the drainage network. The HEC-GeoHMS was used to carry out most of these activities. A Digital Elevation Model (DEM) is required as input for watershed processing.

Digital Elevation Model (DEM)

Scanned topographic maps scaled 1:50,000 were used to generate DEM. First the maps were digitized to extract maps of drainage, contour, roads and other important topographic or physical features. ArcView GIS 3.3 for digitizing and ILWIS 3.3 Academic were used to generate the DEMs.

Figure 3-2 shows the steps followed to generate DEM from scanned topographic maps. Figures 3-3 through 3-5 present the DEM for each of the watershed.

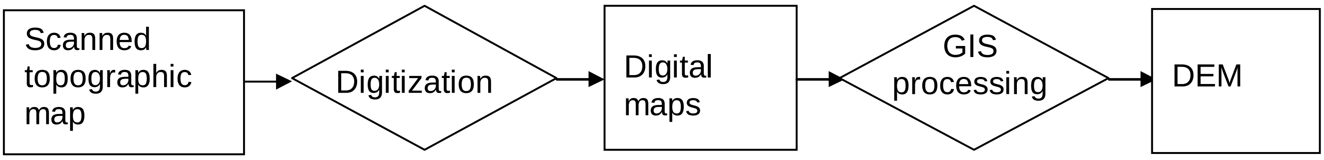

Figure 3-2: Schematic representation of Digital Elevation Model (DEM) preparation

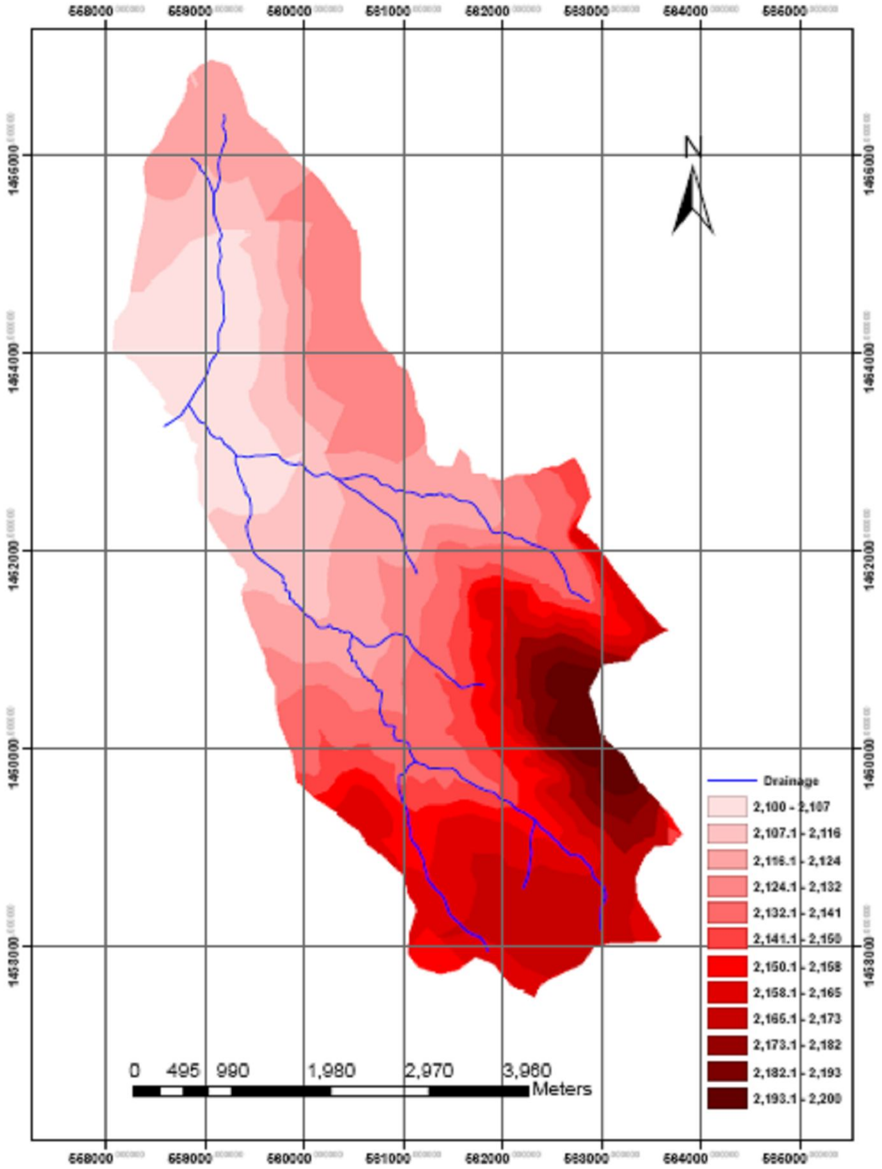

Figure 3-3: Digital Elevation Model (DEM) for GumSelassa watershed

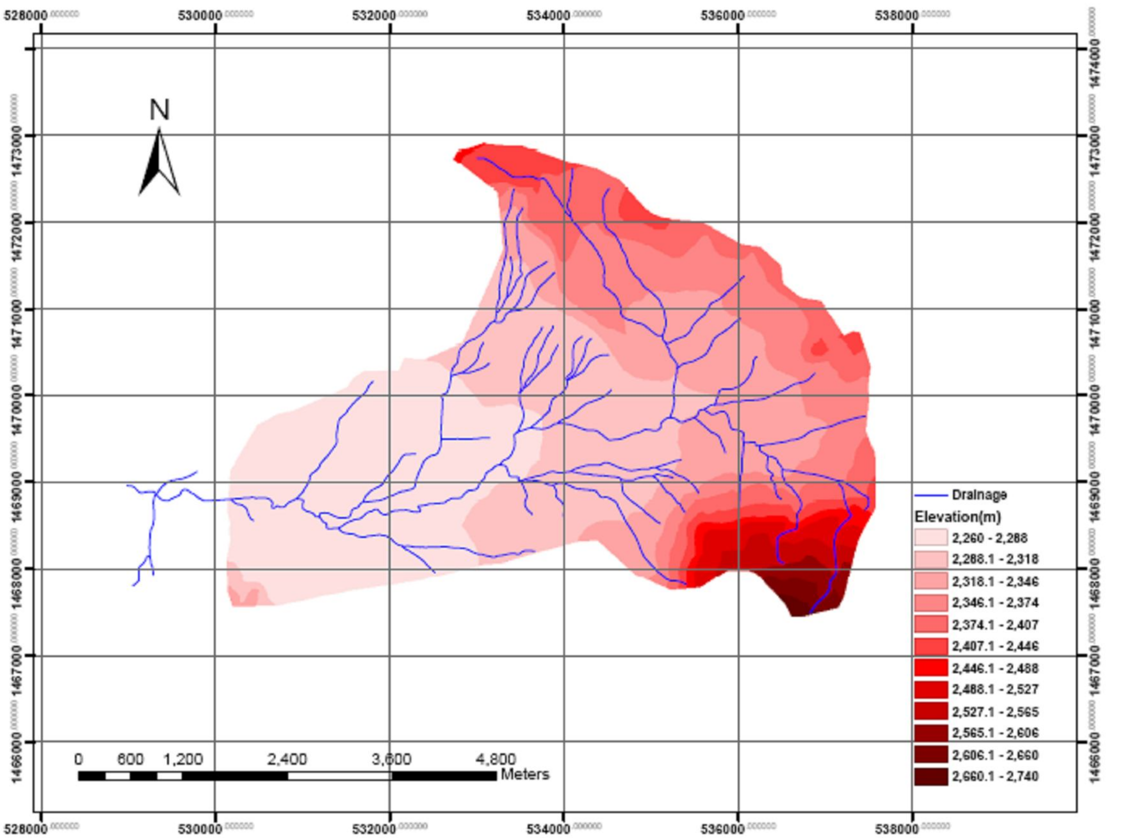

Figure 3-4: Digital Elevation Model (DEM) for Haiba watershed

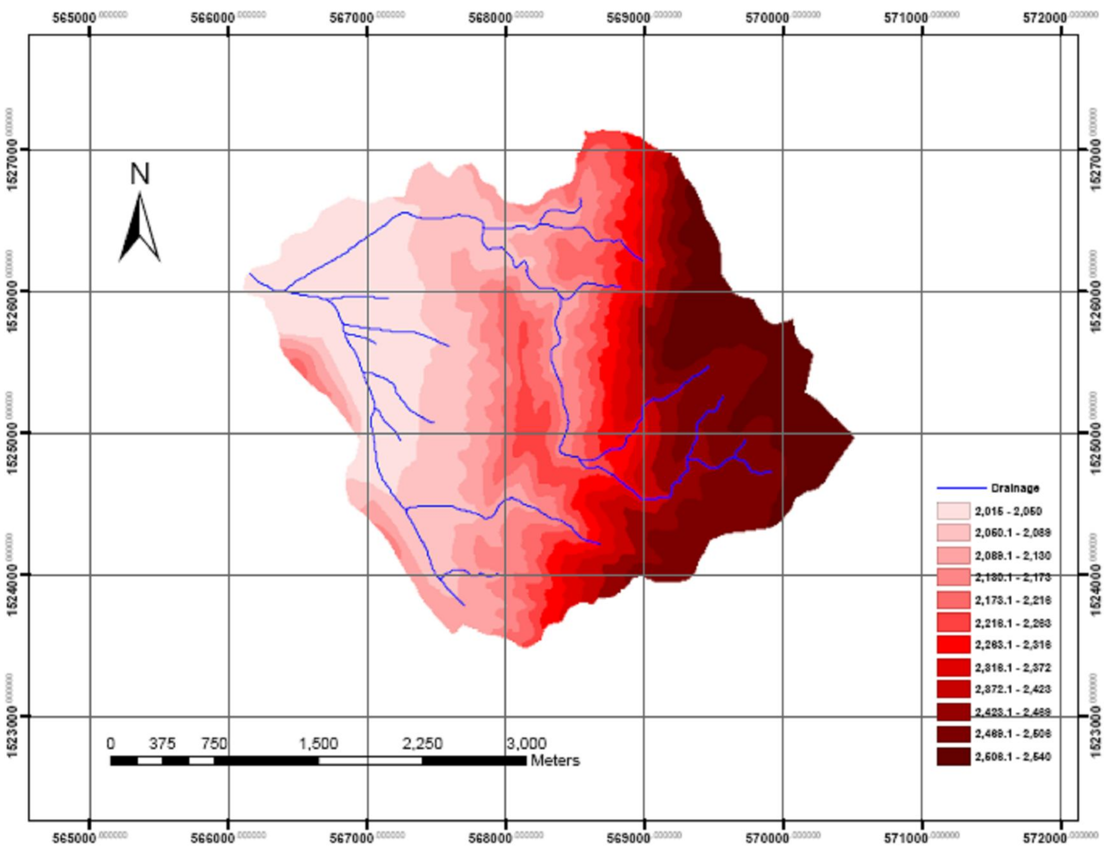

Figure 3-5: Digital Elevation Model (DEM) for Laelay Wukro watershed

Watershed processing

HEC-GeoHMS was used to prepare the required input parameters for the hydrological models from DEM following the procedures depicted in Figure 3-6.The program features are terrain preprocessing, basin processing, hydrologic parameter estimation and Hydrologic Engineering Center Hydrologic Modeling Systems (HEC-HMS) model support. The first two features are accomplished through a number of procedural steps. Terrain preprocessing includes filling sinks, assigning flow direction and flow accumulation, defining stream and sub-watershed area sizes. In the basin processing parameters like river slope, river length, watershed centroid, and longest and centroidal flow path were also determined.

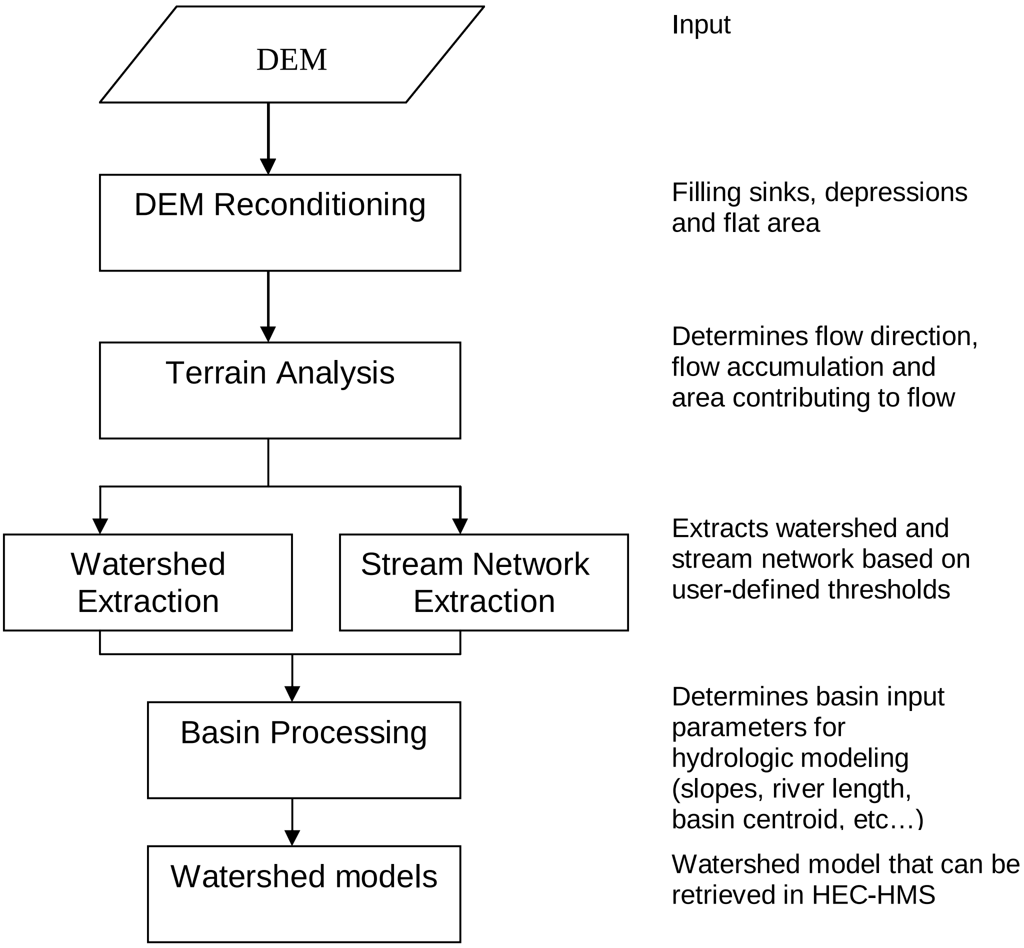

Figure 3-6: Flow chart for watershed processing in HEC-GeoHMS

3.1.2.3 Watershed slopes

This study uses a DEM to estimate slopes for all pixels within a catchment, then constructs a cumulative frequency distribution of slopes from which slope indices S_x are derived. S_x denotes a slope value for which x% of the pixels in a basin are equal to or less than this value. Berger and Entekhabi (2001) and Mazvimavi (2003) used the median slope (S_{50}) instead of average slopes, which is also adapted in this study.

Figure 3-7 shows the distribution of slopes across the watershed. It can be seen that GumSelassa watershed is dominated with flat slopes accounting about 88% less than 5% and Laelay Wukro with steep slopes. Watershed topography can be also compared taking different indexes like (S_{25}, S_{50} and S_{75}). The median slopes are 25.7% for Laelay Wukro, 5.6% for Haiba and 3.2% for GumSelassa watersheds. The median slopes determined for the total and sub-watersheds are used to estimate watershed lag.

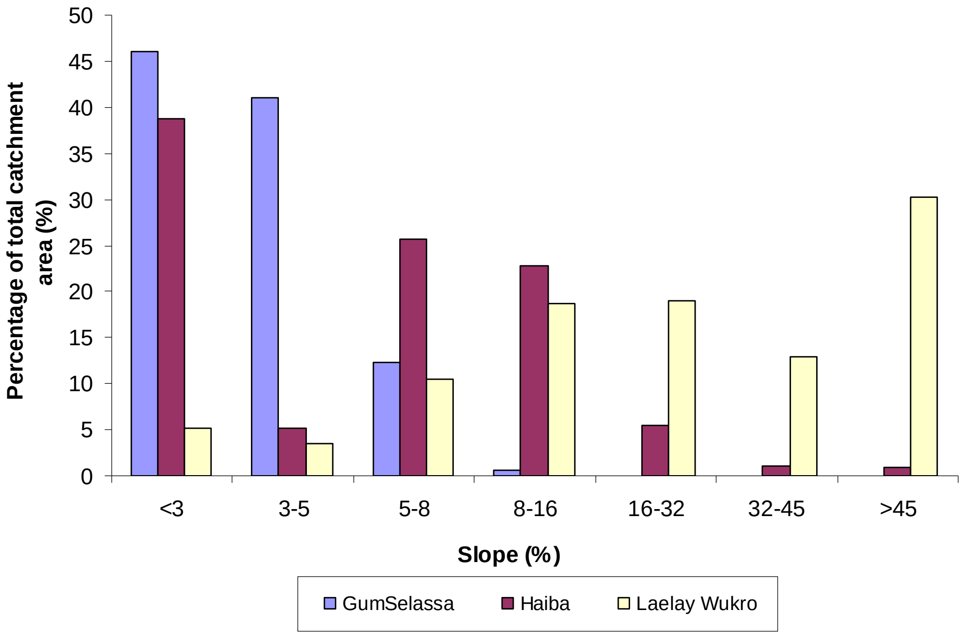

Figure 3-7: Distribution of slopes across the catchment

3.1.2.4 Soils and landuse

The spatial variability in soil properties and land use are important to the hydrological response of a catchment and should be incorporated into the catchment representation (Marėchal and Holman, 2005). Most of the parameters used in hydrological modeling are derived from soil properties and land covers. Often soil and landuse date are acquired from specialized institutions. Unfortunately there were no soil and landuse maps available that can be readily used for this study. Thus it was mandatory to prepare new maps following field work, sampling and laboratory analysis.

Soils

The amount of runoff water moving across the field has a lot to do with soil characteristics such as texture, organic mater content and water holding capacity. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration (Mark and Marek, 2009). An attempt has been made to prepare new soil maps for the three selected watersheds combining deskwork, field sampling, laboratory testing and mapping with appropriate GIS software.

Soil samplings were undertaken that would enable to describe the different types of soils in each watershed. The collected samples were analyzed to identify both, the engineering and physical properties of soils. The United States Department of Agriculture (USDA) soil texture classification was used as a basis for classification. Some of important results achieved after laboratory testing and analysis are the saturated hydraulic conductivity (k_S), field capacity, wilting point, and soil gradation curves. Summary of the results are indicated in Appendix 3.1.2.4 with respect to each watershed including hydraulic conductivity analysis. Details of the laboratory analysis results are reported in Appendix 3.1.2.4b in the accompanying compact disc (CD)

Beven (2004) noted that a soil map by itself is not an end result; it should be transferred into usable classification based on some interpretation mechanism like a pedotransfer-function as suggested by Rawls and Braken-Seik (1989). The pedotransfer-function basically uses a regression analysis to correlate the soil texture with hydraulic conductivity. The U.S. Department of Agriculture and Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS) divides soils into four hydrologic soil groups, based on infiltration rates (groups A-D). The classification is summarized in Table 3-3.

Table 3-3: Summary table for hydrologic soil classification proposed by NRCS

Hydrologic soil group	Minimum Infiltration range (mm/h)	Soil types	Remark
group A	7.6 – 11.4	Deep sands, deep loess, and aggregated silts	Low runoff potential due to high infiltration
group B	3.8 – 7.6	Shallow loess, sandy loam	Moderately low runoff due to moderate infiltration rate when saturated
group C	1.3 – 3.8	clay loam, shallow sandy loams, soils with low organic content	Moderately high runoff potential due to slow infiltration rates, soils with moderately fine to fine texture
group D	Less than 1.3	Soils with clay pan or clay layer at or near the surface, Heavy plastic soils	Soils have high runoff potential sue to slow infiltration rates, shallow soils over nearly impervious parent material

The minimum infiltration rate can be described by the permeability or transmissibility (hydraulic conductivity) of soils (Mishra and Singh, 2003). Thus the hydrologic soil group of the soils found in all watersheds is classified based on the hydraulic conductivity estimated at the laboratory and the classification recommendations proposed by NRCS. Accordingly the hydrologic soil groupings of the soils are mapped as shown in Figures 3-8, 3-9 and 3-10.

The soils of the GumSelassa and Haiba watersheds are dominated by clay and fine-textured soils, with saturated hydraulic conductivity (8.9 x 10^{-9}m/s to 1.12 x 10^{-7}m/s), which makes them fit into hydraulic soil group D. However in the Laelay Wukro watershed silty clay loam and silt loam are found dominantly, with saturated hydraulic conductivity ranging from 1.18 x 10^{-7}m/s to 8.3 x 10^{-7}m/s, resulting in soil types C and D (Figure 3-10).

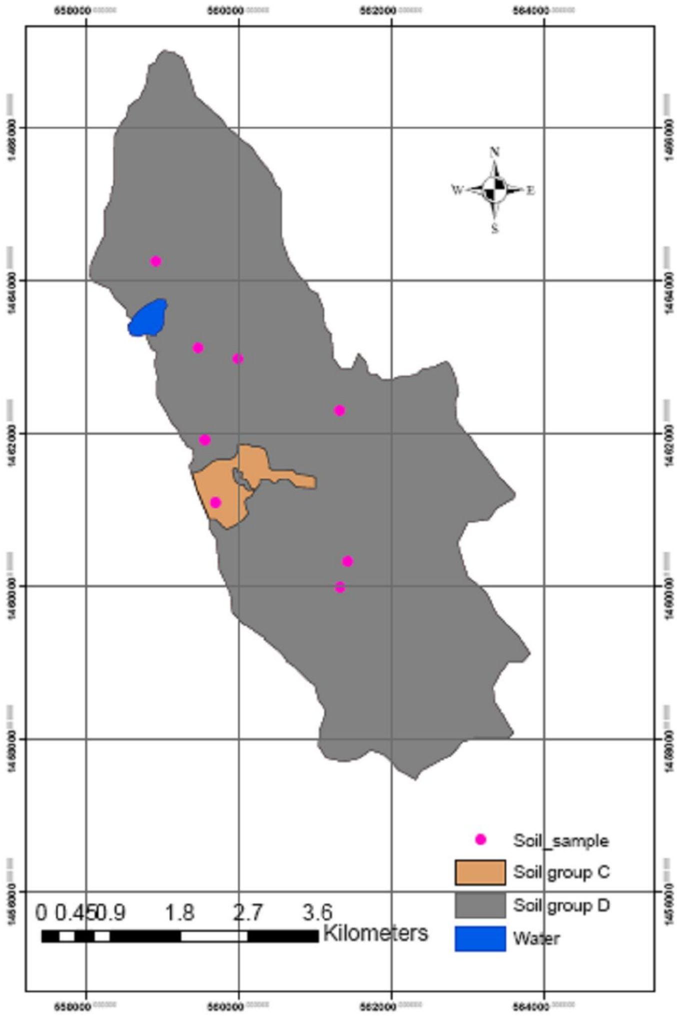

Figure 3-8: Hydrologic soil group classification for GumSelassa watershed

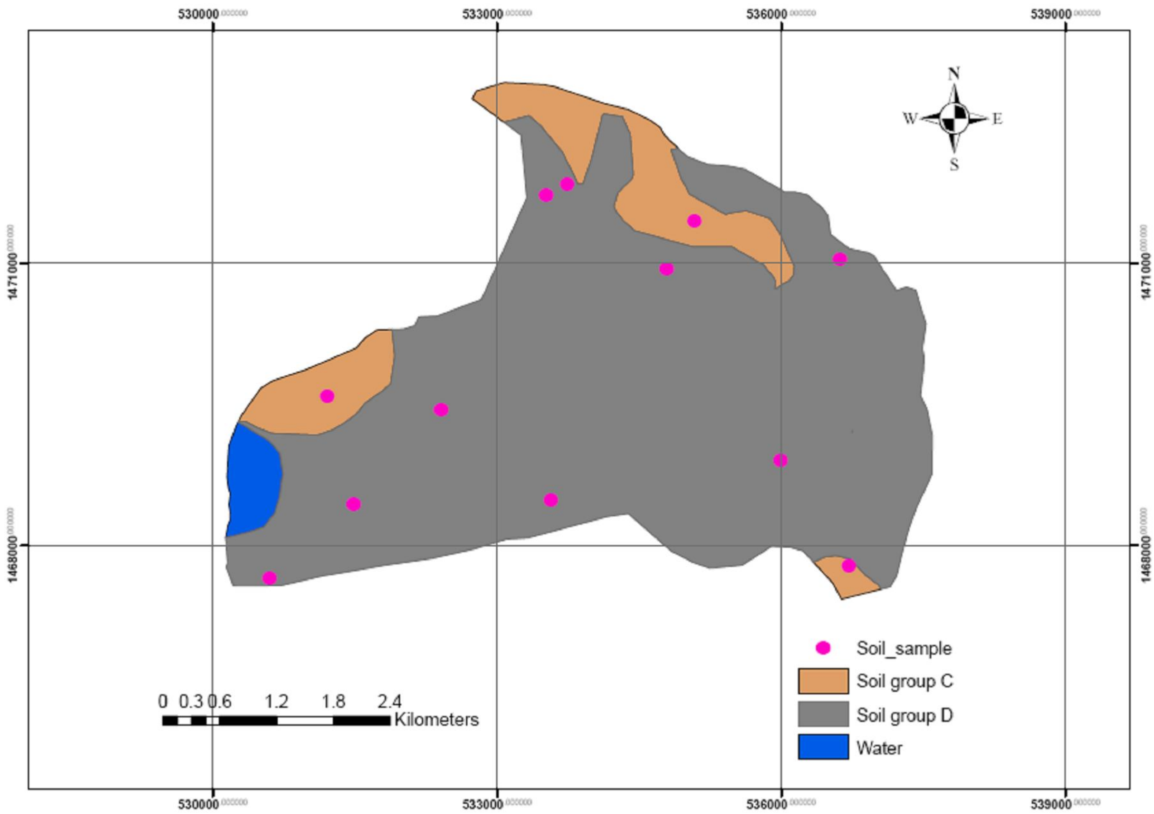

Figure 3:9: Hydrologic soil group classification for Haiba watershed

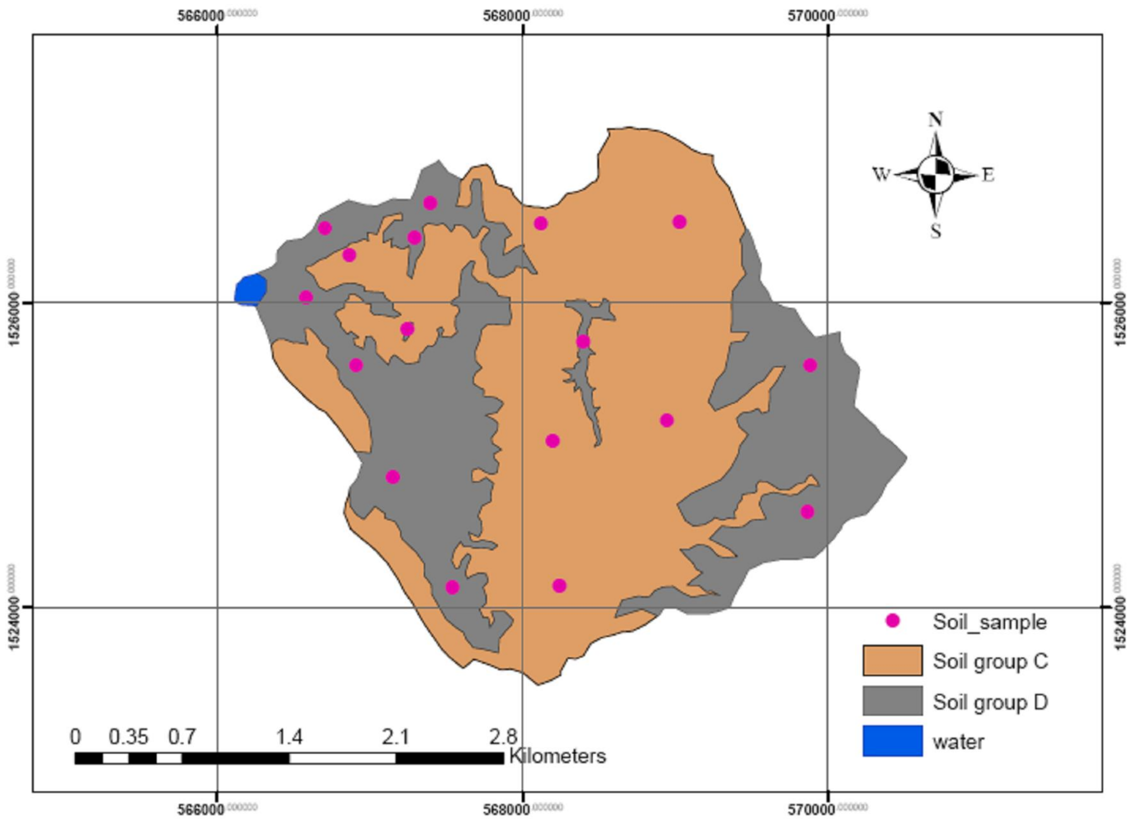

Figure 3-10: Hydrologic soil group classification for Laelay Wukro watershed

Land use / Land cover

Land use has been shown in several studies to affect runoff (Kosmas et al., 1997; Rietz et. al., 2000; Dunjó et al., 2004; Wang J. et al., 2005; Hurni et al., 2005, 2008). In this study the land uses found in the watersheds were mapped and used in the runoff estimation. There is no clearly demarcated landuse classification system adapted in Tigray region, but, according to the watershed studies of the micro-dam irrigation projects in the region, the major land use units used for classification are

- cultivated land,
- forest/bush land,
- grazing land,
- homesteads, and
- miscellaneous.

In these projects cultivated land takes the lion share (Table 3-4) and the cultivation practice is dominantly small grain with minor soil and water conservation practices at farm level.

Table 3-4: Landuse/Land covers distribution for some watershed of the irrigation projects in Tigray region

Watershed	Land use/Land covers distribution in a watershed (%)					
	Cultivated land	Grazing land	Bush land	Forest	Homesteads	Miscellaneous
Meila	86.0	13.0	0.0	0.0	1.0	0.0
Betqua	45.2	7.1	39.4	0.0	5.3	3.0
Gereb Mehiz	54.4	4.0	29.0	0.0	3.6	9.0
Teghane	45.4	22.4	0.0	27.7	0.0	4.5
Mainugus	51.3	18.9	0.0	16.4	11.0	2.4
GumSelassa*	92.0	0.0	2.0	0.4	0.6	5.0
Haiba*	68.9	11.4	0.0	1.9	15.5	2.3
Laelay Wukro*	40.4	1.8	38.9	18.1**	0.7	0.1

*Source= CoSAERT respective watershed design report, *= data from this study, **= Forest for Laelay Wukro watershed mean enclosure and grass lands.*

The landuse is defined based on aerial photography interpretation, complementary field verification with Geographical Positing System (GPS) and mapping using appropriate GIS software. The different landuse/ land cover found in each watershed are reported from Figures 3-11 through 3-13.

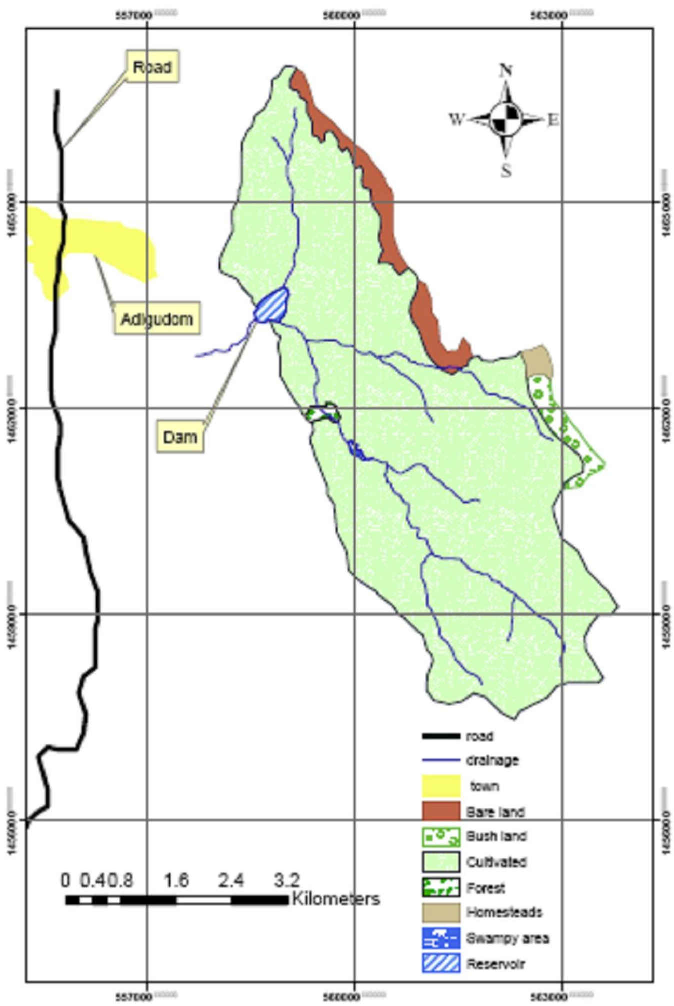

Figure 3-11: Landuse map for GumSelassa watershed

The watershed area of GumSelassa is dominated by cultivated land, which constitutes about 92%, and for Haiba 68.9% of the land is cultivated. In contrast the Laelay Wukro watershed is composed of cultivated land (40.4%); bush land (38.9%) and grass and exclosure land together (18.1%).

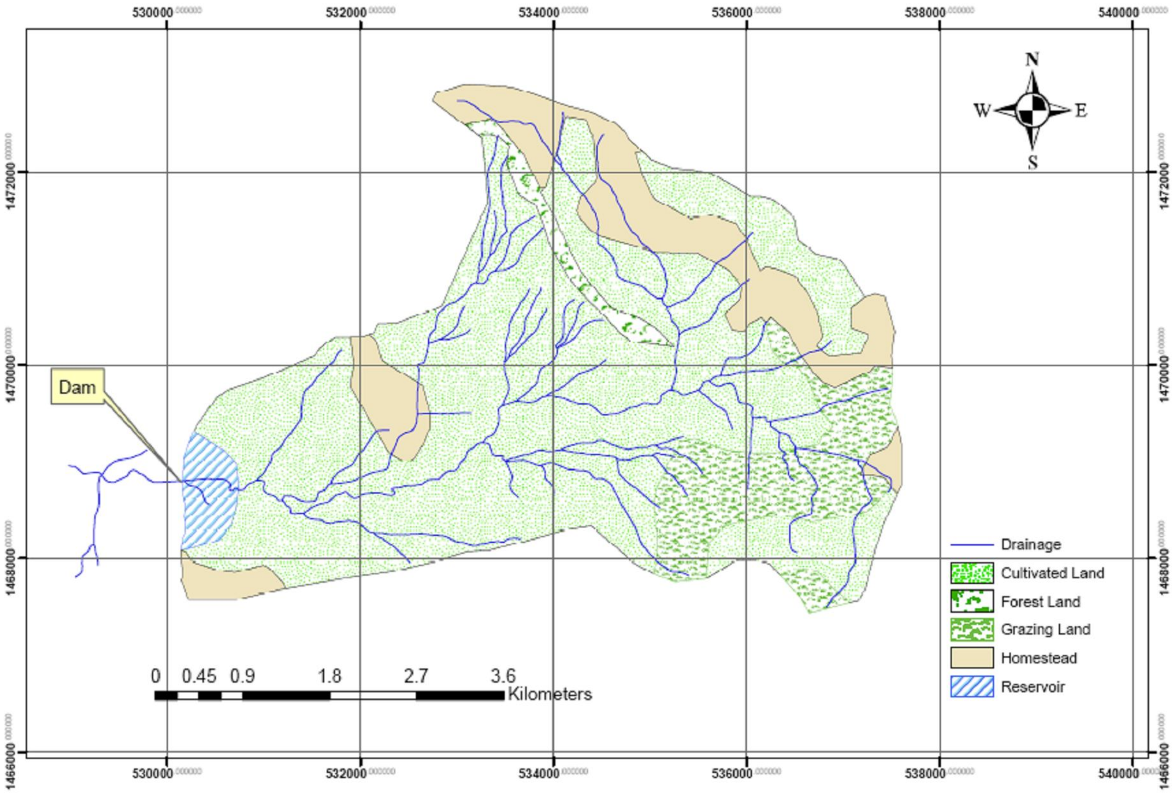

Figure 3-12: Landuse map for Haiba watershed

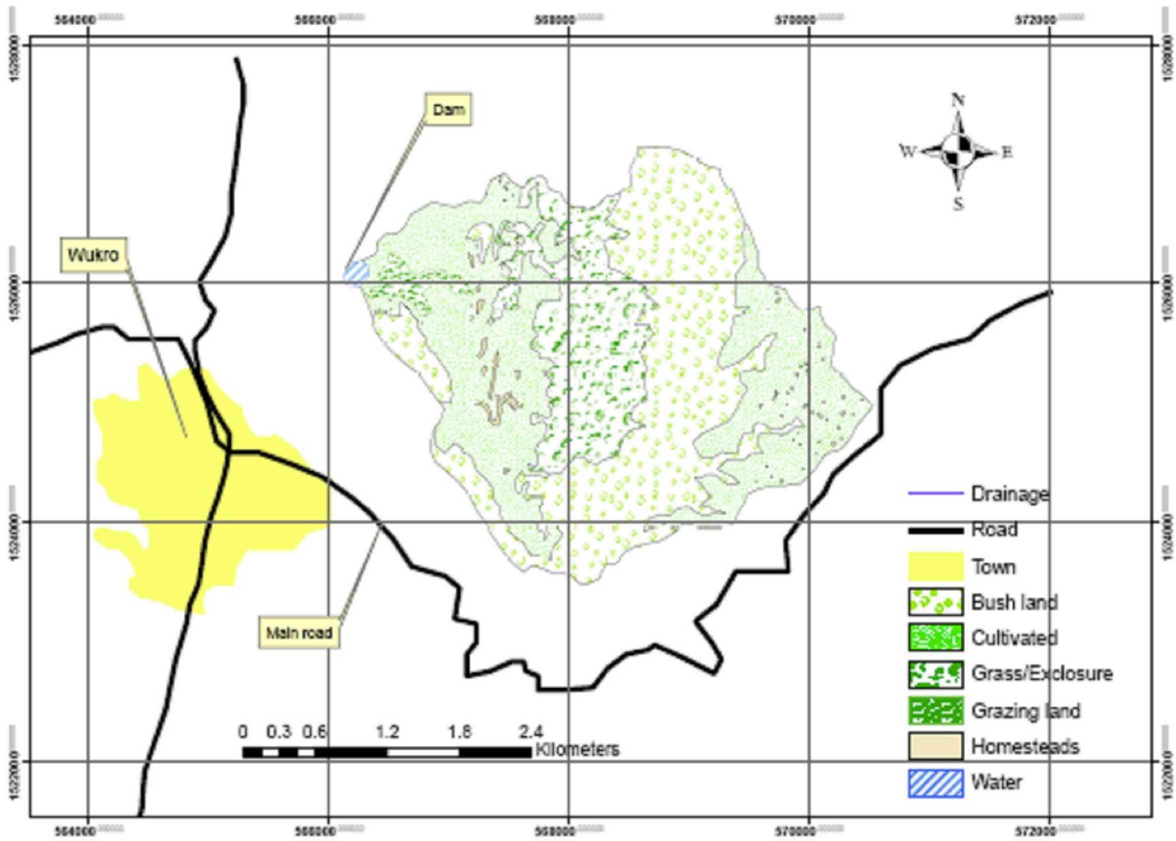

Figure 3:13: Landuse map for Laelay Wukro watershed

3.1.3 Rainfall and other meteorological data

3.1.3.1 Rainfall

Rainfall is the driving force for runoff generation. The accuracy and distribution of the rain gauge network can have a significant effect on the outcome of the simulation. In 2007, when the first field campaign was undertaken, it was wet season with recorded annual rainfall 538.2 mm/a, 700.7 mm/a and 702.8 mm/a in GumSelassa, Haiba and Laelay Wukro watersheds respectively. In contrast in 2008, it was generally a dry year with annual rainfalls 244.8 mm/a and 440 mm/a in GumSelassa and Laelay Wukro. In most cases more than 85% of the annual rainfall occurs during the months of June, July, August and September.

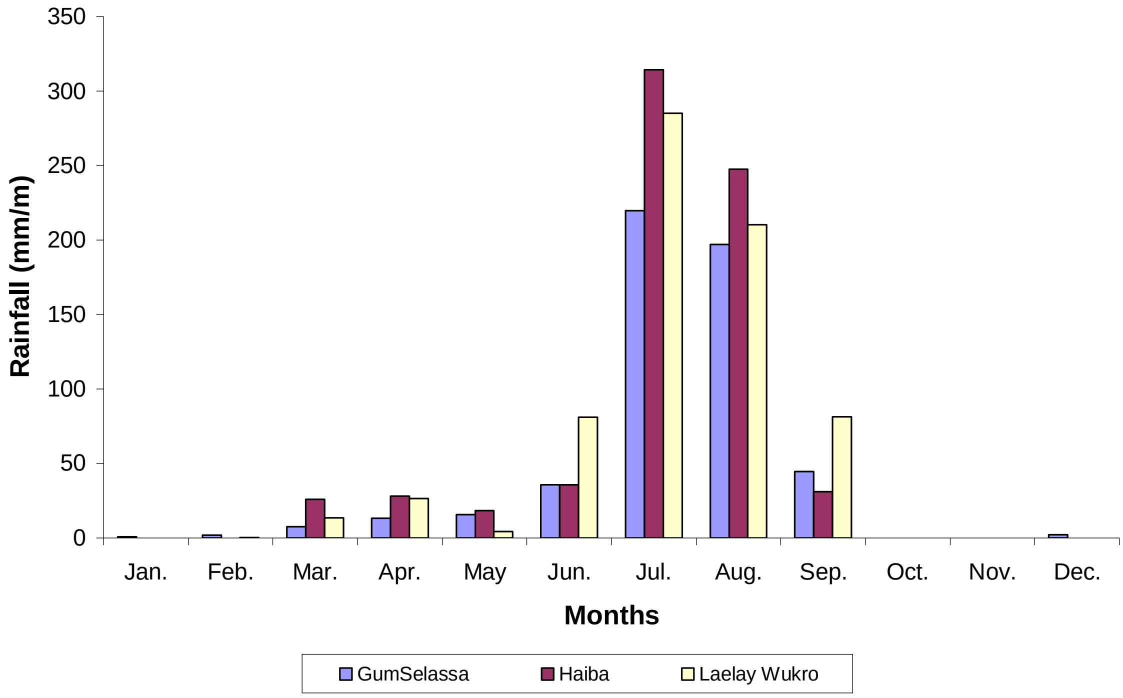

Figure 3-14: Monthly rainfall recorded at each watershed in 2007

Rainfall recording

The rainfall records used in this study are measurements made with intervals of 5 min duration using automatic tipping bucket rain gauges. The equipments were partly installed by this study in 2007 and were partly already existing facilities installed by CoSAERT/TBWREM in the project areas. In 2007 there was one rain gauge per watershed for GumSelassa and for Laelay Wukro, whereas in Haiba there was one automatic tipping bucket at the outlet and one manual recorder nearly at the centroid of the watershed. In 2008, additional rain gauges were installed at GumSelassa and Laelay Wukro watersheds.

Figure 3-15: Automatic weather station

Mean areal rainfall depth

The required watershed precipitation depth can be inferred from the depths at the gauges using an average scheme (HEC, 2000).

$$h_{PMAP} = \frac{\sum_i \left\{ w_i \times \sum_i h_{Pi}(t) \right\}}{\sum_i w_i} \qquad (3.1)$$

Where

h_{PMAP}	= total storm mean areal precipitation over the watershed (mm)
$h_{Pi}(t)$	= precipitation measured at time *t* at gauge *i* (mm)
w_i	= weighting factor assigned to gauge *i* (1)

For watersheds having only one rain gauge, the weighting factor (w_i) is taken as 1 and (w_i) is computed by the Thiessen polygon method for more than one gauge in the watershed. The Thiessen polygon works based on the assumption that the depth at any point within a watershed is the same as the precipitation depth at the nearest gauge in or near the watershed.

The mean areal rainfall varies according to storm characteristics which depend on watershed characteristics such as watershed size, shape and geographical location Asquith and Famiglietti (2000). The watershed topography of both, GumSelassa and Haiba, is dominantly flat terrain and thus rainfall variability across the watershed is believed to be not significant. Laelay Wukro, which has significant elevation variation, is considered as small watershed since its area is less than 10 km^2. An additional rain gauge placed in Laelay Wukro in 2008 showed that the two gauges recorded almost similar depths for events July 25 and July 26. Nevertheless in order to avoid the uncertainty propagation on runoff estimation resulting from erroneous rainfall estimation, high percentage of variation up to $\pm$20% is considered during uncertainty analysis (Chapter 3.5).

Temporal distribution of rainfall

In order to evaluate the flow hydrograph it is necessary to provide the variation of the mean areal rainfall with time.

$$h_{pMAP}(t) = \left\{ \frac{h_{Ppattern}(t)}{\sum h_{Ppattern}(t)} \right\} \times h_{PMAP} \tag{3.2}$$

Where

$h_{pMAP}(t)$ = watershed areal rainfall at time *t*(mm) and the $h_{Ppattern}(t)$ is given by:

$$h_{Ppattern}(t) = \frac{\left\{ \sum w_i(t) \times h_{Pi}(t) \right\}}{\sum w_i(t)} \tag{3.3}$$

For a single recording gauge, the resulting hyetograph of the watershed will be directly the measurement of the recording gauge and thus for the Haiba watershed, having both, recording and non recording gauges, the time distribution of the recording gauge will represent the hyetograph of the total watershed. Summary of daily rainfall used for analysis are reported in Appendix 3.1.3.1.

3.1.3.2 Other meteorological data

Temperature

The temperature data available at or nearby the area of interest is directly used for evapotranspiration analysis. But where data lacks, a data transferring technique has been used using the altitude relationship to generate the temperature data. The temperature data used for further analysis are reported in Table 3-5.

Table 3-5: Mean monthly temperature (^{o}C) for GumSelassa, Haiba and Laelay Wukro watersheds

Watershed	Year	Months											
		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
GumSelassa	2007	16.8	18.8	19.5	20.0	21.6	21.2	18.6	18.5	18.2	17.3	16.7	15.9
	2008	17.8	17.2	19.2	20.1	21.4	21.0	19.3	19.3	18.6	18.1	16.9	16.3
Haiba	2001	15.5	17.2	18.2	20.0	21.2	19.3	17.8	17.5	18.3	18.1	16.1	16.4
	2007	16.0	18.0	18.7	19.2	20.8	20.4	17.7	17.7	17.4	16.5	15.9	15.1
Laelay Wukro	2001	17.2	18.1	20.1	21.6	20.9	20.9	19.3	19.0	19.1	18.9	17.5	16.6
	2007	18.3	20.5	21.3	22.5	24.5	23.1	21.0	19.0	18.8	18.7	17.7	16.5
	2008	18.0	17.6	19.4	21.3	22.1	21.0	19.1	18.4	18.7	19.2	18.2	17.6

The Laelay Wukro monitoring station has a complete weather station for recording years 2007 and 2008, which can measure rainfall at 5 min interval, a measurement facility for other parameters like temperature, sunshine hours, wind speed and relative humidity. However the remaining two monitoring stations are equipped only with rain gauges and no other measuring facilities. Thus for the estimation of evapotranspiration an alternative methods have been adapted as explained in the next subtopic.

Evapotranspiration

Evapotranspiration is an important component of the water budget. As many studies indicate that the Penman-Monteith equation (FAO-56 PM, equation 3.4) consistently gives reasonably accurate estimates at various ecological zones of the world (Allen et al., 1998). Availability and quality of data sets are the biggest challenges that usually hinder the use of FAO-56 PM equation during the planning and operation of a project especially in the developing world. In this study the applicability of different empirical equations was tested and compared with the estimates of FAO-56 PM equation for three weather stations found in Tigray region, (namely Quiha Airport, Sinkata and

Adigirat) having relatively good sets of data from 2002-2008. Refer to Appendix 3.1.3.2a for location map and Appendix 3.1.3.2b for data sets obtained from the three weather stations.

The most common methods for estimation of evapotranspiration are introduced here.

i. FAO-56 PM equation (Allen et al., 1998)

$$\dot{h}_{ETpPen} = \frac{0.408 \times \Delta \times (R_n - G) + \gamma \times \left(\frac{900}{\vartheta_{mean} + 273}\right) \times v_2 \times (e_S - e_a)}{\Delta + \gamma \times (1 + 0.34 \times v_2)} \qquad (3.4)$$

Where

$$\frac{1\text{mm}}{2.45\text{MJ/m}^2} = \frac{0.408\text{mm} \times \text{m}^2}{\text{MJ}}$$

$\dot{h}_{ETpPen}$	=FAO-56 PM reference evapotranspiration rate (mm/d)
R_n	=net radiation at the crop surface (MJ/(m^{2}day))
G	= soil heat flux density (MJ/(m^{2}day))
ϑ_{mean}	= mean air temperature (^{o}C)
v_2	= wind speed at 2 m height (m/s)
e_S	=saturation vapour pressure (kPa)
e_a	=actual vapour pressure (kPa)
$(e_S - e_a)$	= saturation vapour pressure deficit (kPa)
Δ	=slope vapour pressure curve (kPa/^{o}C)
γ	=psychrometric constant (kPa/ ^{o}C)

The saturation vapour pressure ((e_S) is computed as

$$e_S = \frac{e(\vartheta_{\max}) + e(\vartheta_{\min})}{2} \qquad (3.5)$$

Where

e	= the saturation vapour function at maximum and minimum temperature (kPa)
$\vartheta_{\max}$, $\vartheta_{\min}$	= mean maximum and mean minimum air temperature (^{o}C)

ii. The Hargreaves and Samani equation

The Hargreaves and Samani (1982) empirical equation is:

$$\dot{h}_{ETpHar} = 0.0023 \times (\vartheta_{mean} + 17.8) \times (\vartheta_{\max} - \vartheta_{\min})^{0.5} \times R_a \qquad (3.6)$$

Where

$\dot{h}_{ETpHar}$ = Hargreaves - Samani reference evapotranspiration rate (mm/d)

R_a = Extraterrestrial radiation (mm/d)

iii. Solar radiation (R_S) based method (Irmak et al., 2003)

$$\dot{h}_{ETpRs} = -0.611 + 0.149 \times R_s + 0.079 \times \vartheta_{mean} \qquad (3.7)$$

Where

$\dot{h}_{ETpRs}$ = solar radiation based potential evapotranspiration rate (mm/d)

R_S = incoming solar radiation (MJ/(m^{2}day))

R_S can be related with the daily sunshine hours and extraterrestrial radiation Ra:

$$R_s = \left(0.25 + 0.5 \times \frac{t_{sact}}{t_{s\max}}\right) \times R_a \qquad (3.8)$$

Where

t_{sact} = actual duration of sunshine per day (h/d)

iv. Net radiation (R_n) based method (Irmak et al., 2003)

$$\dot{h}_{ETpRn} = 0.489 + 0.289 \times R_n + 0.023 \times \vartheta_{mean} \qquad (3.9)$$

Where

$\dot{h}_{ETpRn}$ = net radiation based potential evapotranspiration rate (mm/d)

R_n = net radiation (MJ/(m^{2}day))

The annual evapotranspiration rate estimated by different methods vary compared to FAO-56 PM estimation.

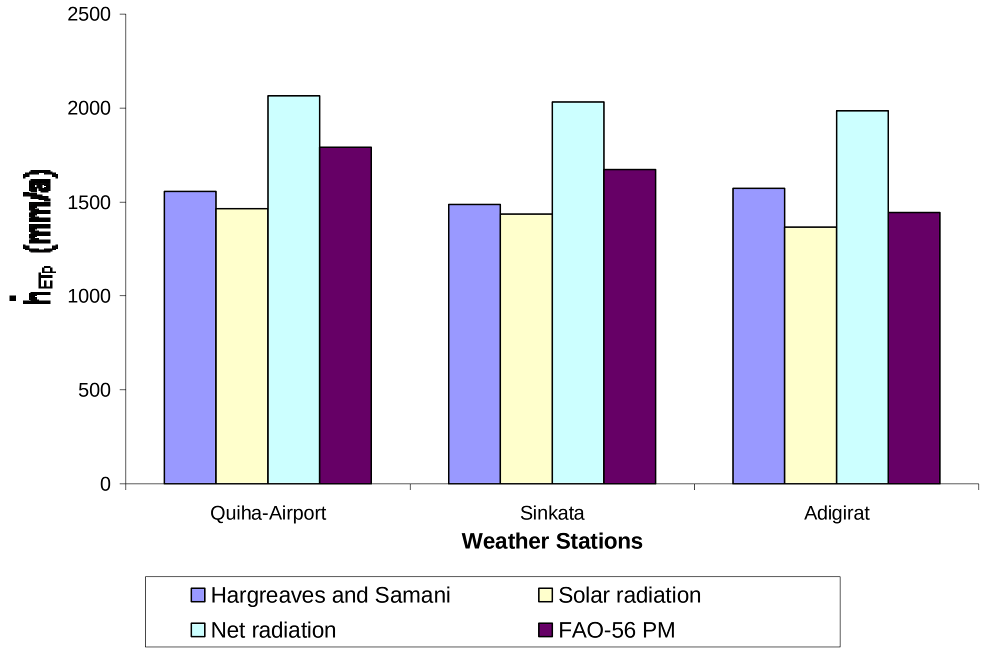

Figure 3-16 Average annual evapotranspiration rates for the three weather stations (2002-2008)

In all stations the net radiation result overestimated the potential evapotranspiration. Curve fitting made with each method results against FAO-56 PM reveals that the solar radiation method gives the best result with correlation coefficient (R^2=0.812). Therefore, in the absence of data for estimation of evapotranspiration by the FAO-56 PM method, the fitted equation (equation 3.10) can be used to estimate potential evapotranspiration in Tigray region. The result of this analysis is inline with the suggestion made by Irmak et al. (2003) as the use of empirical equations for estimating potential evapotranspiration with lesser data inputs but acceptable outputs.

$$\dot{h}_{ETpadj} = 0.4867 \times (\dot{h}_{ETpRs})^{1.6261} \qquad (3.10)$$

Where

$\dot{h}_{ETpadj}$ = adjusted potential evapotranspiration rate (mm/d)

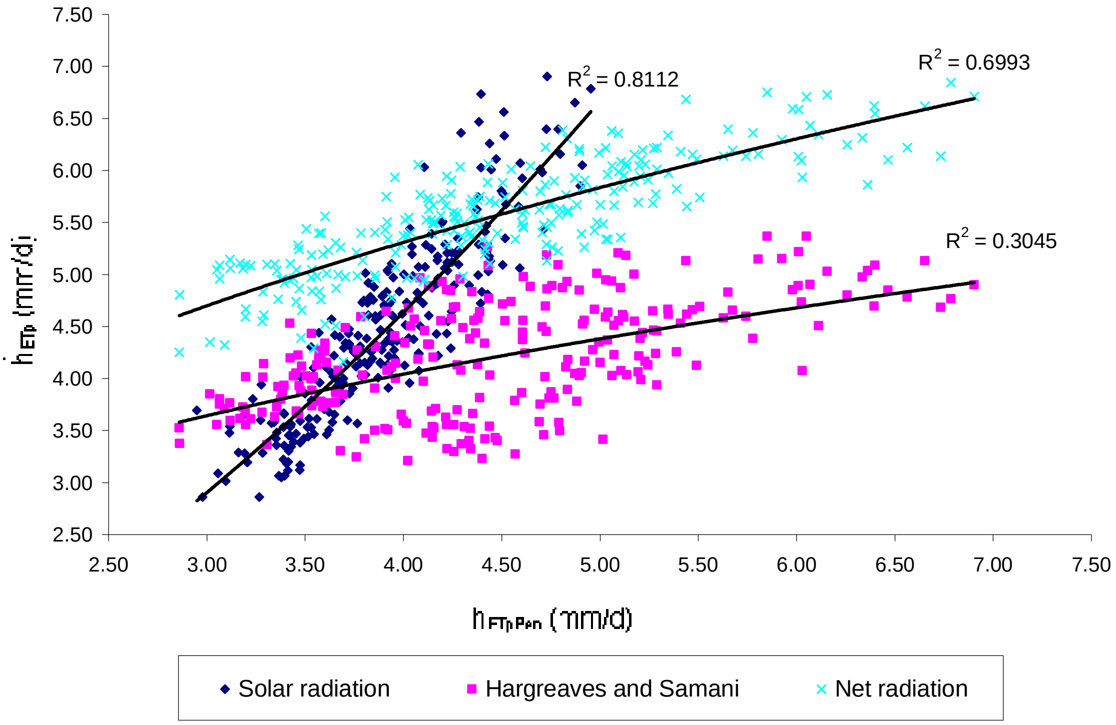

Figure 3-17: Scatter plot for comparison of potential evapotranspiration rates by different methods against FAO-56 PM method

FAO-56 PM equation is used to estimate the potential evapotranspiration for Laelay Wukro watershed because the weather station has sufficient data and the fitted equation (equation 3.10) for GumSelassa and Haiba watersheds. The results of the potential evapotranspiration are shown in Table 3-6.

Table 3-6: Estimated monthly mean potential evapotranspiration (mm/d)

Watershed	Year	Months											
		Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
GumSelassa	2007	4.2	4.9	5.6	6.0	6.0	4.8	3.8	3.9	4.5	5.3	4.6	4.3
	2008	4.5	5.2	6.3	6.2	5.8	4.4	3.5	3.7	4.6	5.2	4.5	4.1
Haiba	2001	4.0	4.8	4.0	5.8	6.1	4.5	3.5	3.6	4.9	4.8	4.4	4.1
	2007	4.0	4.8	5.5	5.8	5.8	4.6	3.6	3.7	4.4	5.1	4.4	4.2
Laelay Wukro	2001	4.3	4.7	5.8	5.9	5.9	4.4	3.6	3.9	4.6	5.1	4.5	4.4
	2007	4.5	5.5	6.8	7.8	6.1	4.7	3.5	3.6	4.2	5.5	5.3	4.3
	2008	4.4	5.5	7.1	7.1	5.8	4.9	3.6	3.5				

3.1.4 Runoff

The observed runoff hydrograph that will be used in this study are generated from the water balance analysis made for each reservoir. The reservoir volume and depth relationship coupled with the water balance equations are used to generate the reservoir inflow.

To avoid deviation of reservoir capacity with time due to sediment accumulation in the reservoirs, new topographic surveys have been carried out for the selected reservoirs and the newly developed reservoir water level and storage relationship are used for further analysis. As an example the topographic map and area capacity curve of Laelay Wukro dam are shown in Figure 3-18 and Figure 3-19, respectively.

The reservoir capacity is calculated using the formula:

$$V = \frac{h}{3} \times (A_{res1} + A_{res2} + (A_{res1} \times A_{res2})^{0.5}) \qquad (3.11)$$

Where

$V_{1,2}$ = reservoir volume between two successive elevations (m^3)

h = elevation difference between successive contours (m)

A_{res1} and A_{res2} = area of reservoir water spread at elevation h_1 and h_2 (m^2)

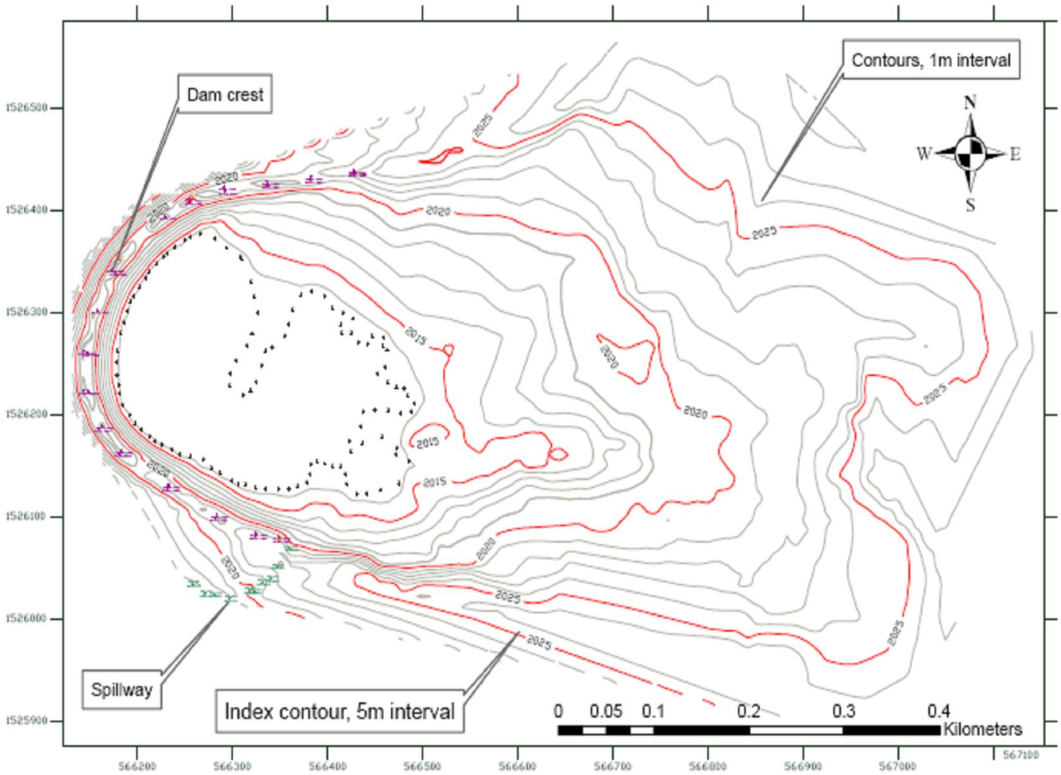

Figure 3-18 Topographic map of Laelay Wukro

The water level was recorded with a pressure transducer inserted into the reservoir and connected to a data logger. The measurement interval is 15 min and 30 min. The atmospheric pressure was compensated with baro divers, installed at each reservoir. Slight variations in level observed during measurements were also corrected first by checking the outliers and then by moving average considering variable durations within a day. The outliers adjustment was done only to the levels where there was no inflow towards the reservoir

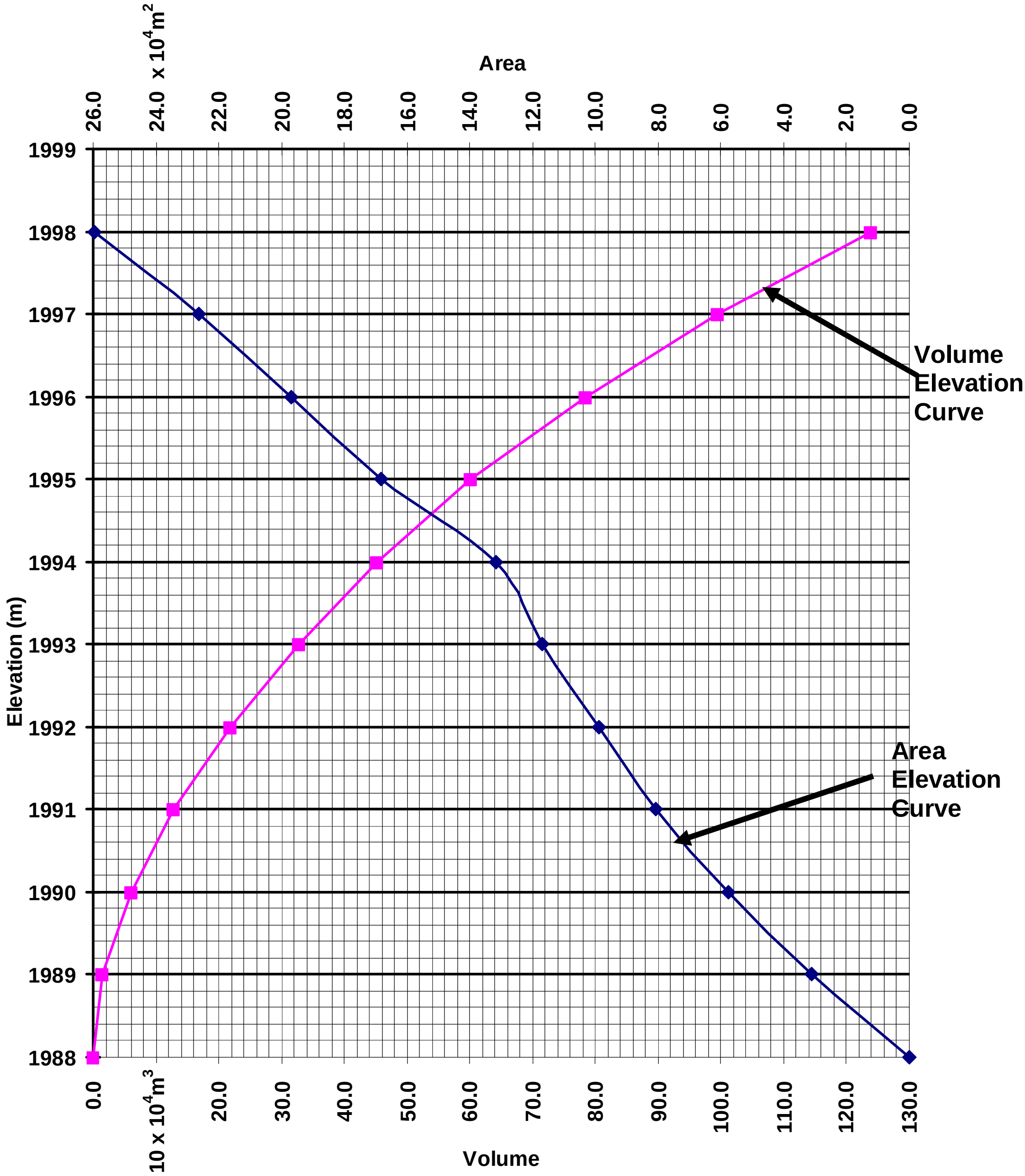

Figure 3-19: Area and volume corresponding to reservoir elevation for Laelay Wukro reservoir

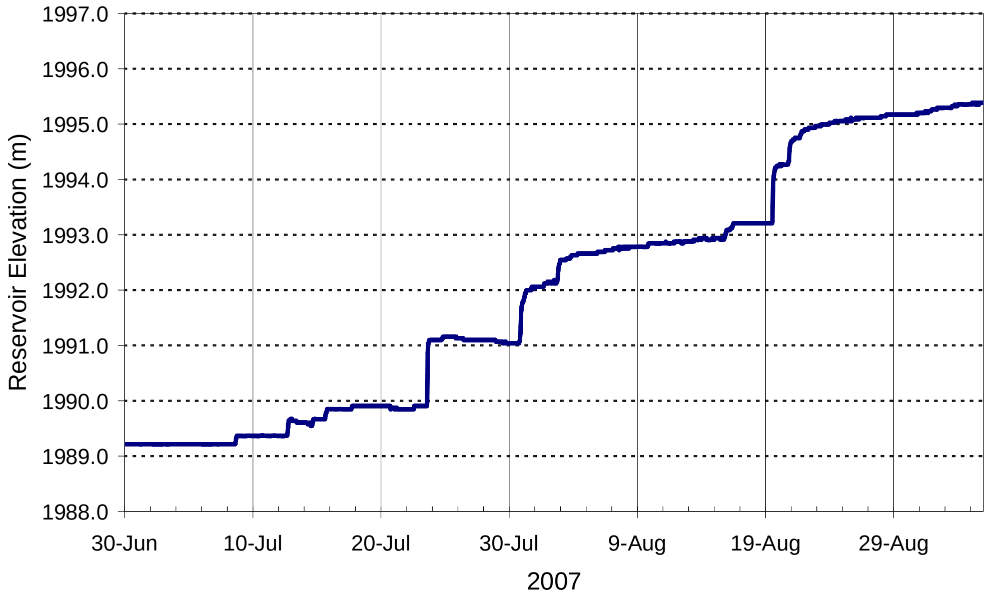

Figure 3-20: Laelay Wukro reservoir water level record (2007)

The adapted reservoir water balance equation is explained hereunder. The basic conservation-of-volume equation for a reservoir or a river reach for a time interval Δt is expressed as:

$$Vs_{t+\Delta t} - Vs_t = \sum_i V_I - \sum_i V_O \tag{3.12}$$

Where

Vs_t = storage volume at the start of interval (m^3)

$Vs_{t+\Delta t}$ = storage volume at the end of interval (m^3)

V_I = inflow volume between successive simulation interval (m^3)

V_O = outflow volume between successive simulation interval (m^3)

The outflows volume includes the evaporation losses, seepage losses, spillway out flow and outlet releases if any. The inflow volume is the combined volume of runoff from the watershed and precipitation fall on the reservoir.

Evaporation loss

The rate of evaporation is influenced by solar radiation, air temperature, vapour pressure, wind and minimally by atmospheric pressure (Nagy et al., 2002). There are very few evaporation pans in the region, neither of them within nor nearby to the monitoring stations. Thus their applicability for open reservoir water evaporation estimation is limited. Thus potential evapotranspiration will be used as alternative estimation.

Mazvimavi (2003) used a factor 1.25 to convert potential evapotranspiration to pan evaporation equivalent in South Africa and Zimbabwe. Penman concluded that potential evapotranspiration is only 0.75 of the free water evaporation. In this study reservoir evaporation is determined by multiplying the potential evapotranspiration depth with a factor 1.33.

$$V_{Evap} = \frac{1.33 \times h_{ETp} \times A_{res}}{1000} \tag{3.13}$$

Where

V_{Evap} = evaporations loss (m^3)

h_{ETp} = potential evapotranspiration for given duration (mm)

A_{res} = reservoir area (m^2)

Seepage loss

Expanding equation 3.12 and working for a time interval Δt gives:

$$Vs_{t+\Delta t} - Vs_t = \sum_{i=t}^{t+\Delta t}(V + V_P) - \sum_{i=t}^{t+\Delta t}(V_{Evap} + V_{Seapage} + Vsp_{outflow} + V_{irr}) \tag{3.14}$$

Where

V = runoff from watershed (m^3)

V_P = precipitation fall on the reservoir surface (m^3)

V_{Evap} = evaporation loss (m^3)

$V_{Seapage}$ = water loss from reservoir due to seepage (m^3)

$Vsp_{outflow}$ = spillway outflow (m^3)

V_{irr} = water released for irrigation (m^3)

$t, t + \Delta t$ = start and end time of the simulation

To minimize the input parameters in equation 3.14, the seepage volume is calculated where there is no runoff from the catchment and no spillway outflow. Besides, during the rainy season there is no irrigation, thus water released for irrigation is zero.
Rewriting the equation for seepage loss gives:

$$V_{Seapage} = \left[\left(\sum_{i=t}^{t+\Delta t} (V + V_P) \right) + (Vs_t) \right] - \left[\left(\sum_{i=t}^{t+\Delta t} (V_{Evap}) \right) + (Vs_{t+\Delta t}) \right] \quad (3.15)$$

Many of the rainfall-runoff events last for few hours. As a result it might not affect significantly the overall inflow volume for a single event. Reservoir water records of Laelay Wukro indicate there is no appreciable variation in water level records for successive days during no rainfall events. Field observation along the downstream reach of the dam for considerable length proved that there is no seepage downstream of the dam, unlike other dams. Thus for all event simulation, seepage loss is considered to be zero for Laelay Wukro reservoir. Meanwhile analysis of the seepage loss with equation 3.15 for GumSelassa during maximum reservoir level revealed that it can be as high as 39.5 l/s and for Haiba reservoirs during maximum reservoir the seepage loss is found to be about 66.6 l/s.

Spillway outflow

The general equation for uncontrolled spillway outflow is expressed with the formula

$$Vsp_{outflow} = (C_d \times l \times h^{3/2}) \times \Delta t \quad (3.16)$$

Where

$Vsp_{outflow}$ = spillway outflow (m^3)

C_d = discharge coefficient ($\sqrt{m}$ /s)

l = spillway crest length (m)

h = head over spillway (m)

Δt = time interval between simulations (s)

Depending on the shape of the crest, the spillway outflow volume was calculated and incorporated in the water balance analysis at times where there is spillway outflow. The coefficients of discharges reported in respective engineering design reports are (2.2), (1.7) and (1.7) for GumSelassa, Haiba and Laelay Wukro respectively.

Rearranging equation 3.14

$$Vs_{t+\Delta t} - Vs_t = \sum_{i=t}^{t+\Delta t}(V + V_P) - \sum_{i=t}^{t+\Delta t}(V_{Evap} + V_{Seapage} + Vsp_{outflow} + V_{irr}) \tag{3.17}$$

$$(Vs_{t+\Delta t} - Vs_t) + \sum_{i=t}^{t+\Delta t}(V_{Evap} + V_{Seapage} + Vsp_{outflow} + V_{irr}) = \sum_{i=t}^{t+\Delta t} V_I \tag{3.18}$$

Accordingly the discharge during the time interval Δt will be

$$\frac{(Vs_{t+\Delta t} - Vs_t)}{\Delta t} + \frac{\sum_{i=t}^{t+\Delta t}(V_{Evap} + V_{seapage} + Vsp_{outflow} + V_{irr})}{\Delta t} = \frac{\sum_{t}^{t+\Delta t} V_I}{\Delta t} \tag{3.19}$$

The output of the aforementioned analysis is considered as observed discharge ($\dot{V}_o$). The reservoir water balance model developed for each dam reservoir is reported in Appendix 3.1.4 in the accompanying CD.

3.2 Calibration and optimization

Air, soil, and water constitute the environment continum, these components are interactive and interactions amongst them are complex and thus should be accomplished within a spatial unit called watershed through modeling (Singh,1995). The watershed models can be divided according to different criteria (Nemec, 1993; Singh, 1995; Wagener et al., 2007). Hydrologic models are mathematical representations of watersheds which describe the natural processes which transpose precipitation into runoff. For detailed classification one can see Singh (1995) but a general trend is seen from lumped conceptual models towards distributed physically-based models (Kite et al. (n.d.); Fortin et al., 2004).

The choice of either a lumped or distributed hydrologic model largely depends on the availability of data, purpose and scope of the study. Several researchers tried to compare the performance of distributed and lumped models. Each of them does have its own advantage and disadvantages. Distributed hydrologic models, which are capable of incorporating a variety of spatially-varying land characteristics and precipitation forcing data, can be powerful in understanding the hydrologic process spatially within a watershed. But uncertainty in the high resolution estimates of precipitation and model parameters may diminish potential gains in prediction accuracy achieved by accounting for the inherent spatial variability (Carpenter and Georgakakos, 2006; Meselhe, 2004). Unlike distributed models the lumped hydrologic models account average parameters over a watershed and compromise the variability of watershed parameters in actual natural system. However, they are relatively simple and the data set requirements for calibration and modeling makes them preferable in areas where data availability is the main constraint.

For this study a lumped hydrologic model is adapted. Comparison is also made by subdividing the total watershed area into sub-watersheds (Chapter 3.6.2.4).The sub-watershed areas were subdivided with due consideration of similar soil and landuse, and drainage. The Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) was used for hydrologic modeling of the three watersheds.

3.2.1 Description of the hydrologic model

The HEC-HMS is designed to simulate the rainfall-runoff of watershed systems (HEC, 2006). The model is developed over 30 years continuous research and improvement. It is available freely in public domain.

The rainfall-runoff simulation process is achieved through four major components:

- the runoff volume component,
- the runoff transform,
- the routing, and
- the base flow components.

In each component there are several models that will allow the user alternative options that suit the watershed conditions. The model can be run as lumped or distributed model.

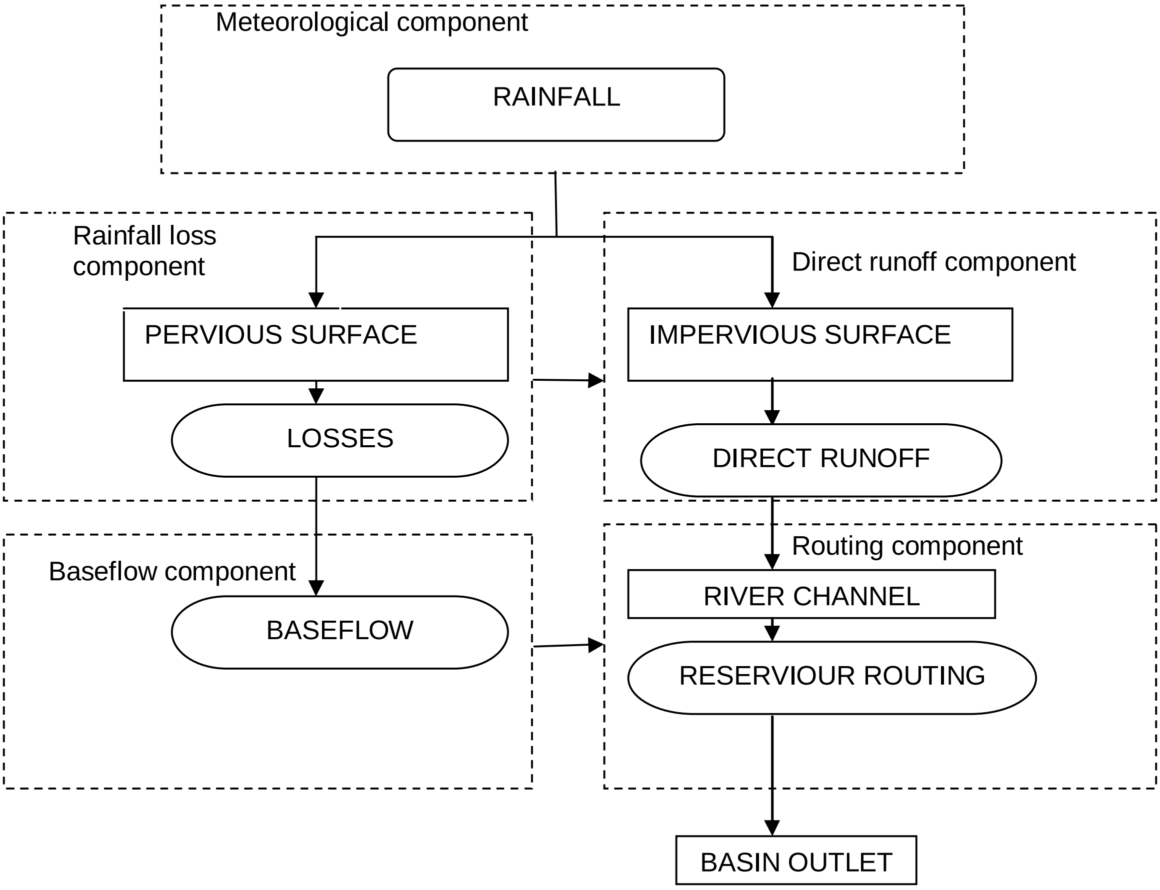

Figure 3-21: Runoff generation process in HEC-HMS

3.2.1.1Model parameters set up

Runoff volume component

The major component in the rainfall-runoff transformation is the runoff volume component. The runoff volume is computed with the Natural Resources Conservation Centre Curve Number (NRCS-CN) and formerly known as the Soil Conservation Service (SCS) model developed by the United States Department of Agriculture (USDA). The method is commonly called SCS, and details of the model are available in SCS (1971). It computes direct runoff through an empirical equation that requires the rainfall and a watershed coefficient inputs (Nayak and Jaiswal, 2003).The general equation for the SCS curve number method is:

$$\frac{h_F}{hs_P} = \frac{h_V}{h_P - h_{Ia}} \qquad (3.20)$$

Where

h_F = cumulative infiltration excluding initial abstraction (mm)

hs_P = potential retention excluding initial abstraction (mm)

h_V = runoff (mm)

h_P = total rainfall (mm)

h_{Ia} = initial abstraction (mm)

From the continuity principle,

$$h_F = (h_P - h_{Ia}) - h_V \tag{3.21}$$

The initial abstraction is expressed in relation to the potential maximum retention (hs_P) by multiplying it with an initial abstraction factor (λ):

$$h_{Ia} = \lambda \times hs_P \tag{3.22}$$

Where

λ = initial abstraction factor (1)

The SCS method defined the value of the initial abstraction factor to be approximately 20% of the watershed storage derived from experimental watersheds.

$$h_{Ia} = 0.2 \times hs_P \tag{3.23}$$

Solving equations 3.20 and 3.21 simultaneously gives

$$h_V = \frac{(h_P - 0.2 \times hs_P)^2}{h_P + 0.8 \times hs_P} \qquad (h_P \geq 0.2 \times hs_P) \tag{3.24}$$

The SCS method also provides the relationship between (hs_P) and the Curve Number (CN) by the formula

$$hs_P = \frac{25400}{CN} - 254 \tag{3.25}$$

The curve number is a hydrologic parameter used to describe the storm water runoff potential for drainage area. It is a dimensionless number which is a function of land use, soil type, and soil moisture. Parameter (hs_P) of the SCS method depends on the soil type, landuse, hydrologic condition, and Antecedent Moisture Condition (AMC), which are manifested in the curve number selection. The initial abstraction accounts for the short losses, such as interception, surface storage, and infiltration. The soil type (hydrologic soil group) and the landuse of the three watersheds have been discussed in chapter 3.1.2.4. The hydrologic condition explains the management practice of the catchment. Three management conditions identified by SCS are poor, fair and good. Poor refers to less management practice and thus high runoff and to the contrary good indicates less runoff resulting from good conservation practices in the watershed.

The AMC is the soil moisture condition of a watershed pre-storm event. The SCS method uses the concept of AMC grouped into three moisture levels:

- AMC I a dry condition,
- AMC II an average condition, and
- AMC III a wet condition

Detailed descriptions about the method are available in the National Engineering Handbook Section 4 (NEH-4).For practical applications NEH-4 provides the derivation of CN based on the amount of antecedent 5-d rainfall which forms an index of moisture before the start of the storm (Mishra and Singh, 2003).

Table 3-7: Antecedent moisture conditions category

AMC	Total 5-day antecedent rainfall mm/5d	
	Dormant season	Growing season
I	< 12.7	< 35.6
II	12.7 – 27.9	35.6 – 53.3
III	> 27.9	> 53.3

The NEH-4 presented CN values based on land use/treatment, hydrologic conditions, and hydrologic soil condition (Appendix 3.2.1.1a). The runoff curve numbers are developed for AMC II and $h_{Ia} = 0.2hs_P$. Either tables or equations are normally used to change from AMC II to AMC I or AMC III. In this study equations (3-26) and (3-27) proposed by Chow (1988) has been used as starting value for calibration. Equations proposed by different researches and output of this study will be further discussed in chapter 3.6.2.

$$CN_I = \frac{4.2 \times CN_{II}}{10 - 0.058 \times CN_{II}} \qquad (3.26)$$

Where

CN_I = curve number for AMC I

CN_{II} = curve number for AMC II

$$CN_{III} = \frac{23 \times CN_{II}}{10 + 0.13 \times CN_{II}} \qquad (3.27)$$

Where

CN_{III} = curve number for AMC III

The direct runoff depth is then calculated for the curve number defined by the above procedures corresponding to the antecedent moisture condition and measured rainfall with the knowledge of the initial abstraction.

Transform

The SCS unit hydrograph is used to transform the excess precipitation into direct runoff. The SCS suggests the unit hydrograph peak and time to peak of the unit hydrograph are related by:

$$q_p = C \times \frac{Ac}{T_p} \qquad (3.28)$$

Where

q_p = unit hydrograph peak ((m^3/s)/cm)

Ac = catchment area (km^2)

C = conversion factor (1), C=2.08

T_p = time to peak (h)

Time to peak is related to the basin lag, defined as the time difference between the center mass of rainfall excess and the peak of the unit hydrograph. The basin lag is usually determined from a number of measured rainfall events or empirical equations. The difference between the centroid of the rainfall hyetograph and the time to peak of the observed discharge is considered as initial value and then later verified by calibration. The use of empirical equations for estimating the basin lag will be discussed in chapter 3.6.2.3.

$$T_P = \frac{\Delta t}{2} + t_{lag} \tag{3.29}$$

Where

Δt = excess precipitation duration (h)

t_{lag} = basin lag (h)

When the basin lag time is specified, peak discharge of the unit hydrograph and the time to peak of the unit hydrograph are calculated with equations 3-28 and 3-29 respectively.

Baseflow

The main rainfall seasons in the region are June, July and August. Except in these months the streams remain dry without any flow. During the dry months the soils are almost dry with high evapotranspiration as discussed in chapter 3.1.3. The stream flow nature in such watersheds is seasonal intermittent flow during the rainy season, with no flow in some days. Hydrographs which have been observed at the three watersheds respond quickly to the onset and fluctuations of rainfall. Moreover the hydrograph peaks are sharp and time to peak is few minutes. Stream flows in arid and semi-arid regions tend to be dominated by rapid responses to intense rainfall (Gallart, 2005). Ward and Trimble (2004) illustrated that where the water table is relatively deep and the watershed is dominated by agricultural landuse with clay soil the proportion of rainfall as overland flow is very high with storm duration in contrary to infiltration which decreases with time. This lead to the idea that the storm flows observed in such watersheds are mainly contributed from direct runoff, perhaps some from interflow but no input from baseflow.

As it can be observed during the field visit there was no flow prior to many of the rainfall-runoff events and equally the runoff stops after a few hours unless another event follows the previous event. The contribution of base flow is considered to be insignificant, only in Laelay Wukro watershed a constant value was considered where baseflow was observed during the month of August in 2007.

Channel Routing

In this study primarily parameter calibration and estimation is done for the average or weighted value that represents the whole watershed. As a result the routing is mainly overland flow or sheet flow, which was already considered as transform. However while dividing the total area into sub-watersheds it is necessary to consider routing as the runoff moves from the upper catchment to downstream catchment through a river reach.

Many methods and models are available for routing the flow length through a watershed. Two methods, the lag method and Muskingum-Cunge have been tested for channel routing. The lag method is used in all three watersheds, where as the Muskingum-Cunge method was only used in Laelay Wukro watershed where the streams do have defined cross section across the watershed. In the lag method routing is modeled with no attenuation. Depending on the length of the reach and bed slope assumed lag time $(t_{lagrouting})$ was given initially which later adjusted by calibration. It gives an acceptable result in cases where flood attenuation is not significant.

The Muskingum-Cunge channel routing technique is a nonlinear coefficient method that accounts for hydrograph diffusion based on physical properties and the inflow. Unlike other routing methods the Muskingum-Cunge parameters are determined from physical conditions of the watershed and thus recommendable for ungauged watersheds. Detailed derivation of the Muskingum–Cunge equations is shown in Appendix 3.2.1.1b.

The data required for running the Muskingum-Cunge method are:

- representative channel cross section,
- reach length,
- Manning's roughness, and
- channel bed slope.

The cross sections and Manning's roughness were determined from field investigation along the river length (Appendix 3.2.1.1c) whereas the reach length and channel bed slopes are generated from digital maps prepared during watershed processing. The only parameter calibrated in this method was the Manning's roughness (N_{man}) which can be approximated following field investigation and literature review.

3.2.2 Calibration

Rainfall-runoff models are a mathematical representation of a physical process happening in a watershed. The outputs of the model simulation are not exactly equal to the measured or observed data. Therefore it is necessary to adjust the parameter inputs to match the measured values. The process of parameter estimation and their adjustment to certain observed values is referred to as calibration (Gupta et al., 1998; Agyei and Hatfield (2006)).The quantitative measure of the match is described by the objective function, which measures the degree of variation between computed and observed hydrographs (Gupta et al., 1999; Cunderlik and Simonovic, 2004).

Selection of the objective function depends on the purpose or objective of the study. Since the major interest in this study is volume assessment at an outlet, primarily Percent Error in Volume (PEV) is used for analysis, and where necessary Sum Squared Residuals (SSR) for refining the shape of the hydrograph.

- Percent Error in Volume (PEV)

$$PEV = 100 \times \left(\frac{V_o - V_m}{V_o} \right) \tag{3.30}$$

Where

V_o = observed volume (m^3)

V_m = modeled volume (m^3)

- Sum of Squared Residuals (SSR)

$$SSR = \sum_{t=1}^{N} (\dot{V}_o(t) - \dot{V}_m(t))^2 \tag{3.31}$$

Where

$\dot{V}_o$ = observed discharge at time t (m^3/s)

$\dot{V}_m$ = modeled discharge at time t (m^3/s)

3.2.2.1 Selection of rainfall-runoff events

Data measurement and field campaign for this study was started in 2007. During this year it was managed to capture many rainfall and runoff events from the three watersheds. Meanwhile the rainy season during the 2008 field campaign was far below average and the rainfall events which generate runoff were very few. In order to increase the number of events previous records from TBWREM were checked and the available records for year 2001 in Haiba and Laelay Wukro were found to be reliable and thus used as inputs for this study. The records in Haiba and Laelay Wukro other than for year 2001 were affected with frequent damage of measuring sensors, inconsistencies and error in equipment reading. In addition some of the events do not generate runoff like it was also observed in Laelay Wukro in 2005 and 2006. Combining the data obtained from the mentioned sources and own measurement, a total of 20 rainfall-runoff events representing different antecedent moisture conditions used in this study are shown in Table 3-8.

Table 3-8: Rainfall-runoff events from three watersheds

Year	Watershed	Date	AMC	Event Name
2001	Laelay Wukro	22-August	AMC I	E-1
2007	Laelay Wukro	23-July	AMC II	E-2
		30-July	AMC I	E-3
		2-August	AMC III	E-4
		20-August	AMC III	E-5
2008	Laelay Wukro	25-July	AMC II	E-6
		26-July	AMC III	E-7
2007	GumSelassa	12-July	AMC I	E-8
		18-July	AMC II	E-9
		19-August	AMC I	E-10
		20-August	AMC II	E-11
		21-August	AMC III	E-12
2008	GumSelassa	22-August	AMC I	E-13
2001	Haiba	25-July	AMC III	E-14
		27-July	AMC III	E-15
		5-August	AMC I	E-16
		10-August	AMC II	E-17
2007	Haiba	16-July	AMC II	E-18
		1-August	AMC III	E-19
		2-August	AMC III	E-20

3.2.2.2 Calibration results

Calibration was carried out manually and automatically. Initially manual calibration was done in order to identify appropriate initial parameters for automated calibration. The parameters calibrated are CN, h_{Ia} and t_{lag} for a single unit considering the total watershed area and channel routing parameters $t_{lagrouting}$ and N_{man} when subdividing the total area into smaller sub-watersheds. The outputs of the calibration included in this subtopic are related to a single unit calibration for the total watershed which is a lumped model. Summary of the optimized parameters is presented in Appendix 3.2.2.2. Parameter derivation and estimation for sub-watersheds will be discussed in chapter 3.6.2.4.

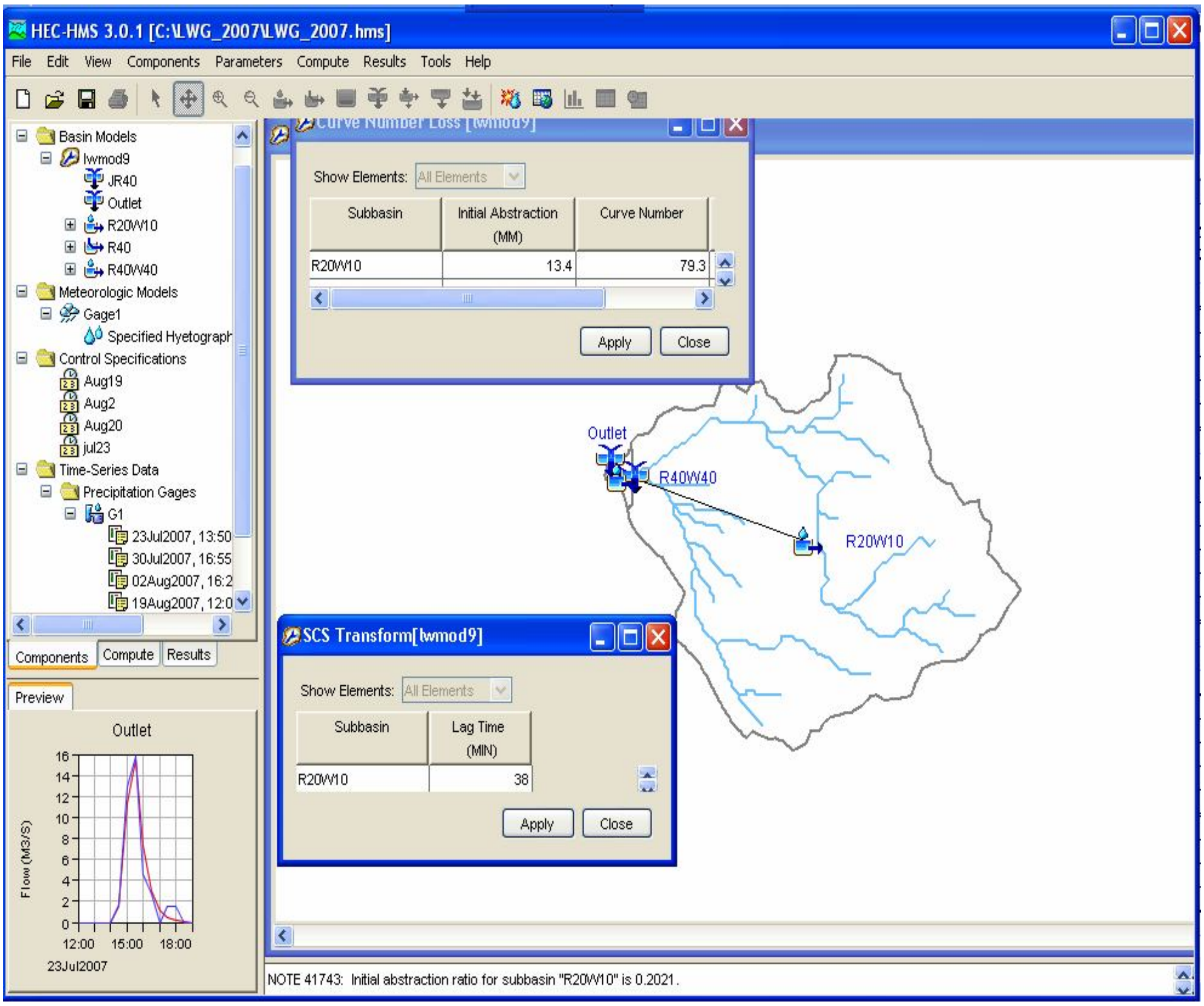

Figure 3-22: HEC-HMS model input parameter and simulation hydrograph for Laelay Wukro watershed of 23/07/2007

Automatic calibration was carried out using Nelder and Mead searching algorithm method available in the program for fine twining the manual calibration results. The Nelder and Mead algorithm evaluates all parameters simultaneously using the downhill simplex method. The optimization output was assessed by means of flow graph comparison, scatter diagram and statistical goodness of fit.

Flow graph comparison

Figures 3-23 through 3-25 show the observed and simulated hydrographs for Laelay Wukro, GumSelassa and Haiba Watersheds. The hydrologic model simulations have remarkably reproduced the observed hydrographs.

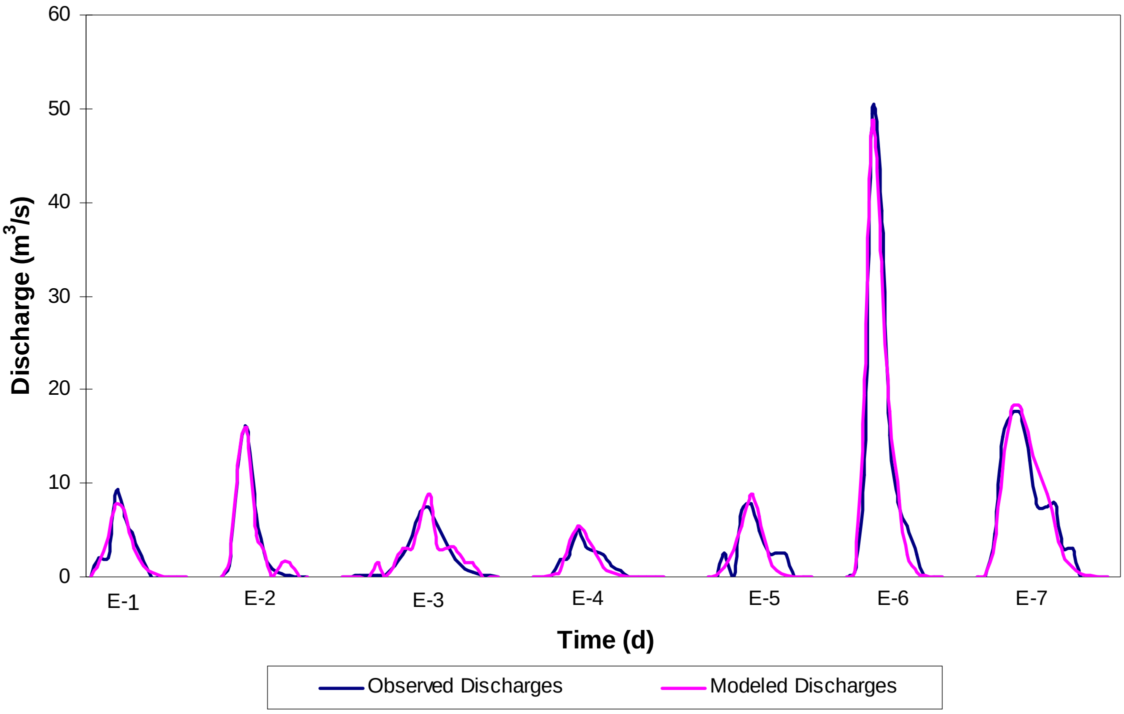

Figure 3-23: Observed and modeled hydrographs for Laelay Wukro watershed

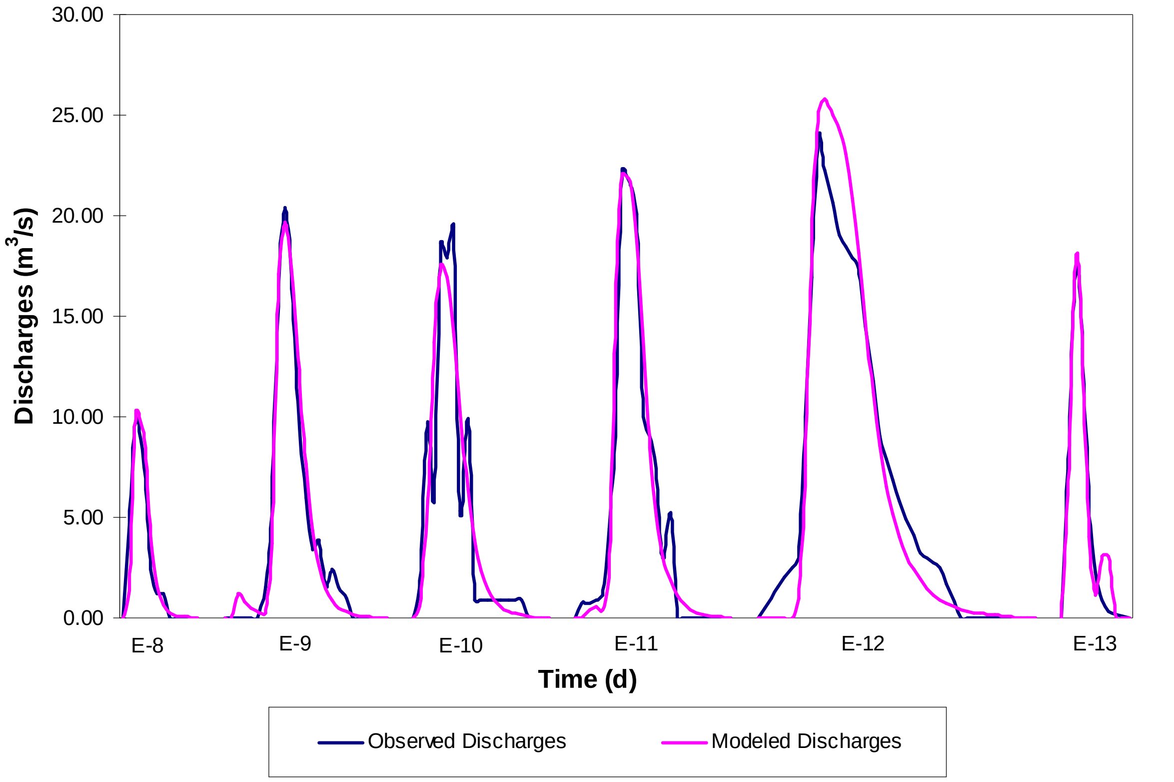

Figure 3-24: Observed and modeled hydrographs for GumSelassa watershed

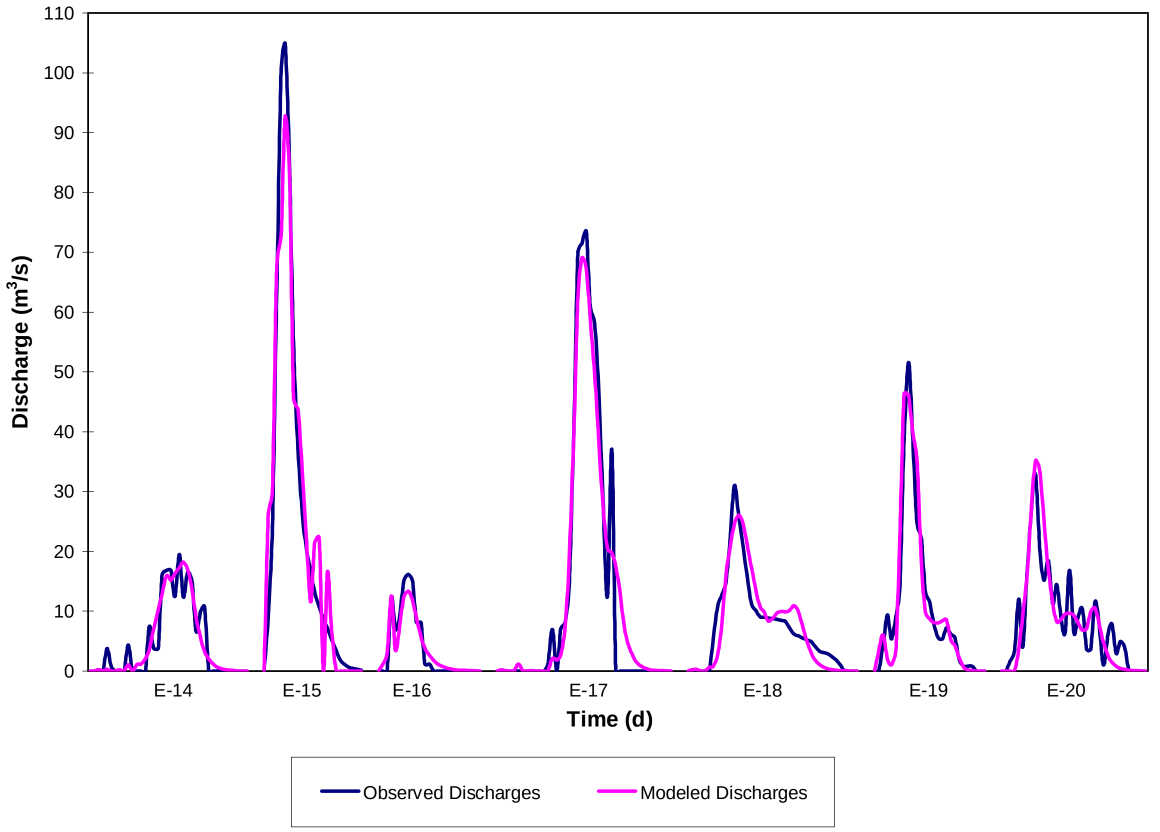

Figure 3-25: Observed and modeled hydrographs for Haiba watershed

Scatter graph

The scatter graph is a plot of observed discharge on one axis and modeled discharge on another axis. Straight line plots represent equality of both, observed and modeled discharges. Figure 3-26 shows the scatter plot diagram for the three watersheds. The first row refers to Laelay Wukro, second row GumSelassa, and third row for Haiba watersheds with respect to AMC and event name. The AMC is shown at the top of each box and event name at the bottom of each box. The lowest correlation (R^2) =0.77 observed in GumSelassa for AMC III condition. 60% of the simulation do have R^2 >0.90, 15%, R^2= (0.85-0.90), 15% R^2 = (0.80 -0.84), 20%, R^2 = (0.75- 79). The x-axis of each graph represents the observed discharge (m^3/s) and the y-axis the modeled discharge (m^3/s).

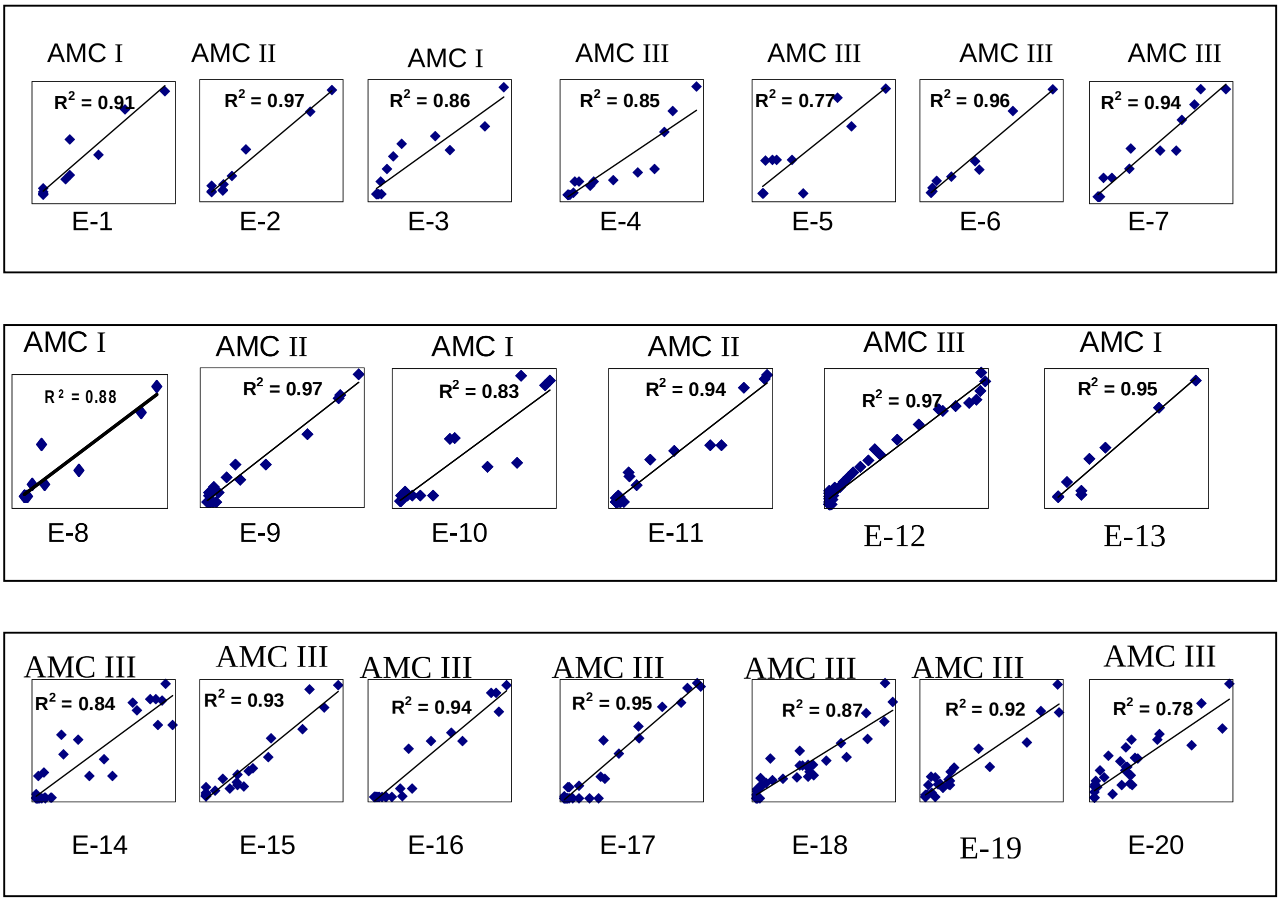

Figure 3-26: Scatter plots for different rainfall-runoff events, for different AMCs.

Statistical goodness-of-fit

Performance of rainfall-runoff models is usually tested using different statistical measures. The Nash and Sutcliffe (1970) is the widely used method in assessing goodness of fit of the modeled result compared to observed value.

Nash-Sutcliffe model efficiency coefficient (NS) is given by:

$$NS = 1 - \frac{\sum_{t=i}^{N} (\dot{V}_o(t) - \dot{V}_m(t))^2}{\sum_{t=i}^{N} (\dot{V}_o(t) - \dot{V}_{Avg})^2} \quad (3.32)$$

Where

$\dot{V}_o$ = observed discharge (m^3/s)

$\dot{V}_m$ = modeled discharge (m^3/s)

$\dot{V}_{Avg}$ = average of observed discharges (m^3/s)

The computed (NS) is greater than 0.8, except for two having NS =0.73 and NS =0.70 respectively. In general it can be concluded that the calibration process is good and satisfactory result was obtained. The computed NS values are reported in Table 3-9.

Table 3-9: Statistical performance assessment of each event

Watershed	Event Date	Event Name	NS
Laelay Wukro	22/08/2001	E-1	0.91
	23/07/2007	E-2	0.97
	30/07/2007	E-3	0.84
	02/08/2007	E-4	0.81
	20/08/2007	E-5	0.73
	25/07/2008	E-6	0.96
	26/07/2008	E-7	0.93
GumSelassa	12/07/2007	E-8	0.86
	18/07/2007	E-9	0.98
	19/08/2007	E-10	0.88
	20/08/2007	E-11	0.94
	21/08/2007	E-12	0.94
	22/08/2008	E-13	0.95
Haiba	25/07/2001	E-14	0.84
	27/07/2001	E-15	0.92
	05/08/2001	E-16	0.91
	10/08/2001	E-17	0.96
	16/07/2007	E-18	0.85
	01/08/2007	E-19	0.94
	02/08/2007	E-20	0.70

3.3 Sensitivity analysis

Rainfall-runoff models can vary from simple to complex. The model output will be dependent on the variation of changes on the input values. One possible way of grouping sensitivity analysis methods is to differentiate three classes: screening methods, local and global sensitivity analyses (Saltelli, 2000). In local sensitivity analyses one parameter is selected to vary and running model with the rest of the parameters kept constant (Lenhart et al., 2002). In the global method a number of variables can be changed simultaneously and, unlike the local method, parameter interaction is possible .However, the local sensitivity analysis still remains quite powerful method in gaining an in sight into the function of the model and whether parameters are adequately represented for the model at interest (Murphy et al., 2006). For this study local sensitivity analysis is used to test the sensitivity of each parameter input to the model output.

Mishra and Singh (2003) analyzed the sensitivity of input parameters to the SCS equation by plotting iso-lines. According to Mishra and Singh (2003) the curve number is the most sensitive parameter especially at lower curve number values. In this study sensitivity analysis is carried out to see overall volume variation at the outlet when the input parameters are varied with a certain percentage, considering the total watershed area as a unit. The optimum parameter values obtained from calibration are considered to be the base line values for estimating the next values and the measured values for rainfall. The HEC-HMS model was run repeatedly with parameter values obtained by multiplying optimal parameters (baseline) with factors 0.9, 0.92, 0.94, 0.96, 0.98, 1.02, 1.04, 1.06, 1.08 and 1.10 to indicate the percentage changes for each parameter one at a time keeping the other parameter at its optimum value. At the end of each model run the PEV and Percentage Error in Peak (PEP) is calculated and plotted as shown in Figure 3-27 and Figure 3-28.

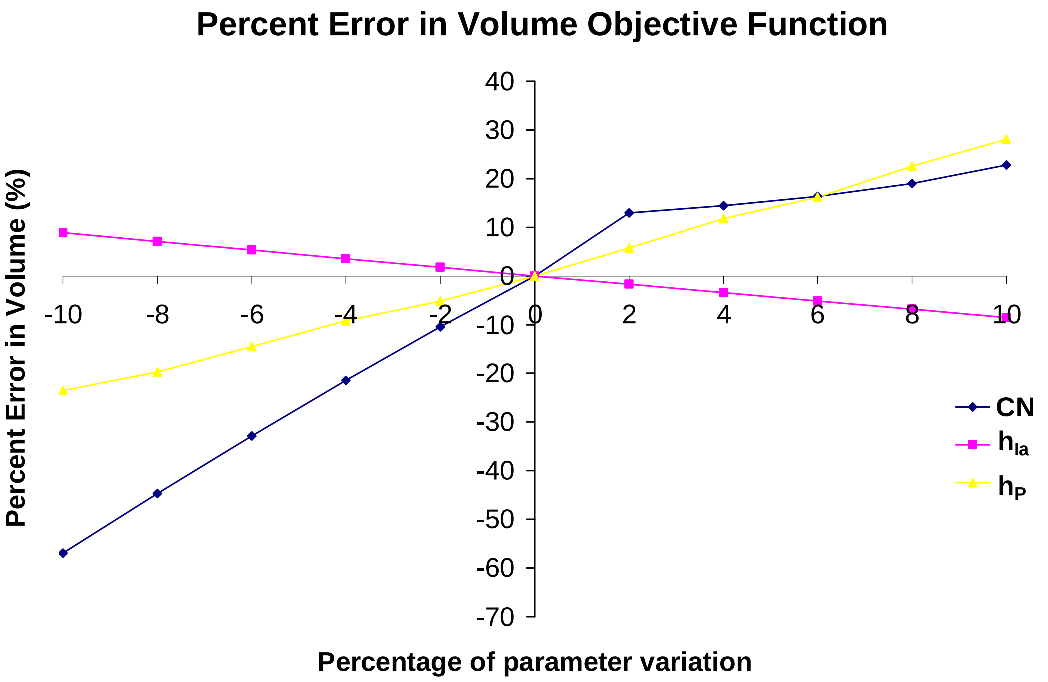

Figure 3-27: PEV variation with parameter change scenario

The volume at the outlet is highly sensitive to changes made on curve numbers. As it is shown in Figure 3-27 there is a sharp decline in volume if the curve number is under estimated. This result is consistent to the conclusions made by Mishra and Singh (2003). An increment in rainfall depth beyond 6% results more runoff to the outlet, which indicates that a higher rainfall depths tend to produce higher runoff coefficients.

Similarly sensitivity analysis done to observe the influence of parameter changes to the peak of the hydrograph showed that the curve number is still the most sensitive followed by the storm rainfall, time lag and initial abstraction in the order of their sensitiveness.

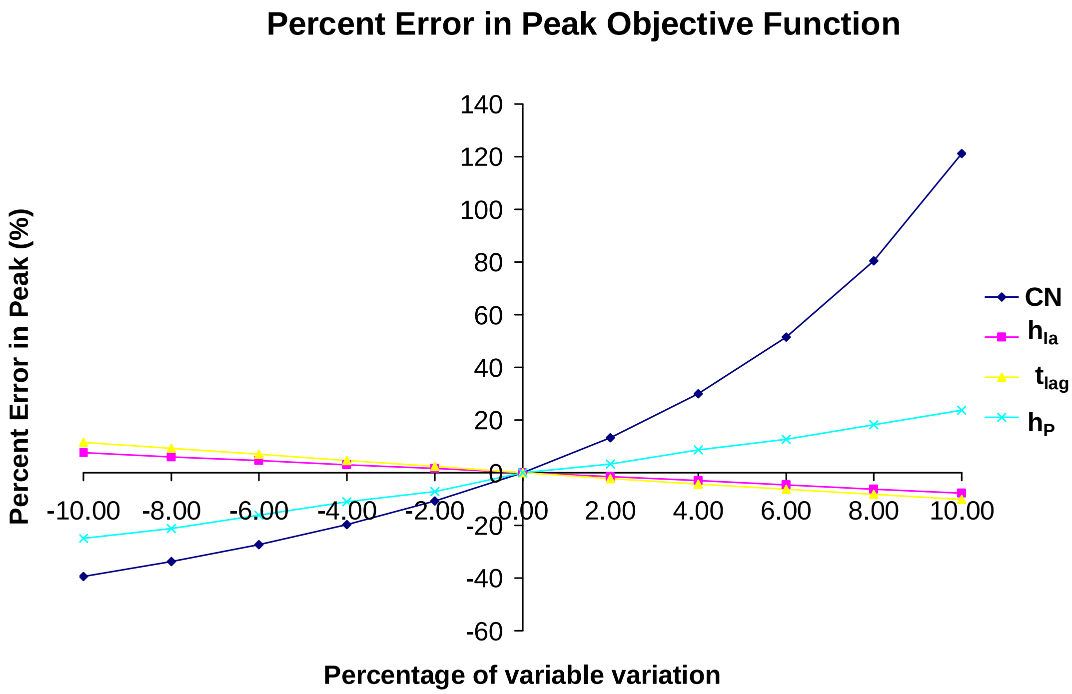

Figure 3-28: PEP variation with parameter change scenario

$$PEP = 100 \times \left(\frac{\dot{V}p_o - \dot{V}p_m}{\dot{V}p_o} \right) \tag{3.33}$$

Where

PEP = percentage error in peak (%)

$\dot{V}p_o$ = observed peak discharge (m^3/s)

$\dot{V}p_m$ = modeled peak discharge (m^3/s)

Figure 3-28 shows that 10% increase in the curve number can lead to over 100% increase on the peak hydrograph and about 40% decrease at the peak discharge with 10% decrease in the curve number values. The time lag and initial abstraction seemed to vary linearly. From Figure 3-27 and Figure 3-28 it is possible to deduce that the curve number which is at the centre of the SCS method should be carefully selected. Underestimation can have a significant impact on volume estimation and overestimation can also significantly over estimate the peak discharge.

3.4 Model validation

It is good practice to conduct a verification test for the reasons that the results of any calibration process are conditional on several factors such as the calibration data, the objective function, and the optimization procedure (Sorooshian et al. in Sing, 1995). Usually split sampling procedure is followed by keeping part of the available record for calibration and the remaining for validation. Unfortunately, historical flow information is highly limited for many ungauged or recently gauged watersheds, and thus synthetic flow generation needs to be implemented (Jia and Culver, 2006). In this study all the data recorded from the three watersheds were used for model calibration; model validation is performed with synthetic data generated by bootstrap sampling. The sampling method works with replacement from a sample and basically relies on own resources as often the only resources a researcher has (Teknomo, 2006).

The following procedures are followed during the model validation

i. Sample generation

Initially the parameters obtained from calibration of each rainfall-runoff event were grouped in accordance with antecedent moisture condition. The dataset from GumSelassa and Haiba were kept in the same group and Laelay Wukro was treated separately. Then sampling carried out with replacement resulting possible scenarios for parameters CN and λ combination. The samples were generated based on the work of Teknomo (2006) which is available on public domain.

ii. Flow volume comparison

For each event scenario (combination of CN and λ) volume is computed with equation 3.30. The computed volume was compared with the observed data of each rainfall-runoff event based on the PEV objective function. The PEV objective function did not consider the magnitude of volume modeled. It gives the same weight for both smaller and bigger volume. In order to give more weight to bigger volumes a proportional weighting (equation 3.34) objective function is preferred while selecting the best scenario events.

$$PEV_{agg} = \frac{\sum_i (PEV)_i \cdot Vo_i}{\sum_i V_{oi}} \qquad (3.34)$$

Where

PEV_{agg} = aggregated PEV (%)

PEV_i = PEV for each rainfall-runoff event (%)

Vo_i = observed volume for each rainfall-runoff event (m^3)

iii. Selecting best scenarios

The generated scenario depends on the number of calibrated data set used for bootstrap sampling. The number of scenarios and related summary statistics are reported in Table 3-10. According to Table 3-10, the aggregated errors in volume are quite low for AMC II and AMC III. Sampling result for AMC I showed slightly higher error compared to the other two moisture conditions. However given the fact that the SCS method is highly sensitive to lower curve number values the result is found to be satisfactory. The scenarios generated are shown in Appendix 3.4.1 in the accompanying CD.

Table 3-10: Summary results of bootstrap sampling

AMC	Watershed	No. of scenarios	PEV_{agg} (%)	
			μ	σ
I	GumSelassa and Haiba	14	13.31	1.09
II	GumSelassa and Haiba	58	4.296	0.291
III	GumSelassa and Haiba	81	2.758	0.133
I	Laelay Wukro	9	6.023	1.963
II	Laelay Wukro	7	2.24	0.704
III	Laelay Wukro	38	4.295	1.352

3.5 Uncertainty analysis

Not all parameters used in any environment model are measurable and at the same time the values to be used in the model estimation are not unique and are subject to error in measurements. The mathematical representation of the model deployed may not also accurately describe the reality. A combination of the mentioned factors leads to uncertainty of the model output. Wu et al. (2006) have indicated a number of techniques such as application of probability theory, Taylor series expression, Monte Carlo simulation, Bayesian statistics and sequential portioning for uncertainty analysis. For this study the Monte Carlo based uncertainty analysis is used and statistical measures, graphs and plots are used to analyze the results of the Monte Carlo simulation (MCS). In this study uncertainty propagation in the evapotranpiration and runoff estimation were studied and are reported as follows.

3.5.1 Uncertainties in evapotranspiration estimation

Evapotranspiration estimation often requires good measurement of climate data. In this study evapotranspiration is estimated by FAO-56 PM method where data is available (for Laelay Wukro watershed), and by fitted equation for GumSelassa watershed (equation 3.10) where data availability is limited. The fitted equation depends on temperature, sunshine hours and extraterrestrial radiation data. Temperature data obtained from Quiha Airport (1960-2008) were used to extrapolate and generate the temperature data set for GumSelassa watershed. MCS random samples were generated considering normal and uniform distribution for the different input parameters used to compute evapotranspiration in the FAO-56 PM and fitted equations (Appendix 3.5.1). About 2500 samples were generated for each month and the spread of the potential evapotranspiration is shown in Figure 3-29.

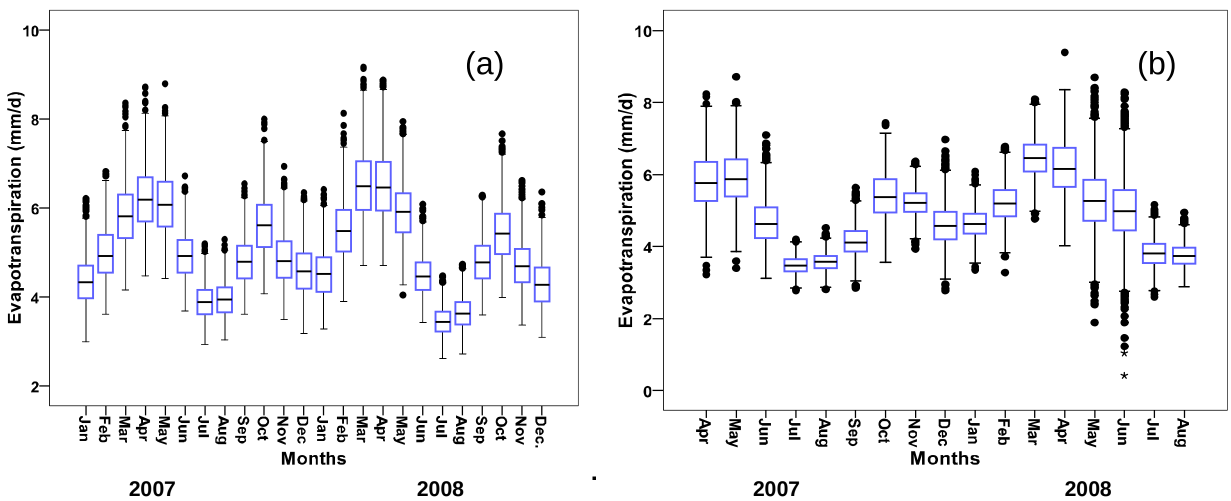

Figure 3-29: Box plot showing the spread and distribution of potential evapotranspiration for each month during the simulation period using the method of (a) fitted equation and (b) FAO-56 PM

Figure 3-29 shows the distribution of the potential evapotranspiration realized from MCS. The boxes represent the 50% confidence interval on the mean and the 1st and 3rd quartiles. The whiskers indicate the minimum and maximum values excluding the outliers. The small dots in the figure are the outliers. In Figure 3-29, (a) the outliers are observed only on upper side which indicates the method is more sensitive to higher temperature than lower temperature which is not the case in Figure 3-29, (b).

Potential evapotranspiration calculated based on the measured data is considered as representative potential evapotranspiration (h_{ETprep}) for comparison. The mean values of the MCS and the h_{ETprep} are almost identical for July and August where the evapotranspiration is low due to high humidity and low wind speed. In contrast during the driest months of the year the range of the estimated potential evapotranspiration is high resulting from variability of wind speed and humidity. However comparison of the h_{ETprep} with the MCS results revealed that the h_{ETprep} are contained consistently within the box, except on May and June 2008 in Laelay Wukro where the h_{ETprep} are slightly outside the box range. Figure 3-30 shows the Relative Error (ER) computed for each method. ER is computed with the following equation:

$$ER = \frac{\sum_i \left| \dot{h}_{ETprep} - \dot{h}_{ETpMCS} \right|}{n_{MCS}} \times \frac{100}{\dot{h}_{ETprep}} \qquad (3.35)$$

Where

ER = relative error (%)

$\dot{h}_{ETprep}$ = potential evapotranspiration computed from measured data (mm/d)

$\dot{h}_{ETpMCS}$ = potential evapotranspiration from each MCS run (mm/d)

n_{MCS} = number of Monte Carlo runs (1)

The relative errors computed by both methods are less than 15%. In fact for many of the months ER is less than 10% or about 10% which insures less uncertainty in the evapotranspiration computed by both methods. Summary of statistical parameters like BIAS, Relative Error (ER) and Coefficient of Variation (CV) computed for each month are reported in Table 3-11.

Table 3-11: summary of mean statistical parameters computed from MCS realizations

Statistical indicator	FAO-56 PM			Fitted equation		
	BIAS	ER (%)	CV	BIAS	ER (%)	CV
μ	0.4	7.67	0.12	0.49	10.05	0.15
σ	0.2	2.78	0.02	0.11	1.24	0.01

μ = monthly average over simulation period, σ = mean monthly standard deviation over simulation period

From all simulations it has been found out that the level of uncertainty in evapotranspiration estimation by the FAO-56 PM equation and the fitted equation are within acceptable range for the majority of the simulation periods. Thus it is concluded that the potential evapotranspiration generated in this study by different approaches can be used to estimate the reservoir evaporation, actual evapotranspiration and estimation of evapotranspiration in the region where data is scarce.

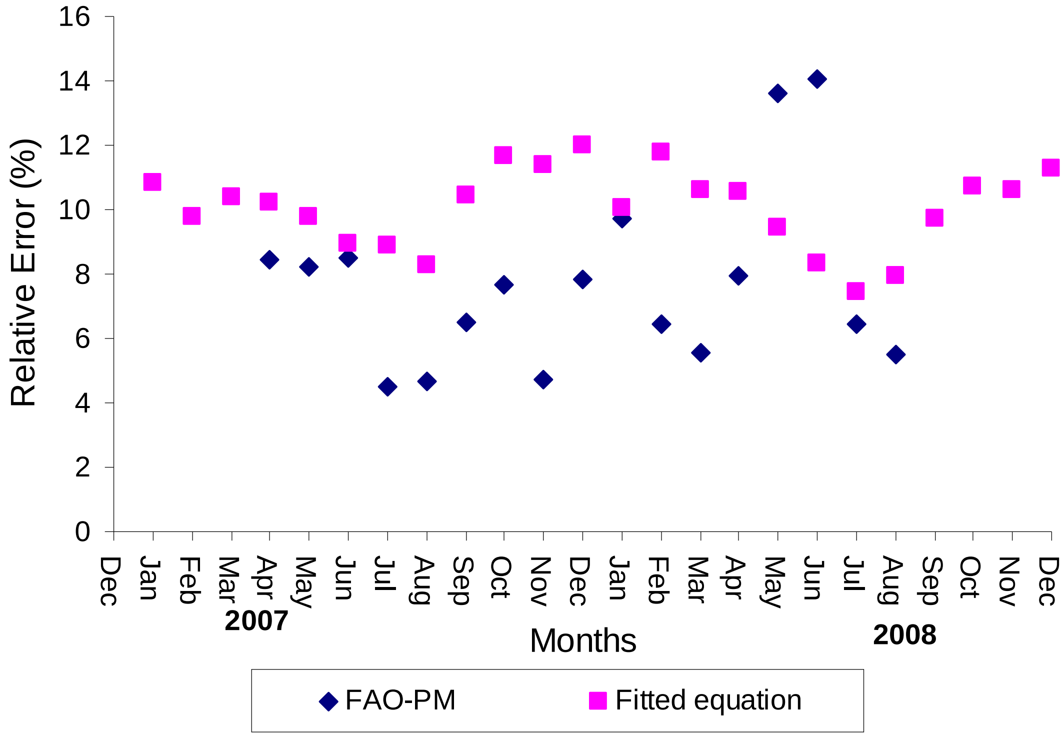

Figure 3-30: Relative error of the MCS realizations computed for each month

3.5.2 Uncertainty in runoff estimation

The rainfall-runoff events selected for calibration were analyzed separately considering the antecedent moisture conditions. GumSelassa and Haiba watersheds do have relatively similar soil group, landuse and relatively flat topography. Thus rainfall-runoff events from both watersheds were analyzed together and Laelay Wukro watershed independently. The Generalized Likelihood Uncertainty Estimation (GLUE) approach (Beven and Binley, 1992) was applied to obtain the uncertainty estimates. This method involves a number of steps namely defining input distribution, sample randomly from input distribution, run simulation with repeated sampling and determine probability distribution for the output.

i. Identification of sampling space for every parameter

In the SCS-method the rainfall depth of a single event is assumed to be distributed uniformly across watershed area. Each rainfall event has a specific pattern, distribution and amount and thus behaves independently from other rainfall event. Thus random sample generation for input parameter rainfall was done with uniform distribution by considering uncertainty in rainfall measurements to a high percentage i.e. $\pm$ 20% for a single event. For the remaining parameters random samples were generated with Gaussian equation for normal distribution considering the antecedent moisture conditions (Appendix 3.5.2).

ii. Sampling of parameter space

10,000 Monte Carlo random samples generated for each event based on the parameter space indicated above.

iii. Selection of a likelihood measure

The likelihood measure is chosen on basis of its appropriateness in relation to the model, the observed data and the objectives of the study (Wu et al., 2006). For this study PEV is used as a likelihood function as it fits with the objective of this study. A likely hood value **(0)** means perfect match between the simulated and observed volume.

iv. Selection of acceptable (behavioural) simulations

The term behavioural is used to signify models that are judged to acceptable or in other word not ruled out on the basis of available data and knowledge (Blasone et al., 2008). The threshold for distinction between behavioural and non-behavioural can be fixed by either the percentage of retained simulation from the total array of distribution or the accepted error for that particular study. For this study $\pm$10% is considered as threshold and all simulations within this range are considered as behavioural and the remaining non-behavioural, thus not considered for posterior distribution.

v. Determining the lower and upper boundary from behavioural distribution

Cumulative Distribution Function (CDF) of each event has been determined to identify the 5% and 95% sample quantiles. The 5% percentile is lower boundary and 95% is upper boundary of the uncertainty estimation as shown in Figure 3-31.

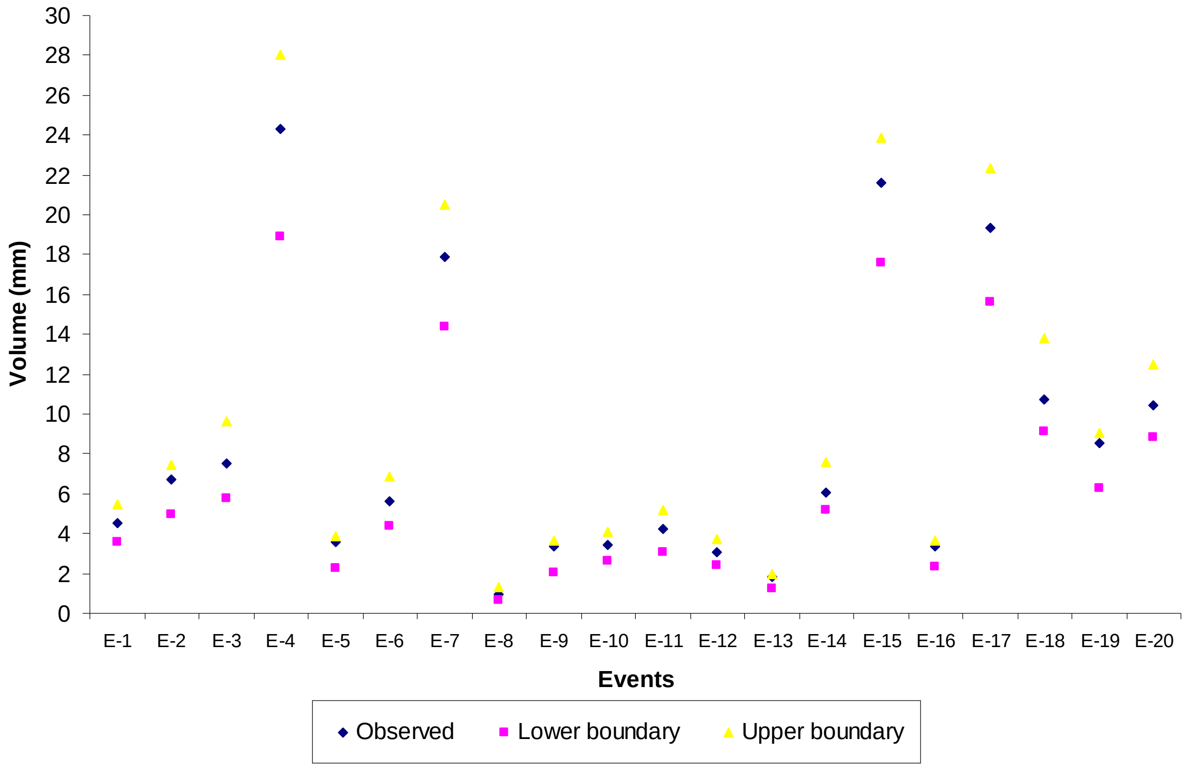

Figure 3-31: Uncertainty boundaries for different simulation events and corresponding volumes of MCS realizations

The ranges of parameters (CN and λ) that give behavioural output are derived from the distribution space of the uncertainty limits are shown in Figure 3-32 and Figure 3-33. The shape of the distribution indicate the degree of uncertainty of the estimates i.e. sharp and peaked distribution are associated with well defined parameters, while flat distributions indicate more uncertain parameters (Balsone et al., 2008). Looking at the distribution of the histograms It might not be possible to single out a well defined parameter but it is clear to see the optimum parameters are skewed to wards the mean resulting a optimum solution.

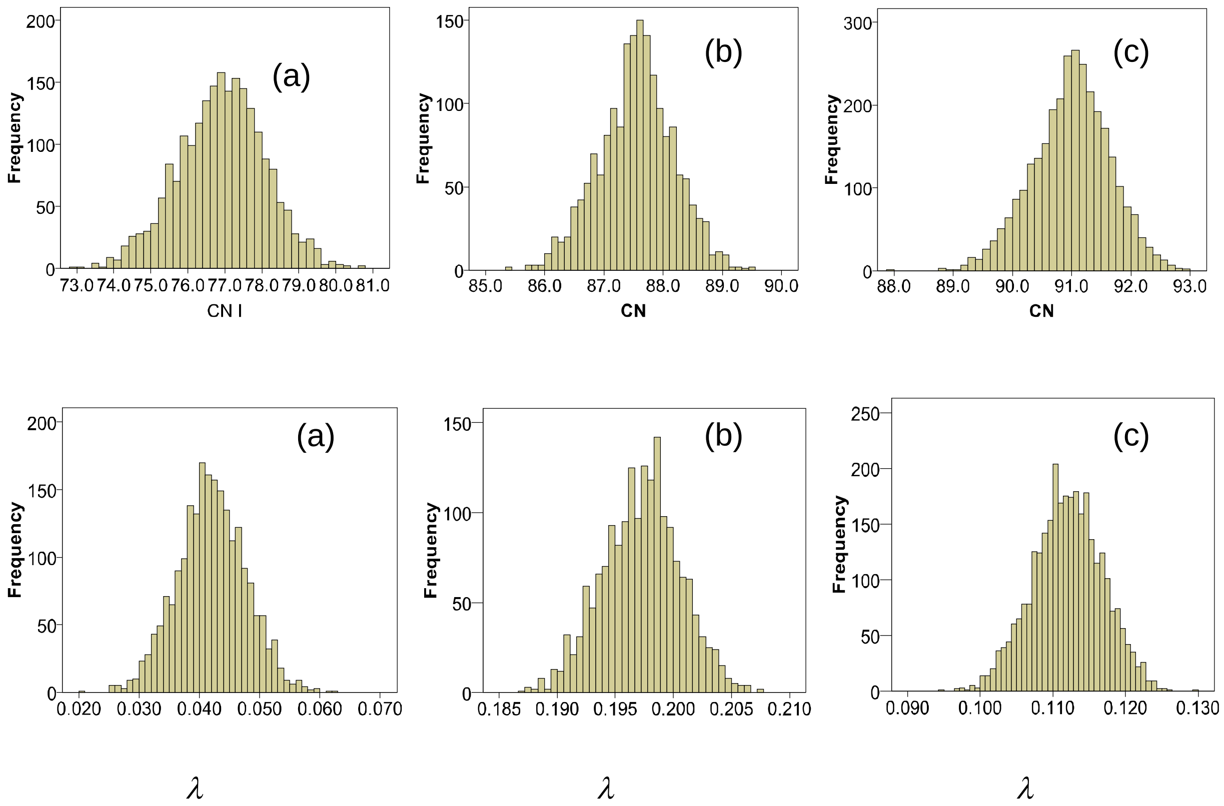

Figure 3-32: Histograms approximating posterior distribution of parameters CN and λ for Haiba and GumSelassa watersheds (a) = AMC I, (b) = AMC II, (c) = AMC III

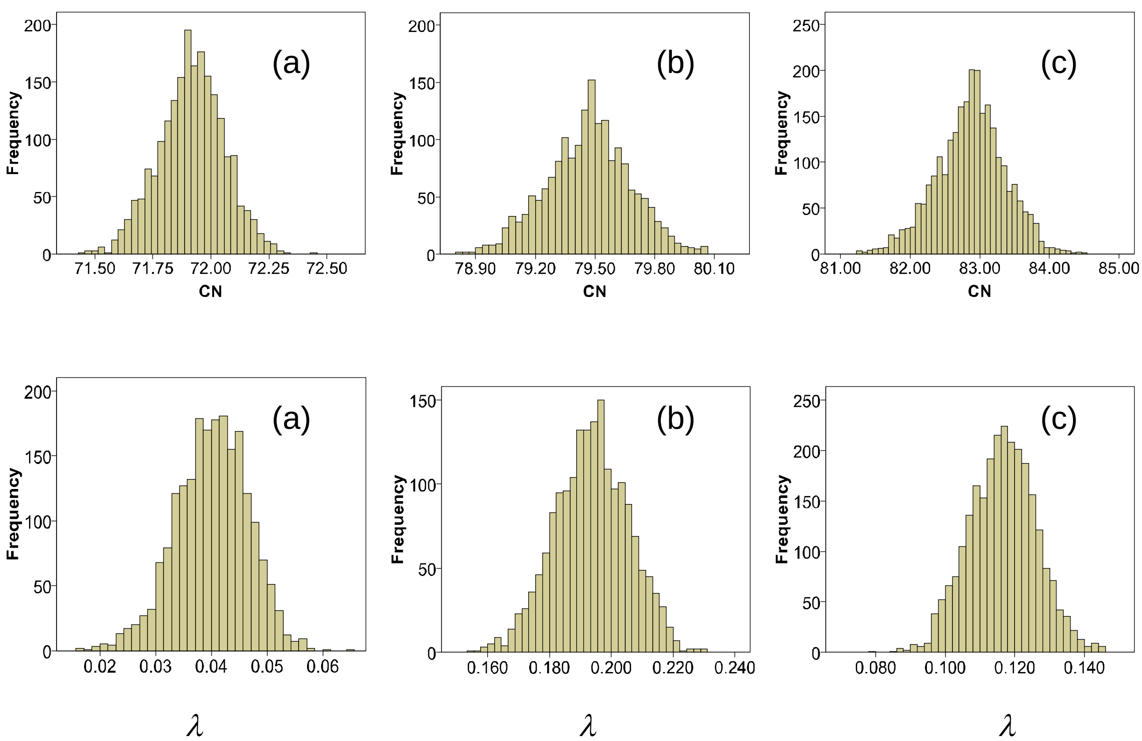

Figure 3-33: Histograms approximating posterior distribution of parameters CN and λ for Laelay Wukro watershed (a) = AMC I, (b) = AMC II, (c) = AMC III

The range of parameters and the likelihood measure for the posterior distribution are reported on Figure 3-34.

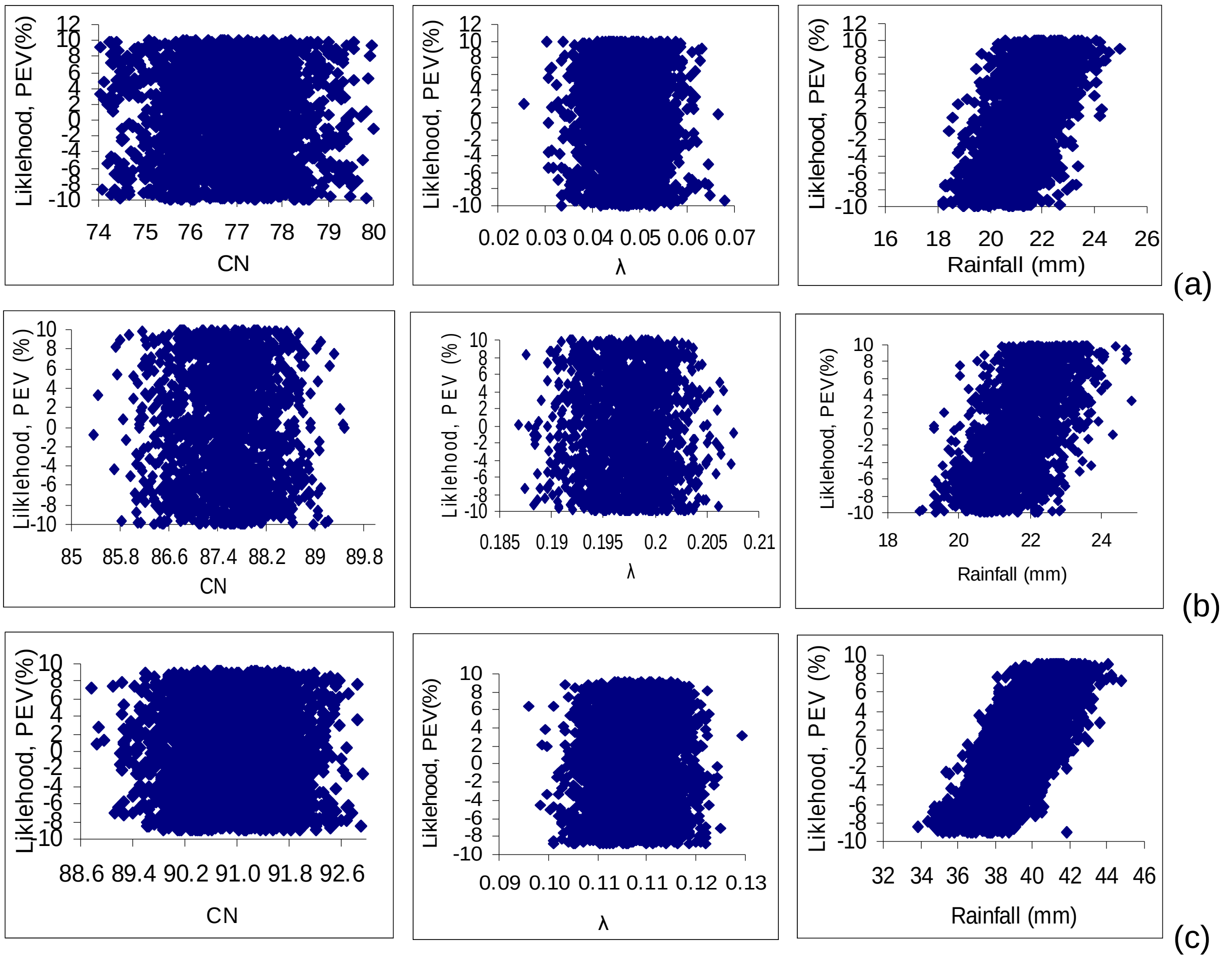

Figure 3-34: Dotty plot of likelihood PEV against parameters for Haiba and GumSelassa watersheds for representative rainfall-runoff events, (a) = AMC I, (b)=AMC II, (c) = AMC III.

The most optimum model output value is distributed across the behavioural parameter range. This might imply the uncertainty is wide spread across the parameter range, but it is also possible to conclude that the possible range of parameters that can contribute towards attaining the most optimum model output. This leads to the idea that many different model structures and many different parameter sets within a chosen model structure can reproduce the observed system (Beven and Freer, 2001) which is usually advocated as equifinality.

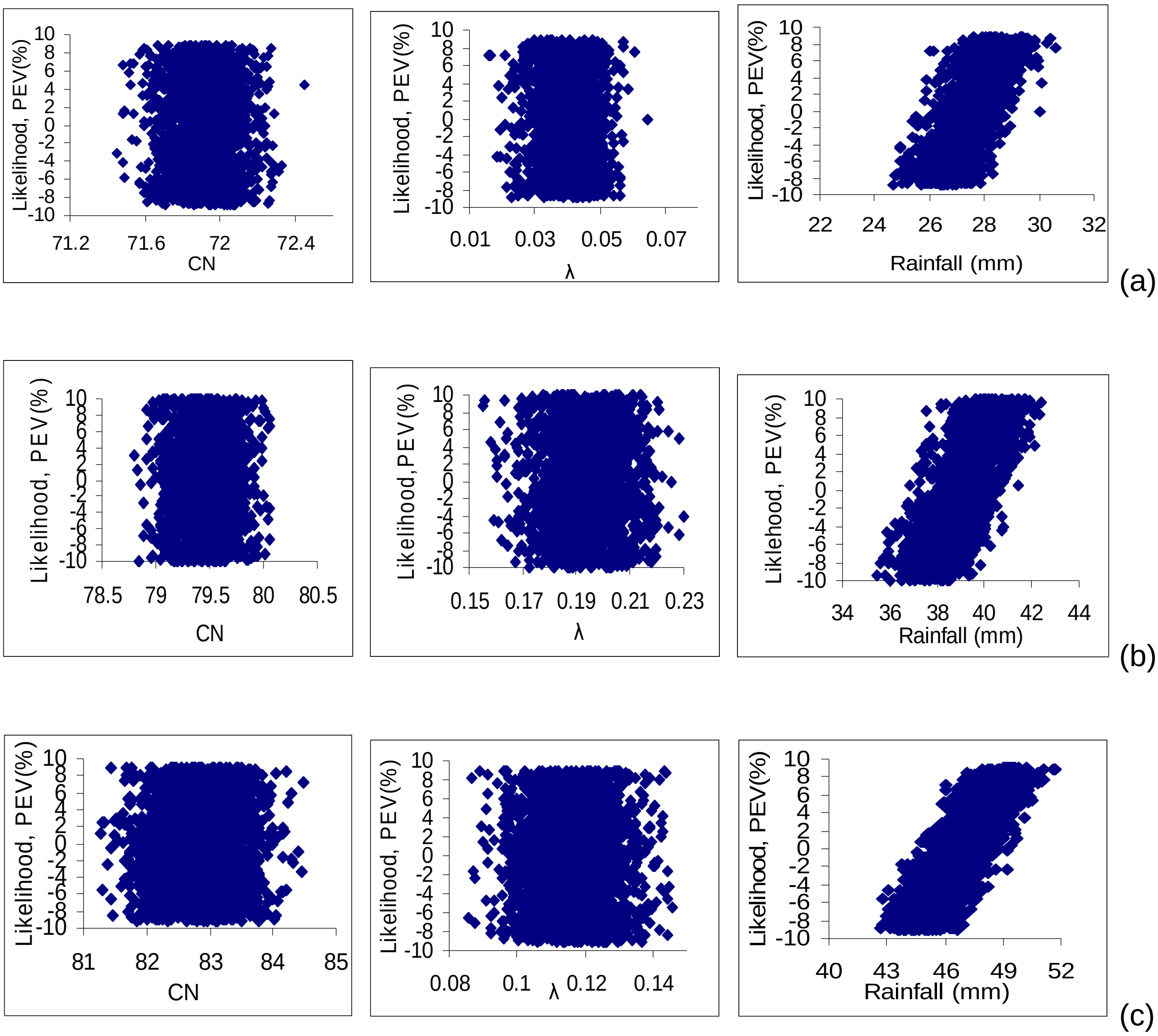

Figure 3-35: Dotty plot of likelihood PEV against parameters for Laelay Wukro watershed for representative rainfall-runoff event, (a) =AMC I, (b) =AMC II and (c) = AMC III

Summary statistics are given in Table 3-12. In general the distribution for Haiba and GumSelassa are relatively dispersed compared to Laelay Wukro, which is also reflected in larger standard deviation.

Table 3-12 Summary statistic of the posterior distribution with respect to AMC

Watershed	Parameter	AMC I		AMC II		AMC III	
		μ	σ	μ	σ	μ	σ
GumSelassa	CN	77.10	0.865	87.55	0.628	91.00	0.652
and Haiba	λ	0.058	0.019	0.197	0.003	0.112	0.0048
Laelay Wukro	CN	71.92	0.141	79.47	0.212	82.87	0.489
	λ	0.040	0.0065	0.194	0.012	0.116	0.0096

μ = average , σ = standard deviation

3.6 Parameter estimation and regionalization for ungauged catchments

Various approaches and techniques were applied that would give a basis for rainfall-runoff modeling of ungauged catchments in Northern Ethiopia. The approaches deployed are:

- derivation of equations through regression,
- regionalization of hydrologic model parameters, and
- monthly water balance approach

3.6.1 Derivation of new equations

Empirical equations derived from physical features of the gauged catchments can give reasonably good estimate. Empirical equations were developed with a multiple regression accounting various catchment characteristics. The relationship between rainfall and runoff is usually considered to be linear. However the initial losses and infiltration losses may result non-linear relationship. This problem can be solved with logarithmic transformation.

$$Log_{10} h_V = b_1 \times Log_{10} X_1 + b_2 \times Log_{10} X_2 + \cdots\cdots + b_n \times Log_{10} X_n + Log_{10} c \qquad (3.36)$$

Or alternatively if transformed it can be expressed as

$$h_V = c \times X_1^{\,b_1} \times X_2^{\,b_2} \cdots\cdots X_n^{\,b_n} \qquad (3.37)$$

Where

h_V = runoff (mm)

X_1, X_2, X_n = catchment characteristics (variable units)

b_1, b_2, b_n = regression coefficients for respective catchment characteristics (1)

c = intercept (1)

The rainfall-runoff events were grouped according to AMC and the predictive regression equations were estimated based on the contribution of each catchment characteristics (model descriptors) to the regression model.

Equations for AMC I: for rainfall depths (20mm - 40mm)

i. $Log_{10}h_V = 2.319 \times Log_{10}h_P + Log_{10}0.002522$ (3.38a)

$$h_V = 0.002522 \times (h_P)^{2.319} \quad (R^2=0.87) \quad (3.38b)$$

Where

h_V = runoff (mm)

h_P = precipitation (mm)

ii $Log_{10}h_V = 3.64 \times Log_{10}h_P + 1.795 \times Log_{10}\dot{h}_{imean} + Log_{10}1.78 \times 10^{-7}$ (3.39a)

$$h_V = 1.79 \times 10^{-7} \times (h_P)^{3.64} \times (\dot{h}_{imean})^{1.795} \quad (R^2=0.95) \quad (3.39b)$$

Where

$\dot{h}_{imean}$ = average rainfall intensity (mm/h)

iii. $Log_{10}h_V = 4.35 \times Log_{10}h_P + 1.522 \times Log_{10}\dot{h}_{imean} + 0.706 \times Log_{10}Ac + Log_{10}5.68 \times 10^{-9}$

(3.40a)

$$h_V = 5.68 \times 10^{-9} \times (h_P)^{4.35} \times (\dot{h}_{imean})^{1.522} \times Ac^{0.706} \quad (R^2=0.99) \quad (3.40b)$$

Where

Ac = catchment area (km^2)

Equations for AMC II: for rainfall depths (18mm – 65mm)

i. $Log_{10}h_V = 1.63 \times Log_{10}h_P + Log_{10}0.028515$ (3.41a)

$$h_V = 0.028515 \times (h_P)^{1.63} \quad (R^2=0.90) \quad (3.41b)$$

ii. $Log_{10}h_V = 2.09 \times Log_{10}h_P + 0.67 \times Log_{10}Ac + Log_{10}0.0008$ (3.42a)

$$h_V = 0.0008 \times (h_P)^{2.09} \times Ac^{0.67} \quad (R^2=0.997) \quad (3.42b)$$

Equations for AMC III: for rainfall depths (13mm – 45mm)

i. $Log_{10}h_V = 1.55 \times Log_{10}h_P + Log_{10}0.05581$ (3.43a)

$$h_V = 0.05581 \times (h_P)^{1.55} \quad (R^2=0.77) \quad (3.43b)$$

ii. $Log_{10}h_V = 1.87 \times Log_{10}h_P + 0.72 \times Log_{10}Ac + Log_{10}0.00253$ (3.44a)

$$h_V = 0.00253 \times (h_P)^{1.87} \times Ac^{0.72} \quad (R^2=0.992) \quad (3.44b)$$

iii. $Log_{10}h_V = 1.84 \times Log_{10}h_P + 0.89 \times Log_{10}Ac + 0.1 \times Log_{10}S + Log_{10}0.00138$ (3.45a)

$$h_V = 0.00138 \times (h_P)^{1.84} \times Ac^{0.89} \times S^{0.10} \quad (R^2=0.993) \quad (3.45b)$$

Where

S : watershed slope (m/m)

The calculated statistical parameters like unbiased estimate of the coefficient of determination, F-statistics, FDIST (F-distribution) and standard error of estimate of the developed regression equations are reported in Table 3-13. These statistical indicators can be calculated with standard statistical software.

$$\overline{R}^2 = 1 - \frac{(1-R^2)\times(n-1)}{df} \qquad (3.46)$$

Where

$\overline{R}^2$ = unbiased coefficient of determination (1)

R^2 = coefficient of determination or correlation (1)

n = number of observations (1)

df = degree of freedom (1)

The degree of freedom is calculated by subtracting the number of variables from the number of events. The F-statistics listed in Table 3-13 are useful to check whether the obtained correlation between the dependent and independent variables occur by chance. High F-statistic value indicates good correlation. This is further checked by FDIST probability estimates and it measures the probability of high correlation occurrence. Equations 3.38a - 3.45b were developed with rainfall depths greater than 15mm for AMC I and AMC II, and greater than 13mm for AMC III.

Table 3-13: Summary statistics of the developed equations

Equation	$\overline{R}^2$	F-statistics	FDIST	S_e for regression coefficients of model descriptors				
				P	$imean$	Ac	S	Intercept*
3.38a/b	0.84	26.54	0.007	0.45	---	---	---	0.61
3.39a/b	0.92	31.16	0.009	0.64	0.76	---	---	---
3.40a/b	0.98	71.12	0.014	0.43	0.43	0.25	---	1.13
3.41a/b	0.88	37.01	0.004	0.27	---	---	---	0.41
3.42a/b	0.99	547.87	0.0001	0.069	---	0.066	---	0.17
3.43a/b	0.74	20.43	0.004	0.34	---	---	---	0.48
3.44a/b	0.99	314.5	0.0000055	0.075	---	0.06	---	0.15
3.45a/b	0.99	179.47	0.0001	0.10	---	0.32	0.19	0.53

*= S_e for the intercept are for real number not for the logarithmic number shown in each equation

Therefore the equations are valid for rainfall depths greater than or equal to the listed rainfall depths and for better results between the values indicated within the ranges indicated for each category. Inclusion of additional rainfall-runoff events and new watersheds can improve the predicting capability of the above listed equations. Nevertheless, the estimated correlations are high and the associated errors are relatively low especially for AMC II and AMC III. The FDIST probability values are low which confirms that the high correlation values are not obtained by chance rather they are results of the existing relationship between catchment characteristics and event runoff. Thus the equations can be used in watersheds in the region having similar catchment characteristics or features.

Due to their low S_e value for regression coefficients the following equation can give better comparative advantage.

- Equations 3.40a or 3.40b for AMC I
- Equations 3.42b or 3.42b for AMC II, and
- Equations 3.44a or 3.44b for AMC III.

3.6.2 Regionalization of selected rainfall-runoff model

Runoff from ungauged catchments can be estimated by using models that include only parameters that can be observed or inferred from measurements, or extrapolate from parameters found for gauged catchments within the same region (USACE, 1994, EM 1110-2-1417). The SCS method is entirely dependent on physical features of the watershed such as hydrologic soil group, landuse and catchment treatment which are necessary for computing the losses. Watershed transform can be also modeled with due consideration of river length, catchment slope besides to soil and landuse information of the watershed. Therefore the SCS method is applicable for ungauged catchments. SCS model features and parameter calibration were discussed in depth in chapter 3.2. In this section the regionalization of the model parameters (curve number, initial abstraction factor and transform) will be presented.

3.6.2.1 Curve Number

AMC II

In the SCS method AMC II is considered to be the optimum moisture condition of a watershed. Table 3-14 shows CN calibrated (CN_{cal}), CN read from NEH-4 table and CN calibration factor (Cf_{CN}) for the selected rainfall-runoff events. For AMC II condition, the calibrated CN value and the standard CN values taken from NEH-4 table are comparable. The mean (μ) and standard deviation (σ) of the calibration factor (Cf_{CN}) are 1.008 and 0.008 respectively. The Cf_{CN} is nearly equal to one for most of the simulations which implies, the NEH -4 table values can give reasonably acceptable runoff estimates for AMC II condition with slight refinement on CN values read from NEH-4 table.

Table 3-14: Comparison of CN values read from table with values obtained from model calibration and summary statistics for AMC II

Event / CN	E-2	E-6	E-9	E-11	E-17	E-18	Cf_{CN} statistics for all events		
							μ	Median	σ
NEH-4 table	79.39	79.39	87.23	87.23	85.89	85.89			
CN_{cal}	79.3	79.6	88.0	87.5	87.8	86.6			
Cf_{CN}	0.999	1.003	1.009	1.003	1.022	1.008	1.007	1.006	0.008

AMC I and AMCIII

The rainfall-runoff events representing the AMC I and AMC III condition are summarized in Table 3-15. According to Table 3-15, the existing NEH-4 conversion table consistently underestimates the curve number for AMC I and overestimates the curve number for AMC III. The factor Cf_{CN} varies from one watershed to other for both AMC I and AMC III. About 16% increment is observed in Laelay Wukro and less than 5% in GumSelassa and Haiba Watersheds for AMC I. Similarly for AMC III condition the variation is 8-9% for Laelay Wukro and less than 4% in GumSelassa and Haiba. The Cf_{CN} closer to 1.0 observed for both AMC I and AMC III conditions confirmed the suitability of the method for watersheds dominated with cultivated land (GumSelassa and Haiba) compared to watershed dominated with bushes, grasses and exclosure.

Table 3-15: Comparison of CN values read from table and obtained from model calibration with summary statistics for AMC I and AMC III

Event	AMC	Watershed	NEH-4 table conversion	CN_{cal}	Cf_{CN}	Cf_{CN}	
						μ	σ
E-1	I	Laelay Wukro	61.81	71.8	1.162	1.163	0.002
E-3	I	Laelay Wukro	61.81	72.0	1.165		
E-8	I	GumSelassa	74.15	76.0	1.025	1.04	0.011
E-10	I	GumSelassa	74.15	77.5	1.045		
E-13	I	GumSelassa	74.15	77.5	1.045		
E-16	I	Haiba	71.89	75.5	1.050		
E-4	III	Laelay Wukro	89.86	83.00	0.923	0.922	0.005
E-5	III	Laelay Wukro	89.86	83.25	0.926		
E-7	III	Laelay Wukro	89.86	82.3	0.916		
E-12	III	GumSelassa	94.02	91.0	0.968	0.973	0.007
E-14	III	Haiba	93.33	90.1	0.965		
E-15	III	Haiba	93.33	91.25	0.978		
E-19	III	Haiba	93.33	91.8	0.984		
E-20	III	Haiba	93.33	90.55	0.970		

Different researchers proposed equations for changing the AMC II (CN_{II}) to AMC I (CN_{I}) and AMC III (CN_{III})

i. Hawkins et al. (1985)

$$CN_{I} = \frac{CN_{II}}{2.281 - 0.01281 \times CN_{II}} \tag{3.47}$$

$$CN_{III} = \frac{CN_{II}}{0.427 + 0.00573 \times CN_{II}} \tag{3.48}$$

ii. Chow et al. (1988)

$$CN_{I} = \frac{4.2 \times CN_{II}}{10 - 0.058 \times CN_{II}} \tag{3.49}$$

$$CN_{III} = \frac{23 \times CN_{II}}{10 + 0.13 \times CN_{II}} \tag{3.50}$$

iii. Neitsch et al. (2002)

$$CN_{I} = CN_{II} - \frac{20 \times (100 - CN_{II})}{\{100 - CN_{II} + \exp[2.533 - 0.0636 \times (100 - CN_{II})]\}} \tag{3.51}$$

$$CN_{III} = CN_{II} \times \exp\{0.00673 \times (100 - CN_{II})\} \tag{3.52}$$

iv. Mishra et al. (2008)

$$CN_{I} = \frac{CN_{II}}{2.2754 - 0.012754 \times CN_{II}} \tag{3.53}$$

$$CN_{III} = \frac{CN_{II}}{0.43 + 0.0057 \cdot CN_{II}} \tag{3.54}$$

(CN_I) and (CN_{III}) determined for each watershed by equations 3.47 to 3.54 showed difference (Figure 3-36) with the calibrated curve numbers for respective antecedent moisture conditions. All equations perform poorly for Laelay Wukro and relatively better results for both GumSelassa and Haiba. In general the applicability of the aforementioned equations to the conditions of the study area was questioned; because all methods derive curve number assuming the initial abstraction factor (λ) equal to 0.2. But calibration and optimization of various rainfall-runoff events made by this study having different antecedent moisture conditions (Chapter 3.6.2.2.) confirmed that it varies according to antecedent moisture condition.

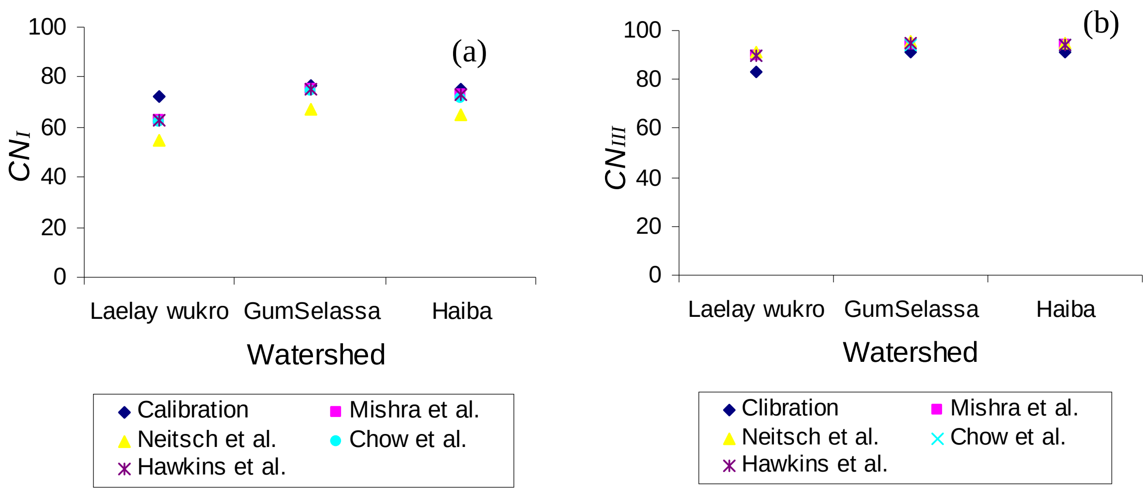

Figure 3-36: Curve numbers computed by different methods and the curve numbers obtained from calibration for each watershed (a) AMC I and (b) AMC III

Thus there is a need to develop new equations that can be representative to the existing conditions of the region. To this end an attempt has been made to relate the calibrated (CN_{II}) with (CN_I) or (CN_{III}) and propose new formulae suitable for the rainfall-runoff condition of the study area. The equations were developed based on the existing rainfall-runoff records from three catchments and are presented with respect to different land uses.

A. Mixed landuse watershed: This type of watershed comprises a combination of different land uses with cultivated land, bushes, exclosure and grass lands are the major land use units.

$$CN_I = \frac{CN_{II}}{1.819 - 0.009 \times CN_{II}} \tag{3.55}$$

$$CN_{III} = \frac{CN_{II}}{1.012 - 0.00068 \times CN_{II}} \tag{3.56}$$

B. Watershed dominated with cultivated land: Watersheds where more than 70% of the total area is covered with cultivated lands are considered in this category.

$$CN_I = \frac{CN_{II}}{1.819 - 0.00786 \times CN_{II}} \tag{3.57}$$

$$CN_{III} = \frac{CN_{II}}{1.012 - 0.00057 \times CN_{II}} \tag{3.58}$$

In summary this study confirmed that the existing NEH-4 table can be used in the study area for estimating runoff for AMC II with minor adjustment on the curve number. But the NEH-4 conversion table or equations 3.47 through 3.54 underestimate the curve number for AMC I and overestimate for AMC III. Therefore for ungauged catchments in the region having similar watershed features like Laelay Wukro, GumSelassa and Haiba the curve number for AMC II can be adjusted by multiplying with either the average or median Cf_{CN} shown in Table 3-14. Equations 3.55 to 3.58 can be used for AMC I and III accordingly. In addition the parameter range identified through uncertainty analysis (Figure 3-32, Figure 3-33, and Table 3-12) can be used as basis to set the possible range of parameter.

3.6.2.2 Initial abstraction factor

The SCS method described with equation 3-24 and commonly used to estimate runoff assumes the initial abstraction factor (λ) equals 0.2. However recent researches (e.g. Hawkins et al., 2002; Descheemaker et al., 2008) showed that λ can vary from storm to storm and watershed to watershed. The initial abstraction factor is determined from equation 3.23.

$$\lambda = \frac{h_{Ia}}{hs_p} \tag{3.59}$$

Where

λ = initial abstraction factor (1)

h_{Ia} = initial abstraction loss (mm)

hs_p = potential infiltration excluding initial abstraction (mm)

AMC II

Rainfall-runoff modeling of the selected events showed that the median factor is about 0.1977.

Table 3-16: Comparison of initial abstraction factor of NEH-4 table with calibrated value for AMC II condition and summary statistics

Event / λ	E-2	E-6	E-9	E-11	E-17	E-18	λ statistics for all events		
							μ	Median	σ
NH4-table	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0
λ_{cal}	0.202	0.186	0.197	0.198	0.193	0.201	0.1962	0.1977	0.006
Cf_{λ}	1.010	0.93	0.985	0.992	0.965	1.005			

λ_{cal} and Cf_{λ} are the calibrated and calibration factors for initial abstraction factor respectively

The calibration factor for E-6 is relatively smaller compared to other events. This might be attributed to the high rainfall depth (65.2mm) recorded for this event. The SCS-method is sensitive to curve number compared to initial abstraction ratio as discussed in sub-topic 3.3. The PEV and root mean square error (RMSE) for all events were recalculated for each event using the calibrated (CN) and (λ) as 0.2, 0.1977.

$$RMSE = \sqrt{\frac{\sum_{i=1}^{N}(V_o - V_m)^2}{N}} \tag{3.60}$$

Where

$RMSE$:root mean square error (mm)

V_o : observed runoff (mm)

V_m : modeled runoff (mm)

N : number of events

Table 3-17: Comparison of PEV (%) for each event considering λ=0.2 and λ =0.1977

Event / λ	E-2	E-6	E-9	E-11	E-17	E-18
λ=0.2	3.38	4.38	2.54	4.86	0.79	2.88
λ =0.1977	2.41	3.96	1.48	4.06	0.47	2.34

The computed PEV are less than 5% and are considered as acceptable results for all practical applications with λ= 0.1977 giving less computed errors. The calculated RMSE are 0.59mm and 0.53mm for λ=0.2 and λ =0.1977 respectively. Therefore λ=0.2 or λ=0.1977 can be used alternatively for AMC II condition as the differences are not significant.

AMC I and AMC III

The initial abstraction factor varies from one event to another with maximum 0.092 and minimum 0.035 for AMC I condition. However for AMC III its range is 0.096-0.127. The mean values are rounded to 0.05 and 0.112 for AMC I and AMC III respectively. The calculated root means square error using $\lambda = 0.05$ for AMC I and $\lambda = 0.112$ for AMC III are 0.604 mm and 0.273 mm respectively. Since the errors are relatively small the mean values can also represent the AMC I and AMC III events of the calibrated rainfall-runoff events. Woodward et al. (2003) concluded that λ=0.05 fits observed rainfall-runoff data much better than does the handbook value of 0.2, which is inline to the findings of this study for AMC I.

Table 3-18: comparison of Initial abstraction factor of NEH-4 table with calibrated value for AMC I condition and summary statistics

Event / λ	E-1	E-3	E-8	E-10	E-13	E-16	λ statistics for all events	
							μ	σ
NH4-table	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0
λ_{cal}	0.045	0.035	0.092	0.047	0.043	0.036	0.05	0.021

Table 3-19: comparison of Initial abstraction factor of NEH-4 table with calibrated value for AMC III condition and summary statistics

Event / λ	E-4	E-5	E-7	E-12	E-14	E-15	E-19	E-20	λ statistics for all events	
									μ	σ
NH4-table	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0
λ_{cal}	0.11	0.123	0.127	0.104	0.114	0.096	0.113	0.111	0.112	0.01

In summary as a guide line for ungauged catchments in the study area, one can use λ=0.2 or λ=0.1977 for AMC II condition and λ=0.05 for AMC I. The wet moisture condition (AMC III) can be modeled considering λ=0.112. Alternatively the optimum parameter range can be also fixed with ($\mu \pm \sigma$) for each antecedent moisture conditions.

3.6.2.3 Transform

The SCS unit hydrograph is used to transform the excess precipitation into runoff. The shape of the hydrograph can be determined once the basin lag is determined. The basin lag can be accurately determined where there is a recording rain gauge and observed hydrograph within the watershed. In cases where either of them is missing empirical equations or physical based models can be alternative options for determining the basin lag.

The SCS lag formula is one of the commonly used empirical formula for estimating the basin lag for watersheds dominated with agricultural land.

$$t_{lag} = \left[\frac{l^{0.8} \times (hs_P + 1)^{0.7}}{1900 \times S^{0.5}} \right] \times 60 \qquad (3.61)$$

Where

t_{lag} = basin lag (min)

l = maximum watershed length (ft)

hs_P = maximum potential retention (in)

S = median catchment (watershed) slope (%)

Values obtained with equation 3.61 were compared with the calibrated basin lag and are shown in Appendix 3.6.2.3. The SCS lag formula is sensitive to watershed slope. It gives better estimate for events having steep watershed slope. But for watersheds having gentle or mild slopes it over estimate the basin lag which can result very small peak discharge. Thus this research confirmed that the exponent for the watershed slope in the formula can not be a fixed value rather it should be variable according to watershed terrain.

Maintaining the original form of the equation the exponent for the watershed slope was regressed and the following three equations were developed and are proposed for the study area.

Slope category 1: watershed slope less than 5%

$$t_{lag} = \left[\frac{l^{0.8} \times (hs_P + 1)^{0.7}}{1900 \times S^a} \right] \times 60 \qquad [R^2=0.75] \qquad (3.62)$$

Where

a = slope exponent (1) and its value is a=1.5

Slope category 2: watershed slope greater than 5% and less than 8%

Estimated value of a is 0.98

$$t_{lag} = \left[\frac{l^{0.8} \times (hs_P + 1)^{0.7}}{1900 \times S^a} \right] \times 60 \qquad [R^2=0.99] \qquad (3.63)$$

Slope category 3: watershed slope greater than 25%

For this category a=0.52

$$t_{lag} = \left[\frac{l^{0.8} \times (hs_P + 1)^{0.7}}{1900 \times S^a} \right] \times 60 \qquad [R^2=0.64] \qquad (3.64)$$

Slope category 4: watershed slope greater than 8 and less than 25%

For slopes (8-25%) the exponent for the watershed slope can be derived from equation 3.65 with additional field verification like flood mark levels.

$$a = 2.4844 \cdot S^{-0.489} \qquad (3.65)$$

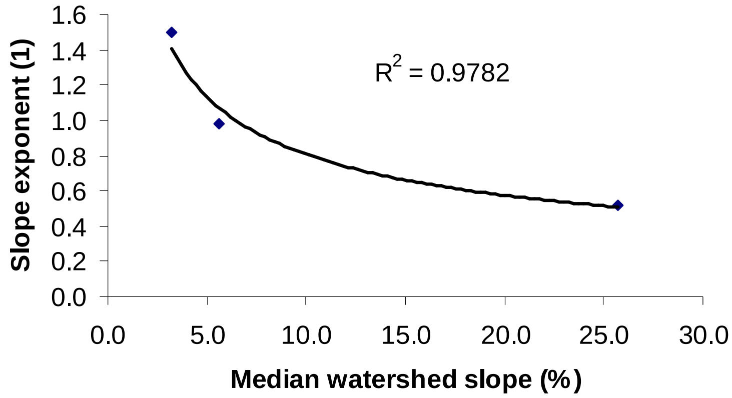

Figure 3-37: variation of slope exponent with watershed median slope

3.6.2.4 Modeling sub-watersheds

The observed flows used for each event analysis are measured at the outlet of each watersheds and parameter estimation were so far carried out for the average watershed value. But with knowledge of the watershed soil and land use it is possible to develop a semi distributed model by using calibration factors developed from total watershed modeling. The factor can be considered constant for all sub-watersheds or allowing few percentage variations based on prior knowledge of the hydrology process of the study area and parameter sensitivity. In this research attempts were made to extract valuable information from existing datasets by using proven scientific concepts and own experiences in the process of modeling. This procedure is useful to evaluate the rainfall-runoff process at the sub-watershed level based on the information developed at the outlet.

The procedures followed in this research are outlined here:

i. calibrate and determine optimum parameters for the total watershed for each rainfall-runoff events,
ii. fix calibration factors for each parameter: the calibration factors for each parameter were developed by diving the calibrated parameter with initial model parameter value. The initial value for CN and λ are obtained from NEH-4 table. For the basin lag equations developed in this study equation 3-62 to 3.65 were used to set the initial parameter value,
iii. divide the total watershed into sub-watersheds based on soil and landuse similarity. The sub-watersheds belong to the same stream network group,

iv. assign parameter value: assign initial parameter value for each watershed based on step two and modify the parameter value by multiplying with the calibration factor for corresponding parameter,

v. fix channel routing parameters: the routing parameters are fixed based on the procedures outlined in chapter 3.2.1.1, channel routing, and

vi. run HEC-HMS model for each rainfall-runoff event and compare the performance similar to methods outlined in chapter 3.2.2.2.

Figure 3-38 shows the model setup taking an example for Laelay Wukro sub-watersheds. The computed runoffs errors are within allowable range of volume difference (0.23 to 3%). 90% of the modeled rainfall-runoff events do have Nash-Sutcliffe model efficiency greater than 0.80 and 55% of them Nash-Sutcliffe greater than 0.9. Thus it is possible to conclude that if the only measuring facility is at the outlet it is possible to evaluate rainfall-runoff process at sub-watershed level following the procedures outlined above. The resulting hydrographs after simulation and optimization are shown from Figure 3-39 through Figure 3-41.

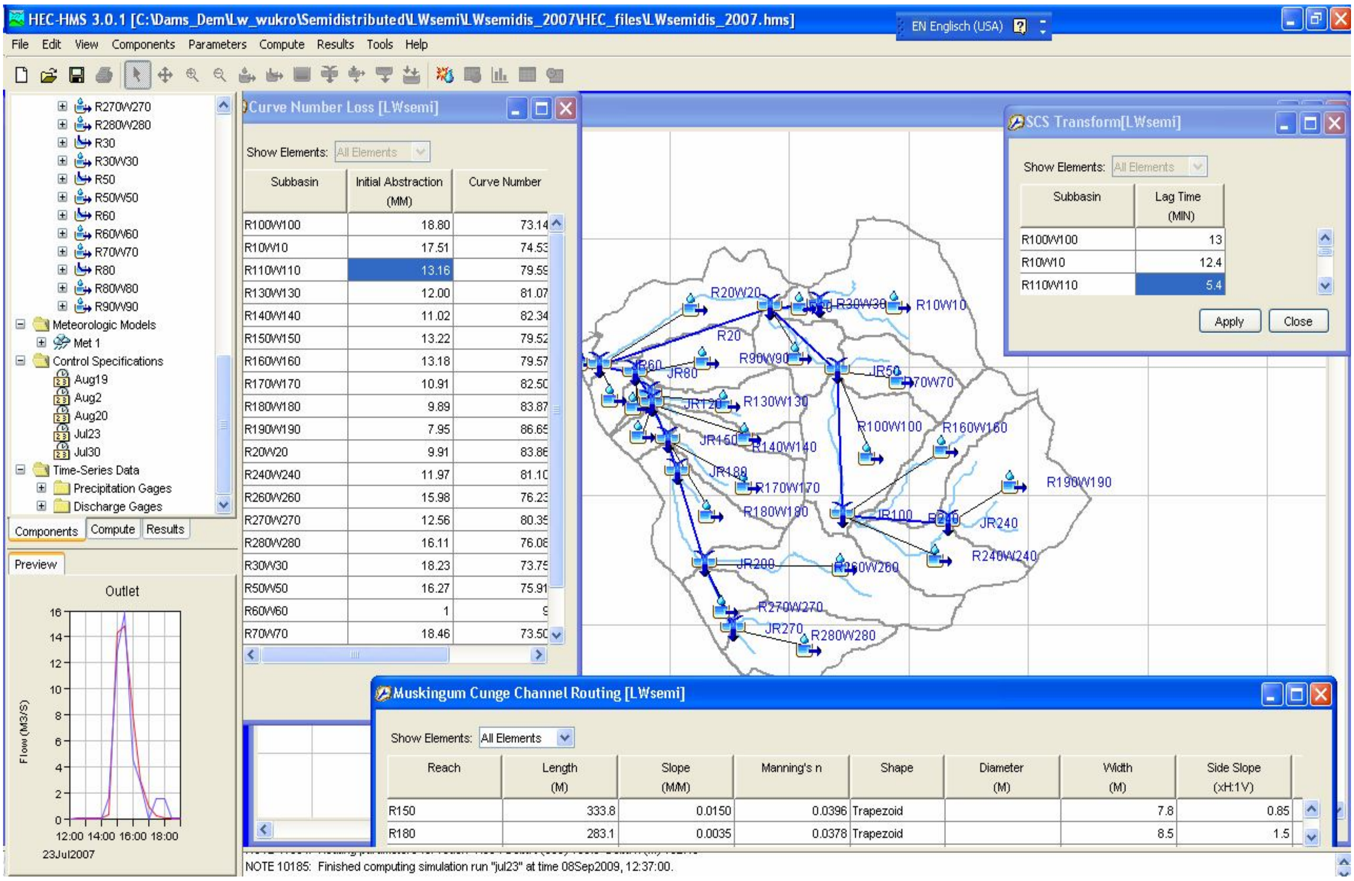

Figure 3-38: sub watershed model and input parameters for Laelay Wukro

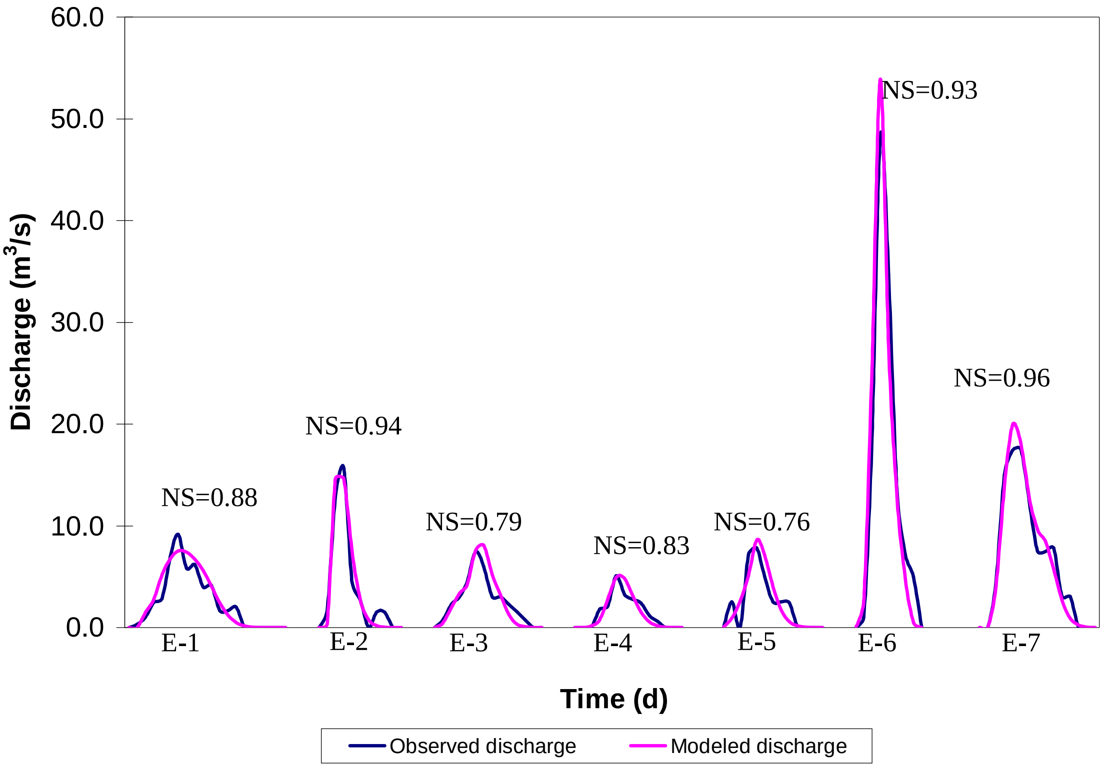

Figure 3-39: Observed and modeled hydrographs for Laelay Wukro watershed

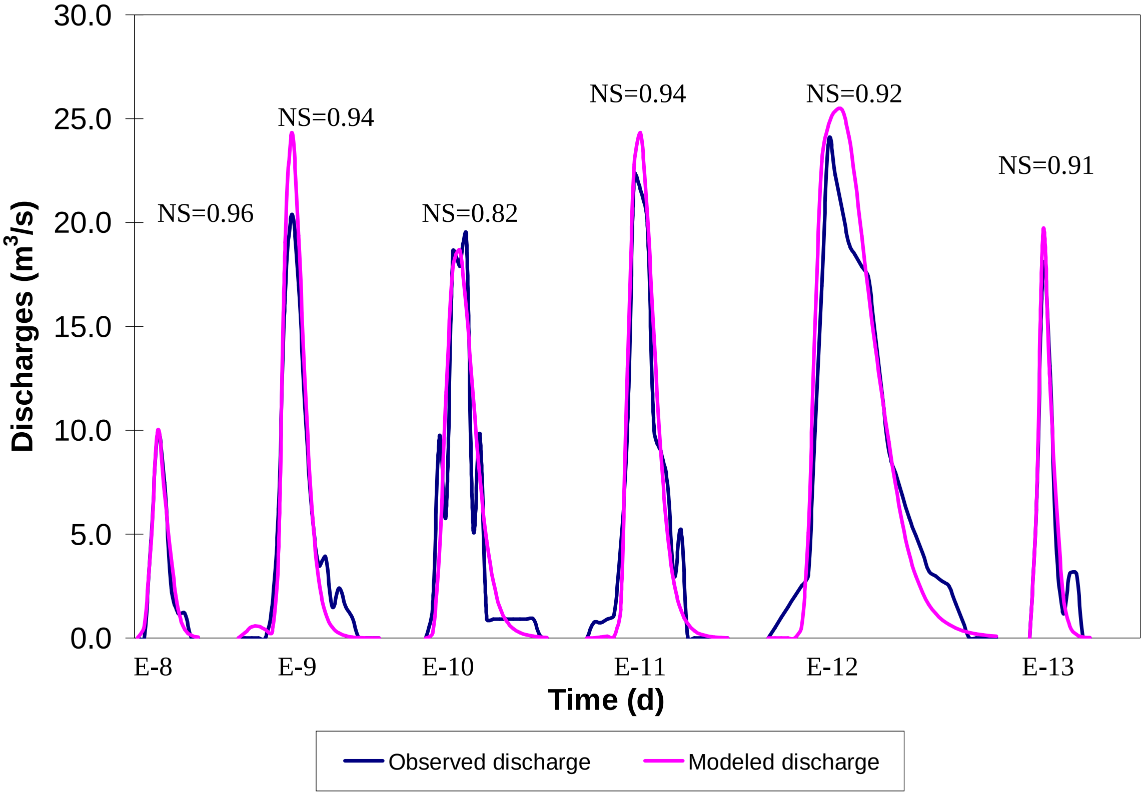

Figure 3-40: Observed and modeled hydrographs for GumSelassa watershed

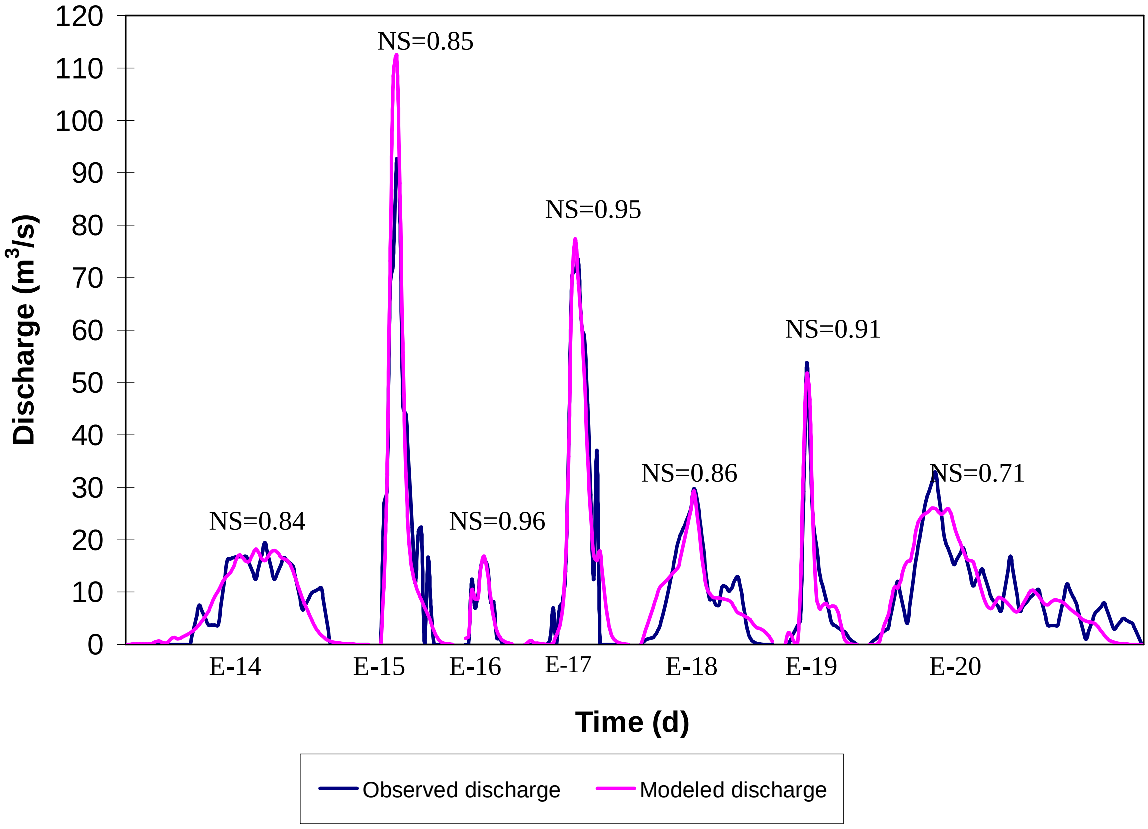

Figure 3-41: Observed and modeled hydrographs for Haiba watershed

3.6.3 Monthly water balance approach

The water resources of a given watershed can be also evaluated with the water balance approach considering different input parameters. Water balance models can be developed at various time scales (e.g. hourly, daily, monthly and yearly) and to varying degrees of complexity (Xu et al., 1998). The larger scales like daily and hourly analysis require more intensive data and the constructed models can have many parameters compared to the monthly or yearly water balance models. The monthly water balance models can be used to address a range of hydrological problems and assessment of climate change impacts (Xu and Singh, 1998; Xiong and Guo, 1999; McCabe and Markstrom, 2007).

A water balance model is developed with precipitation, evapotranspiration and soil moisture storage as model variables. The schematic representation of the developed model is shown in Figure 3-42.

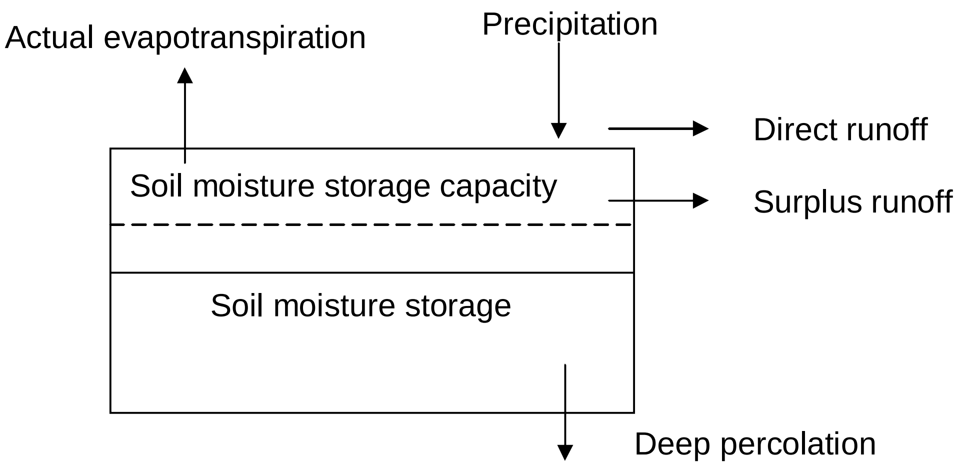

Figure 3-42: Schematic representation of monthly water balance model (modified after McCabe and Markstrom., 2007)

The total runoff at the watershed outlet is the summation of the direct runoff and surplus runoff

$$h_{Vm} = h_{DR} + h_{SR} \qquad (3.66)$$

Where

h_{Vm} = modeled runoff (mm)

h_{DR} = direct runoff (mm)

h_{SR} = surplus runoff (mm)

3.6.3.1 Direct runoff component

The direct runoff is resulted from infiltration excess overland flow or impervious surfaces. It is expressed as a certain percentage of the total rainfall. The part of rainfall that contributes immediately to the direct runoff should be deducted before analyzing the actual evapotranspiration.

$$h_{DR} = h_{P\,total} \times DR_{frac} \qquad (3.67)$$

Where

h_{DR} = direct runoff (mm)

$h_{P\,total}$ = total precipitation (mm)

DR_{frac} = direct runoff fraction (1)

3.6.3.2 Surplus runoff

The surplus runoff is generated after meeting the soil moisture storage and the actual evapotranspiration. The difficult aspect in the determination of the surplus runoff is estimation of the actual evapotranspiration. For this research the soil moisture balance approach is used to estimate actual evapotranspiration on monthly basis (McCabe and Markstrom, 2007, Steenhuis et al. (in press)).The required parameters to determine actual evapotranspiration are mean monthly rainfall, mean monthly potential evapotranspiration, water holding capacity of the dominant soil type and monthly soil moisture storage. The landuse and soil maps of each watershed were aggregated to get a combined landuse and soil for each watershed. Depending on the soil type, the available soil moisture per depth for each landuse was fixed, considering an acceptable rooting depth for the corresponding landuse. The monthly soil moisture was determined with the assumption that soil moisture storage withdrawal linearly decreases with decreasing soil moisture, i.e. as the soil becomes dry it becomes more difficult to remove water from the soil and hence less water is available for actual evapotranspiration.

$$h_{SM\,i} = h_{SM\,i-1} - \left\{ \left| h_{P\,i} - h_{ETp\,i} \right| \times \frac{h_{SM\,i-1}}{h_{TAM}} \right\} \qquad (3.68)$$

Where

$h_{SM\,i}$ = soil moisture content at month, i (mm)

$h_{SM\,i-1}$ = soil moisture content of the previous month (mm)

$h_{P\,i}$ = precipitation at month, i (mm)

$h_{ETp\,i}$ = potential evapotranspiration at month, i (mm)

h_{TAM} = available soil moisture content for a given soil type (mm)

Equation 3.68 is the general equation for estimating the soil moisture for most of the months in the year. But for months where precipitation exceeds the potential evapotranspiration the soil moisture is estimated with the following equations.

$$h_{SM\,i} = \left\{ h_{P\,i} - h_{ETp} \right\}_i + h_{SM\,i-1} \qquad \left\{ h_{SM} < h_{TAM} \right\} \qquad (3.69)$$

$$h_{SM\,i} = h_{TAM} \qquad \left\{ h_{P\,i} - h_{ETp} \Big|_i + h_{SM\,i-1} > h_{TAM} \right\} \qquad (3.70)$$

Potential evapotranspiration will be equal to actual evapotranspiration when the monthly rainfall is greater than the potential evapotranspiration. However for dry months where potential evapotranspiration exceeds the monthly rainfall it will be the summation of the monthly precipitation and the soil moisture storage difference for that particular month. The actual evapotranspiration of the total watershed is determined by adding the weighted actual evapotranspiration for a single unit having a specific landuse and soil.

$$h_{ETac\,i} = h_{ETp\,i} \qquad \{h_{P\,i} > h_{ETp\,i}\} \tag{3.71}$$

$$h_{ETac\,i} = h_{P\,i} + \Delta h_{SM\,i} \qquad \{h_{P\,i} < h_{ETp\,i}\} \tag{3.72}$$

$$h_{ETac(\text{watershed})} = \frac{\sum_{i}^{n}(h_{ETac\,i} \times Ac_i)}{\sum_{i}^{n} Ac_i} \tag{3.73}$$

Where

$h_{ETac(watershed)}$ = actual evapotranspiration of total watershed (mm)

$h_{ETac\,i}$ = actual evapotranspiration of a single unit with specific soil and landuse(mm)

Ac_i = area of a single watershed area with specific soil and landuse (km^2)

3.6.3.3 Runoff estimation

The runoff process computation was done in iterative way by first assuming direct runoff fraction to determine the direct runoff. Then with the remaining rainfall the actual evapotranspiration and surplus runoff is computed for each month where the soil moisture exceeds its moisture holding capacity. The summation of both direct runoff and surplus runoff is compared with the monthly observed runoff. The process will continue till the observed volume is equal to the simulated volume.

The surplus runoff can only happen when the soil is completely saturated. The soil moisture in excess of the saturation can be available as surplus runoff or lost as deep percolation, which can be considered as permanent loss from the watershed.

$$h_{SR} = SR_{frac} \times \{(h_P - h_{ETact})_i - \Delta h_{SM\,i}\} \qquad [h_P > (h_{ETact} + \Delta h_{SM})] \tag{3.74}$$

Where

SR_{frac}: surplus runoff fraction (1)

Details of the estimation process are shown in Appendix 3.6.3. The computed actual evapotranspiration significantly varies equalling the potential evapotranspiration during rainy season, July and August. It can be as low as less than 1% of the potential evapotranspiration during dry months, where the soil moisture is almost dry. Comparing annually for year 2007, the annual actual evapotranspiration is about 26% and 32% of the potential evapotranspiration for GumSelassa and Laelay Wukro respectively.

McCabe and Markstrom(2007) stated that the direct runoff fraction is specified based on previous year water balance analyses and 0.05 is typical value as reported in Wolock and McCabe (1999). Meanwhile this study showed that the direct runoff fraction is not uniform. It varies with the watershed characteristics and the storm condition. GumSelassa which is characterized with impervious soils and dominantly agricultural landuse resulted high fraction when the rainfall intensity was high even during dry soil moisture condition. In contrast Laelay Wukro watershed mostly yields relatively low direct runoff fraction as 58% of the watershed is covered with exclosure and bushes. However when two consecutive rains follow one another in 2008 (Jul25 and July 26) the direct runoff fraction is found to be high resulting from soil wetness. On moist or wet soils, which can be saturated during a short burst of rainfall, immediate saturation overland flow (SOF) is expected (Naef et al., 2002).

The surplus runoff fraction for GumSelassa in 2007 and 2008 was found to be zero. The former year rainfall was nearly equal to annual average rainfall but it was not sufficient enough to meet the actual evapotranspiration and exceed the soil moisture holding capacity of the dominant landuse. However 2008 rainfall season was far below average and thus surplus runoff is not expected. For Laelay Wukro watershed the rainfall season in 2007 was above average and surplus runoff was determined assuming 50% of the surplus water will be available for runoff and the remaining 50% will be lost through deep percolation($h_{D_{per}}$) to recharge the groundwater.

Table 3-20: Water balance analysis summary for Laelay Wukro watershed

Year	Month	h_{Ptotal} (mm)	DR_{frac} (1)	h_{DR} (mm)	h_P (mm)	Δh_{SM} (mm)	h_{ETact} (mm)	$h_{D_{per}}$ (mm)	SR_{frac} (1)	h_{SR} (mm)	h_{Vm} (mm)
2007	July	285.2	0.032	9.1	276.1	148.3	108.9	9.4	0.5	9.4	18.5
	August	210.4	0.043	9.0	201.4	26.35	112.76	31.1	0.5	31.1	40.1
2008	July	231.7	0.222	51.3	179.9	66.9	112.65	0.17	0.5	0.17	51.47
	August	143.2	0.05	9.60	133.6	21.6	108.3	1.83	0.5	1.83	11.43

ΔSM *:part of precipitation added to the soil moisture storage*

Table 3-21: Water balance analysis summary for GumSelassa watershed

Year	Month	h_{Ptotal} (mm)	DR_{frac} (1)	h_{DR} (mm)	h_P (mm)	Δh_{SM} (mm)	h_{ETact} (mm)	$h_{D_{per}}$ (mm)	SR_{frac} (1)	h_{SR} (mm)	h_{Vm} (mm)
2007	July	219.6	0.257	56.00	163.16	46.06	116.19	0.67	0.5	0.67	56.67
	August	197.0	0.16	31.72	165.09	43.1	119.67	1.23	0.5	1.23	32.95
2008	July	113.2	0.0035	0.4	112.80	6.2	106.6	0.0	0.0	0.0	0.4
	August	95.2	0.021	2.0	93.2	-6.2*	99.4	0.0	0.0	0.0	2.0

* (-ve) sign indicates the soil moisture from previous month is consumed by evapotranspiration

The developed water balance is found to be useful for estimating the actual evapotranspiration and simulate runoff for each month. The method is easily adaptable and requires less input data. It can be used for planning purposes and clearly see the different hydrologic cycle components (like potential evapotranspiration, actual evapotranspiration, surface runoff and percolation into ground water). The data are mainly generated from existing maps or newly produced maps (soil and landuse) after some field work and laboratory analysis. Potential evapotranspiration and rainfall are the basic input data.

The parameters which are required to be estimated are the direct runoff and the surplus runoff fractions. The direct runoff can be calibrated if observed flow is available. With the number of events used in this study it was difficult to conclude the fraction that can be used for ungauged catchments in the region. But the following information can be used as a starting guideline for fixing the direct runoff fraction.

Watersheds dominated with cultivated land and impervious soils

- Within a given month if 2 or more rainfall events with AMC III condition are recorded immediate saturation overland flow can dominate the runoff process. In such cases DR_{frac} =0.20 - 0.25 can be used

- When AMC I and AMC II dominate the month DR_{frac} = 0.10 - 0.15 can be considered as starting values
- For dry years and months dominated with no rainfall events DR_{frac} is generally very low. The DR_{frac} =0.0 - 0.02 can be considered.

Watersheds dominated with bushes and exclosure

- In most cases DR_{frac} is less than 0.05. But it can be in the range of 0.15 - 0.20 when two or more events follow one another resulting AMC III conditions and soil wetness.
- For dry years and months dominated with no rainfall events, DR_{frac} = 0.0 - 0.02 can be considered as starting values.

The surplus runoff is mostly observed when the seasonal rainfall is above average. Obviously when it is dry year the surplus runoff can be considered nearly to zero. Knowledge of the soils hydraulic conductivity, landscape, depth of water table and local geology can be useful indicators while selecting the surplus runoff fraction. Shallow depth and impervious soils can have SR_{frac} greater than 0.5. With the availability of additional data from different watersheds in the future, the surplus runoff fraction can be regionalized. For the watersheds understudy SR_{frac} =0.5 seems to give acceptable results. Thus in the absence of better data for ungauged catchments having similar watershed features like Laelay Wukro and GumSelassa one can use SR_{frac} =0.5 with the assumption that 50% of the surplus runoff will be lost through deep percolation.

4. RUNOFF COEFFICENT GENERATION AND DEVELOPMENT

The runoff coefficient (C) used in the lumped model during planning and design of all micro-dam irrigation projects was adapted from literature based on soil, landuse and slopes of the watershed. One seasonal value is considered for potential assessment. The comparative assessment made in this study showed that the synthesized seasonal runoff coefficient (from rainfall and runoff records) is low compared to the value adapted during design as reported in Table 2-3. Therefore in this research from the rainfall-runoff records of the three monitoring stations a new and more applicable runoff coefficients were developed that can be used for planning and design purposes in the region. The approach followed in this research was to determine the event, decadal (10days), seasonal runoff coefficients and finally propose new set of runoff coefficients in relation to rainfall, antecedent soil moisture conditions and watershed features.

4.1 Event analysis

Different rainfall-runoff events have been considered during the rainy season. Tables 4.1, 4.2 and 4.3 show summarized information from the rainfall measurement and water balance analysis model of each reservoir. The runoff coefficient (C) is the ratio of the observed inflow volume to the corresponding measured rainfall. Not all rainfall events occurred in the watershed will generate runoff to the outlet. Usually the runoff events recorded during AMC I are either from big rainfall depths or high rainfall intensity exceeding the soil infiltration capacity resulting direct runoff. The rainfall events were grouped according to watershed for better visualization of runoff coefficients among watersheds.

Table 4.1: Event based analysis for Laelay Wukro watershed

Designation	AMC	Event inflow (mm)	Event rainfall (mm)	Runoff coefficient
E-1	AMC I	4.52	27.4	0.16
E-2	AMC II	7.55	39.2	0.19
E-3	AMC I	6.71	32.0	0.21
E-4	AMC III	3.58	20.4	0.18
E-5	AMC III	5.59	26.2	0.21
E-6	AMC II	24.34	65.28	0.37
E-7	AMC III	17.87	46.23	0.39

The maximum runoff coefficient observed was 0.39 when the watershed was wet i.e. at AMC III. The second highest runoff coefficient, 0.37 for AMC II condition is almost double to the runoff coefficient obtained at event E-2 having similar antecedent moisture condition. This is due to the high rainfall depth recorded during this event. This entails the runoff coefficient depends on the rainfall depth and the antecedent moisture conditions.

Table 4-2: Event based analysis for GumSelassa watershed

Designation	AMC	Event inflow (mm)	Event rainfall (mm)	Runoff coefficient
E-8	AMC I	0.97	16.0	0.06
E-9	AMC II	3.33	18.7	0.18
E-10	AMC I	3.46	20.8	0.17
E-11	AMC II	4.22	21.5	0.20
E-12	AMC III	3.09	13.2	0.23
E-13	AMC I	1.79	15.0	0.12

The runoff coefficients observed at GumSelassa were low resulting from low rainfall depths recorded during two years analysis period. The runoff coefficient generally increases as the watershed soil moisture increases during the rainy season. The high runoff coefficient value for AMC I with low rainfall depth observed from E-13 compared to E-8 was mainly caused by the maximum rainfall intensity ($\dot{h}_{i\max} =$103mm/h) occurred during this particular event.

Table 4-3: Event based analysis for Haiba watershed

Designation	AMC	Event inflow (mm)	Event rainfall (mm)	Runoff coefficient
E-14	AMC III	6.08	19.0	0.32
E-15	AMC III	21.61	38.2	0.57
E-16	AMC I	3.37	19.8	0.17
E-17	AMC II	19.35	44.4	0.44
E-18	AMC II	10.75	34.0	0.32
E-19	AMC III	8.56	21.0	0.41
E-20	AMC III	10.47	25.3	0.41

Unlike GumSelassa, the runoff coefficients observed at Haiba watershed were high resulting from high rainfall events recorded during events E-15 and E-17/E-18 for AMC III and AMC II respectively**.** Therefore for watersheds dominated with cultivated land and hydrologic soil group D (like Haiba) about 57% of the measured rainfall depth could be changed into runoff when the watershed was wet. It is also interesting to see that comparing E-14 and E-16 for almost similar rainfall depth the runoff coefficients for AMC III is about 1.88 times the runoff coefficient for AMC I. Also the same runoff coefficients could be observed for AMC III condition with rainfall depth about only 56% of the rainfall depth for AMC II condition. This generalization might not be always true but they are helpful to stress the dependability of runoff coefficient with depth of rainfall and antecedent moisture condition while selecting appropriate runoff coefficients.

4.2 Decadal analysis

The decadal analysis was done based on the rainfall records measured within 10 days and the observed runoff obtained from water balance analysis of each reservoirs. During the rainy season the runoff coefficient increased from one decade to the next decade with increasing catchment soil moisture. At the start of the rainy season i.e. the first two decades in July the runoff coefficient was very low, but it could also be as high as 0.29 if rainfall events with relatively high magnitude happed in consecutive days (July 25/2008 and July 26/2008) which was the case in D-9 Table 4-4.

Table 4-4: Decadal analysis Laelay Wukro watershed

Designation	Date	Year	Decadal inflow (mm)	Decadal rainfall (mm)	Runoff coefficient
D-1	1-10July	2007	0.327	50.2	0.006
D-2	11-20July	2007	2.218	76.4	0.029
D-3	21-31July	2007	15.72	158.6	0.10
D-4	1-10August	2007	8.006	84.4	0.095
D-5	11-20August	2007	23.213	106.2	0.218
D-6	21-31August	2007	2.081	19.8	0.105
D-7	1-10July	2008	0.019	30.4	0.0006
D-8	11-20July	2008	0.317	25.2	0.01
D-9	21-31July	2008	51.03	175.6	0.29
D-10	1-10August	2008	9.84	108.8	0.09
D-11	11-20August	2008	1.33	34.0	0.039
D-12	21-31August	2008	0.0	0.4	0.00

The runoff coefficient could be low during August when the numbers of days with out rainfall were high and the recorded rainfall depths are low (D-10 and D-11) in Table 4-4.

Table 4-5: Decadal analysis GumSelassa watershed

Designation	Date	Year	Decadal inflow (mm)	Decadal rainfall (mm)	Runoff coefficient
D-1	1-10july	2007	2.67	43.6	0.0613
D-2	11-20july	2007	14.1	85.4	0.1652
D-3	21-31july	2007	39.84	90.6	0.4397
D-4	1-10August	2007	13.03	67.0	0.1945
D-5	11-20August	2007	11.2	63.2	0.1772
D-6	21-31August	2007	8.79	66.8	0.1315
D-7	1-10july	2008	0.377	66.8	0.0056
D-8	11-20July	2008	0.0	23.8	0.0
D-9	21-31July	2008	0.0	22.6	0.0
D-10	1-10August	2008	0.0	27.4	0.0
D-11	11-20August	2008	0.0	29.6	0.0
D-12	21-31August	2008	2.146	38.2	0.056

The high runoff coefficient in D-3 of Table 4-5 was attributed to the nearly continuous rainfall events recorded from July 29 though July 31. Low runoff coefficients observed in 2008 are resulted from low rainfall recorded during each decade.

4.3 Seasonal analysis

The runoff coefficient was estimated considering the rainfall-runoff events from July – August, which were basically the main rainy months in the region. The estimated seasonal runoff coefficient for GumSelassa was higher than Laelay Wukro watershed as shown in Table 4-6.

Table 4-6 Seasonal runoff coefficient comparison for GumSelassa and Laelay Wukro watersheds

Watershed	Year	Seasonal inflow (mm)	Seasonal rainfall(mm)	Seasonal runoff coefficient(1)	Designed runoff coefficient*(1)
Laelay Wukro	2007	58.40	495.6	0.12	0.329
	2008	62.92	374.9	0.17	0.329
GumSelassa	2007	89.63	416.6	0.21	0.3
	2008	2.523	208.4	0.012	0.3

= COSAERT respective design document

The seasonal runoff coefficient, C observed for Laelay Wukro was by far less than the design estimate. The maximum runoff coefficient observed during the analysis periods was about 52% of the design estimate. For GumSelassa watershed the runoff coefficient for wet year was about 70% and about 4% of the design estimate for dry year. The high runoff coefficient with lesser rainfall depth for Laelay Wukro watershed in 2008 was resulted from runoff recorded in D-9 of Table 4-4 accounting 81.1% of the total observed runoff during that year.

In summary, GumSelassa watershed resulted higher runoff coefficient compared to Laelay Wukro. This low runoff coefficient for Laelay Wukro was attributed to the part of the watershed covered with bush land, exclosure and grazing land. In addition every year biological and physical conservation measures were in place in the watershed that would increase the soil infiltration opportunity time, and thus decrease the surface runoff. Unlike Laelay Wukro, more than 90% of GumSelassa watershed is cultivated land with fine textured soils which are less impervious and can generate more runoff.

4.4 Evaluation of event, decadal and seasonal runoff coefficient analysis

Comparison of the different analysis methods showed that the runoff coefficient is dependent on depth of rainfall, antecedent moisture condition and watershed characteristics such as landuse and soil type (hydrologic soil group). Not all rainfall events recorded generate runoff specially when the watershed is dry or the numbers of days without rain are high. As can be seen from Table 4-4 and Table 4-5, the decadal runoff coefficient was variable across the decades and it is dependent on the individual rainfall events occurred during that particular decade. As a result it is not possible to fix a certain value for a specific decade. Thus regionalization of decadal runoff coefficient was found not realistic

The existing method being adapted in the region considers a single runoff coefficient value and dependable rainfall for annual runoff estimation. This method has got two draw back:

i. runoff coefficient can not be a single value since it varies with rainfall depth and antecedent moisture condition of the watershed, and
ii. the method considers a lumped rainfall depth, but in reality only limited rainfall events can only generate runoff. This can be clearly observed on the results of GumSelassa watershed for 2007 and 2008.

In contrast to the decadal and seasonal analysis, the event based analysis can minimize the draw backs sited above and can give better runoff coefficient estimation. The event based analysis is superior to other methods because:

i. it identifies the events which only generate runoff and thus rainfall below a specified amount (Chapter 4.5) will not generate runoff but can contribute to the soil moisture storage,
ii. it considers the effect of antecedent moisture condition and is high for wet antecedent moisture condition and low for dry condition, and
iii. it accounts the variation of runoff coefficient with changes in rainfall depth for a given antecedent moisture condition.

Thus for these reasons a new event based runoff coefficients are proposed for the study area that considers

- rainfall depth,
- antecedent moisture conditions, and
- watershed characteristics

The watershed characteristics are expressed with land use land cover and hydrologic soil group (HSG).

4.5 Proposed new runoff coefficients

The proposed runoff coefficients were developed from analysis of rainfall-runoff events outlined in previous chapters. The outcomes of the sub-watershed modeling discussed in chapter 3 are the basis for estimation of the newly proposed runoff coefficients. In line to the previous chapters discussions GumSelassa and Haiba watersheds will represent agricultural land uses and hydrologic soil group D. Where as the runoff coefficients generated from Laelay Wukro will represent mixed land uses mainly composed of cultivated lands, bushes, area exclosures and grazing lands. Different working graphs developed from each sub-watershed analysis were reported from Figure 4-1 though Figure 4-3 that would give an alternative options for selecting suitable runoff coefficients for a given watershed.

The rainfall depths indicated for each curve corresponds to a single rainfall-runoff event. One can easily determine the runoff coefficient for similar rainfall depths shown on the graphs for given AMC and CN value. For rainfall depth other than the stated depths it is possible to interpolate in between two curves as the curves are nearly parallel. Besides for the given rainfall depth the curve can be extended for lower and higher curve numbers within a given antecedent moisture condition. The fitted equations that can be used for interpolating and extending the plotted curves are shown on Appendix 4.5.

The required inputs for estimation of runoff coefficients are the rainfall depth, AMC condition, curve numbers, hydrologic soil group and landuse. Once the runoff coefficients are selected runoff can be estimated using equation ($V = 1000 \times C \times h_P \times A_C$) in which h_P in (mm) and A_c in (km^2).The following steps can be used as a guide line for estimation of runoff:

i. determine daily rainfall from daily records. It is advisable to use many years recording for better result,
ii. determine the antecedent moisture condition for each event,
iii. according to the rainfall-runoff analysis of the three watersheds, for AMC I conditions, all rainfall events less than 15mm will not generate runoff and thus the runoff coefficients is zero for AMC I less than 15mm rainfall,
iv. determine the curve number of the watershed based on AMC, landuse and hydrologic soil group. The curve number must be inline with the outcomes of this research discussed in chapter three,
v. for each event read runoff coefficient (C) from respective graph which corresponds the rainfall depth, AMC and watershed features,
vi. determine the runoff volume with equation $V = 1000 \times C \times h_P \times A_C$ for each event, and
vii. the decadal, monthly and seasonal runoff can be fixed by adding each event within the decade, month and season respectively.

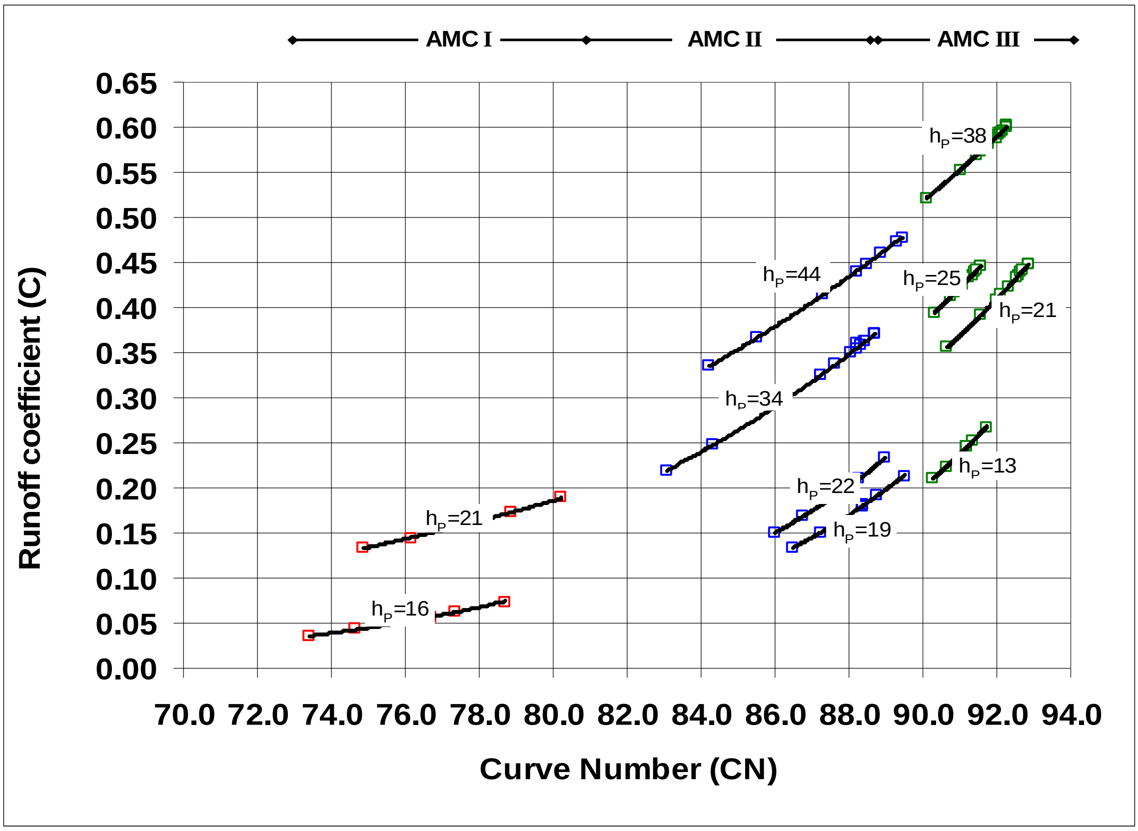

Figure 4-1: Runoff coefficients for cultivated land with hydrologic soil group D for variable rainfall depths (mm) and different AMC

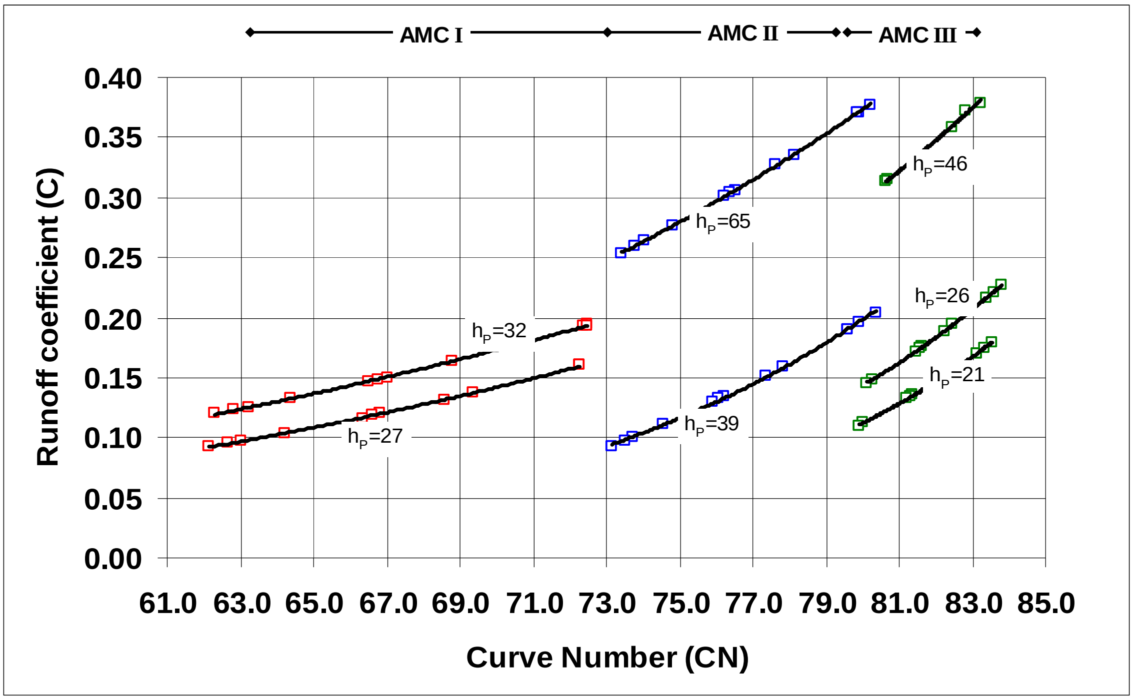

Figure 4-2: Runoff coefficients for variable rainfall depths and different AMC (Mixed landuse cultivated lands constitute up to 50% of the total watershed area

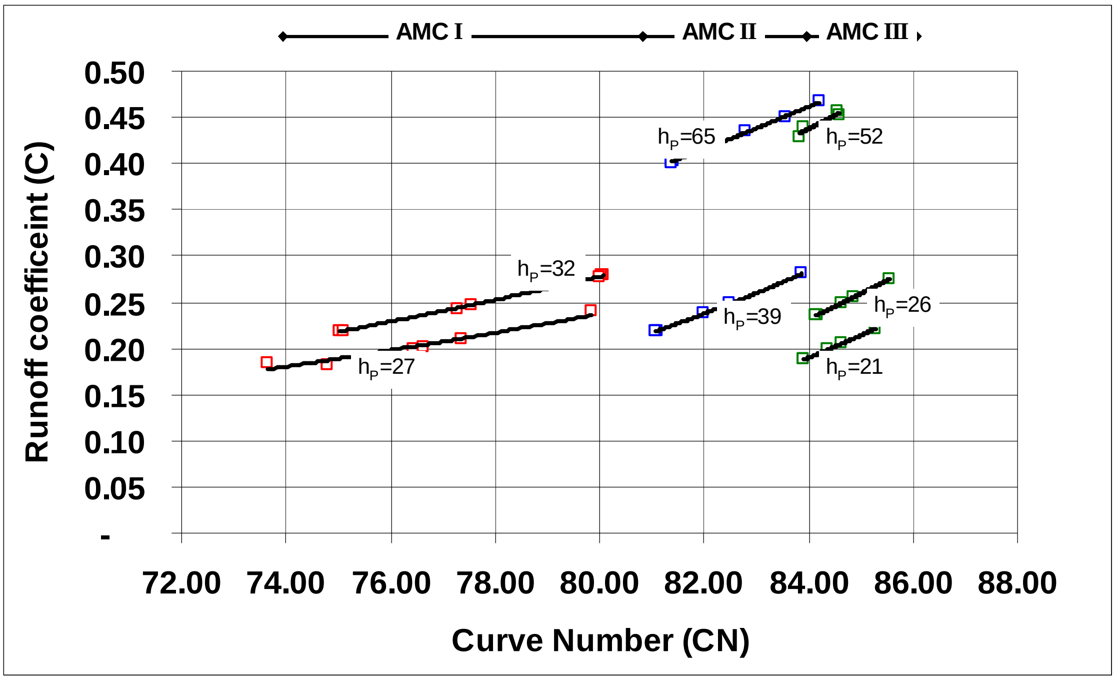

Figure 4-3: Runoff coefficients for variable rainfall depths and different AMC (Mixed landuse cultivated lands constitute more than 50% of the total watershed area)

The runoff coefficients for AMC I (h_P =27mm and h_P =32mm) in Figure 4-2 and 4-3 are greater than AMC II (h_P =39mm). This situation happened because λ_{cal} for AMC I is very much less than λ_{cal} for AMC II as shown in Table 4-7. The smaller λ is the smaller initial abstraction loss. Therefore rainfall in excess of loss can generate more direct runoff. The generated runoff is divided by the total rainfall to determine runoff coefficient, which resulted less C values for E-2. So for better results and good estimation of runoff coefficients one has to stick with the findings of this research concerning the initial abstraction ratio and curve numbers for the three different antecedent moisture conditions.

Table 4-7: Selected features of rainfall-runoff events

Event	AMC	h_P (mm)	λ_{cal} (1)	$h_{Ia_{cal}}$ (mm)
E-1	AMC I	27	0.045	4.5
E-3	AMC I	32	0.035	3.5
E-2	AMC II	39	0.20	13.4

Figures 4.1 to Figure 4.3 are useful tools to determine the runoff coefficients with respect to curve numbers, watershed landuse and soil properties. In addition to this

attempts were also made to derive the runoff coefficients according to different landuse groups and hydrologic soil groups. The runoff coefficients from a single sub-watershed having more than one landuse are proportioned based on each landuse contribution to the total area. Runoff coefficients generated from sub-watersheds covered with a single landuse and soil group will be considered as an input to derive runoff coefficients from other sub-watersheds. The major landuse land cover and hydrologic soil groups found in the three monitoring stations were used as basis for classification.

The developed runoff coefficients are reported in Table 4-8. Similar to previous discussions the runoff coefficient increases with AMC and cultivated lands generated more runoff compared to other land uses, which is inline to the findings of Girmay et al. (2009). Bush lands, grass or exclosure did not generate runoff from smaller rainfall depths due to interception.

Table 4-8: Runoff coefficients for different land uses and hydrologic soil groups with respect to AMC and variable rainfall depths

Landuse	AMC	AMC I					AMCII						AMCIII					
	P (mm) / HSG	16	20	21	28	32	19	22	34	39	44	65	13	19	20	25	26	38
Cultivated land	D	0.06	0.18	0.17			0.18	0.2	0.37		0.49		0.25	0.34		0.45		0.6
	C	0.02		0.11	0.20	0.25	0.1	0.12		0.25		0.45	0.2		0.21		0.27	
Bush land/ Forest/ Enclosure	D	0.0	0.08	0.09			0.0	0.0	0.21		0.33		0.02	0.25		0.34		0.5
	C	0.0			0.09	0.13	0.0	0.0		0.09		0.25	0.0		0.11		0.14	
Homesteads/ Miscellaneous	D		0.14	0.1			0.1	0.15	0.31		0.43		0.2	0.3		0.41		0.56
	C		0.08						0.19		0.31			0.22		0.32		0.49
Grazing land	D	0.0	0.04						0.09		0.19			0.15		0.25		0.42
	C	0.0	0.03		0.04	0.06			0.03	0.06	0.12	0.22		0.08	0.09	0.17	0.13	0.32
Bare land/ Fallow	D	0.1		0.23			0.27	0.29					0.3					
	C	0.0		0.06				0.1										

Note=

The values indicated in Table 4-8 are concluded from the down scaling analysis of the calibrated rainfall-runoff events for each sub-watershed. The existing data sets for bare land are small in number, but one can take the runoff coefficients for cultivated lands provided that the area of bare land is small compared to the total watershed area. For rainfall depths other than listed in the table interpolation can be made where possible with good engineering judgment. Curve fittings prepared from data sets of Table 4-8 are shown in Appendix 4.6 and the corresponding equations are used for data filling and reproduce Table 4-9.The runoff coefficients for homesteads and grazing land, AMC I condition can be proportioned or adjusted following the trend in AMC II for respective landuse and rainfall depth.

Table 4-9: Re-produced runoff coefficients for different land uses and hydrologic soil groups with respect to AMC and variable rainfall depths

	AMC	AMC I					AMCII						AMCIII					
Landuse	P (mm) / HSG	16	20	21	28	32	19	22	34	39	44	65	13	19	20	25	26	38
Cultivated land	D	0.06	0.18	0.17	0.35	0.45	0.18	0.2	0.37	0.42	0.49		0.25	0.34	0.35	0.45	0.45	0.6
	C	0.02	0.04	0.11	0.20	0.25	0.1	0.12	0.21	0.25	0.28	0.45	0.2	0.22	0.21	0.25	0.27	0.3
Bush land/ Forest/	D	0.0	0.08	0.09	0.22	0.30	0.0	0.0	0.21	0.26	0.33		0.02	0.25	0.25	0.34	0.30	0.5
Enclosure	C	0.0	0.03	0.04	0.09	0.13	0.0	0.0	0.07	0.09	0.13	0.25	0.0	0.07	0.11	0.14	0.14	0.28
Homesteads/	D		0.14	0.1			0.1	0.15	0.31	0.36	0.43		0.2	0.3	0.31	0.41	0.40	0.56
Miscellaneous	C		0.08						0.19	0.25	0.31		0.14	0.22	0.24	0.32	0.32	0.49
Grazing land	D	0.0	0.04						0.09	0.14	0.19		0.07	0.15	0.17	0.25	0.25	0.42
	C	0.0	0.03	0.03	0.04	0.06	0.0	0.0	0.03	0.06	0.12	0.22	0.0	0.08	0.09	0.17	0.13	0.32

5. SUMMARY AND CONCLUSIONS

Ethiopia is a tropical country which is predominantly inhabited by peasant farmers who are dependent on rainfed agriculture and natural resource for their livelihoods. However, rainfall is so unevenly distributed, with good rainfall in the southwest of the country to scanty rainfall in the north and southeastern parts of the country, causing frequent droughts. In countries like Ethiopia where widespread poverty, poor health, low farm productivity and degraded natural resources are major problems, development of irrigated agriculture is vital. The importance of introducing irrigated agriculture into the economy of developing countries is based on the fact that rainfed agriculture is not capable of supplying the desired amount of production to feed the increasing population. Irrigation is not only required to supplement the deficit from annual crop demand, but also to adjust seasonal variations or erratic nature of rainfall distribution. This inadequacy of moisture will inevitably lead to considerable yield reduction. Irrigated agriculture is one of the means to achieve the agricultural development-led industrialization and food security strategy of the national government. The struggle to secure food in the country can be greatly assisted by increasing production using irrigation water from small-scale, medium or large-scale irrigation schemes

In line with the development policy of the country, with the objective of increasing and stabilizing food production, the Regional Government of Tigray has been promoting irrigation development. To fulfil this target the regional government established an organisation called Commission for Sustainable Agriculture and Environmental Rehabilitation in Tigray (CoSAERT) in 1994. The mandate of CoSAERT was to construct 500 dams and irrigate 50,000 ha in 10 years. Since the establishment of CoSAERT, many micro-dams and river diversions have been built or rehabilitated. Other non-governmental organizations were also involved in the development of water resources activities with the same objectives. Introduction of irrigated agriculture has played a significant role in increasing farmers income compared to non-irrigating house holder. Using the data from Tigray region, Gebrehaweria et al. (2009) reported that the average income of non-irrigating households is less than that of the irrigating households by about 50%. In an earlier study (Mintesinot, 2002) reported that with the use of irrigation the house hold income on average have increased by three fold compared to rainfed cultivation The stored water also provides water for livestock and household uses as well. Irrigation is still a top priority of the regional government and huge capital is allocated for new projects. The regional Agriculture Bureau reported that the 2008/2009 agricultural produce through irrigation has surpassed the year before production by 44% (Walta Information Centre (WIC), WIC, 2009).

Despite these positive impacts the performance of the irrigation projects had diminished due to many reasons and the target set by CoSAERT could not be achieved. In 2007, a survey was made on 44 micro-dam irrigation projects in Tigray region. The problems observed during the survey were broad and consisted of both technical and non-technical issues. The major technical problems identified were insufficient inflow into the reservoirs, excessive seepage from reservoirs, reservoir early sedimentation, poor irrigation water application and management, structural and dam stability, and social and institutional related problems.

The reservoirs of each dam were sized assuming a certain amount of inflow to come every year. The dam height, slopes, other component of the dam and the irrigation infrastructure were designed and constructed corresponding to this storage. However most of the dams failed to store the expected storage due to low incoming flow. As a result of which only farmers on the head reach would get water but much of the land remains without irrigation. Insufficient inflow towards the reservoirs may safely be attributed to the use of poor data sets used during the planning phase. The water resources potential of all reservoirs in Tigray region was estimated by a lumped rainfall abstraction model which uses parameter inputs which are not calibrated for the watershed conditions of the region. Use of such empirical models was the only option, because there was no measured runoff data that can be used for potential assessment and evaluation. The problem of runoff prediction for ungauged catchments is a very important research question which needs to be addressed as more and more irrigation projects are currently under design and also planned in the future. Evaluation of the current design procedure and calibrating runoff coefficients for the watershed condition of the region is another research question of practical relevance. Thus these two questions were dealt in depth in this PhD research and will be presented in the later part of this chapter.

Excessive seepage from reservoirs through reservoir rims and dam foundation should be seen from economic and dam safety point of view. Excessive seepage means less water available for irrigation and uncontrolled seepage through dam foundation or dam body can be also a series treat for the dam safety if dam body or foundation materials are washed away with the seeping water. Several reasons has been described for excessive seepage, but the author believes the complexity of the geology where the dams are located and the method of investigation coupled with lack of experience in data collection and analysis are the major factors attributed to this problem. Dam safety and monitoring should be part of the routine task of the Bureau of Water Resources Energy and Mines. Dams having excessive seepage should be closely monitored and remedial measures should be in place before severe damage or failure occurs. During the field visit it was possible to observe the seeping water at the downstream of Rubafeleg was not clean. Thus the author strongly recommends undertaking in detail geo-physical survey to understand the seepage lines and its impact on the safety of the dam.

Reservoir sedimentation is another constraint in the development of irrigation in the region. The bottom outlets of some dams are filling prior to service life estimated during planning. As a result of this gravity irrigation is no more possible and lifting irrigation already started as means of utilizing the stored water for irrigation (e.g. Hizati Wedicheber, Adikenafiz, and Filiglig). Few of them are totally abounded and the reservoir is already started to serve as farm land (e.g. Majae). Except to scanty information there was no documented study on the sediment yield of watersheds in Tigray region. Recent studies made on selected watersheds indicated the possible range of sediment yield. The sediment yield varies from watershed to watershed depending on differences in lithology, ground cover, extent of bank gullies and human activities (Nigussie et al., 2005). Despite to the high variability among watersheds the research outputs are useful for planning and design purposes. Experience and good judgment are necessary while using the mentioned research outputs, because the adapted methods are subjective can lead to wrong assumptions or conclusions.

Design and construction of micro-dam irrigation schemes could be one achievement but proper operation and periodic maintenance of the schemes is also equally important. Otherwise the project will not be sustainable. From my field the author noticed that the sustainability of the implemented projects is in question for the following reasons

- Early reservoir sedimentation: reservoir sedimentation will replace the usable storage of the reservoir and thus less water will be available for irrigation. When it is worse inlet structure clogging will be the ultimate fate leaving the dam without its prime objective. Thus Once the dams are constructed the watershed rehabilitation measures (biological and physical) should be implemented with the objective of minimizing the incoming sediments to wards the reservoirs there by increase its service life.
- Percentage of area under cultivation: analysis of the actual irrigated land compared to the designed land revealed that the median range is 25% to 65%, which is very low performance. This low performance will have an impact to the motivation of farmers and less investment towards irrigation.
- Maintenance: the irrigation canal network and infrastructure needs to be maintained periodically to minimize wastage of water and undesirable effects coming from seepages. Likewise the dam appurtenance structures (spillway and inlet outlet) are deteriorating with time; it seems that once the construction is completed no body pays an attention after that. The classic examples are spillways of Rubafeleg and Adikenafiz where the structure is seriously damaged by spillway overflows. Unless immediate measure is taken the maintenance cost could be highly inflated or the spillway may collapse which endangers the safety of the dam.
- Irrigation water application and management: uncontrolled irrigation can have negative impact like water logging, salinity build-up and favourable condition for mosquito breeding. Irrigation scheduling, frequency and irrigation stream size are more or less are fixed based on the common experience of the farmers. Therefore it should be backed with research to fill the knowledge gaps.
- Agriculture and extension: It is obvious the farmers do have rich knowledge in rainfed agriculture and in irrigated agriculture where there are traditional irrigation schemes. But in places where new technology is introduced continuous assistance is necessary from agriculture bureau. Sharing

experience among the farmers can be one means to bridge this knowledge gaps.

- Market and infrastructure facility: road networks are important for the farmers in order to transport their agricultural produce into near by cities or towns. Storage and agro-processing facility also useful when there is excess produce.

All issues discussed above are important and need to be tackled at different level with different expertise. This PhD research focused mainly in the areas of rainfall-runoff which is the principal component of inflow estimation towards the reservoirs. As indicated above there was no measuring facility that would be used for estimating the reservoir inflow, alternatively empirical equation were often used leading in accurate estimation during planning. Design of more projects in similar ungauged catchments will still continue and the same mistake can be repeated unless better runoff estimation methods are used. Thus prediction models for ungauged catchments are very much necessary for successful estimation of runoff and accordingly use it for design purposes. Therefore in this research different approaches have been used that would give basis for runoff prediction for ungauged catchments in the Northern part of Ethiopia.

This kind of research is new in its kind in the region and thus most of the data are collected from the facilities established for this research. Considerable time and energy was spent to install, monitor and collect necessary input data from the established stations. The basic input data required for this research were landuse land cover, soil properties, rainfall, reservoir water level measurements and reservoir topography details. In order to determine the hydrologic soil groups found in each watershed many soil samples were collected and analyzed in the laboratory resulting important engineering and physical soil properties. Likewise with the use of areal photo, topographic maps and field verification the landuse maps were prepared. The rainfall measurement was done with tipping bucket rain gauges installed at each watershed. The reservoir water level measurement installed at each reservoir enabled us to determine the reservoir water level at different time intervals. Detailed reservoir water balance analysis has been carried out for each recording year in order to determine the observed hydrograph for each event. To minimize the effect of reservoir sedimentation new topographic maps produced by this study were used as input. Once the basic data are collected a number of software packages were used to compile, analyze, and finally get meaning full data sets for further analysis.

Flow prediction for ungauged catchments is firstly done by relating runoff with different catchment characteristics (e.g. precipitation, catchment area, slope, etc…).The prediction equations developed for each antecedent moisture conditions were found to be satisfactory with very good correlation for AMC I, AMC II and AMC III respectively (chapter 3 equations 3.38a to 3.45b).

Hydrologic models where their model parameters are dependent on watershed physical features are often used for modeling ungauged catchments. The SCS method is one of the models where its parameters are entirely dependent on catchment characteristics. Watershed model for each catchment developed by HEC-GeoHMS was used as an input for HEC-HMS hydrologic model. For the selected rainfall-runoff events calibration and optimization were carried out. The Nash-Sutcliffe model efficiency obtained from calibration and optimization with 60% of the events above 0.9, 30% of the events (0.8-0.9) and two events with NS=0.73 and 0.70 respectively. The sensitivity analysis made for each model parameter reveals that the curve number is the most sensitive for runoff volume and peak discharge and the second most sensitive parameter is the precipitation depth. Thus during calibration processes the curve number has to be selected with great caution as changes in CN values can affect significantly the simulation output especially at lower CN values. Uncertainty in the model output can be attributed to various factors. In this study uncertainty resulting from parameter estimation is studied. Random samples generated with Monte Carlo Simulation were analyzed with GLUE method in order to determine the behavioural simulations from the total array of distribution. A likelihood of 10% PEV objective function is used to select behavioural simulations. Posterior analysis made on behavioural simulations identified the most possible parameter distribution for different antecedent moisture conditions and respective watersheds.

It is common practice to use part of the data set for calibration and the remaining data set for validation. But in recently gauged catchments the existing flow data are few in number, thus splitting the already few data will not serve both purposes. Thus in this study all data were used for calibration and using bootstrap sampling technique synthetic data were generated for model validation. The aggregated percent error in volume is less than 12% for AMC I, less than 6%, and less than 5% for AMC II and III respectively. The errors observed for AMC II and AMC III are low and 12% for AMC I is also encouraging result. Because modeling AMC I condition is very difficult since the curve number is very much sensitive at lower curve number.

The parameters that need to be regionalized in the SCS-method are the curve number, initial abstraction factor, transform (basin lag) and routing parameters. The original SCS method assumes the initial abstraction factor $(\lambda = 0.2)$ and a table of CN values (NEH-4) for AMC II for different soil groups and antecedent moisture conditions. Besides to this conversion table and number of equations are available to change AMC II into AMC I and AMC III. This research identified the curve number for AMC II proposed in the NEH-4 can give acceptable runoff estimate for most of the calibrated rainfall-runoff events. The median factor (Cf_{CN}) obtained for the calibrated events is about 1.006, which is nearly equal to one. Therefore for modeling AMC II condition, the existing CN read from NEH-4 table can be either directly used or modified it with the calibration factor Cf_{CN} =1.006.

A comparison assessment made between the calibrated CN for AMC I and AMC III with the equations proposed by different researchers revealed that CN_I is underestimated and CN_{III} is overestimated by all equations. Thus use of those equations or the conversion table proposed by SCS will not work for the rainfall-runoff conditions of the region. Thus the author proposed a new formula derived from the calibrated rainfall-runoff events observed from the monitoring stations. The equations are reported in chapter 3 (equations 3.55 to 3.58).

Another important parameter in the SCS method is the abstraction loss factor. Similar to CN_{II} for AMC II, the median λ for the calibrated events is 0.1977. Thus either λ =0.2 or λ = 0.1977 can be used alternatively as the calculated root mean square error in both cases are less than 5%. Recently there are some researches which conclude λ=0.05 represent the field data compared to the text λ=0.2 (e.g. Hawkins et al., 2002; Descheemaker et al., 2008).This PhD study finding is inline with the mentioned studies for AMC I, but for AMC III the median value for the calibrated rainfall-runoff events is found to be λ=0.112. Therefore for ungauged catchments in the study area the author believed that λ=0.2 can work for AMC II, but runoff can be estimated more accurately if λ=0.05 is used for AMC I conditions and λ=0.112 for AMC III conditions.

The shape of the inflow hydrograph can be modeled accurately with the properly estimated watershed transform parameter. The SCS lag formula proposed for watersheds dominated with agricultural lands over estimates the time lag for mild and flat topography. This study concluded that the exponent to the watershed slope in the SCS lag equation should be variable according to watershed topography. The different formula developed for variable topographic condition while computing watershed lag time are reported in chapter 3 with equations (3.63 to 3.65).

Modeling at sub-watershed level can be done based on the calibration factors developed at the outlet for the total watershed area. The calibration factor can be constant or with minor percentage variation depending on parameter sensitivity. Channel routing in sub-watershed modeling is done with Muskingum-Cunge method which needs calibrating only Manning roughness (N_{man}), but other parameters can be determined either from field measurement, maps and GIS processing. In the absence of defined channel cross section the time lag method can give similar results for smaller reaches where flood attenuation in the channels is not significant.

Another approach deployed in this study for estimation of runoff for ungauged catchments is the use of monthly water balance model. This approach is useful to evaluate the different components of the total rainfall-runoff process like potential and actual evapotranspiration, soil moisture storage, runoff (direct and surplus) on monthly basis. Potential evapotranspiration can be estimated with reasonable accuracy by FAO-56PM method provided that good set of data are available. But since this is not usually the case this study has come up with an empirical equation developed from regression analysis of potential evapotranspiration estimated by different methods for standard stations having good set of data (chapter 3, equation 3.10). Uncertainty analysis made for potential evapotranspiration with Monte Carlo simulation showed that the level of uncertainty is within acceptable limits and thus the estimated potential evapotranspiration can be used for actual evapotranspiration estimation. Evaluation of actual evapotranspiration for two watersheds indicated that it is equal to the potential evapotranspiration during the rainy seasons and nearly zero during the dry months of the year.

The developed watershed water balance model requires calibration of two parameters i.e. direct runoff fraction (DR_{frac}) and surplus runoff fractions (SR_{frac}). A guide line on how to set the two parameters are indicated in the report that can serve as initial values for estimation. Best results can be obtained if there is measured runoff at the watershed. In order to regionalize those parameters more rainfall-runoff data from different watersheds are necessary to successfully calibrate and regionalize them with catchment characteristics.

The uses of empirical equations and runoff coefficients have been discussed in depth in this study. Attempt has been made to evaluate the existing method and determine the event, decadal and seasonal runoff coefficients from reservoir water balance analysis. This study concluded that the runoff coefficient couldn't be a single value rather it should be variable across the season. To this end a new set of runoff coefficients were developed in relation to curve numbers, antecedent moisture conditions and rainfall depths. Different graphs were developed which will serve for watersheds dominated with cultivated lands and another graph for mixed land uses with cultivate land, bushes and exclosure as major landuse land units. Furthermore with downscaling principles runoff coefficients were developed for different major land uses with respect to rainfall depth. Also guide lines on how to estimate runoff with the newly proposed runoff coefficients were also presented.

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Appendices

Appendix 2: Appendix to Chapter 2

Appendix 2.1: Dam inspection sheet used for pilot survey for the micro-dam irrigation projects in Tigray region

Project Name: ------------------

1. General- ____
2. Dam and infrastructure data sheet
    - Owner____________________________________
    - Designed by:______________________________
    - Constructed by: ___________________________
    - Year of completion:_________________________
    - Region:__________________________________
    - Woreda:__________________________________
    - Kebele:__________________________________
    - GPS location:______________________________
    - GPS elevation:_____________________________
    - Purpose::_________________________________
    - Type: ___________________________________
    - Height:__________________________________
    - Length:__________________________________
    - Crest width:_______________________________
    - Upstream slope:____________________________
    - Downstream slope:__________________________
    - Full Supply Level (FSL or NWL) ________________
    - Gross capacity of reservoir:____________________
    - Dead storage:______________________________
    - Catchment area:____________________________
    - Type of Spillway:_____________________
    - Spillway discharge Capacity (at zero free board):________
    - Freeboard (above FSL):________________________
    - Inlet description:____________________________
    - Scour Outlet:______________________________
    - Hazard Rating:_____________________________
    - Operation and Maintenance Manual:______________
    - Dam Safety Emergency Plan:____________________
    - Designed irrigable area:_______________________
    - Irrigation canal network details:
      ____
    - Irrigation structures (Drops, Division box, Culvert, aqueduct, siphon, road crossing, etc…):________________________________
3. Operational Status at time of inspection
    - Dam inspection:
      ____
      ____
    - Date of surveillance report:___________________________
    - Reservoir level: ____________________________
    - Release outlet____________________________
    - Weather Condition
      ____
4. Inspection team
    - Name: - Affiliation:
5. Hydrology
   ____
   ____
   ____
6. Embankment

   7.1 General Description

   ____
   ____

   7.2 Upstream Face

- Beaching: ______________________
- Signs of Movement and/or slope: ______________________
- Settlement: not observed ______________________
- Slope Stability: ______________________
- Cracks and sink holes: Not observed ______________________
- Debris: ______________________

7.3 Crest

- Signs of movement and/or Slips: ______________________
- Sink holes______________________
- Cracks: ______________________
- Settlement: ______________________
- Horizontal movement: ______________________
- Camber: ______________________
- Crest capping: ______________________
- Erosion: ______________________
- Channelisation: ______________________
- Vegetation: ______________________
- Access: ______________________
- Surface Condition: ______________________

7.4 Downstream face

- Surface Condition: ______________________
- Signs of Movement and/or slips: ______________________
- Settlement: ______________________
- Wet areas: ______________________
- Slope Stability: ______________________
- Crakes, sink holes and animal burrows: ______________________
- Erosion: ______________________
- Toe ______________________

7. Instrumentation: ______________________
8. Inlet structure ______________________
9. Outlet structure
   - Overall condition ______________________
   - Regulating valve: ______________________
   - Sealing/leakages: ______________________
   - Erosion: ______________________
10. Spillway ______________________
11. Reservoir perimeter
   - Shoreline Landslide: ______________________
   - Shoreline Erosion: ______________________
   - Shoreline Protection:______________________

13. Irrigation canals and structures
   - Siltation:

____
- Erosion:
____
- Weed growth:
____
- Slopes:
____
- Seepage:
____

14. Consequences
    14.1 Building
    - Closest house:
    ____
    - No of effected houses-____
    14.2 Transient Risk to Life
    - Farming:____
    14.3 Environmental
    - Significant losses:
    ____
    14.4 Social
    - Heritage:____
    - Significant effect:____
    14.5 Economic Loss
    - Supply reliance:____
    - Alternatives:____
    - Industry:____
15. other comments
____
____

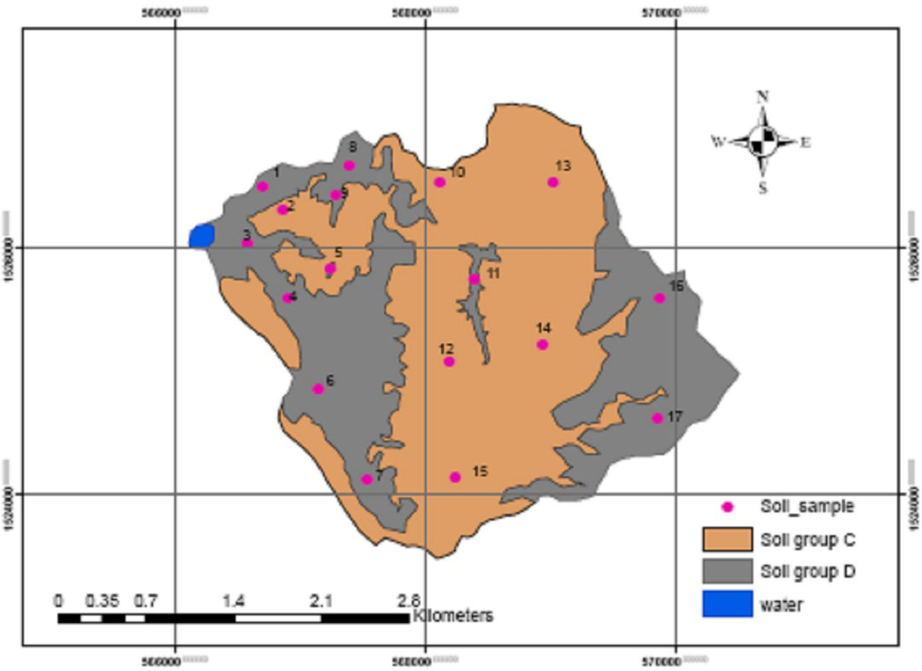
Appendix 3: Appendix to Chapter 3

Appendix 3.1.2.4: Summary of soil properties with respect to each watershed are presented in the following table. Raw data and detailed analysis of the laboratory results are reported in the accompanying CD .

Appendix 3.1.2.4a: Soil properties summary for Laelay Wukro watershed

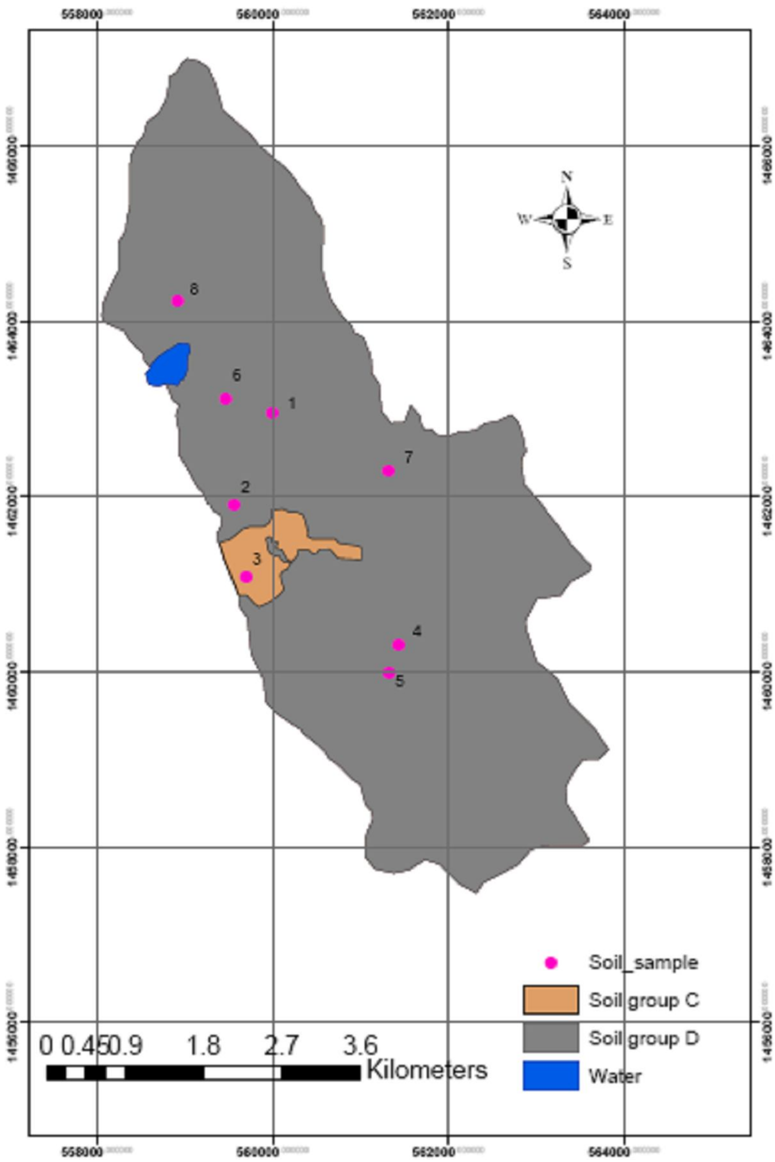
Sample code	Texture		BD	SG	Ksat	HSG
	Clay (%)	Silt (%)	(gm/cc)	(gm/cc)	(mm/h)	
1	35.20	37.75	1.38	2.69	0.10	D
2	13.96	54.97	1.63	2.54	2.06	C
3	25.74	54.61	1.49	2.63	0.34	D
4	17.02	68.86	1.26	2.52	0.10	D
5	9.35	58.29	1.50	2.62	3.00	C
6	20.68	52.58	1.53	2.67	0.12	D
7					0.11	D
8					0.31	D
9	21.59	58.58	1.35	2.64	0.36	D
10					2.06	C
11	24.47	62.11	1.29	2.34	0.44	D
12					1.24	C
13					2.13	C
14					1.33	C
15					1.24	C
16					0.35	D
17	32.75	49.48	1.48	2.54	0.10	D

Bd= Bulk density, SG= Specific gravity, Ksat=Saturated hydraulic conductivity, HSG =Hydrologic soil group



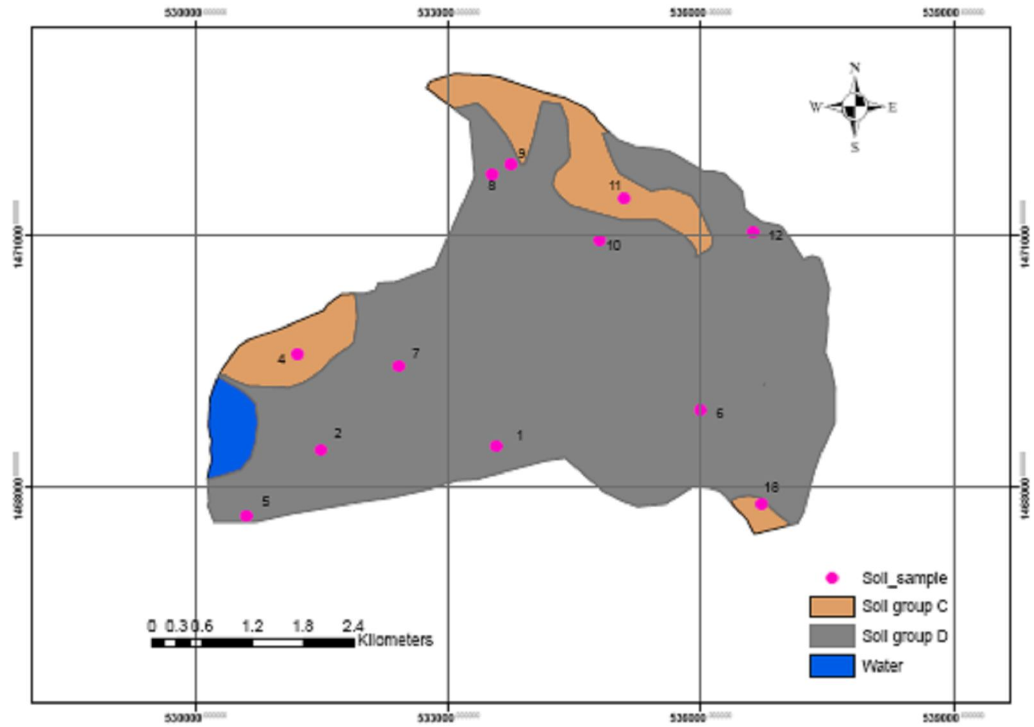
Appendix 3.1.2.4 b: Soil properties summary for GumSelassa watershed

Sample code	Texture		BD	SG	Ksat	HSG
	Clay (%)	Silt (%)	(gm/cc)	(gm/cc)	(mm/h)	
1	60.36	19.19	1.27	2.55	0.05	D
2	62.06	29.63	1.13	2.53	0.03	D
3	49.43	45.22	1.37	2.69	1.49	C
4	27.54	38.76	1.58	2.66	0.40	D
5	48.45	24.19	1.29	2.48	0.33	D
6	35.09	26.99	1.44	2.74	0.63	D
7	28.75	42.17	1.45	2.58	0.26	D
8	34.65	54.88	1.45	2.62	0.57	D



Appendix 3.1.2.4 c: Soil properties summary for Haiba watershed

Sample code	Texture		BD	SG	Ksat	HSG
	Clay (%)	Silt (%)	(gm/cc)	(gm/cc)	(mm/h)	
1	83.71	16.29	1.73	2.68	0.033	D
2	52.06	27.70	1.54	2.73	0.035	D
4	21.39	40.45	1.56	2.66	2.406	C
5	11.50	47.95	1.59	2.85	0.744	D
6	54.70	44.58	1.71	2.44	0.023	D
7	53.75	32.20	2.05	2.74	0.032	D
8	44.05	38.88	1.50	2.73	0.182	D
9	24.38	41.46	1.78	2.83	0.456	D
10	55.92	39.81	1.46	2.72	0.064	D
11	11.63	55.84	1.69	2.51	2.652	C



Appendix 3.1.2.4b: Raw data and analysis results of soil samples collected from each watershed.

The procedures adapted to determine the saturated hydraulic conductivity and hydrometer analysis for texture analysis is presented taking the soil samples collected from Haiba watershed as an example

More detailed results for each samples is presented in the accompanying CD.

Saturated hydraulic conductivity determination

The formulas used for calculation are briefly indicated as follows.

$$As = \frac{\pi \times D^2}{4}, \; Ast = \frac{\pi \times d^2}{4}, \; Vm = As \times l$$

Where

As = Area of specimen (cm^2) — D = diameter of specimen (cm)

Ast = Area of stand pipe (cm^2) — d = diameter of stand pipe (cm)

Vm = Volume of mold (cm^3) — l = length of specimen (cm)

$$Ksat = \frac{2.303 \times l \times Ast}{(As \times t_{elapsed}) \times Log\left(\frac{h_{initial}}{h_{final}}\right)}$$

Where

$Ksat$ = saturated hydraulic conductivity at test temperature (cm/s)

$t_{elapsed}$ = test duration (s), $h_{initial}$ = initial head at start of the test observation,

h_{final} = final head at the end of the observation

The saturated hydraulic conductivity will be converted into saturated hydraulic conductivity at 20 ^oC by multiplying with the ration of viscosity at the test temperature and 20 ^oC.

Haiba raw data and analysis results

	Temperature (^{o}c)	Viscosity
η_t, Viscosity of water at temperature	20.0	1.00
η_t, Viscosity of water at temperature	21.0	0.97
η_t, Viscosity of water at temperature	22.0	0.95
η_t, Viscosity of water at temperature	23.0	0.93
η_t, Viscosity of water at temperature	24.0	0.91

Diameter of specimen(cm),D=	10.2	Area of Specimen(cm^2)=As	81.71
Length of Specimen(cm), l=	11.55	Area of Standpipe(cm^2)=Ast	0.79
Diameter of Standpipe(cm),d	1.00	Volume of mold(cm^3)=Vm	943.78

Sample code *1*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^{o}c)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/ η_{20})	Ksat at 20 ^{o}c (cm/s)	Ksat (mm/h)
1	50.30	50.2	20	240	9.20689E-07	1	9.20689E-07	
2	50.20	50.1	20	240	9.22524E-07	1	9.22524E-07	
3	50.10	50.0	20	240	9.24368E-07	1	9.24368E-07	
4	50.30	50.0	20	720	9.22527E-07	1	9.22527E-07	
					9.22527E-07		9.22527E-07	0.033211

Sample code *2*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^{o}c)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^{o}c (cm/s)	Ksat (mm/h)
1	46.35	46.25	21	240	9.99235E-07	0.97	9.69258E-07	
2	46.25	46.15	21	240	1.0014E-06	0.97	9.71356E-07	
3	46.15	46.05	21	240	1.00357E-06	0.97	9.73463E-07	
4	46.35	46.05	21	720	1.0014E-06	0.97	9.71359E-07	
					1.0014E-06		9.71359E-07	0.034969

Sample code *4*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^{o}c)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^{o}c (cm/s)	Ksat (mm/h)
1	50.50	46.8	22	120	7.04056E-05	0.95	6.68853E-05	
2	46.80	43.4	22	120	6.9789E-05	0.95	6.62995E-05	
3	43.40	40.2	22	120	7.08704E-05	0.95	6.73268E-05	
4	50.50	40.2	22	360	7.0355E-05	0.95	6.68372E-05	
					7.0355E-05		6.68372E-05	2.40614

Sample code *5*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^{o}c)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^{o}c (cm/s)	Ksat (mm/h)
1	50.00	48.8	21	120	2.24778E-05	0.97	2.18035E-05	
2	48.80	47.7	21	120	2.10956E-05	0.97	2.04628E-05	
3	47.70	46.7	21	120	1.96044E-05	0.97	1.90162E-05	
4	46.70	45.6	21	120	2.20557E-05	0.97	2.1394E-05	
					2.13084E-05		2.06691E-05	0.744089

Sample code *6*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^{o}c)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^{o}c (cm/s)	Ksat (mm/h)
1	51.35	50.45	21	3058	6.42035E-07	0.97	6.22774E-07	
2	50.45	50.05	21	1448	6.10404E-07	0.97	5.92092E-07	
3	50.05	48.9	21	3894	6.6282E-07	0.97	6.42935E-07	
4	48.90	48.8	21	330	6.88782E-07	0.97	6.68118E-07	
					6.5101E-07		6.3148E-07	0.022733

Sample code *7*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^{o}c)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^{o}c (cm/s)	Ksat (mm/h)
1	50.35	50.2	20	360	9.20231E-07	1	9.20231E-07	
2	50.20	50.1	20	280	7.90735E-07	1	7.90735E-07	
3	50.10	49.95	20	380	8.76155E-07	1	8.76155E-07	
4	49.95	49.75	20	480	9.28077E-07	1	9.28077E-07	
					8.788E-07		8.788E-07	0.031637

Sample code *8*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^oc)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^oc (cm/s)	Ksat (mm/h)
1	50.28	49.92	20	180	4.43255E-06	1	4.43255E-06	
2	49.92	49.55	20	180	4.58912E-06	1	4.58912E-06	
3	49.55	49.10	20	180	5.62776E-06	1	5.62776E-06	
4	49.10	48.55	20	224	5.58389E-06	1	5.58389E-06	
5	50.28	48.55	20	764	5.0886E-06	1	5.0886E-06	
					5.06438E-06		5.06438E-06	0.182318

Sample code *9*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^oc)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^oc (cm/s)	Ksat (mm/h)
1	50.20	49.5	20	120	1.29933E-05	1	1.29933E-05	
2	49.50	48.9	20	120	1.12842E-05	1	1.12842E-05	
3	48.90	47.8	20	180	1.40347E-05	1	1.40347E-05	
4	50.20	48.9	20	240	1.21387E-05	1	1.21387E-05	
5	50.20	47.8	20	420	1.29513E-05	1	1.29513E-05	
					1.26804E-05		1.26804E-05	0.456495

Sample code *10*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^oc)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^oc (cm/s)	Ksat (mm/h)
1	50.50	50.3	21	240	1.8359E-06	0.97	1.78082E-06	
2	50.30	50.1	21	240	1.84321E-06	0.97	1.78792E-06	
3	50.10	50.0	21	120	1.84874E-06	0.97	1.79327E-06	
4	50.50	50.0	21	600	1.84139E-06	0.97	1.78615E-06	
					1.84231E-06		1.78704E-06	0.064333

Sample code *11*

Obs.No.	Initial head (cm)	Final head (cm)	Test temp (^oc)	Elapsed time (s)	Ksat at test Temp. (cm/s)	Viscosity Ratio (η_t/η_{20})	Ksat at 20 ^oc (cm/s)	Ksat (mm/h)
1	50.50	47	21	95	8.39492E-05	0.97	8.14307E-05	
2	47.00	45.1	21	60	7.6365E-05	0.97	7.4074E-05	
3	45.10	42.7	21	85	7.14325E-05	0.97	6.92895E-05	
4	42.70	39.5	21	120	7.20786E-05	0.97	6.99162E-05	
					7.59563E-05		7.36776E-05	2.652394

Hydrometer: Sedimentation analysis for one sample

The calculation procedure is based on the Geotechnical laboratory manual prepared at Mekelle University Department of Civil Engineering. Similar procedures were followed for each sample and the results are shown with the accompanying CD

Haiba raw data and analysis results

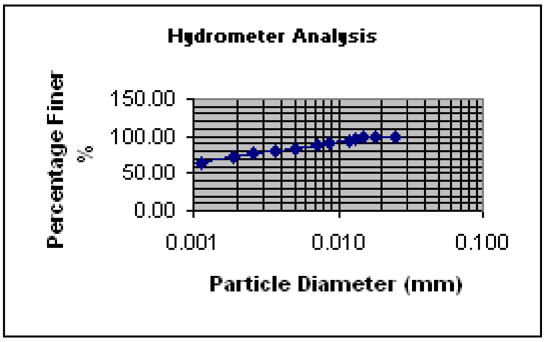
Hydrometer Analysis (Texture)

(Sedimentation Method)

Oven dry mass of soil (gm)	50
Volume of suspension(cc)	1000
Specific gravity of soil	2.6788
Specific gravity of water	1
Unit wt of water g/cc	1

Sample code *1*

Observation	Reading Time	Elapsed time t	Hydrometer Reading	Temperature	Composite correction	Corrected hydrometer reading	Effective depth L	L/t	$(L/t)^{1/2}$	Correction factor K_f	Particle Diameter D	Particle Finer
		(min)		(^{o}c)			(mm)	mm/s	$(mm/s)^{1/2}$		(mm)	%
1	3:20	0										
2	3:22	2	1.037	20	0.0027	1.0343	6.5	3.250	1.803	0.014	0.024	100.00
3	3:24	4	1.035	20	0.0027	1.032	7.0	1.750	1.323	0.014	0.018	100.00
4	3:26	6	1.034	20	0.0027	1.0313	7.3	1.217	1.103	0.014	0.015	99.89
5	3:28	8	1.033	20	0.0027	1.0303	7.6	0.950	0.975	0.014	0.013	96.70
6	3:30	10	1.032	20	0.0027	1.0293	7.8	0.780	0.883	0.014	0.012	93.51
7	3:40	20	1.031	20	0.0027	1.0283	8.1	0.405	0.636	0.014	0.009	90.31
8	3:50	30	1.030	20	0.0027	1.0273	8.4	0.280	0.529	0.014	0.007	87.12
9	4:20	60	1.029	20	0.0027	1.0263	8.6	0.143	0.379	0.014	0.005	83.93
10	5:20	120	1.028	21	0.0025	1.0255	8.9	0.074	0.272	0.013	0.004	81.38
11	7:20	240	1.027	21	0.0025	1.0245	9.2	0.038	0.196	0.013	0.003	78.19
12	11:20	480	1.025	21	0.0025	1.0225	9.7	0.020	0.142	0.013	0.002	71.80
13	3:20	1440	1.023	20	0.0027	1.0203	10.2	0.007	0.084	0.014	0.001	64.78



		Clay (%)	Silt(%)
0.001	2.55306		
0.001	2.33279	83.71	16.28837
			16.28837

Soil type **Clay**

Appendix 3.1.3.1 Daily rainfall used for analysis for rainfall-runoff events from the three monitoring stations

Appendix 3.1.3.1a Daily rainfall for Laelay Wukro watershed

Year	Day / Month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
2001	July	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	**0.00**	**3.60**	**17.8**	**17.4**	**16.8**	**12.6**	**13.6**	**2.4**	**10.4**
	August	2.8	1.0	24.2	6.2	3.0	8.0	21.6	9.4	1.4	25.8	61.4	0.2	19.6	12	12.8	1.2	0.0	0.0	0.8	0.0	0.2	1.4	28.2	7.4	0.0	0.0	0.4	10.8	22.2	12.2	11.8
2007	June	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	5.2	0.0	0.0	0.0	0.0	1.4	3	10.8	4.4	2	36.2	6.2	5.2	3.8	2.2	1.4	3
	July	1.8	0.4	0.0	0.6	11.8	5.2	0.0	20.4	3.4	6.6	7.2	9.6	9	7.6	11.6	4.6	22.8	1.6	2.2	0.2	0.8	29	39.4	13.6	0.0	0.0	0.0	0.0	0.8	55	20
	August	18.8	23.6	16.4	1.2	6	5.8	4.8	7.6	0.2	0.0	2.4	6.4	9.8	0.0	11	4.8	0.0	0.0	41	30.8	10.2	0.2	2	0.0	0.2	0.2	0.0	1.6	0.0	0.2	5.2
2008	June	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	4.8	24.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0
	July	0.0	0.0	0.0	0.0	0.0	6.8	7.6	3.0	5.8	7.2	0.2	4.6	0.0	0.0	0.0	4.0	2.4	2.2	0.0	11.8	9.4	0.6	7.4	1.2	45.2	57.8	0.2	4.4	19.4	29.6	0.4
	August	10.2	10.2	4.2	0.0	8.8	33.8	5.8	1.8	33.4	0.6	0.6	0.2	22.4	5.2	1.8	1.0	2.0	0.6	0.2	0.0	0.2	0.0	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.00

Note: Measuring facility became functional installed by Tigray Bureau of Water Resources Energy and Mines through WHIST (Water Harvesting and Institutional strengthening) project since 23/07/2001

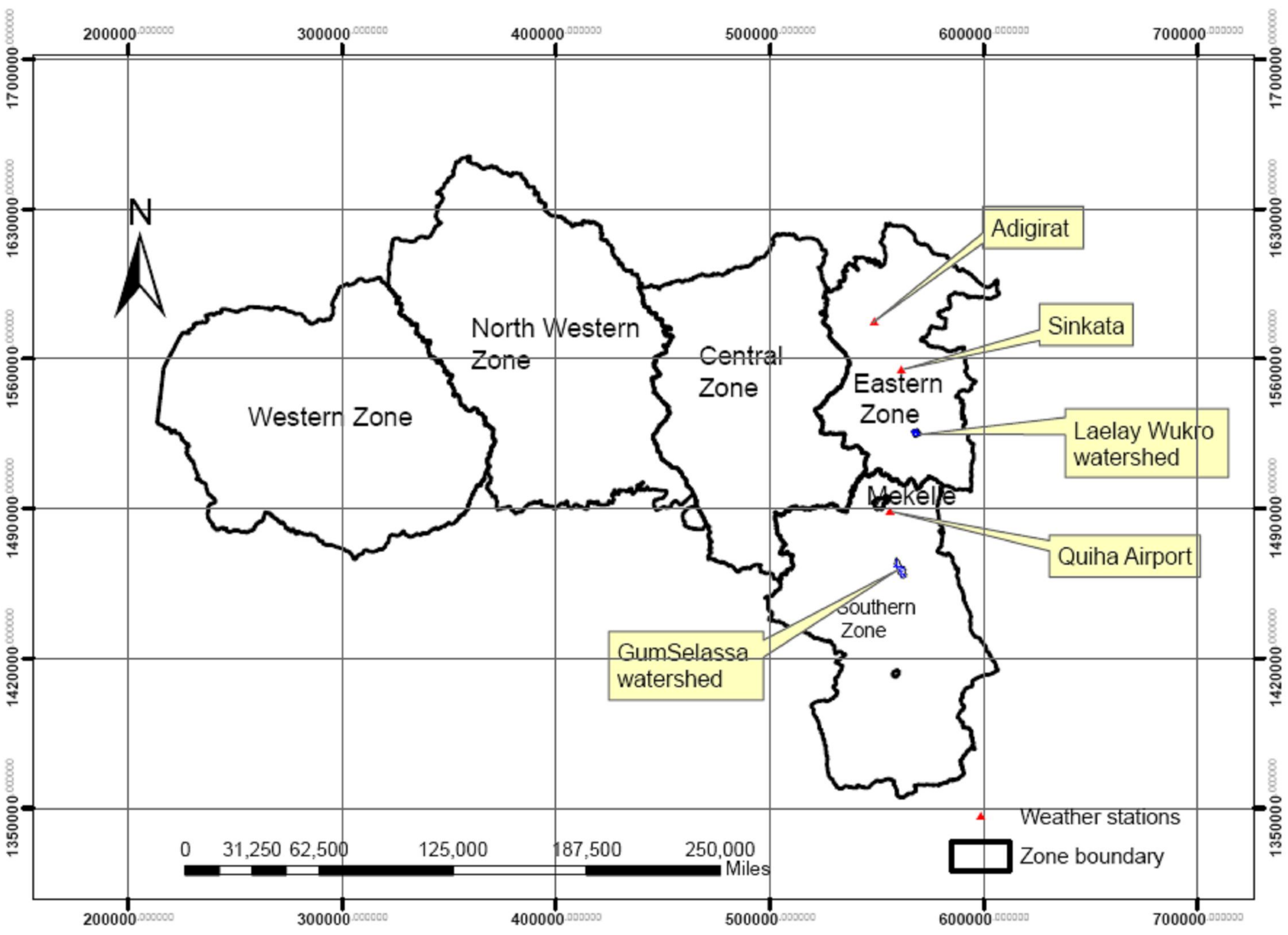
Appendix 3.1.3.1b Daily rainfall for GumSelassa watershed

Year	Day / Month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
2007	June	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5.4	3.2	4.1	0	10.6	0	0	0	0	0	0	0	0
	July	13.6	0.0	0.6	9.2	0.2	3.8	5.6	0.4	10.2	0.0	0.0	16.4	7.8	12.2	0.8	13.0	13.6	15.6	0.0	6.0	8.4	7.2	0.2	1.8	0.2	0.0	0.6	10.6	18.6	30.6	12.4
	August	2.2	6.0	0.2	0.0	22.4	0.8	11.8	1.6	15.6	6.4	9.0	2.2	1.2	0.0	2.2	2.6	0.2	0.0	28.4	17.4	18.0	1.6	9.0	1.2	0.2	0.6	0.0	15.2	0.0	12.2	8.8
2008	June	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3.2	0	0	0
	July	10.6	20.2	0.0	0.0	0.4	0.0	1.6	11.4	15.4	7.2	0.6	7.8	1.4	1.2	0.0	4.8	10.6	20.2	0.0	0.0	0.4	0.0	1.6	11.4	15.4	7.2	0.6	7.8	1.4	1.2	0.0
	August	1.8	6.2	0.0	0.0	0.0	0.0	0.2	0.0	0.0	3.4	1.2	1.6	1.0	10.4	4.8	2.8	3.2	1.6	0.2	8.4	8.4	0.2	0.2	1.6	0.8	10.6	1.2	0.2	4.0	4.8	7.4

Appendix 3.1.3.1c Daily rainfall for Haiba watershed

Year	Day / Month	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
2001	July	6.2	2.2	0	0	0	6.3	2.6	8.1	1.1	3.9	0	25	3.2	4.3	2.8	0	5.8	0	16.2	20.4	10.6	26.2	2.8	16.2	9.2	25.6	28.6	25.2	17.4	3.6	4.6
	August	2.8	3.4	1.6	1.2	18.2	12.2	0.2	3.4	8.0	20.0	15.4	6.2	14.6	25.4	5.6	3.2	3.2	29.2	10.2	3.2	15.6	0.2	4.6	2.0	0.0	2.4	0.0	19.2	0.8	0.0	5.8
2007	June	0	0	0	0	0	0	4.6	0	0	0.4	0	0	0	0	0	2.4	0	0	0	8.5	0	0	0.4	0	0	0.6	0	6.3	9.6	2.8	0
	July	19.2	0.4	1.7	9.8	0.2	1.9	10.8	0	0.2	9.9	25.8	0.2	14.6	4.6	7.4	33.8	2.8	5.8	0.6	5.2	1.8	15.4	6.8	3.4	5	0.2	0.4	16	2.2	52	19.8
	August	19.8	21.6	1	2	42.8	28.6	0.4	11.2	8.8	2	1.8	3.2	19.8	0.6	12.4	0	1.1	0	9.9	12.5	31.5	12.3	0.4	0	3.4	0	11.2	25.3	5.6	24.2	5.6

Appendix 3.1.3.2a Location map for three weather stations found in Tigray region (Quiha Airport, Sinkata and Adigirat)



Appendix 3.1.3.2b: Meteorological data for Adigirat, Sinkata and Quiha Airport weather stations

Station: Adigirat weather station
Elevation, Z = 2497m
Location: 39^{o}27' North, 14^o 16' East

Year	Month	$\vartheta_{\max}$ (^{o}c)	$\vartheta_{\min}$ (^{o}c)	RH (%)	Z (m)	v_2 m/s	t_{Sact} h/d	$t_{S\max}$ h/d	R_a (MJ/m^{2}d	ϑ_{mean} (^{o}c)	$\vartheta_{mean_{i-1}}$ (^{o}c)
2002	1	23.10	7.50	71.00	2497.00	1.00	9.30	11.25	29.81	15.30	14.80
	2	24.70	7.80	58.00	2497.00	1.20	9.50	11.54	32.93	16.25	15.30
	3	24.60	10.10	68.00	2497.00	1.50	8.40	11.94	36.25	17.35	16.25
	4	24.90	10.20	76.00	2497.00	1.60	9.30	12.35	38.12	17.55	17.35
	5	26.50	10.80	72.00	2497.00	1.50	9.80	12.68	38.41	18.65	17.55
	6	24.90	12.40	57.00	2497.00	2.60	7.90	12.84	38.16	18.65	18.65
	7	24.00	11.60	78.00	2497.00	1.90	5.90	12.75	38.13	17.80	18.65
	8	22.20	9.20	87.00	2497.00	1.40	5.90	12.46	37.96	15.70	17.80
	9	23.90	7.10	72.00	2497.00	1.30	7.90	12.06	36.57	15.50	15.70
	10	23.50	6.52	73.00	2497.00	1.20	7.10	11.66	33.65	15.01	15.50
	11	23.00	7.90	74.00	2497.00	1.10	8.40	11.32	30.41	15.45	15.01
	12	22.10	7.50	81.00	2497.00	1.10	9.30	11.16	28.76	14.80	15.45
2003	1	23.40	6.58	73.00	2497.00	1.22	9.30	11.25	29.81	14.99	14.80
	2	25.30	9.41	69.00	2497.00	1.21	9.50	11.54	32.93	17.36	14.99
	3	25.00	10.14	70.00	2497.00	1.22	8.40	11.94	36.25	17.57	17.36
	4	24.90	11.67	76.00	2497.00	1.30	9.30	12.35	38.12	18.29	17.57
	5	26.40	13.12	69.00	2497.00	1.42	9.80	12.68	38.41	19.76	18.29
	6	25.00	13.43	65.00	2497.00	2.17	7.90	12.84	38.16	19.22	19.76
	7	21.20	13.05	77.00	2497.00	1.90	5.90	12.75	38.13	17.13	19.22
	8	22.10	12.29	88.50	2497.00	1.80	5.90	12.46	37.96	17.19	17.13
	9	24.00	9.06	65.00	2497.00	1.14	7.90	12.06	36.57	16.53	17.19
	10	23.00	7.93	81.00	2497.00	1.16	7.10	11.66	33.65	15.47	16.53
	11	23.20	7.71	71.00	2497.00	1.15	8.40	11.32	30.41	15.46	15.47
	12	23.40	6.55	65.00	2497.00	1.03	9.30	11.16	28.76	14.98	15.46
2004	1	24.07	8.44	71.00	2497.00	1.20	9.44	11.25	29.81	16.25	14.98
	2	24.17	8.12	58.00	2497.00	1.22	9.37	11.54	32.93	16.14	16.25
	3	24.95	9.15	62.00	2497.00	1.27	8.83	11.94	36.25	17.05	16.14
	4	24.81	11.07	80.00	2497.00	1.28	8.70	12.35	38.12	17.94	17.05
	5	26.22	10.89	58.00	2497.00	1.62	10.77	12.68	38.41	18.55	17.94
	6	24.51	12.09	68.00	2497.00	1.69	7.35	12.84	38.16	18.30	18.55
	7	22.33	11.92	76.00	2497.00	2.51	6.32	12.75	38.13	17.12	18.30
	8	21.75	10.68	90.00	2497.00	1.33	6.38	12.46	37.96	16.21	17.12
	9	24.20	8.04	51.00	2497.00	1.23	8.02	12.06	36.57	16.12	16.21
	10	22.58	6.76	53.67	2497.00	1.17	8.73	11.66	33.65	14.67	16.12
	11	23.04	6.17	54.33	2497.00	1.00	8.15	11.32	30.41	14.61	14.67
	12	23.06	5.90	72.33	2497.00	0.91	9.29	11.16	28.76	14.48	14.61
2005	1	23.10	6.31	63.31	2497.00	1.20	9.18	11.25	29.81	14.70	14.48
	2	25.88	7.44	54.82	2497.00	1.17	10.09	11.54	32.93	16.66	14.70
	3	25.51	9.19	55.67	2497.00	1.13	8.34	11.94	36.25	17.35	16.66
	4	24.75	9.60	65.23	2497.00	1.29	9.84	12.35	38.12	17.18	17.35
	5	25.02	10.04	58.35	2497.00	1.26	9.72	12.68	38.41	17.53	17.18
	6	25.85	10.96	57.38	2497.00	1.95	8.63	12.84	38.16	18.40	17.53
	7	21.76	10.23	75.28	2497.00	1.62	5.40	12.75	38.13	16.00	18.40
	8	22.39	9.91	80.07	2497.00	1.05	5.50	12.46	37.96	16.15	16.00
	9	24.06	7.71	61.24	2497.00	0.93	7.88	12.06	36.57	15.88	16.15
	10	23.23	7.05	63.45	2497.00	0.98	10.42	11.66	33.65	15.14	15.88

	11	22.95	7.20	63.77	2497.00	0.92	8.40	11.32	30.41	15.07	15.14
	12	22.94	6.61	64.83	2497.00	0.87	9.30	11.16	28.76	14.78	15.07
2006	1	24.39	4.48	45.51	2497.00	0.98	10.04	11.25	29.81	14.44	14.78
	2	25.85	5.98	44.64	2497.00	0.98	9.64	11.54	32.93	15.92	14.44
	3	24.82	6.26	56.73	2497.00	0.95	8.17	11.94	36.25	15.54	15.92
	4	24.39	7.44	54.83	2497.00	0.97	8.13	12.35	38.12	15.92	15.54
	5	24.50	8.69	50.95	2497.00	1.09	8.08	12.68	38.41	16.60	15.92
	6	25.23	9.68	48.76	2497.00	2.09	7.37	12.84	38.16	17.46	16.60
	7	21.81	10.05	73.43	2497.00	1.39	4.91	12.75	38.13	15.93	17.46
	8	21.55	10.41	75.04	2497.00	1.08	5.01	12.46	37.96	15.98	15.93
	9	23.28	7.45	57.05	2497.00	0.71	6.14	12.06	36.57	15.37	15.98
	10	23.78	8.05	56.66	2497.00	0.71	7.95	11.66	33.65	15.92	15.37
	11	22.79	8.41	58.45	2497.00	0.68	8.60	11.32	30.41	15.60	15.92
	12	22.57	8.50	56.37	2497.00	0.56	8.22	11.16	28.76	15.53	15.60
2007	1	23.26	8.89	65.23	2497.00	0.78	8.63	11.25	29.81	16.07	15.53
	2	24.67	9.65	58.54	2497.00	0.79	8.44	11.54	32.93	17.16	16.07
	3	26.22	9.59	50.35	2497.00	0.89	9.22	11.94	36.25	17.91	17.16
	4	25.08	10.62	63.47	2497.00	0.86	8.51	12.35	38.12	17.85	17.91
	5	26.48	10.15	47.94	2497.00	0.99	8.43	12.68	38.41	18.31	17.85
	6	24.61	10.38	55.37	2497.00	2.22	6.91	12.84	38.16	17.49	18.31
	7	21.17	9.84	78.61	2497.00	0.77	5.61	12.75	38.13	15.51	17.49
	8	22.04	9.84	73.71	2497.00	0.74	6.31	12.46	37.96	15.94	15.51
	9	22.95	7.79	62.57	2497.00	0.79	6.84	12.06	36.57	15.37	15.94
	10	23.30	6.00	54.42	2497.00	0.73	8.91	11.66	33.65	14.65	15.37
	11	22.73	5.79	62.53	2497.00	0.69	8.81	11.32	30.41	14.26	14.65
	12	23.59	4.60	50.94	2497.00	0.73	9.33	11.16	28.76	14.09	14.26
2008	1	24.04	7.78	54.13	2497.00	0.72	8.30	11.25	29.81	15.91	14.09
	2	24.74	6.58	40.76	2497.00	0.87	10.24	11.54	32.93	15.66	15.91
	3	26.35	8.21	26.94	2497.00	0.97	9.35	11.94	36.25	17.28	15.66
	4	25.19	10.55	41.07	2497.00	1.25	8.80	12.35	38.12	17.87	17.28
	5	25.10	11.10	52.23	2497.00	0.91	8.11	12.68	38.41	18.10	17.87
	6	24.60	12.10	50.17	2497.00	1.48	7.52	12.84	38.16	18.35	18.10
	7	21.70	12.00	68.61	2497.00	1.86	5.62	12.75	38.13	16.85	18.35
	8	22.00	10.39	66.19	2497.00	1.16	5.93	12.46	37.96	16.20	16.85
	9	23.73	7.86	59.82	2497.00	1.02	7.45	12.06	36.57	15.79	16.20
	10	23.23	7.05	61.98	2497.00	0.99	8.37	11.66	33.65	15.14	15.79
	11	22.95	7.20	62.28	2497.00	0.92	8.46	11.32	30.41	15.07	15.14
	12	22.94	6.61	63.32	2497.00	0.87	9.12	11.16	28.76	14.78	15.07

Station: Sinkata weather station
Elevation, Z = 2437m
Location: 14° 04' North, 39° 34' East

Year	Month	$\vartheta_{\max}$ (°c)	$\vartheta_{\min}$ (°c)	RH (%)	Z (m)	v_2 m/s	t_{Sact} h/d	$t_{S\max}$ h/d	R_a (MJ/m²d)	ϑ_{mean} (°c)	$\vartheta_{mean_{i-1}}$ (°c)
2002	1	23.80	9.56	68.73	2437.00	1.67	9.30	11.26	29.91	16.68	16.62
	2	26.00	10.83	51.07	2437.00	1.98	10.10	11.55	33.01	18.41	16.68
	3	25.60	12.39	60.45	2437.00	1.92	9.00	11.94	36.29	19.00	18.41
	4	26.00	12.66	63.39	2437.00	2.72	10.30	12.34	38.11	19.33	19.00
	5	27.70	14.51	47.61	2437.00	2.67	10.20	12.67	38.37	21.10	19.33
	6	26.70	13.92	62.21	2437.00	2.09	6.90	12.83	38.10	20.31	21.10
	7	25.00	13.74	84.72	2437.00	1.61	5.70	12.74	38.09	19.37	20.31
	8	22.60	12.50	80.32	2437.00	1.38	6.10	12.45	37.94	17.55	19.37
	9	24.40	12.82	58.13	2437.00	2.74	8.50	12.06	36.60	18.61	17.55
	10	23.50	11.28	62.68	2437.00	2.37	8.80	11.66	33.72	17.39	18.61
	11	23.30	10.47	66.50	2437.00	1.91	9.50	11.33	30.50	16.89	17.39
	12	23.00	9.75	74.47	2437.00	2.57	10.07	11.17	28.87	16.37	16.89
2003	1	23.80	9.90	49.56	2437.00	1.81	10.56	11.26	29.91	16.85	16.37
	2	25.20	11.30	63.59	2437.00	1.96	9.94	11.55	33.01	18.25	16.85
	3	25.60	12.60	65.54	2437.00	2.22	9.56	11.94	36.29	19.10	18.25
	4	25.60	13.40	71.95	2437.00	2.33	9.01	12.34	38.11	19.50	19.10
	5	26.30	15.00	69.55	2437.00	2.40	9.18	12.67	38.37	20.65	19.50
	6	25.30	13.60	45.85	2437.00	2.00	6.67	12.83	38.10	19.45	20.65
	7	22.00	12.60	71.45	2437.00	2.40	3.48	12.74	38.09	17.30	19.45
	8	21.00	13.50	74.66	2437.00	2.70	4.71	12.45	37.94	17.25	17.30
	9	23.70	12.30	45.98	2437.00	2.30	8.09	12.06	36.60	18.00	17.25
	10	22.80	11.20	47.18	2437.00	2.10	9.45	11.66	33.72	17.00	18.00
	11	23.00	10.00	70.32	2437.00	1.90	10.50	11.33	30.50	16.50	17.00
	12	21.80	9.10	48.96	2437.00	2.20	10.17	11.17	28.87	15.45	16.50
2004	1	24.54	10.89	39.84	2437.00	1.75	10.02	11.26	29.91	17.71	15.45
	2	24.35	10.66	35.86	2437.00	2.08	10.07	11.55	33.01	17.51	17.71
	3	24.79	11.58	36.48	2437.00	2.20	9.74	11.94	36.29	18.19	17.51
	4	24.24	13.14	51.50	2437.00	2.18	9.11	12.34	38.11	18.69	18.19
	5	26.52	14.63	26.52	2437.00	3.07	11.13	12.67	38.37	20.57	18.69
	6	24.79	13.42	45.90	2437.00	2.03	6.78	12.83	38.10	19.11	20.57
	7	21.93	12.22	71.26	2437.00	1.38	5.53	12.74	38.09	17.07	19.11
	8	22.07	12.03	77.48	2437.00	1.34	5.69	12.45	37.94	17.05	17.07
	9	24.13	12.52	42.33	2437.00	2.49	8.45	12.06	36.60	18.32	17.05
	10	22.39	9.98	46.87	2437.00	2.42	8.99	11.66	33.72	16.18	18.32
	11	23.07	9.52	47.37	2437.00	1.90	9.39	11.33	30.50	16.29	16.18
	12	23.22	9.42	43.48	2437.00	2.20	10.15	11.17	28.87	16.32	16.29
2005	1	23.37	10.18	41.97	2437.00	1.92	9.98	11.26	29.91	16.77	16.32
	2	26.12	10.96	31.43	2437.00	2.27	10.05	11.55	33.01	18.54	16.77
	3	25.72	12.35	48.32	2437.00	2.03	9.66	11.94	36.29	19.03	18.54
	4	24.52	12.82	47.93	2437.00	2.48	9.23	12.34	38.11	18.67	19.03
	5	25.39	14.07	48.71	2437.00	2.18	9.12	12.67	38.37	19.73	18.67
	6	26.01	14.36	43.93	2437.00	2.37	6.85	12.83	38.10	20.19	19.73
	7	21.38	12.67	77.29	2437.00	1.30	4.66	12.74	38.09	17.03	20.19
	8	22.26	12.46	74.94	2437.00	1.43	5.20	12.45	37.94	17.36	17.03
	9	24.21	12.56	49.20	2437.00	2.08	7.92	12.06	36.60	18.38	17.36
	10	22.94	10.91	46.32	2437.00	2.64	9.08	11.66	33.72	16.92	18.38
	11	22.90	9.83	55.51	2437.00	1.90	9.78	11.33	30.50	16.37	16.92
	12	23.21	9.81	48.96	2437.00	2.20	10.16	11.17	28.87	16.51	16.37

2006	1	24.12	10.19	36.77	2437.00	1.90	10.53	11.26	29.91	17.16	16.51
	2	30.15	11.30	41.86	2437.00	1.99	10.27	11.55	33.01	20.73	17.16
	3	25.60	12.10	48.58	2437.00	2.39	8.91	11.94	36.29	18.85	20.73
	4	24.76	12.83	51.40	2437.00	2.28	8.15	12.34	38.11	18.80	18.85
	5	24.35	14.07	50.94	2437.00	2.39	8.16	12.67	38.37	19.21	18.80
	6	25.79	13.74	42.57	2437.00	2.17	7.13	12.83	38.10	19.77	19.21
	7	21.66	12.92	79.97	2437.00	1.27	3.96	12.74	38.09	17.29	19.77
	8	20.90	12.71	84.52	2437.00	1.65	3.74	12.45	37.94	16.80	17.29
	9	23.44	12.39	51.13	2437.00	1.98	6.94	12.06	36.60	17.92	16.80
	10	23.19	11.39	53.13	2437.00	2.45	8.37	11.66	33.72	17.29	17.92
	11	22.88	10.02	54.30	2437.00	1.88	9.56	11.33	30.50	16.45	17.29
	12	22.71	9.94	60.00	2437.00	2.21	9.70	11.17	28.87	16.33	16.45
2007	1	23.09	9.85	48.84	2437.00	1.69	9.71	11.26	29.91	16.47	16.33
	2	25.11	11.44	49.36	2437.00	1.85	9.34	11.55	33.01	18.28	16.47
	3	25.95	12.14	42.65	2437.00	2.26	10.03	11.94	36.29	19.04	18.28
	4	24.70	13.34	49.93	2437.00	2.44	9.17	12.34	38.11	19.02	19.04
	5	26.18	14.53	41.55	2437.00	2.71	7.86	12.67	38.37	20.35	19.02
	6	24.61	13.91	56.20	2437.00	2.05	6.50	12.83	38.10	19.26	20.35
	7	21.26	12.39	81.48	2437.00	1.23	4.80	12.74	38.09	16.83	19.26
	8	22.00	12.44	79.06	2437.00	1.40	5.78	12.45	37.94	17.22	16.83
	9	23.61	12.29	55.50	2437.00	1.84	7.61	12.06	36.60	17.95	17.22
	10	22.82	10.68	46.32	2437.00	2.51	9.78	11.66	33.72	16.75	17.95
	11	22.26	9.15	49.63	2437.00	2.21	9.94	11.33	30.50	15.70	16.75
	12	25.31	10.85	36.32	2437.00	1.86	10.70	11.17	28.87	18.08	15.70
2008	1	23.75	10.67	38.84	2437.00	1.89	9.73	11.26	29.91	17.21	18.08
	2	24.28	10.21	30.31	2437.00	2.20	10.57	11.55	33.01	17.24	17.21
	3	27.80	10.70	27.16	2437.00	2.36	10.75	11.94	36.29	19.25	17.24
	4	24.30	13.00	41.30	2437.00	2.99	9.67	12.34	38.11	18.65	19.25
	5	25.50	13.80	44.58	2437.00	2.18	8.21	12.67	38.37	19.65	18.65
	6	25.00	13.50	47.30	2437.00	1.98	7.11	12.83	38.10	19.25	19.65
	7	22.21	12.75	74.54	2437.00	2.21	4.50	12.74	38.09	17.48	19.25
	8	21.80	12.61	77.04	2437.00	1.65	5.20	12.45	37.94	17.21	17.48
	9	23.91	12.48	48.43	2437.00	2.24	7.92	12.06	36.60	18.20	17.21
	10	22.94	10.91	48.03	2437.00	2.41	9.08	11.66	33.72	16.92	18.20
	11	22.90	9.83	54.33	2437.00	1.95	9.78	11.33	30.50	16.37	16.92
	12	23.21	9.81	48.70	2437.00	2.21	10.16	11.17	28.87	16.51	16.37

Station: Quiha-airport weather station
Elevation, Z = 2257m
Location: 13^{o} 28'North, 39^{o} 31' East

Year	Month	$\vartheta_{\max}$ (^{o}c)	$\vartheta_{\min}$ (^{o}c)	RH (%)	Z (m)	v_2 m/s	t_{Sact} h/d	$t_{S\max}$ h/d	R_a (MJ/m^{2}d)	ϑ_{mean} (^{o}c)	$\vartheta_{mean_{i-1}}$ (^{o}c)
2002	1	22.30	10.55	51.57	2257.00	3.18	9.20	11.29	30.21	16.43	16.96
	2	24.64	10.74	43.78	2257.00	3.53	9.90	11.57	33.23	17.69	16.43
	3	25.82	12.43	45.97	2257.00	3.18	8.90	11.94	36.40	19.12	17.69
	4	26.51	12.19	46.68	2257.00	3.49	10.68	12.33	38.10	19.35	19.12
	5	28.65	14.09	35.97	2257.00	2.46	10.68	12.64	38.25	21.37	19.35
	6	27.25	13.76	42.25	2257.00	1.97	7.36	12.79	37.94	20.51	21.37
	7	25.49	13.75	69.25	2257.00	1.56	5.76	12.71	37.95	19.62	20.51
	8	23.29	12.67	76.55	2257.00	1.52	6.49	12.43	37.89	17.98	19.62
	9	24.84	12.22	55.42	2257.00	1.93	8.43	12.06	36.67	18.53	17.98
	10	24.82	11.62	47.51	2257.00	3.01	10.10	11.68	33.91	18.22	18.53
	11	23.50	11.06	50.66	2257.00	3.31	9.86	11.36	30.78	17.28	18.22
	12	23.41	10.52	39.62	2257.00	3.07	9.45	11.21	29.19	16.96	17.28
2003	1	24.46	8.71	55.60	2257.00	2.98	9.90	11.29	30.21	16.58	16.96
	2	25.60	11.72	53.51	2257.00	3.29	9.40	11.57	33.23	18.66	16.58
	3	25.67	12.29	56.14	2257.00	3.82	9.40	11.94	36.40	18.98	18.66
	4	26.62	13.61	50.89	2257.00	3.59	8.90	12.33	38.10	20.12	18.98
	5	28.24	15.15	35.98	2257.00	3.20	10.40	12.64	38.25	21.70	20.12
	6	26.86	13.47	47.40	2257.00	2.01	6.90	12.79	37.94	20.17	21.70
	7	23.38	13.78	75.74	2257.00	1.95	3.80	12.71	37.95	18.58	20.17
	8	22.28	12.74	82.61	2257.00	1.98	4.10	12.43	37.89	17.51	18.58
	9	24.29	11.68	51.97	2257.00	2.51	7.80	12.06	36.67	17.99	17.51
	10	23.65	10.86	49.73	2257.00	3.16	10.30	11.68	33.91	17.25	17.99
	11	22.86	10.55	51.75	2257.00	3.47	10.10	11.36	30.78	16.70	17.25
	12	22.04	9.31	63.57	2257.00	3.57	9.90	11.21	29.19	15.67	16.70
2004	1	24.96	10.11	49.48	2257.00	3.08	9.82	11.29	30.21	17.54	15.67
	2	23.96	9.75	40.90	2257.00	4.50	10.03	11.57	33.23	16.86	17.54
	3	24.95	11.58	35.34	2257.00	4.83	10.06	11.94	36.40	18.27	16.86
	4	25.93	13.45	44.90	2257.00	3.82	8.82	12.33	38.10	19.69	18.27
	5	28.24	13.18	20.94	2257.00	2.69	10.86	12.64	38.25	20.71	19.69
	6	26.49	13.16	36.08	2257.00	1.85	6.69	12.79	37.94	19.83	20.71
	7	24.78	12.98	56.60	2257.00	2.01	5.85	12.71	37.95	18.88	19.83
	8	22.88	12.98	70.74	2257.00	1.49	5.79	12.43	37.89	17.93	18.88
	9	25.06	11.64	55.42	2257.00	1.98	7.78	12.06	36.67	18.35	17.93
	10	23.49	10.04	47.51	2257.00	3.06	10.15	11.68	33.91	16.77	18.35
	11	25.93	10.82	50.66	2257.00	3.96	10.18	11.36	30.78	18.38	16.77
	12	24.81	12.98	39.62	2257.00	3.57	10.10	11.21	29.19	18.89	18.38
2005	1	26.17	10.55	49.63	2257.00	3.36	9.40	11.29	30.21	18.36	18.89
	2	28.15	10.74	36.93	2257.00	4.58	10.38	11.57	33.23	19.44	18.36
	3	26.11	12.43	46.42	2257.00	4.51	9.37	11.94	36.40	19.27	19.44
	4	26.29	12.19	44.24	2257.00	4.46	9.49	12.33	38.10	19.24	19.27
	5	26.35	14.09	51.00	2257.00	2.95	9.32	12.64	38.25	20.22	19.24
	6	27.35	13.76	43.27	2257.00	3.78	8.45	12.79	37.94	20.56	20.22
	7	23.20	13.75	75.42	2257.00	2.11	5.75	12.71	37.95	18.48	20.56
	8	23.31	12.67	76.29	2257.00	1.48	5.89	12.43	37.89	17.99	18.48
	9	24.59	12.22	58.87	2257.00	1.45	7.26	12.06	36.67	18.40	17.99
	10	23.77	11.62	45.29	2257.00	2.82	9.99	11.68	33.91	17.69	18.40
	11	22.67	11.06	49.57	2257.00	3.47	9.60	11.36	30.78	16.87	17.69
	12	22.36	10.52	15.67	2257.00	3.57	9.80	11.21	29.19	16.44	16.87

2006	1	23.56	7.63	48.87	2257.00	3.98	10.55	11.29	30.21	15.59	16.44
	2	25.21	11.10	54.82	2257.00	4.26	10.29	11.57	33.23	18.16	15.59
	3	25.52	11.47	52.48	2257.00	3.79	8.09	11.94	36.40	18.49	18.16
	4	25.02	12.86	52.37	2257.00	4.04	8.99	12.33	38.10	18.94	18.49
	5	26.01	13.01	48.03	2257.00	2.91	9.90	12.64	38.25	19.51	18.94
	6	27.14	12.82	44.61	2257.00	2.00	6.57	12.79	37.94	19.98	19.51
	7	23.57	13.44	74.19	2257.00	1.71	4.91	12.71	37.95	18.51	19.98
	8	22.31	12.98	82.39	2257.00	1.65	3.97	12.43	37.89	17.65	18.51
	9	24.52	10.98	59.13	2257.00	1.47	7.28	12.06	36.67	17.75	17.65
	10	23.87	11.08	56.35	2257.00	3.15	9.55	11.68	33.91	17.48	17.75
	11	22.71	9.98	60.93	2257.00	3.81	10.29	11.36	30.78	16.35	17.48
	12	22.43	10.45	67.42	2257.00	3.85	9.39	11.21	29.19	16.44	16.35
2007	1	22.72	9.39	66.06	2257.00	3.86	9.41	11.29	30.21	16.05	16.44
	2	24.82	11.35	58.75	2257.00	4.10	9.42	11.57	33.23	18.08	16.05
	3	26.15	11.38	44.97	2257.00	4.63	9.73	11.94	36.40	18.76	18.08
	4	26.10	12.42	48.83	2257.00	4.12	9.83	12.33	38.10	19.26	18.76
	5	27.78	14.03	42.42	2257.00	2.80	9.49	12.64	38.25	20.90	19.26
	6	26.83	14.13	60.00	2257.00	2.56	7.22	12.79	37.94	20.48	20.90
	7	22.62	13.03	79.39	2257.00	1.72	5.59	12.71	37.95	17.82	20.48
	8	22.85	12.75	78.42	2257.00	1.47	5.77	12.43	37.89	17.80	17.82
	9	23.87	11.07	62.33	2257.00	1.40	7.67	12.06	36.67	17.47	17.80
	10	23.35	9.82	46.94	2257.00	2.55	10.60	11.68	33.91	16.58	17.47
	11	22.14	9.77	54.87	2257.00	3.57	10.34	11.36	30.78	15.96	16.58
	12	21.82	8.49	54.58	2257.00	3.53	10.69	11.21	29.19	15.16	15.96
2008	1	24.10	10.00	61.48	2257.00	3.40	9.88	11.29	30.21	17.05	15.16
	2	23.90	9.00	45.79	2257.00	4.18	10.60	11.57	33.23	16.45	17.05
	3	26.30	10.70	31.83	2257.00	4.22	11.12	11.94	36.40	18.50	16.45
	4	25.60	13.10	44.17	2257.00	4.24	10.38	12.33	38.10	19.35	18.50
	5	27.60	13.70	39.55	2257.00	2.57	9.11	12.64	38.25	20.65	19.35
	6	27.60	13.00	44.90	2257.00	1.89	6.26	12.79	37.94	20.30	20.65
	7	24.20	13.00	70.97	2257.00	2.21	4.50	12.71	37.95	18.60	20.30
	8	23.50	13.70	75.13	2257.00	1.67	5.05	12.43	37.89	18.60	18.60
	9	24.53	11.63	57.19	2257.00	1.79	7.70	12.06	36.67	18.08	18.60
	10	23.82	10.84	49.28	2257.00	2.96	10.11	11.68	33.91	17.33	18.08
	11	23.30	10.54	53.07	2257.00	3.60	10.06	11.36	30.78	16.92	17.33
	12	22.81	10.38	46.75	2257.00	3.53	9.89	11.21	29.19	16.59	16.92

Appendix 3.1.4 Reservoir water balance model for the three watersheds

The tables of the water balance models can not be accommodated within this page. Therefore for further reference please see the CD accompanying this thesis. Sample water balance models for year 2007 are shown for each reservoir.

Appendix 3.2.1.1 Curve Number values for cultivated and uncultivated land with respect to hydrologic condition and hydrologic soil groups (Source Mishra and Singh (2003))

No	Landuse description/treatment	Hydrologic condition	Hydrologic Soil Groups			
			A	B	C	D
	Agricultural					
1.	**Cultivated land**:					
1.1	Fallow					
	Bare soil Straight row		77	86	91	94
	Crop residue cover	Poor	76	85	90	93
		Good	74	83	88	90
1.2	Row crops:					
	……………………….Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Crop residue cover Straight row	Poor	71	80	87	90
	Crop residue cover Straight row	Good	64	75	82	85
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Crop residue cover Contoured	Poor	69	78	83	87
	Crop residue cover Contoured	Good	64	74	81	85
	Contoured and terraced	Poor	66	74	80	82
	Contoured and terraced	Good	62	71	78	81
	Crop residue cover Contoured and terraced	Poor	65	73	79	81
	Crop residue cover Contoured and terraced	Good	61	70	77	80
1.3	Small grain					
	Straight row	Poor	65	76	84	88
	Straight row	Good	63	75	83	87
	Crop residue cover Straight row	Poor	64	75	83	86
	Crop residue cover Straight row	Good	60	72	80	84
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Crop residue cover Contoured	Poor	62	73	81	84
	Crop residue cover Contoured	Good	60	72	80	83
	Contoured and terraced	Poor	61	72	79	82
	Contoured and terraced	Good	59	70	78	81
	Crop residue cover Contoured and terraced	Poor	60	71	78	81
	Crop residue cover Contoured and terraced	Good	58	69	77	80
1.4	Close-seeded legumes[1] or rotation meadow:					
	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
	Contoured and terraced	Good	51	67	76	80
2.	Uncultivated lands:					
2.1	Pasture or range	Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
2.2	Meadow-continuous grass, protected from grazing, and generally mowed for hay	Good	30	58	71	78
	Brush-brush weed grass mixture with brush being the major element	Poor	48	67	77	83
		Fair	35	56	70	77
		Good	30	48	65	73
	Farmsteads-buildings, lanes, driveways, and surrounding lots	-----	59	74	82	86
Woods and forests						
1.	Humid rangelands or agricultural uncultivated lands					
1.1	Woods or forest land	Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
1.2	Woods-grass combination (orchard or tree farm)	Poor	57	73	82	86
		Fair	43	65	76	82

		Good	32	58	72	79
2.	Arid and Semiarid rangelands[2]					
2.1	Herbaceous	Poor		80	87	93
		Fair		71	81	89
		Good		62	74	85
2.2	Oak-aspen	Poor		66	74	79
		Fair		48	57	63
		Good		30	41	48
2.3	Pinyon-juniper	Poor		75	85	89
		Fair		58	73	80
		Good		41	61	71
2.4	Sagebrush with grass under story	Poor		67	80	85
		Fair		51	63	70
		Good		35	47	55
2.5	Desert shrub	Poor	63	77	85	88
		Fair	55	72	81	86
		Good	49	68	79	84

Appendix 3.2.1.1b: Derivation of the Muskingum-Cunge equations

The original Muskingum channel routing is given by:

$$\dot{V}_{Ot} = \left(\frac{\Delta t - 2KX}{2K(1-X)+\Delta t}\right)\dot{V}_{It} + \left(\frac{\Delta t + 2KX}{2K(1-X)+\Delta t}\right)\dot{V}_{It-1} + \left(\frac{2K(1-X)-\Delta t}{2K(1-X)+\Delta t}\right)\dot{V}_{Ot-1} \qquad \text{(A-3.2.1)}$$

Where

$\dot{V}_O$: outflow from the routing reach (m³/s); $\dot{V}_I$:inflow to the routing reach (m³/s); K :travel time of the flood wave through the reach (h); X :dimensionless weighting factor, ranging from 0.0 to 0.5 (1);and Δt :time interval (t)

If the inflow and outflow at a given reach are known it is possible to determine K and X for that particular reach. Generally for small catchments measured inflow and outflow hydrographs are not available. Thus K and X must be approximated (Haan et al., 1994) and such problems are minimized in Muskingum-Cunge method. It is independent to travel time and demands less parameter for calibration which makes it suitable for ungauged catchments.

The Muskingum-Cunge equation is developed combining the continuity equation and the diffusion form of the momentum equation.

$$\frac{\partial A_f}{\partial t} + \frac{\partial \dot{V}}{\partial x} = (\dot{V})'_L \qquad \text{(A-3.2.2)}$$

$$S_f = S_o - \frac{\partial Y}{\partial x} \qquad \text{(A-3.2.3)}$$

Combining equations (A-3.2.1) and (A-3.2.2) Miller and Cunge, 1975 formulated an equation:

$$\frac{\partial \dot{V}}{\partial t} + \dot{h}_c \frac{\partial \dot{V}}{\partial x} = \mu_{diff} \frac{\partial^2 \dot{V}}{\partial x^2} + \dot{h}_c (\dot{V})'_L \qquad \text{(A-3.2.4)}$$

Where

$\dot{V}$ **:** Discharge (m³/s); A_f **:**flow area (m²); t :time (s); x :distance along the channel (m); Y :depth of flow (m); $(\dot{V})'_L$: lateral inflow per unit channel length (m³/sm); S_f :friction slope (m/m); S_o :bed slope (m/m); $\dot{h}_c$:wave celerity (m/s);and μ_{diff} :hydraulic diffusivity (m³/s/m)

The wave celerity and the hydraulic diffusion are defined with the following equations:

$$\dot{h}_c = \frac{d\dot{V}}{dA} \qquad \text{(A-3.2.5)}$$

$$\mu_{diff} = \frac{\dot{V}}{2 \times B \times S_o} \qquad \text{(A-3.2.6)}$$

Where

B : top width of the water surface (m)

From Muskingum, the storage, V_S (m^3) in the channel reach is:

$$V_S = K\left[XV_I + (1-X)V_O\right] \tag{A-3.2.7}$$

A finite difference approximation of the partial derivatives, combined with equation (A -3.2.1)

$$V_{O_t} = C_1 V_{I_{t-1}} + C_2 I_t + C_3 V_{O_{t-1}} + C_4 ((\dot{V})'_L \Delta x) \tag{A-3.2.8}$$

The coefficients are:

$$C_1 = \frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2(1-X)} \tag{A-3.2.9}$$

$$C_2 = \frac{\frac{\Delta t}{K} - 2X}{\frac{\Delta t}{K} + 2(1-X)} \tag{A-3.2.10}$$

$$C_3 = \frac{2(1-X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1-X)} \tag{A-3.2.11}$$

$$C_4 = \frac{2\left(\frac{\Delta t}{K}\right)}{\frac{\Delta t}{K} + 2(1-X)} \tag{A-3.2.12}$$

The parameters K and X are determined as follows (Cunge, 1969; Ponce, et al., 1978):

$$K = \frac{\Delta x}{c} \tag{A-3.2.13}$$

$$X = \frac{1}{2}\left(1 - \frac{\dot{V}_{ref}}{BS_o \dot{h}_c \Delta x}\right) \tag{A-3.2.14}$$

Where

$\dot{V}_{ref}$: reference flow (m^3/s)

The reference flow is basically the flow at the mid of the hydrograph and can be estimated as:

$$\dot{V}_{ref} = \dot{V}_B + 0.5(\dot{V}_p - \dot{V}_B) \quad \text{(A-3.2.15)}$$

Where

$\dot{V}_B$:base flow discharge (m^3/s) and $\dot{V}_p$: peak flow discharge (m^3/s)

Since the base flow is zero for most of the rainfall-runoff events then equation A-3.2.15 will be simplified to

$$\dot{V}_{ref} = 0.5\dot{V}_p \quad \text{(A-3.2.16)}$$

Appendix 3.2.1.1c: Sample input parameters for Muskingum Cunge for Laelay Wukro watershed

Reach	Length (m)	Slope (m/m)	Cross section	N_{man}	Bed width (m)	Side slope (H:V)
R100	1437.4	0.0564	Trapezoid	0.04	5.5	1.5
R110	225.6	0.0000	Rectangle	0.04	15	
R120	78.3	0.0000	Trapezoid	0.04	7.8	0.85
R150	333.8	0.0150	Trapezoid	0.04	7.8	0.85
R180	283.1	0.0035	Trapezoid	0.04	8.5	1.5
R20	1747.5	0.0269	Rectangle	0.05	30	
R200	841.8	0.0403	Trapezoid	0.04	2.7	1
R240	1138.1	0.2302	Trapezoid	0.04	5	1
R270	607.7	0.0297	Trapezoid	0.04	5	1
R30	451.4	0.0620	Trapezoid	0.04	6.2	0.5
R50	989.8	0.0485	Trapezoid	0.04	7.8	0.85
R80	324.9	0.0062	Rectangle	0.04	15	

N_{man} *= Initial values for Manning's roughness and refined by calibration*

Appendix 3.2.1.1c: Sample input parameter for time lag routing for Laelay Wukro watershed

Reach	R100	R110	R120	R150	R180	R20	R200	R240	R270	R30	R50	R60	R80
t_{lag} (min)	7.5	7.5	1.0	3.0	3.0	15.0	5.0	7.5	5.0	5.0	7.5	3.0	7.5

t_{lag} *= initial value and refined by calibration*

Appendix 3.2.2.2: Summary table for rainfall-runoff events and parameter calibration results

Watershed	Year	Date	AMC	CN from table (1)	CN_{cal} (1)	$(h_{Ia})_{cal}$ (mm)	$(t_{lag})_{cal}$ (min)	CN_{cf} (1)	$(h_{Ia})_{cf}$ (1)	$(t_{lag})_{cf}$ (1)	λ (1)	h_P (mm)	$h_{V\,o}$ (mm)
Laelay Wukro	2001	22-Aug	AMCI		71.80	4.50	45.00	1.1616	0.16	0.90	0.0451	27.4	4.52
	2007	23-Jul	AMC II	79.39	79.30	13.40	35.00	0.9988	0.98	0.86	0.2021	39.2	7.55
	2007	30-Jul	AMC I		72.00	3.50	60.00	1.1649	0.65	1.11	0.0354	32.0	6.71
	2007	2-Aug	AMCIII		83.00	5.25	35.00	0.9237	1.00	1.13	0.1100	20.4	3.58
	2007	20-Aug	AMC III		83.25	6.30	33.00	0.9264	1.31	0.84	0.1233	26.2	5.59
	2008	25-Jul	AMC-II	79.39	79.60	12.10	35.00	1.0026	0.91	0.88	0.1859	65.3	24.34
	2008	26-Jul	AMC-III		82.30	7.00	40.00	0.9159			0.1267	46.2	17.87
GumSelassa	2007	12-Jul	AMC I		76.00	7.40	23.00	1.0249	0.40	0.53	0.0923	16.0	0.97
	2007	18-Jul	AMC II	87.23	88.00	6.50	38.00	1.0088	0.87	0.88	0.1970	18.7	3.33
	2007	19-Aug	AMC I		77.50	3.50	45.00	1.0452	0.28	0.64	0.0475	20.8	3.46
	2007	20-Aug	AMCII	87.23	87.50	7.20	40.00	1.0031	0.97	0.93	0.1984	21.5	4.22
	2007	21-Aug	AMC III		91.00	2.62	30.00	0.9679	1.02	0.93	0.1043	13.2	3.09
	2008	22-Aug	AMC I		77.50	3.15	24.00	1.0452	0.22	0.45	0.0427	15.0	1.79
Haiba	2001	25-jul	ANC III		90.10	3.20	40.00	0.9654	0.99	1.19	0.1147	19.0	6.08
	2001	27-Jul	AMC III		91.25	2.23	40.00	0.9777	0.83	1.19	0.0916	38.2	21.61
	2001	5-Aug	AMC I		75.50	3.00	65.00	1.0502	0.24	0.76	0.0364	19.8	3.37
	2001	10-Aug	AMC II	85.89	87.80	6.80	43.00	1.0222	0.81	0.84	0.1930	44.4	19.35
	2007	16-Jul	AMC II	85.89	86.60	7.90	45.00	1.0082	0.95	0.78	0.2010	34.0	10.75
	2007	1-Aug	AMC III		91.80	2.35	36.00	0.9836	0.54	1.04	0.1132	21.0	8.56
	2007	2-Aug	AMC III		90.55	2.95	40.00	0.9702	0.91	1.09	0.1110	25.3	10.47

Appendix: 3.4.1: Boot strap samples generated for model validation

(Please see the accompanying CD)

Appendix 3.5.1 Monte Carlo Simulation input data for evapotranspiration

Potential evapotranspiration with FAO-56PM (e.g. one MCS run)

Parameter	**Designation**	**Min**	**Max.**	**Value**	**default Value**	**Average**	**Stn. Dev.**	**Distribution**
Mean $_{max.}$ temp.	ϑ max			26.5	27.4	27.43	2.01	normal
Mean $_{min.}$ temp.	ϑ min			14.6	14.5	14.52	1.38	normal
Humidity	RH			61.5	70.8	70.77	13.18	normal
Altitude	Z	1942.8	2147.3	2048.0	2045.0	2045.00	0.00	uniform
Wind speed	v_2			4.9	3.5	2.5424263	1.353941	normal
Actaul daily sunshine hours	t_{sact}	5.9	7.2	6.1	6.5	6.5	0.00	uniform
Max. possible day light hours	t_{smax}	11.5	13.5	11.8	12.8	12.8	0.00	uniform
Exra. Terristerial radiation	Ra	34.2	39.9	35.9	38.0	38.0	0.00	unifrom
Mean$_{meani}$. temp	ϑ_{meani}			19.6	21.0	20.97	1.31	normal
Mean$_{meani-1}$. temp	$\vartheta_{meani-1}$			20.6	22.1	22.10	1.01	normal
	ϑ mean				20.53			
Slope of saturation vapour pressure	Δ				0.15			
Pressure	P				79.32			
Psychrometeric constant	γ				0.05			
	$(1+0.34v_2)$				2.67			
	$\gamma*(1+0.34v_2)$				0.14			
	$\Delta+\gamma*(1+0.34v_2)$				0.29			
	$900*v_2/(\vartheta\text{ mean}+273)$				15.02			
Saturation vapour pressure at ϑ max	e_{sTmax}				3.46			
Saturation vapour pressure at ϑ min	e_{sTmin}				1.66			
Saturation vapour pressure	e_s				2.56			
Actual vapour pressure	e_a				1.021688			
Vapour pressure deficit	e_s-e_a				1.54			
Relative sun shine hour	t_{sact}/t_{smax}				0.52			

Solar radiation	Rs	18.23	
Clear sky solar radiation	Rso	28.36	
Relative short wave radiation	Rs/Rso	0.64	
Net sloar radiation	Rns	14.03	
Stefan-Boltzmann constant	σ	4.903×10^{-9}	
	σ_{TmaxK}	39.33	
	σ_{TminK}	33.54	
Net longwave radiation	Rnl	3.74	
Net radiation	Rn	10.29	
Soil hear flux	G	-0.14	
	Rn-G	10.43	
	0.408*Δ*(Rn-G)	0.63	
	$\gamma*(900*v_2/(\vartheta\ mean+273))*(e_s-e_a)$	1.22	
	$(\Delta+\gamma(1+0.34v_2))$	0.29	
Potential Evapotranspiration	ETo	**6.39**	mm/day

Appendix 3.5.2. Monte Carlo Simulation input data for runoff

Parameter	Designation	Min	Max.	Value	default Value	Average	Stn. Deviation	Distribution
Precipitation	h_P	14.960	22.440	16.7	18.70			uniform
Initial abstraction factor	λ			0.2	0.197	0.19725	0.003304038	normal
Curve Number	CN			86.6	88.00	87.475	0.618465844	normal
Runoff	h_V			1.63				
Potential maximum retention	h_{sp}			39.24887				

Note: for initial abstraction and curve number parameters the samples are generated with normal distribution and for precipitation uniform distribution

$$h_V = \frac{(h_P - \lambda \times h_{Sp})^2}{(h_P + h_{Sp} - \lambda \times h_{Sp})}$$

$$h_{Sp} = \frac{25400}{CN} - 254$$

Where

hs_P = potential retention excluding initial abstraction (mm)

h_V = runoff (mm)

h_P = total rainfall (mm)

CN = Curve Number (1)

Appendix 3.6.2.3: comparison of time lag calculated by the equation proposed by SCS method and calibrated time for each event

The SCS lag formula proposed for agricultural watersheds is given by:

$$t_{lag} = \left[\frac{l^{0.8} \times (hs_P + 1)^{0.7}}{1900 \times S^{0.5}} \right] \times 60$$

Where

t_{lag} : basin lag (min)

l : maximum watershed length (ft)

hs_P : maximum potential retention (in)

S : median catchment (watershed) slope (%)

As it is shown in the following table the time lag estimated by the SCS formula is almost 3times to that of calibrated time lag for mild or gentle slopes. The length indicated in the table is converted into equivalent length (ft) before applied in the above equation.

Appendix 3.6.2.3: comparison of time lag calculated by the equation proposed by SCS method and calibrated time for each event

Event date	CN_{cal} (1)	S_{50} (m/m)	Max. River length (m)	h_{Sp} (in)	$(t_{lag})_{SCS}$ (min)	$(t_{lag})_{cal}$ (min)	Ration (%)
22-Aug	71.80	25.7	20344.8	3.928	53.2	45.0	118.3
23-Jul	79.30	25.7	20344.8	2.610	42.8	35.0	122.3
30-Jul	72.00	25.7	20344.8	3.889	52.9	60.0	88.2
2-Aug	83.55	25.7	20344.8	1.969	37.3	35.0	106.6
20-Aug	83.25	25.7	20344.8	2.012	37.7	33.0	114.3
25-Jul	79.60	25.7	20344.8	2.563	42.4	35.0	121.2
26-Jul	82.30	25.7	20344.8	2.151	38.9	40.0	97.3
18-Jul	88.50	3.2	27076.3	1.299	111.2	38.0	292.6
19-Aug	77.50	3.2	27076.3	2.903	161.0	45.0	357.8
20-Aug	87.50	3.2	27076.3	1.429	115.5	40.0	288.8
21-Aug	91.00	3.2	27076.3	0.989	100.4	30.0	334.8
25-26jul	90.10	5.6	32373.3	1.099	91.0	40.0	227.4
27-Jul	91.10	5.6	32373.3	0.977	87.2	40.0	218.1
5-Aug	75.50	5.6	32373.3	3.245	148.9	65.0	229.1
10-Aug	87.80	5.6	32373.3	1.390	99.6	43.0	231.6
16-Jul	86.60	5.6	32373.3	1.547	104.2	45.0	231.5
1-Aug	91.80	5.6	32373.3	0.893	84.6	36.0	235.1
2-Aug	90.70	5.6	32373.3	1.025	88.7	39.0	227.5

Appendix 3.6.3a Watershed water balance computation table for GumSelassa (2007)

Month	**July**	**August**		
h_{Ptotal}(mm)	219.6	197.0		
DR_{frac}	0.255	0.161		
h_{DR}(mm)	56.0	31.7		
h_P(mm)	163.6	165.3	Error (%)	Error (%)
$h_{Vo}(m)^3$	1403515	818401	-0.65	-0.94
$h_{Vm}(m)^3$	1394324	810711		

Soil water available (Clay), for cultivated land soil depth = 100cm and available moisture 225mm/m 225.0

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	July			August		
														h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	5985742	335189.5	31.72	5985742	189849.78
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
$\|h_P$-$h_{ETp}\|$ (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	93.0	2.3	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	45.6	90.7	2.3	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	135.3	2.3	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		h_{SR}	Area	V_{SR}	h_{SR}	Area	V_{SR}
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	SR	(mm)	(m^2)	(m^3)	(mm)	(m^2)	(m^3)
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	100.0	1.4	0.0	1.7		0.00	5985742	0	0.00	5985742	0

Soil water available (Clay), for cultivated land soil depth = 100cm and available moisture 200mm/m 200.0

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	8049590	450760.9	31.72	8049590	255308.8
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
|h_P-h_{ETp}| (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	93.0	2.3	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	45.6	90.7	2.3	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	135.3	2.3	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0							
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	SR						
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	100.0	1.4	0.0	1.7		0.00	8049590	0	0.00	8049590	0

Soil water available (Clay loam, Silty clay loam, Silty clay and Loam), for cultivated land soil depth = 100cm and available moisture 150mm/m 150.0

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	8637098	483660.2	31.72	8637098	273942.84
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
|h_P-h_{ETp}| (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	93.0	2.3	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	45.6	90.7	2.3	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	135.3	2.3	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0							
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	SR						
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	100.0	1.4	0.0	1.7		0.00	8637098	0.0	0.00	8637098	0

Soil water available (Clay loam), for bush land soil depth = 150cm and available moisture 150mm/m 225.0

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	505232	28292.0	31.72	505232	16024.4
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
|h_P-h_{ETp}| (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	93.0	2.3	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	45.6	90.7	2.3	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	135.3	2.3	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0							
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	SR						
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	100.0	1.4	0.0	1.7		0.00	505232	0	0.00	505232	0

Soil water available (Loam), for bare land soil depth = 15cm and available moisture 150mm/m 22.5

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	1178283	65981.41	31.72	1178283	37371.6
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
|h_P-h_{ETp}| (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	22.5	22.5	0.6	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	22.5	0.0	21.9	0.6	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	66.5	0.6	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	24.9	45.6	0.0	0.0	0.0	0.0							
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	12.5	22.8	0.0	0.0	0.0	0.0	SR						
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	49.2	0.3	0.0	1.7		12.45	1178283	14673.8	22.8	1178283	26872.2

Soil water available (Clay), for forest land soil depth = 150cm and available moisture 200mm/m 300.0

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	94409	5286.7	31.72	94409	2994.3703
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
|h_P-h_{ETp}| (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	93.0	2.3	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	47.4	45.6	90.7	2.3	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	135.3	2.3	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0							
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	SR						
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	100.0	1.4	0.0	1.7		0.00	94409	0	0.0	94409	0

Soil water available (Clay loam), for homesteads soil depth = 15cm and available moisture 150mm/m 22.5

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.9	2.0	7.5	13.2	15.6	35.6	163.6	165.3	44.6	0.0	0.0	2.2		56.00	153084	8572.4	31.72	153084	4855.3652
h_{ETp}(mm)	128.1	137.1	174.1	177.8	184.3	143.1	116.2	119.7	135.3	163.0	136.4	133.1							
h_P-h_{ETp}(mm)	-127.2	-135.1	-166.6	-164.6	-168.7	-107.5	47.4	45.6	-90.7	-163.0	-136.4	-130.9							
|h_P-h_{ETp}| (mm)	127.2	135.1	166.6	164.6	168.7	107.5	47.4	45.6	90.7	163.0	136.4	130.9	DR						
h_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	22.5	22.5	0.6	0.0	0.0	0.0							
Δh_{SM} (mm)	0.0	0.0	0.0	0.0	0.0	0.0	22.5	0.0	21.9	0.6	0.0	0.0							
h_{ETact} (mm)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	66.5	0.6	0.0	2.2							
Excess	0.0	0.0	0.0	0.0	0.0	0.0	24.9	45.6	0.0	0.0	0.0	0.0							
h_{SR}=50%exces (mm)	0.0	0.0	0.0	0.0	0.0	0.0	12.5	22.8	0.0	0.0	0.0	0.0	SR						
(h_{ETact}/h_{ETp}) (%)	0.7	1.5	4.3	7.4	8.5	24.9	100.0	100.0	49.2	0.3	0.0	1.7		12.45	153084	1906.4	22.8	153084	3491.2
Δh_{SM}(watershed)	0.0	0.0	0.0	0.0	0.0	0.0	46.1	43.1	87.0	2.2	0.0	0.0							
h_{ETact}(watershed)	0.9	2.0	7.5	13.2	15.6	35.6	116.2	119.7	131.6	2.2	0.0	2.2							
Excess (watershed)	0.0	0.0	0.0	0.0	0.0	0.0	1.3	2.5	0.0	0.0	0.0	0.0							
h_{SR}(watershed)	0.0	0.0	0.0	0.0	0.0	0.0	0.7	1.23	0.0	0.0	0.0	0.0							

Appendix 3.6.3b: Watershed water balance computation table for Laelay Wukro (2007)

Month	**July**	**Aug**		
h_{Ptotal}(mm)	285.2	210.4		
DR_{frac}	0.032	0.043		
h_{DR}(mm)	9.1264	9.0472		
h_P(mm)	276.0	201.35	Error (%)	Error (%)
$h_{Vo}(m)^3$	176993.7	388869.9	-0.57	0.013
$h_{Vm}(m)^3$	178002.2	388810.6		

Soil water available, for cultivated land (Silt loam, Silt clay loam, Clay loam), soil depth = 100cm and available moisture175mm/m 175mm

														July			August		
MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.0	0.4	13.4	26.6	4.2	81.2	276.1	201.4	81.4	0.0	0.0	0.0		9.1	3875631	35370.6	9.0	3875631	35063.6
h_{ETp}(mm)	138.6	152.8	210.9	232.7	188.5	142.4	108.9	112.8	127.4	171.8	160.0	134.8							
h_P-h_{ETp}(mm)	-138.6	-152.4	-197.5	-206.1	-184.3	-61.2	167.2	88.6	-46.0	-171.8	-160.0	-134.8							
$\|h_P$-$h_{ETp}\|$ (mm)	138.63	152.40	197.50	206.14	184.33	61.24	167.20	88.59	46.02	171.76	160.04	134.81	DR						
h_{SM} (mm)	0.01	0.00	0.00	0.00	0.00	0.00	167.20	175.00	128.98	2.39	0.20	0.05							
Δh_{SM} (mm)	0.04	0.01	0.00	0.00	0.00	0.00	167.20	7.80	46.02	126.59	2.18	0.16							
h_{ETact} (mm)	0.04	0.41	13.40	26.60	4.20	81.20	108.87	112.76	127.42	126.59	2.18	0.16							
Excess	0.00	0.00	0.00	0.00	0.00	0.00	0.00	80.79	0.00	0.00	0.00	0.00		h_{SR}	Area	V_{SR}	h_{SR}	Area	V_{SR}
h_{SR}=50%exces (mm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	40.40	0.00	0.00	0.00	0.00	SR	(mm)	(m^2)	(m^3)	(mm)	(m^2)	(m^3)
(h_{ETact}/h_{ETp}) (%)	0.03	0.27	6.35	11.43	2.23	57.01	100.00	100.00	100.00	73.70	1.36	0.12		0.00	3875631	0	40.40	3875631	156563

Soil water available, for bush land (Silt loam, Silt clay loam), soil depth = 150cm and available moisture 150mm/m 225mm

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.0	0.4	13.4	26.6	4.2	81.2	276.1	201.4	81.4	0.0	0.0	0.0		9.1	3734336	34081.0	9.0	3875631	35063.6
h_{ETp}(mm)	138.6	152.8	210.9	232.7	188.5	142.4	108.9	112.8	127.4	171.8	160.0	134.8							
h_P-h_{ETp}(mm)	-138.63	-152.40	-197.50	-206.14	-184.33	-61.24	167.20	88.59	-46.02	-171.76	-160.04	-134.81							
|h_P-h_{ETp}| (mm)	138.63	152.40	197.50	206.14	184.33	61.24	167.20	88.59	46.02	171.76	160.04	134.81	DR						
h_{SM} (mm)	0.01	0.00	0.00	0.00	0.00	0.00	167.20	225.00	165.83	3.07	0.26	0.06							
Δh_{SM} (mm)	0.05	0.01	0.00	0.00	0.00	0.00	167.20	57.80	59.17	162.76	2.80	0.20							
h_{ETact} (mm)	0.05	0.41	13.40	26.60	4.20	81.20	108.87	112.76	140.57	162.76	2.80	0.20							
Excess	0.00	0.00	0.00	0.00	0.00	0.00	0.00	30.79	13.15	0.00	0.00	0.00		h_{SR}	Area	SR	h_{SR}	Area	V_{SR}
h_{SR}=50%exces (mm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	15.40	6.57	0.00	0.00	0.00	SR	(mm)	(m^2)	(m^3)	(mm)	(m^2)	(m^3)
(h_{ETact}/h_{ETp}) (%)	0.03	0.27	6.35	11.43	2.23	57.01	100.00	100.00	110.32	94.76	1.75	0.15		0.00	3734336	0	15.40	3875631	59672

Soil water available, for grass land (Silt loam), soil depth = 45cm and available moisture 150mm/m 67.5mm

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.0	0.4	13.4	26.6	4.2	81.2	276.1	201.4	81.4	0.0	0.0	0.0		9.1	169039	1542.7	9.0	169039	1529.33
h_{ETp}(mm)	138.6	152.8	210.9	232.7	188.5	142.4	108.9	112.8	127.4	171.8	160.0	134.8							
h_P-h_{ETp}(mm)	-138.63	-152.40	-197.50	-206.14	-184.33	-61.24	167.20	88.59	-46.02	-171.76	-160.04	-134.81							
|h_P-h_{ETp}| (mm)	138.63	152.40	197.50	206.14	184.33	61.24	167.20	88.59	46.02	171.76	160.04	134.81							
h_{SM} (mm)	0.00	0.00	0.00	0.00	0.00	0.00	67.50	67.50	49.75	0.92	0.08	0.02	DR						
Δh_{SM} (mm)	0.01	0.00	0.00	0.00	0.00	0.00	67.50	0.00	17.75	48.83	0.84	0.06							
h_{ETact} (mm)	0.01	0.40	13.40	26.60	4.20	81.20	108.87	112.76	99.15	48.83	0.84	0.06							
Excess	0.00	0.00	0.00	0.00	0.00	0.00	99.70	88.59	0.00	0.00	0.00	0.00		h_{SR}	Area	V_{SR}	h_{SR}	Area	V_{SR}
h_{SR}=50%exces (mm)	0.00	0.00	0.00	0.00	0.00	0.00	49.85	44.30	0.00	0.00	0.00	0.00	SR	(mm)	(m^2)	(m^3)	(mm)	(m^2)	(m^3)
(h_{ETact}/h_{ETp}) (%)	0.01	0.26	6.35	11.43	2.23	57.01	100.00	100.00	77.81	28.43	0.53	0.04		49.85	169039	8426.5	44.30	169039	7487.92

Soil water available, for pasture land (Silt clay loam, Silt loam), soil depth = 50cm and available moisture 150mm/m 75mm

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.0	0.4	13.4	26.6	4.2	81.2	276.1	201.4	81.4	0.0	0.0	0.0		9.1	1667198	15215.5	9.0	1667198	15083.5
h_{ETp}(mm)	138.6	152.8	210.9	232.7	188.5	142.4	108.9	112.8	127.4	171.8	160.0	134.8	DR						
h_P-h_{ETp}(mm)	-138.63	-152.40	-197.50	-206.14	-184.33	-61.24	167.20	88.59	-46.02	-171.76	-160.04	-134.81							
$\lvert h_P-h_{ETp}\rvert$ (mm)	138.63	152.40	197.50	206.14	184.33	61.24	167.20	88.59	46.02	171.76	160.04	134.81							
h_{SM} (mm)	0.00	0.00	0.00	0.00	0.00	0.00	75.00	75.00	55.28	1.02	0.09	0.02							
Δh_{SM} (mm)	0.02	0.00	0.00	0.00	0.00	0.00	75.00	0.00	19.72	54.25	0.93	0.07							
h_{ETact} (mm)	0.02	0.40	13.40	26.60	4.20	81.20	108.87	112.76	101.12	54.25	0.93	0.07							
Excess	0.00	0.00	0.00	0.00	0.00	0.00	92.20	88.59	0.00	0.00	0.00	0.00		h_{SR}	Area	V_{SR}	h_{SR}	Area	V_{SR}
h_{SR}=50%exces (mm)	0.00	0.00	0.00	0.00	0.00	0.00	46.10	44.30	0.00	0.00	0.00	0.00	SR	(mm)	(m^2)	(m^3)	(mm)	(m^2)	(m^3)
(h_{ETact}/h_{ETp}) (%)	0.00	0.00	0.06	0.11	0.02	0.57	1.00	1.00	0.79	0.32	0.01	0.00		46.10	1667198	76857.4	44.30	1667198	73851.9

Soil water available, for forest land (Silty clay loam), soil depth = 200cm and available moisture 150mm/m 300mm

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
h_P(mm)	0.0	0.4	13.4	26.6	4.2	81.2	276.1	201.4	81.4	0.0	0.0	0.0		9.1	76383	697.1	9.0	76383	691.0
h_{ETp}(mm)	138.6	152.8	210.9	232.7	188.5	142.4	108.9	112.8	127.4	171.8	160.0	134.8							
h_P-h_{ETp}(mm)	-138.63	-152.40	-197.50	-206.14	-184.33	-61.24	167.20	88.59	-46.02	-171.76	-160.04	-134.81							
$\lvert h_P-h_{ETp}\rvert$ (mm)	138.63	152.40	197.50	206.14	184.33	61.24	167.20	88.59	46.02	171.76	160.04	134.81							
h_{SM} (mm)	0.01	0.00	0.00	0.00	0.00	0.00	167.20	255.79	188.52	3.49	0.30	0.07							
Δh_{SM} (mm)	0.05	0.01	0.00	0.00	0.00	0.00	167.20	88.59	67.27	185.04	3.19	0.23	DR						
h_{ETact} (mm)	0.05	0.41	13.40	26.60	4.20	81.20	108.87	112.76	148.67	185.04	3.19	0.23							
Excess	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	21.25	0.00	0.00	0.00		h_{SR}	Area	V_{SR}	h_{SR}	Area	V_{SR}
h_{SR}=50%exces (mm)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	10.62	0.00	0.00	0.00	SR	(mm)	(m^2)	(m^3)	(mm)	(m^2)	(m^3)
(h_{ETact}/h_{ETp}) (%)	0.04	0.27	6.35	11.43	2.23	57.01	100.00	100.00	116.68	107.73	1.99	0.17		0.00	76383	0	0.00	76383	0

Soil water available, for homesteads (Clay loam), soil depth = 15cm and available moisture 150mm/m 22.5mm

MONTH	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
h_P(mm)	0.0	0.4	13.4	26.6	4.2	81.2	276.1	201.4	81.4	0.0	0.0	0.0
h_{ETp}(mm)	138.6	152.8	210.9	232.7	188.5	142.4	108.9	112.8	127.4	171.8	160.0	134.8
h_P-h_{ETp}(mm)	-138.63	-152.40	-197.50	-206.14	-184.33	-61.24	167.20	88.59	-46.02	-171.76	-160.04	-134.81
$\|h_P$-$h_{ETp}\|$ (mm)	138.63	152.40	197.50	206.14	184.33	61.24	167.20	88.59	46.02	171.76	160.04	134.81
h_{SM} (mm)	0.00	0.00	0.00	0.00	0.00	0.00	22.50	22.50	16.58	0.31	0.03	0.01
Δh_{SM} (mm)	0.00	0.00	0.00	0.00	0.00	0.00	22.50	0.00	5.92	16.28	0.28	0.02
h_{ETact} (mm)	0.00	0.40	13.40	26.60	4.20	81.20	108.87	112.76	87.32	16.28	0.28	0.02
Excess	0.00	0.00	0.00	0.00	0.00	0.00	144.70	88.59	0.00	0.00	0.00	0.00
h_{SR}=50%exces (mm)	0.00	0.00	0.00	0.00	0.00	0.00	72.35	44.30	0.00	0.00	0.00	0.00
(h_{ETact}/h_{ETp}) (%)	0.00	0.26	6.35	11.43	2.23	57.01	100.00	100.00	68.53	9.48	0.18	0.01
Δh_{SM}(watershed)	0.04	0.01	0.00	0.00	0.00	0.00	148.34	26.35	45.94	126.37	2.18	0.16
h_{ETact}(watershed)	0.04	0.41	13.40	26.60	4.20	81.20	108.87	112.76	127.34	126.37	2.18	0.16
Excess (watershed)	0.00	0.00	0.00	0.00	0.00	0.00	18.85	62.24	5.29	0.00	0.00	0.00
h_{SR}(watershed)	0.00	0.00	0.00	0.00	0.00	0.00	9.43	31.12	2.64	0.00	0.00	0.00

Runoff component	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)	h_{DR} (mm)	Area (m^2)	V_{DR} (m^3)
DR	9.1	71326	650.95	9.0	71326	645.3
	h_{SR} (mm)	Area (m^2)	V_{SR} (m^3)	h_{SR} (mm)	Area (m^2)	V_{SR} (m^3)
SR	72.35	71326	5160.4	44.30	71326	3159.53

Appendix 4: Appendix to Chapter 4

Appendix 4.5: Curve fitting for runoff coefficients for Figure 4-1, Figure 4-2 and Figure 4-3

Appendix 4.5a: Curve fitting for runoff coefficient extension for the given precipitation within the same AMC group for Figure 4.1.

AMC	Precipitation (mm)	Curve fitting	
		Equation	R^2
AMC I	16	$C = 1.108\times10^{-21}\times(CN)^{10.457}$	0.999
AMC I	21	$C = 0.4834\times10^{-11}\times(CN)^{5.0364}$	0.999
AMC II	19	$C = 3.95\times10^{-28}\times(CN)^{13.696}$	0.999
AMC II	22	$C = 2\times10^{-26}\times(CN)^{12.822}$	0.999
AMC II	34	$C = 0.98\times10^{-16}\times(CN)^{7.9974}$	0.999
AMC II	44	$C = 0.1325\times10^{-12}\times(CN)^{5.922}$	1.000
AMC III	13	$C = 4.6\times10^{-31}\times(CN)^{15.167}$	0.999
AMC III	21	$C = 4.808\times10^{-20}\times(CN)^{9.6396}$	0.999
AMC III	25	$C = 1.853\times10^{-18}\times(CN)^{8.8602}$	0.999
AMC III	38	$C = 1.37\times10^{-12}\times(CN)^{5.9245}$	0.999

Note: the fitted curves can be also used for interpolation of rainfall depths in between the indicated depths

Appendix 4.5b: Curve fitting for runoff coefficient extension for the given precipitation within the same AMC group for Figure 4.2.

AMC	Precipitation (mm)	Curve fitting	
		Equation	R^2
AMC I	27	$C = 2.803\times10^{-8}\times(CN)^{3.6338}$	0.999
AMC I	32	$C = 2.295\times10^{-7}\times(CN)^{3.1856}$	0.999
AMC II	39	$C = 3.3591\times10^{-17}\times(CN)^{8.2873}$	0.999
AMC II	65	$C = 1.0795\times10^{-9}\times(CN)^{4.4869}$	0.999
AMC III	21	$C = 2.127\times10^{-22}\times(CN)^{10.889}$	0.999
AMC III	26	$C = 2.034\times10^{-20}\times(CN)^{9.9043}$	0.999
AMC III	46	$C = 4.596\times10^{-13}\times(CN)^{6.2068}$	0.997

Appendix 4.5c: curve fitting for runoff coefficient extension for the given precipitation within the same AMC group for Figure 4.3.

AMC	Precipitation (mm)	Curve fitting	
		Equation	R^2
AMC I	27	$C = 4.6381\times10^{-8}\times(CN)^{3.5252}$	0.952
AMC I	32	$C = 2.0839\times10^{-8}\times(CN)^{3.7437}$	0.999
AMC II	39	$C = 1.58\times10^{-15}\times(CN)^{7.4076}$	1.000
AMC II	65	$C = 1.963\times10^{-9}\times(CN)^{4.35047}$	0.999
AMC III	21	$C = 4.554\times10^{-21}\times(CN)^{10.196}$	0.999
AMC III	26	$C = 2.2439\times10^{-19}\times(CN)^{9.3615}$	1.000
AMC III	52	$C = 6.319\times10^{-12}\times(CN)^{5.6335}$	0.881

Appendix 4.6: Curve fitting for filling runoff coefficients for Table 4.8, Table 4-9

Appendix 4.6a: Curve fitting for filling runoff coefficients for different landuse, soil and precipitation combinations (Curves used to fill gaps for Table 4.8)

AMC	Landuse	HSG	Curve fitting	
			Equation	R^2
AMC I	Cultivated land	D	$C = 0.0243 \times h_P - 0.3248$	0.93
AMC I	Cultivated land	C	$C = 0.0142 \times h_P - 0.1987$	0.99
AMC II	Cultivated land	D	$C = 0.0145 \times h_P - 0.1356$	0.98
AMC II	Cultivated land	C	$C = 0.0076 \times h_P - 0.0467$	1.00
AMC III	Cultivated land	D	$C = 0.014 \times h_P + 0.0767$	0.99
AMC III	Cultivated land	C	$C = 0.0053 \times h_P + 0.1229$	0.82

Appendix 4.6b: Curve fitting for filling runoff coefficients for different landuse, soil and precipitation combinations (Curves used to fill gaps for Table 4.9)

AMC	Landuse	HSG	Curve fitting	
			Equation	R^2
AMC I	Bushes, Forest and Exclosure	D	$C = 0.0186 \times h_P - 0.2962$	0.99
AMC I	Bushes, Forest and Exclosure	C	$C = 0.008 \times h_P - 0.1288$	0.99
AMC II	Bushes, Forest and Exclosure	D	$C = 0.0141 \times h_P - 0.2845$	0.98
AMC II	Bushes, Forest and Exclosure	C	$C = 0.0056 \times h_P - 0.1169$	0.99
AMC III	Bushes, Forest and Exclosure	D	Interpolation with adjacent data	
AMC III	Bushes, Forest and Exclosure	C	$C = 0.0109 \times h_P + 0.1311$	0.93

Erklärung des Studenten

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