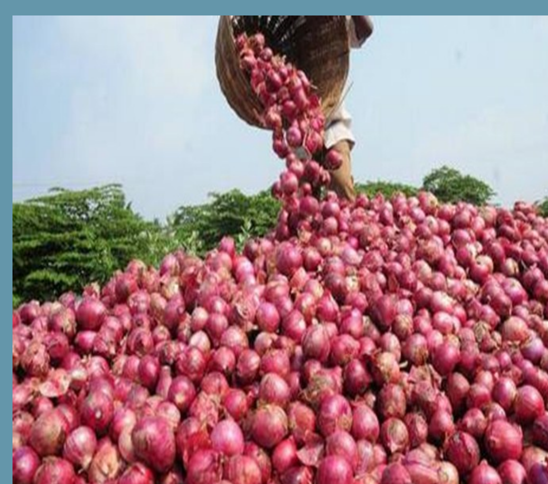
SSIGL 3

NATIONAL GUIDELINES

For Small Scale Irrigation Development in Ethiopia

Hydrology and Water Resources Planning



November 2018

Addis Ababa

MINISTRY OF AGRICULTURE

National Guidelines for Small Scale Irrigation Development in Ethiopia

SSIGL 3: Hydrology and Water Resources Planning

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First Edition 2018

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Financed by Agricultural Growth Program (AGP)

FORWARD

Ministry of Agriculture, based on the national strategic directions is striving to meet its commitments in which modernizing agriculture is on top of its highest priorities to sustain the rapid, broad-based and fair economic growth and development of the country. To date, major efforts have been made to remodel several important strategies and national guidelines by its major programs and projects.

While efforts have been made to create access to irrigation water and promoting sustainable irrigation development, several barriers are still hindering the implementation process and the performance of the schemes. The major technical constrains starts from poor planning and identification, study, design, construction, operation, and maintenance. One of the main reasons behind this outstanding challenge, in addition to the capacity limitations, is that SSIPs have been studied and designed using many ad-hoc procedures and technical guidelines developed by various local and international institutions.

Despite having several guidelines and manuals developed by different entities such as MoA (IDD)-1986, ESRDF-1997, MoWIE-2002 and JICA/OIDA-2014, still the irrigation professionals follow their own public sources and expertise to fill some important gaps. A number of disparities, constraints and outstanding issues in the study and design procedures, criteria and assumptions have been causing huge variations in all vital aspects of SSI study, design and implementation from region to region and among professionals within the same region and institutions due mainly to the lack of agreed standard technical guidelines. Hence, the SSI Directorate with AGP financial support, led by Generation consultant (GIRDC) and with active involvement of national and regional stakeholders and international development partners, these new and comprehensive national guidelines have been developed.

The SSID guidelines have been developed by addressing all key features in a comprehensive and participatory manner at all levels. The guidelines are believed to be responsive to the prevalent study and design contentious issues; and efforts have been made to make the guidelines simple, flexible and adaptable to almost all regional contexts including concerned partner institution interests. The outlines of the guidelines cover all aspects of irrigation development including project initiation, planning, organizations, site identification and prioritization, feasibility studies and detail designs, contract administration and management, scheme operation, maintenance and management.

Enforceability, standardization, social and environmental safeguard mechanisms are well mainstreamed in the guidelines, hence they shall be used as a guiding framework for engineers and other experts engaged in all SSI development phases. The views and actual procedures of all relevant diverse government bodies, research and higher learning institutions, private companies and development partners has been immensely and thoroughly considered to ensure that all stakeholders are aligned and can work together towards a common goal. Appropriately, the guidelines will be familiarized to the entire stakeholders working in the irrigation development. Besides, significant number of experts in the corresponding subject matter will be effectively trained nationwide; and the guidelines will be tested practically on actual new and developing projects for due consideration of possible improvement. Hence, hereinafter, all involved stakeholders including government & non-governmental organizations, development partners, enterprises, institutions, consultants and individuals in Ethiopia have to adhere to these comprehensive national guidelines in all cases and at all level whilst if any overlooked components are found, it should be documented and communicated to MOA to bring them up-to-date.

Therefore, I congratulate all parties involved in the success of this effort, and urge partners and stakeholders to show a similar level of engagement in the implementation and stick to the guidelines over the coming years.

H.E. Dr. Kaba Urgessa
State Minister, Ministry of Agriculture

SMALL SCALE IRRIGATION DEVELOPMENT VISION

Transforming agricultural production from its dependence on rain-fed practices by creating reliable irrigation system in which smallholder farmers have access to at least one option of water source to increase production and productivity as well as enhance resilience to climate change and thereby ensure food security, maintain increasing income and sustain economic growth.

ACKNOWLEDGEMENTS

The preparation of SSIGLs required extensive inputs from all stakeholders and development partners. Accordingly many professionals from government and development partners have contributed to the realization of the guidelines. To this end MOA would like to extend sincere acknowledgement to all institutions and individuals who have been involved in the review of these SSIGLs for their comprehensive participation, invaluable inputs and encouragement to the completion of the guidelines. There are just too many collaborators involved to name exhaustively and congratulate individually, as many experts from Federal, regional states and development partners have been involved in one way or another in the preparation of the guidelines. The contribution of all of them who actively involved in the development of these SSIGLs is gratefully acknowledged. The Ministry believes that their contributions will be truly appreciated by the users for many years to come.

The Ministry would like to extend its appreciation and gratitude to the following contributors:

- Agriculture Growth Program (AGP) of the MoA for financing the development and publication of the guidelines.
- The National Agriculture Water Management Platform (NAWMP) for overseeing, guidance and playing key supervisory and quality control roles in the overall preparation process and for the devotion of its members in reviewing and providing invaluable technical inputs to enrich the guidelines.
- Federal Government and Regional States organizations and their staff for their untiring effort in reviewing the guidelines and providing constructive suggestions, recommendations and comments.
- National and international development partners for their unreserved efforts in reviewing the guidelines and providing constructive comments which invaluably improved the quality of the guidelines.
- Small-scale and Micro Irrigation Support Project (SMIS) and its team for making all efforts to have quality GLs developed as envisioned by the Ministry.

The MOA would also like to extend its high gratitude and sincere thanks to AGP's multi development partners including the International Development Association (IDA)/World Bank, the Canada Department of Foreign Affairs, Trade and Development (DFATD), the United States Agency for International Development (USAID), the Netherlands, the European Commission (EC), the Spanish Agency for International Development (AECID), the Global Agriculture and Food Security Program (GAFSP), the Italy International Development Cooperation, the Food and Agriculture Organization (FAO) and the United Nations Development Program (UNDP).

Moreover, the Ministry would like to express its gratitude to Generation Integrated Rural Development Consultant (GIRDC) and its staff whose determined efforts to the development of these SSIGLs have been invaluable. GIRDC and its team drafted and finalized all the contents of the SSIGLs as per stakeholder suggestions, recommendations and concerns. The MoA recognizes the patience, diligence, tireless, extensive and selfless dedication of the GIRDC and its staff who made this assignment possible.

Finally, we owe courtesy to all national and International source materials cited and referred but unintentionally not cited.

Ministry of Agriculture

DEDICATIONS

The National Guidelines for Small Scale Irrigation Development are dedicated to Ethiopian smallholder farmers, agro-pastoralists, pastoralists, to equip them with appropriate irrigation technology as we envision them empowered and transformed.

LIST OF GUIDELINES

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ACRONYMS

AGP	Agricultural Growth Program
cms	cubic meter per second
DEM	Digital Elevation Model
EFA	Environmental Flow Assessment
EMA	Ethiopian Mapping Agency
ERH	Rainfall Excess Hyetograph
FAO	Food and Agriculture Organization
FDC	Flow Duration Curve
GDP	Gross Domestic Product
GIS	Geographic Information System
GPS	Global Positioning System
HFR	High Flow Requirement
HWSD	Harmonized World Soil Database
IDF	Rainfall Intensity Duration
ITCZ	Inter-tropical Convergence Zone
LFR	Low Flow Requirement
MAF	Mean Annual Flow
MOWIE	Ministry of Water, Irrigation and Electricity
NCEP	National Center for Environmental Prediction
NRCS	Natural Resources Conservation Service
RFE	Rainfall estimates
RSCZ	Red Sea Convergence Zone
RUSLE	Revised Universal Soil Loss Equation
SDR	Sediment Delivery Ratio
SNNPR	Southern Nations, Nationalities and Peoples' Region
SSI	Small Scale Irrigation
SSID	Small Scale Irrigation Development
SSIGL	Small Scale Irrigation Guideline
SSIP	Small Scale Irrigation Project
SSIS	Small Scale Irrigation Scheme
STJ	Subtropical Jet
SY	Sediment Yield
TEJ	Tropical Easterly Jet
TRMM	Tropical Rainfall Measuring Mission

PREFACE

While irrigation development is at the top of the government's priority agendas as it is key to boost production and improve food security as well as to provide inputs for industrial development. Accordingly, irrigated land in different scales has been aggressively expanding from time to time. To this end, to enhance quality delivery of small-scale irrigation development planning, implementation and management, it has been decided to develop standard SSI guidelines that must be nationally applied. In September 2017 the Ministry of Agriculture (MoA) had entrusted Generation Integrated Rural Development Consultant (GIRDC) to prepare the National Small-scale Irrigation Development Guidelines (SSIGLs).

Preparation of the SSIGLs for enhancing development of irrigated agriculture is recognized as one of the many core initiatives of the MoA to improve its delivery system and achieve the targets in irrigated agriculture and fulfill its mission for improving agricultural productivity and production. The core objective of developing SSIGLs is to summarize present thinking, knowledge and practices to enable irrigation practitioners to properly plan, implement and manage community managed SSI schemes to develop the full irrigation potential in a sustainable manner.

As the SSIGLs are prepared based on national and international knowledge, experiences and practices, and describe current and recommended practice and set out the national standard guides and procedures for SSI development, they serve as a source of information and provide guidance. Hence, it is believed that the SSIGLs will contribute to ensuring the quality and timely delivery, operation and maintenance of SSI schemes in the country. The SSIGLs attempt to explain and illustrate the important concepts, considerations and procedures in SSI planning, implementation and management; and shall be used as a guiding framework for professionals engaged in SSI development. Illustrative examples from within the country have been added to enable the users understand the contents, methodologies presented in the SSIGLs.

The intended audiences of the SSIGLs are government organizations, NGOs, CSOs and the private sector involved in SSI development. Professionally, the SSIGLs will be beneficial for experienced and junior planners, experts, contractors, consultants, suppliers, investors, operators and managers of SSI schemes. The SSIGLs will also serve as a useful reference for academia and researchers involved and interested in SSI development. The SSIGLs will guide to ensure that; planning, implementation and management of SSI projects is formalized and set procedures and processes to be followed. As the SSIGLs provide information and guides they must be always fully considered and applied by adapting them to the local specific requirements.

In cognizance with the need for quality SSIGLs, the MoA has duly considered quality assurance and control during preparation of the guidelines. Accordingly, the outlines, contents and scope of the SSIGLs were thoroughly discussed, reviewed and modified by NAWMP members (senior professionals from public, national and international stakeholder) with key stakeholders in many consultative meetings and workshops. Moreover, at each milestone of SSIGL preparation, resource persons from all stakeholders reviewed and confirmed that SSIGLs have met the demands and expectations of users.

Moreover, the Ministry has mobilized resource persons from key Federal, National Regional States level stakeholders and international development partners for review, validation and endorsement of the SSIGLs.

Several hundreds of experienced professionals (who are very qualified experts in their respective fields) from government institutions, relevant private sector and international development partners have significantly contributed to the preparation of the SSIGLs. They have been involved in all aspects of the development of SSIGLs throughout the preparation process. The preparation process included a number of consultation meetings and workshops: (i) workshop to review inception report, (ii) workshop on findings of review of existing guidelines/manuals and proposed contents of the SSIGLs, (iii) meetings to review zero draft SSI GLs, (iv) review workshop on draft SSI GLs, (v) small group review meetings on thematic areas, (vi) small group consultation meetings on its final presentation of contents and layout, (vii) consultation mini-workshops in the National States on semi-final versions of the SSIGLs, and (viii) final write-shop for the appraisal and approval of the final versions of SSIGLs.

The deliberations, concerns, suggestions and comments received from professionals have been duly considered and incorporated by the GIRD Consultant in the final SSIGLs.

There are 34 separate guidelines which are categorized into the following five parts concurrent to SSI development phases:

Part-I. Project Initiation, Planning and Organization Guideline which deals with key considerations and procedures on planning and organization of SSI development projects.

Part-II. Site Identification and Prioritization Guideline which treats physical potential identification and prioritization of investment projects. It presents SSI site selection process and prioritization criteria.

Part-III. Feasibility Study and Detail Design Guidelines for SSID dealing with feasibility study and design concepts, approaches, considerations, requirements and procedures in the study and design of SSI systems.

Part-IV. Contract Administration and Construction Management Guidelines for SSI development presents the considerations, requirements, and procedures involved in construction of works, construction supervision and contract administration.

Part-V. SSI Scheme Management, Operation and Maintenance Guidelines which covers SSI Scheme management and operation.

Moreover, Tools for Small Scale Irrigation development are also prepared as part of SSIGLs.

It is strongly believed and expected that; the SSIGLs will be quickly applied by all stakeholders involved in SSI development and others as appropriate following the dissemination and familiarization process of the guidelines in order to ensure efficient, productive and sustainable irrigation development.

The SSIGLs are envisioned to be updated by incorporating new technologies and experiences including research findings. Therefore, any suggestions, concerns, recommendations and comments on the SSIGLs are highly appreciated and welcome for future updates as per the attached format below. Furthermore, despite efforts in making all types of editorial works, there may still errors, which similarly shall be handled in future undated versions.

UPDATING AND REVISIONS OF GUIDELINES

The GLs are intended as an up-to-date or a live document enabling revisions, to be updated periodically to incorporate improvements, when and where necessary; may be due to evolving demands, technological changes and changing policies, and regulatory frameworks. Planning, study and design of SSI development interventions is a dynamic process. Advancements in these aspects are necessary to cope up with the changing environment and advancing techniques. Also, based on observation feedbacks and experiences gained during application and implementation of the guidelines, there might be a need to update the requirements, provisions and procedures, as appropriate. Besides, day-by-day, water is becoming more and more valuable. Hence, for efficient water development, utilization and management will have to be designed, planned and constructed with a new set up of mind to keep pace with the changing needs of the time. It may, therefore, be necessary to take up the work of further revision of these GLs.

This current version of the GLs has particular reference to the prevailing conditions in Ethiopia and reflects the experience gained through activities within the sub-sector during subsequent years. This is the first version of the SSI development GLs. This version shall be used as a starting point for future update, revision and improvement. Future updating and revisions to the GLs are anticipated as part of the process of strengthening the standards for planning, study, design, construction, operation and management SSI development in the country.

Completion of the review and updating of the GLs shall be undertaken in close consultation with the federal and regional irrigation institutions and other stakeholders in the irrigation sub-sector including the contracting and consulting industry.

In summary, significant changes to criteria, procedures or any other relevant issues related to technological changes, new policies or revised laws should be incorporated into the GLs from their date of effectiveness. Other minor changes that will not significantly affect the whole nature of the GLs may be accumulated and made periodically. When changes are made and approved, new page(s) incorporating the revision, together with the revision date, will be issued and inserted into the relevant GL section.

All suggestions to improve the GLs should be made in accordance with the following procedures:

I. Users of the GLs must register on the MOA website: Website: www.moa.gov.et
II. Proposed changes should be outlined on the GLs Change Form and forwarded with a covering letter or email of its need and purpose to the Ministry.
III. Agreed changes will be approved by the Ministry on recommendation from the Small-scale Irrigation Directorate and/or other responsible government body.
IV. The release date of the new version will be notified to all registered users and authorities.

Users are kindly requested to present their concerns, suggestions, recommendations and comments for future updates including any omissions and/or obvious errors by completing the following revisions form and submitting it to the Ministry. The Ministry shall appraise such requests for revision and will determine if an update to the guide is justified and necessary; and when such updates will be published. Revisions may take the form of replacement or additional pages. Upon receipt, revision pages are to be incorporated in the GLs and all superseded pages removed.

Suggested Revisions Request Form (Official Letter or Email)

To: --
From: ---
Date: --

Description of suggested updates/changes: Include GL code and title, section title and # (heading/subheading #), and page #.

GL Code and Title	Date	Sections/ Heading/Subheading/ Pages/Table/Figure	Explanation	Comments (proposed change)

Note that be specific and include suggested language if possible and include additional sheets for comments, reference materials, charts or graphics.

GLs Change Action

Suggested Change	Recommended Action	Authorized by	Date

Director for SSI Directorate**: ___________________________Date: ___________________**

The following table helps to track initial issuance of the guidelines and subsequent Updates/Versions and Revisions (Registration of Amendments/Updates).

Revision Register

Version/Issue/Revision No	Reference/Revised Sections/Pages/topics	Description of revision (Comments)	Authorized by	Date

1 INTRODUCTION

1.1 GENERAL

Ethiopia has good irrigation potential both in terms of suitable land and availability of fresh water resources. However, irrigation development in the country is in its infancy stage and does not significantly contribute its share to the growth of the agriculture sector and the national GDP. Irrigated agriculture is performing low due to a number of factors one of which is poor system design due to inadequate hydrological analysis which, in turn, is due to limited hydro-meteorological data and appropriate methodology for undertaking such analysis.

Hydrological analysis is required for estimating available and reliable of water resources for a specific purpose such as for irrigation, municipality water supply or for other purposes, flood peaks and probability of their occurrence for the design of civil or hydraulic structures such as dams, diversions weir, bridges and others. Poor hydrological analysis due to lack of accurate and adequate data or inappropriate methodology will result in wrong design hydrological parameters for the design of civil/hydraulic structures consequently causing their failure. Many irrigation schemes have totally failed or partially function due to poor hydrological design of the head works and canal structures.

Therefore, this Guideline is prepared as part of the many guidelines for small-scale irrigation development to enable undertaking hydro-meteorological analysis and provide hydrologic design parameters. The Guideline is framed with the understanding of all the complexity in relation to the hydrological quantification as input for the design of hydraulic structures. The guideline covers:

- Analysis of Catchment features;
- Techniques with steps for analysis of climatic and rainfall data
- Methods of stream and spring flows and design floods
- Water balance analysis linking upstream and downstream conditions
- Sediment analysis
- Drainage-modules

1.2 OBJECTIVE OF THE GUIDELINE

Analysis of peak rate of runoff, volume of runoff and time distribution of flow is fundamental for the design of any hydraulic structures for different purposes. Therefore, the objective of this hydrological guideline is to provide appropriate and simple methodology and procedures for water resources assessment and quantification of hydro-meteorological parameters for the design small-scale irrigation systems including micro-dams, diversion weirs and spring development as well as drainage of irrigation command areas. It provides an understanding of all the complexity of the hydrological cycle and their quantification as input for the design of hydraulics structures.

1.3 SCOPE OF THE GUIDELINE

The scope of the guideline is limited to hydro-meteorological analysis of small catchments that supply water for small-scale irrigation systems. The guideline covers delineation and hydrological analysis of project catchment area, description of the catchment parameters characteristics (location, topographic, land cover, soil condition, and climatic features), analysis of climatic and rainfall, stream and spring flows, drainage-modules, sediment analysis, upstream and downstream linkage for water balance assessment. The guideline also provides different software’s and Excel package tools that support the different hydrological analyses within the scope of the guideline. The guideline presents different software’s and Excels package tools to support the different hydrological computations.

1.4 DEFINITIONS OF TERMINOLOGIES

Catchment features: the features of a catchment that including area, river network, flow length, catchment slope, and others topographical parameters to be input for hydrological analysis

10-85 percent points: the point on the main channel of a catchment that use its elevations and length between them to compute the catchment average slope. It is preferred to avoid the extreme high and extreme low elevations influence on the channel slope computation

CMORPH (CPC MORPHing technique): It produces global precipitation analyses at very high spatial and temporal resolution. This technique uses precipitation estimates that have been derived from low orbiter satellite microwave observations exclusively, and whose features are transported via spatial propagation information that is obtained entirely from geostationary satellite IR data

Tropical Rainfall Measuring Mission (TRMM): satellite based rainfall products developed with a joint mission of NASA and the Japan Aerospace Exploration Agency, was launched in 1997 to study rainfall for weather and climate research

Rainfall estimates (RFE): Satellite based rainfall developed as the part of the Early Warning Focus Area at the USGS Earth Resources Observation and Science (EROS) by the USGS FEWS NET Project,

RAINFALL – INTENSITY – DURATION (IDF): is a mathematical relationship between rainfall intensity, duration and return period that was developed in several parts of the world for its practical use

2 ANALYSIS OF CATCHMENT FEATURES

Catchment features including, area, channel network, ponds depressions, reservoirs, and other natural or constructed features, in a catchment affect the runoff from it. Therefore, data that describe catchment features, which are mainly a function of topography, soil and land use/cove have significant role on the hydrological analysis for irrigation system design. Therefore, it is essential to follow step by step approach to analyze such catchment features of project area hydrological analysis.

2.1 CATCHMENT DELINEATION

2.1.1 Data sources and tools

Digital Elevation Model (DEM) and outlet point location are the only data inputs for catchment delineation. The DEM with 30m resolution or possible smaller resolution can be accessed from Ethiopian Mapping Agency (EMA), from Ethio-GIS Dataset, SRTM website[1] or ASTER website[2]. All the DEM data access from different sources or websites may have quality limitations, which needs quality check and verification using specific location GPS or Surveying data before using for catchment delineation. The location of outlet points possible get from GPS reading of field observation, 1:50,000 topographic map of EMA, or Google Earth site view. Catchment delineation can be made using GIS and hydrological software's including ArcHydrotools, Global Mapper, QGIS, SWAT Delineator, HECGEOHMS, or any other surface analysis tools.

2.1.2 Procedures for catchment delineation

The procedure of the catchment feature analysis is explained with practical illustrations using GPS location of the catchment of Bench Maji Small Scale Irrigation project in SNNPR as presented in Table 2-1 below.

Table 2-1 : Identified. Bench Maji SSPI (2015)

Wereda	Site Name	Easting	Northing	Wereda	Site Name	Easting	Northing
North Bench	Petu	793,103.16	788,943.06	Me'ent Goldiya	Moli	814,804.35	752,404.74
	Dama	794,451.99	784,596.83		Olmu	814,354.74	739,785.68
	Gacheb	790,045.81	777,433.04		Shawel	817,232.24	747,099.34
South Bench	Zelmi	777,366.80	757,290.50	Shewe Bench	Bayni	804,403.36	770,958.66
	Shor	783,751.27	757,080.69		Kashu	804,553.23	758,789.20
	Kaki	788,577.08	754,952.53		Yem	807,610.58	760,557.67

Terrain Preprocessing: use to identify the surface drainage pattern and process with a sequential steps including DEM manipulation, flow direction, flow accumulation, stream definition, stream segmentation, catchment grid delineation, catchment polygon processing, drainage line processing, and Adjoint Catchment Processing (Figures 2.1 and 2.2).

[1] https://lta.cr.usgs.gov/SRTM
[2] https://asterweb.jpl.nasa.gov

Note
Terrain preprocessing has to be proceeded sequentially until Adjoint Catchment Processing. It does not do particular outlet based analysis rather it does for all drainage features for all DEM data loaded. Stream definition needs fixing area extent threshold to define streams. If one considers small stream networks, the threshold has to be small area (cell number). As these sequential steps may not be available sequentially for all GIS packages as given above, users have to take care of the procedures in relation to the GIS packages used.

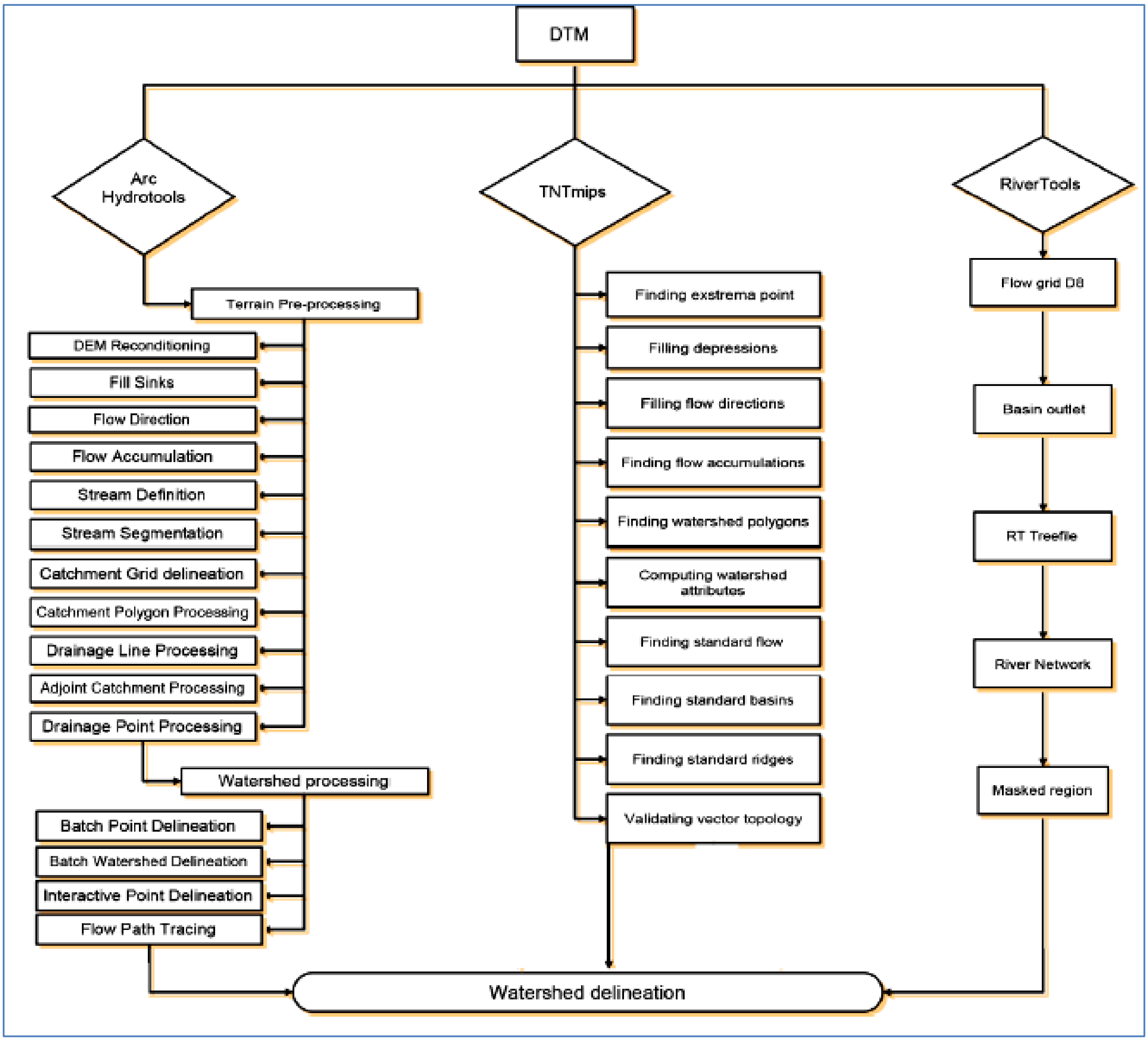


Figure 2-1: Sequential steps of catchment delineation in different tools

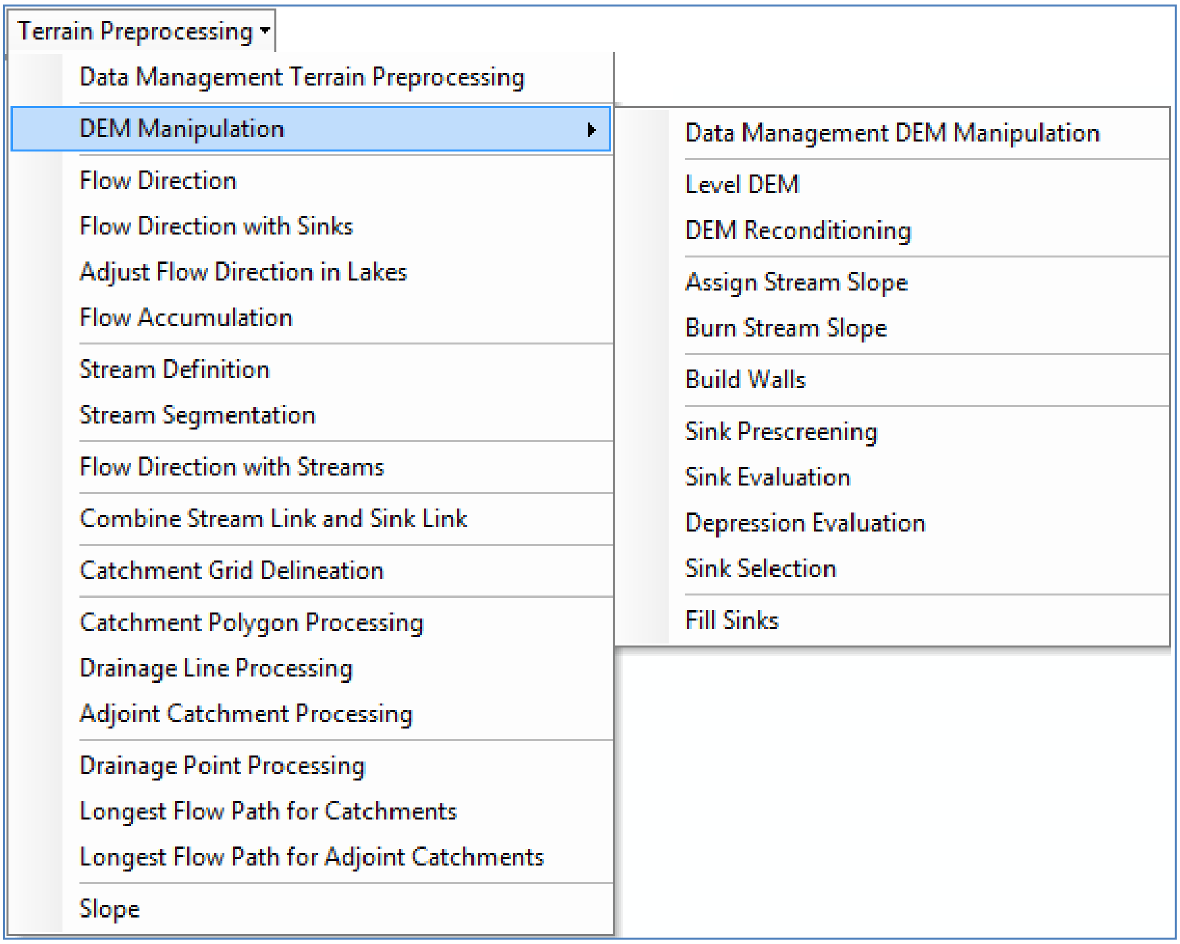


Figure 2-2: Terrain preprocessing sequential steps using ArcHydro

Watershed Processing: Delineation of target catchments can be made by using the preprocessed DEM and the point location of outlets. The preprocessing functions partition terrain into manageable units to allow fast delineation operations. Catchment delineation can be performed as point delineation using one catchment at time or batch delineation to process multi-catchments at a time. The watershed processing menu can perform other catchment attributes like drainage Area Centroid, longest flow path, flow slope and other parameters (Figure 2-3).

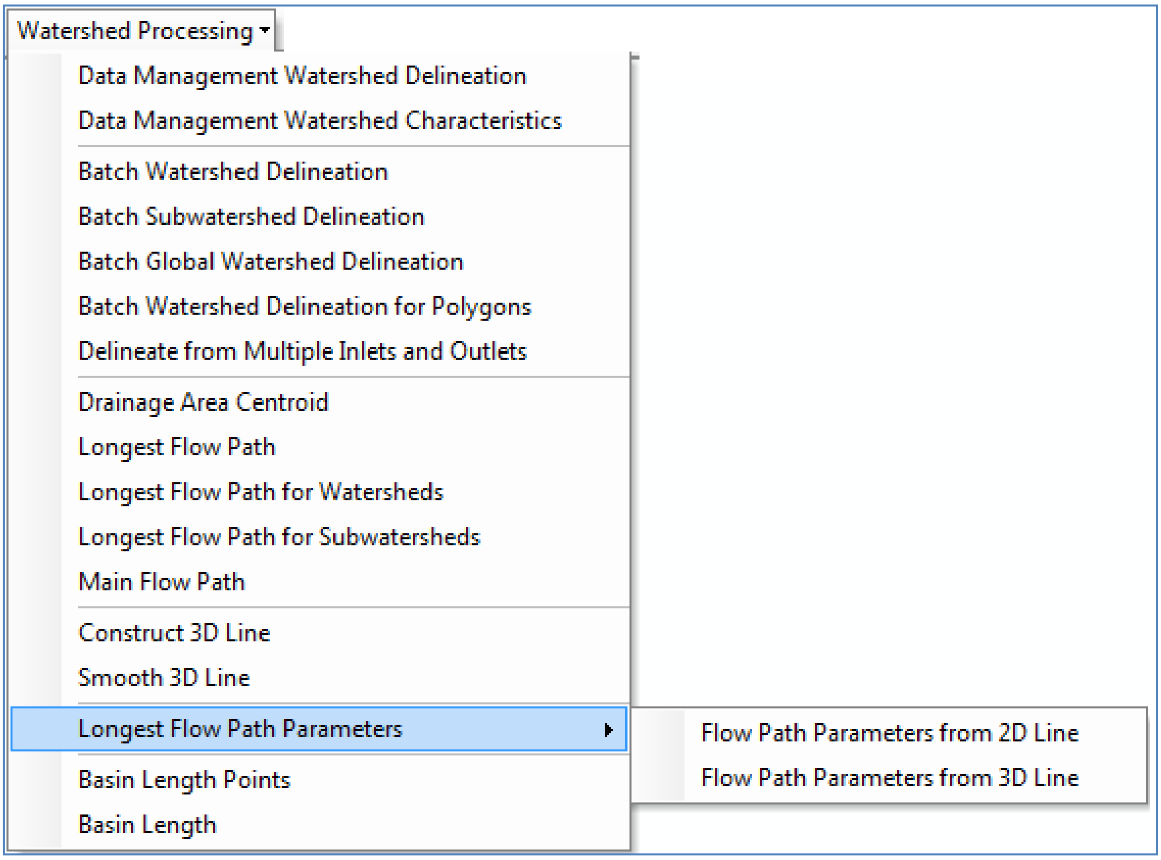


Figure 2-3: Functions and steps in watershed processing in ArcHydro Tools

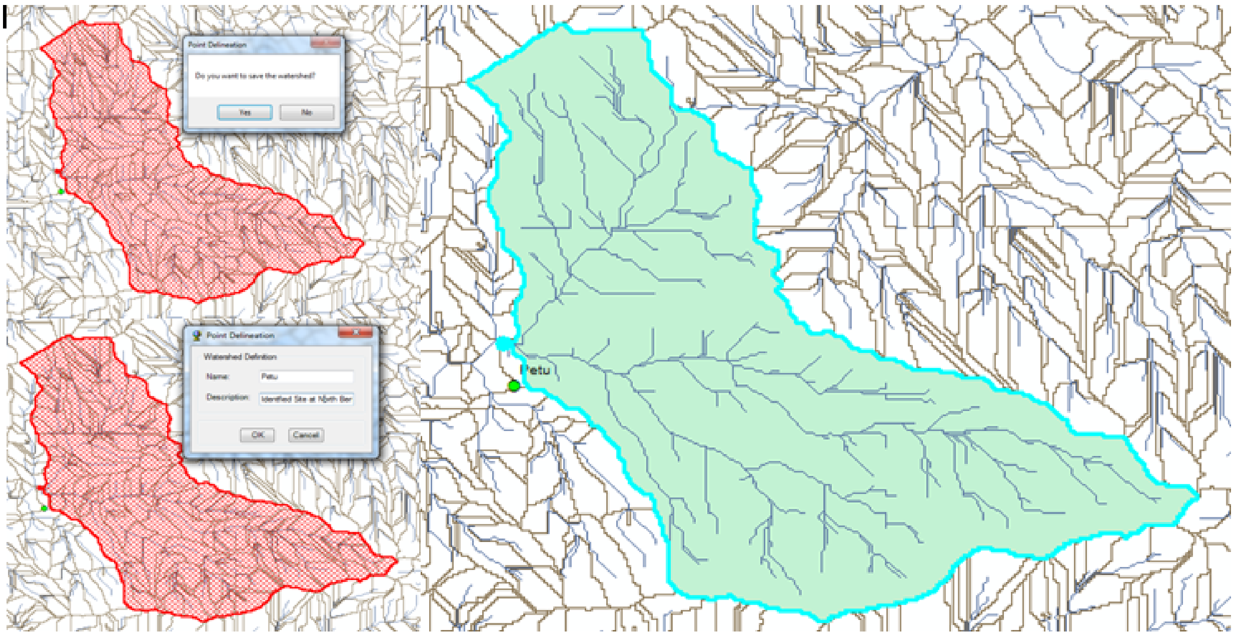


Figure 2-4: Example for catchment delineation: Petu SSIP Sites at Bench Maji Zone

Flow length computation: the longest flow length of the catchment as one of the input attribute for hydrological analysis has to be determined using the watershed processing tools. See the example for Petu SSI site at Bench Maji Zone.

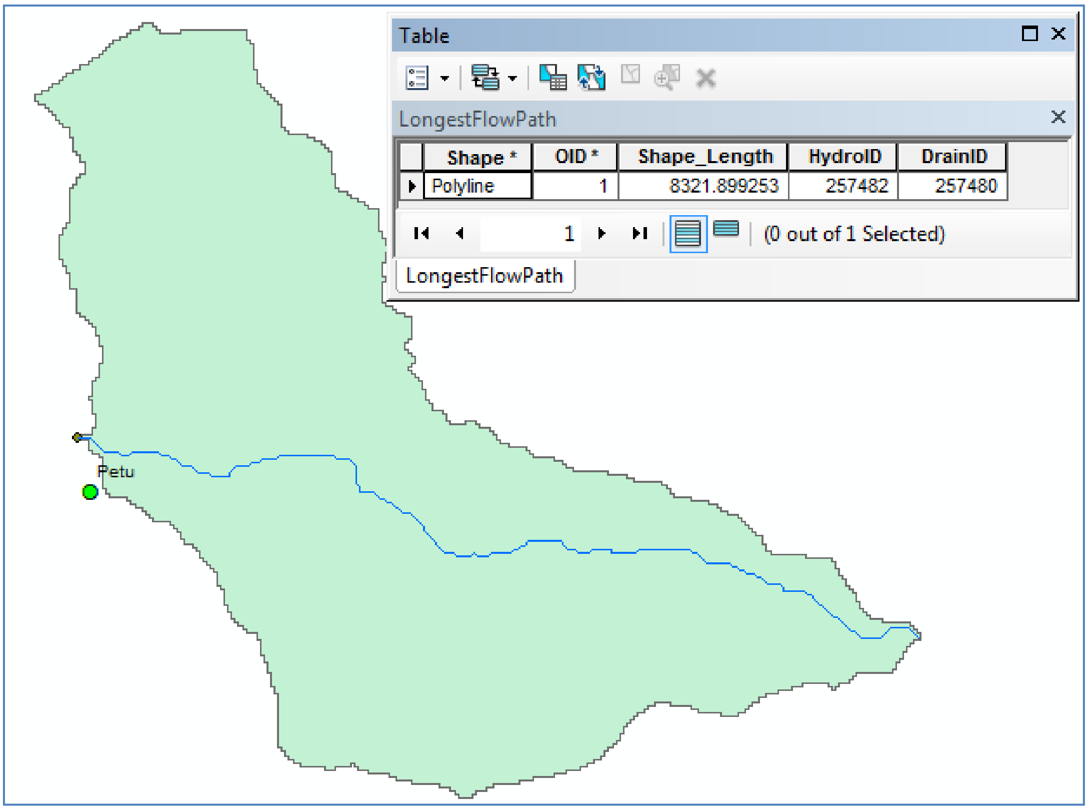


Figure 2-5: Longest flow computation in watershed processing at Petu site

Flow parameters like catchment slope from upstream to downstream points and at 10-85 points also computed as inputs for hydrological analysis (Figure 2.6).

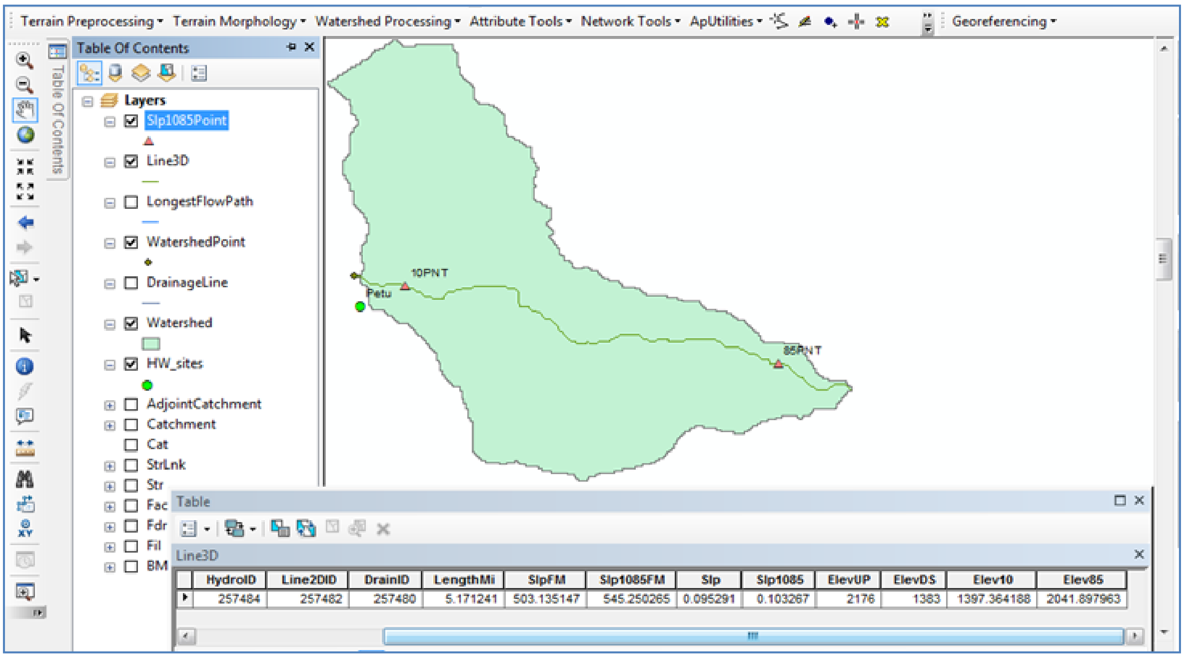


Figure 2-6: Flow parameters computation watershed processing tools at Petu site

2.2 SOIL DATA AS INPUT FOR HYDROLOGICAL ANALYSIS

The soil data for the identified project catchment area can be accessed from different sources with different scales. There are a number of previous studies that analyzed the soils of Ethiopia in different map scales and extents. The temporal variation of soils has no significance influence on analysis of catchment features. However, one has to take care for spatial variability of soil information. Therefore, good spatial resolution dataset has to be used to assess potential sources of soil information. The key for hydrological analysis is selection of a reasonable, good scale soil data set that is available. Some potential sources are given below:

- Ethiopian soil dataset of 1: 1 million scale from the Ministry of Agriculture and Natural Resources data sources developed from the two national studies- Ethiopian Agro-Ecological classification (FAO, 2001) and Woody biomass inventory studies (MoA, 1998).
- Harmonized World Soil Database (HWSD, version 1.2) (FAO/IIASA/ISRIC/ISS-CAS/JRC, 2012): This database has good spatial soil information at a scale of 1:500,000 for the Ethiopia case. It is freely accessed from the public domain access viewer[3]
- Ethiopian geo-Soil Database (Berhanu, et.al 2013): This database has good hydrological soil attributes organized in shape file sources with 1:250,000 scale. It is available at School of Civil and Environmental Science in Addis Ababa Institute of Technology and it is accessible with registered application.
- WLRC 2016, compiled national soil dataset
- It is also possible to use others reliable and possible improved data sources

2.3 LAND USE/COVER ANALYSIS AS INPUT FOR HYDROLOGICAL ANALYSIS

The land use/cover of a project site with significant spatial and temporal variability has a significance role for the hydrological analysis of the site. There are different potential sources of land use/cover information in Ethiopia for the use for studying irrigation development projects. However, all the land use/cover sources have to be supported by site observation and Ground truth.

- **Landsat 8 image data:** The Landsat Program provides repetitive acquisition of high resolution multispectral data of the Earth's surface on a global basis. All Landsat data in the USGS archive, freely accessed from in its websites[4]
- **ESA CCI Land Cover (MERIS)**: It has a new release of 300 m global land cover and 150 m water products (v.1.6.1). This new dataset includes the three land cover maps corresponding to the different epochs 2000 (from 1998 to 2002), 2005 (from 2003 to 2007), and 2010 (from 2008 to 2012) and can be
- Other national data sources, like Ethio GIS, and Ethiopian Mapping Agency

[3] http://harmonized-world-soil-database.software.informer.com/1.2 and http://webarchive.iiasa.ac.at/Research/LUC/External-World-soil-database/ HWSD_Documentation.pdf
[4] https://landsatlook.usgs.gov

3 RAINFALL ANALYSIS

3.1 CLIMATIC MECHANISMS AND WEATHER SYSTEM IN ETHIOPIA

Rainfall is the core element of the hydrological cycle that drives energy circulation in the atmosphere (Kumar *et al.*, 2006). Many studies highlight the importance of the spatial and temporal variability of precipitation (Fiener and Auerswald, 2009; Haile *et al.*, 2009), which has been proved to affect the accuracy of runoff prediction in gauged and ungauged catchments (Goodrich *et al.*, 1995; Schuurmans and Bierkens, 2007). Rainfall in Ethiopia is the result of multi-weather systems that include Subtropical Jet (STJ), Inter-tropical Convergence Zone (ITCZ), Red Sea Convergence Zone (RSCZ), Tropical Easterly Jet (TEJ), and Somali Jet (NMA 1996). The intensity, position, and direction of these weather systems lead the variability of the amount and distribution of seasonal and inter-annual rainfall in the. Moreover, the spatial distribution of rainfall in Ethiopia is significantly influenced by topographical variability of the country (NMA 1996; Camberlin 1997) and this makes the rainfall system of the country more complex. The influence of the ITCZ movement in led to the classification of Ethiopian climate into four main seasons as described below.

March-April: The ITCZ is located south of Ethiopia and moving northwards and this cause the tropical easterlies to have two components: (i) the moist easterly and southeasterly air currents in eastern and southern parts of the country, and (ii) dry northerly air currents in the northwestern quadrant.

May-July: In May, the ITCZ starts moving rapidly northwards, and during June and July, it reaches its northern most position. Hence, the southeastern Ethiopia air masses are in general subsiding and dry when blowing towards the Horn, after losing their moisture on the East African Highlands.
August-October: In August, the ITCZ starts moving rapidly south from its north position. It is located in central and south-central Ethiopia in September and October. In southwest the contribution of the Atlantic equatorial westerlies to autumn rainfall is more than their contribution to spring rainfall. Hence, the southeast part of Ethiopia receives the year's secondary maxima rainfall in autumn from the Indian Ocean easterlies.

November- February: In November, the ITCZ shifts southwards towards the equator and in December-February, the main pressure systems which determine surface air circulation and direction in Ethiopia are the anticyclones over Egypt and Arabia, and the low pressure area over the Lake Victoria-southwest Ethiopia. It leads to dry climatic situation in Ethiopia.

3.2 SOURCES OF RAINFALL DATA IN ETHIOPIA

I. Ground observed data set

- Sources: Ethiopia National Meteorological Agency[5]
- Spatial Extent: randomly distributed above 1,200 rainfall gauged stations
- Temporal extent: vary according to the year establishment
- Time Scale: commonly available with daily and monthly
- Quality: observed data is realistic but has temporal and spatial coverage gaps

[5] www.nma.gov.et

II. Blending Grid data

- Sources: Ethiopia National Meteorological Agency
- Spatial Extent: Evenly distributed 4km Spatial Resolution grids
- Temporal Extent: available since 1980
- Time Scale: Daily, Monthly and Annually
- Quality: fill coverage gaps of observed data

III. Satellite observed Rainfall estimates (RFE) data set

- Sources: FEWS NET Data Portals6
- Spatial Extent: Evenly distributed 8km Spatial Resolution grids
- Temporal Extent: available since 2001
- Time Scale: Daily, Decadal, Monthly and Annually
- Quality: requires bias correction

IV. Satellite observed Tropical Rainfall Measuring Mission (TRMM)

- Sources: NASS portal7
- Spatial Extent: Evenly distributed in 8 km Spatial Resolution grids
- Temporal Extent: Starts from 1998
- Time Scale: Daily, Decadal, monthly and Annually
- Quality: requires bias correction

V. Satellite observed CMORPH

- Sources: National Centre for Environmental Prediction (NCEP)[8]
- Spatial Extent: Evenly distributed in 28 km Spatial Resolution grids
- Temporal Extent: Starts from 2002
- Time Scale: 3hr and daily
- Quality: requires bias correction

VI. Satellite based Reanalysis CFSR data set

- Sources: National Centre for Environmental Prediction (NCEP)9
- Spatial Extent: Evenly distributed 38 km Spatial Resolution grids
- Temporal Extent: available since 1979
- Time Scale: Daily and sub-daily data with 3, 6, 12 hr data set
- Quality: requires bias correction; Addis Ababa Institute of Technology (AAiT) has a bias corrected dataset

Note
While all remote sensing based rainfall datasets have good coverage and even distribution, the data has to be bias-corrected and validated with ground observed dataset with local scale before their use for any scale and type of project design

3.3 CHECKING ADEQUACY OF RAIN GAUGES AND DATA RECORD

The basic instrument for point rainfall measurement is rain gauge. To compute areal rainfall for a given project area, there should be an optimal number of rain gauges. Statistics has been used in determining the optimum number of rain gauges required for a given catchment. The basis behind

[6] http://www.cpc.ncep.noaa.gov/products/fews/rfe.shtml
[7] http://disc.sci.gsfc.nasa.gov/airquality/services/opendap/TRMM/trmm.html
[8] https://climatedataguide.ucar.edu/climate-data/cmorph-cpc-morphing-technique-high-resolution-precipitation-60s-60n
[9] http://rda.ucar.edu/pub/cfsr.html

such statistical calculations is that a certain number of rain gauge stations are necessary to give average rainfall with a certain percentage of error. If the allowable error is more than the calculated error, lesser number of gauges will be required.

The optimum number of rain gauges (N) can be estimated using Equation 3.1:

$$N = \left[\frac{C_V}{E}\right]^2 \quad (3.1)$$

Where: Cv = Coefficient of variation of rainfall based on the existing rain gauge stations
E = Allowable percentage error in the estimate of basic mean rainfall

The allowable percentage error (E) should be limited to below 20% or the case of project work while the maximum allowable percentage of error for research work should not exceed from 10%.

In addition the number of data records in the given rain gauges also has significance important. If the number of observation increases then the standard deviation and mean value decreases. Therefore, a data series could be considered reliable and adequate if the coefficient of variation, CV is less than or equal to 10%.

$$cv = \frac{\delta_p}{\mu} \quad (3.2)$$

Where δ_P is the standard deviation and μ is the mean

Note To get optimal rain station network for the given project area, it is possible to combine the observed gauge stations' data with satellite data or re-analysis data grid stations if the quality of the satellite dataset is validated and the bias correction done.

3.4 FILLING DATA GAP

Rainfall data records occasionally are incomplete due to different reasons like the absence of observer, instrumental failure and social disorders. In such cases, one can estimate the missing data by using the nearest stations rainfall data. There are a number of methods to fill the gaps of the records. The following four methods are suggested for different conditions. Optimal distribution

Table 3-1: Gap filling methods and their applicability

Name	Formula	Condition of Applicability
1. Arithmetic mean	$P_X = \frac{1}{M}[P_1 + P_2 + P_3 + \ldots..P_m]$	For the coefficient of variation among stations less than 10%
2. Normal Ratio	$P_X = \frac{N}{M}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_3} + \ldots..\frac{P_m}{N_m}\right]$	If the amount of variation has no significance if the stations are in the same rainfall regimes)
3. Inverse distance weightage	$P_x = \sum_{i=1}^{n} w_i p_i \quad w_i = \frac{1/d_i^2}{\sum_{i=1}^{n} 1/d_i^2}$	If the amount of variation has no significance and the stations are in short distance
4. Regression equation	Locally developed	If there are significant correlation

3.5 TEST FOR CONSISTENCY

Rainfall data a station may not be consistent over the period of observation. Inconsistency may occur due to due to different reasons: shifting of the rain gauge to new location; changes in the ecosystem due to calamites, such as forest fires, landslides…etc.; significant construction works that may change the surrounding or occurrence of observational error of a certain data. Therefore, testing and adjusting the inconsistency have to be made in the rainfall data processing, which is often carried out with double mass curve technique. The double mass curve technique is a graphical approach where the accumulated rainfall at the doubtful gauge will be plotted as ordinate versus the concurrent accumulated mean rainfall of nearby

Procedures

- Arrange the data with inverse chronological order (the latest data first) and compute the cumulative amounts of annual rainfall for suspect gauge (gauge X)
- Compute the mean annual rainfall of the nearby stations with reliable dataset (gauges 1, 2, 3 …..n)
- Arrange the mean annual data with inverse chronological order and compute the cumulative amounts of mean annual rainfall of the group of stations
- Plot cumulative rainfall for suspect gauge on y-axis versus mean cumulative rainfall for the check gauges in x-axis (Figure 3-1)
- Attempt to construct a straight line through the data points; if there is consistency on the suspect gauge, all points will fall on a straight line. Accept the data as it is.
- If there is a divergence from a straight line, it indicates an inconsistency in the data of suspect gauge and the data has to be corrected.
- The year where the divergence started is marked and slopes before divergence (S1) and after (S2) are computed. A correction factor (k) that adjusts the slope to S1 is computed by dividing slopes (k=s1/s2)
- Correct the annual rainfall of the suspect station after the break of consistency test by multiplying the value by the correction factor to avoid the inconsistency in the observed data set

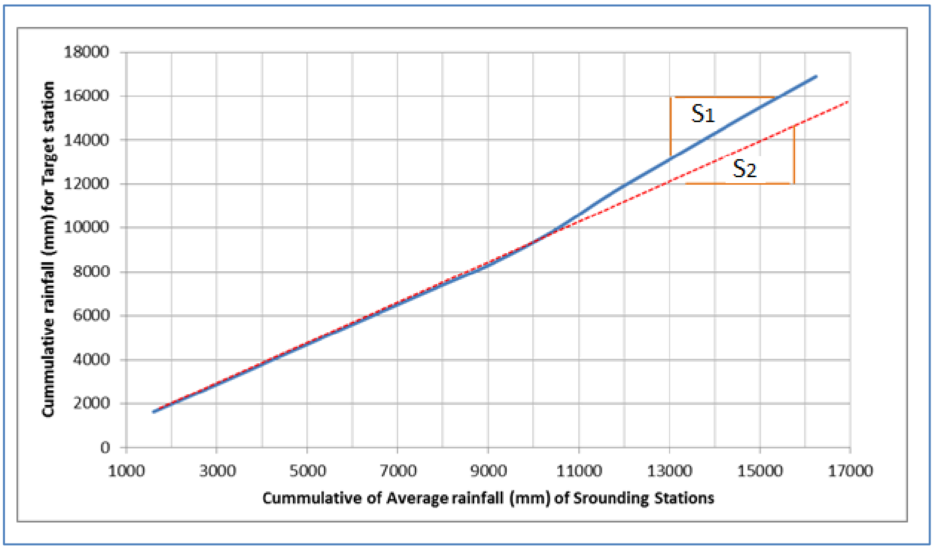


Figure 3-1: Examples of rainfall inconsistency test chart

3.6 AREAL RAINFALL

In order to estimate the catchment runoff of a given area, it is often necessary to estimate the average areal rainfall for a catchment using data from number of observed stations within and around catchment. Depending on the density and distribution of rainfall gauging stations and the topography of the catchment, an appropriate method has to be selected to estimate the average areal rainfall for a catchment (some methods are listed in Table 3-2). Procedure as follows:

1. Compute the Thiessen polygon of the stations using proximity analysis and clip it to its catchment area
2. Compute the weightage of each station by dividing the area of individual polygon by the total catchment area.
3. Multiply the rainfall within each polygon by its respective weightage
4. The average areal rainfall of the catchment will be the Sum all weighted rainfall.
5. Similarly, GIS tool can be used to compute average areal rainfall using Isohyetal method. Following the next steps:
6. Construct or development Isohyetal map using one of the interpolation methods to interpolate rainfall of stations at the surrounding of the catchment.
7. Compute isohyets using the contour tool in GIS, and convert the line to polygon by combing it with the catchment polygon.
8. T Compute the average rainfall of the two consecutive isohyets and multiply this by the area between the isohyets to get the weighted rainfall for each isohyets polygon.
9. Sum all the weighted rainfall and divide it by the total catchment area to get the average areal rainfall of the catchment.

Table 3-2: Different areal rainfall methods

Methods	Formula	Applicability
1. Arithmetic mean	$\overline{P} = \frac{1}{M}[P_1 + P_2 + P_3 + \ldots.P_m]$	The coefficient of variations among stations less than 10%
2. Thiessen Polygon	$\overline{P} = \frac{A_1P_1 + A_2P_2 + \ldots.A_mP_m}{A}$	More accurate If there is no orographic variation among stations (if the stations are in the same face of the mountains and high hills)
3. Isohyetal	$\overline{P} = \frac{a_1\left(\frac{P_1+P_2}{2}\right) + a_2\left(\frac{P_2+P_3}{2}\right)\ldots a_{n-1}\left(\frac{P_{n-1}+P_n}{2}\right)}{A}$	More accurate than for the above two methods if all the stations are in windward directions
$\overline{P} =$ Mean Areal Rainfall , P_1, P_2 …P_m rainfall at different stations , A_1, A_2….A_m, area of the Thiessen Polygon for different stations , a_1, a_2,…. a_{n-1}, area between different isohyets , A total catchment area		

3.7 RAINFALL FREQUENCY ANALYSIS

The term frequency analysis refers to the techniques whose objective is to analyze the occurrence of hydrologic variable (e.g rainfall) within statistical framework. In many hydraulic-engineering applications, the probability of occurrence of a particular extreme rainfall, e.g. a 24-h maximum rainfall, will be of paramount importance. There is a definite relation between the frequency of occurrences and magnitude; the ordinary events occurring almost regularly than the severe storms. The reasonable length of record for frequency analysis should be more than 30 years. However, a record of up to 20 years can be used as sample data set for frequency analysis if data for longer record is not available. General frequency equation 3.3 (Chow (1951) can be used to analysis the frequency of annual 24-hr maximum rainfall of the project area.

$$X_T = X_{av} \pm \sigma K_T \qquad 3.3$$

Where: $X_{T:=}$the event (magnitude) at return period of T years
Xav =the mean of the sample data
σ =the standard deviation and
K= frequency factor, which is different for different distribution and is dependent on the recurrence interval, sample number, and skewness of the distribution.

Therefore, the major question in this frequency analysis is getting the fitted distribution for the catchment rainfall dataset.
Follow procedure to compute design or maximum 24-hr rainfall:

Procedure

1. Extract the annual maximum time series data from the daily rainfall dataset (it has to be > 15 years of data record)
2. Compute the mean (X_{av}) and standard deviation (σ) of the data
3. If there is outlier in the annual maximum data series, remove it (them) from the data series
4. Find the fitted probability distribution for the available sample data. The fitting of the probability distribution can be evaluated with statistical goodness fit tests (Kolmogorov Smirnov, Anderson Darling, Chi-Squared) using Easy-Fit Excel Add-in or moment diagram as graphical approach.
5. Using the parameters of the best fitted probability function, compute the K_T values for different return periods using the K_T equations of each probability distribution
6. Compute the X_T for the different return periods using Chow general frequency equation

Note: An Excel application for moment diagram fit test and quantile computation for the most common 10 probability distributions are attached with this guideline. With 15 rainfall regimes of Ethiopia the X_T values for return periods 5 -200 years are also included.

3.8 RAINFALL – INTENSITY – DURATION (IDF)

The IDF relationship is a mathematical relationship between rainfall intensity, duration and return period that was developed in several parts of the world for its practical use. The design rainfall-intensity-duration relationships are obtained directly from the time distribution of rainfall, simply by converting the rainfall during a given duration to rainfall intensity in millimeters per hour. Maximum Intensity in mm per hour is derived for various durations and return periods for short durations of rainfall ranging from 5 minutes to 120 minutes. The value of rainfall intensity is computed by considering the proportion of the rainfall at the time of concentration using the of 24-hr rainfall. Rainfall intensity is the average rainfall rate during the time of concentration. Based on this definition, it can be calculated with the equation.

$$i = \frac{R_{tc}}{t_c} \qquad (3.4)$$

Where: I= rainfall intensity (mm/hr),
R_{tc}=the amount of rainfall during the time of concentration (mm)
t_c = the time of concentration for the sub basin (hr) for its detail refer section5.3.2.2

An analysis of rainfall data collected by Hershfield (1961) for different durations and frequencies showed that the amount of rain falling during the time of concentration was proportional to the amount of rain falling during the 24-hr period.

$$R_{tc} = \alpha_{tc} R_{day} \qquad (3.5)$$

Where R_{tc} = the amount of rain falling during the time of concentration (mm of water),

α_{tc} = the fraction of daily rainfall that occurs during the time of concentration, R_{day} = the amount of rain falling during 24-hr (mm of water).

The Transport and Road Research Laboratory (TRRL, UK), Laboratory Report 623, Prediction of Storm Rainfall in East Africa (D. Fiddes, J.A. Forsegate and A.O. Grigg) give a valuable regional study on storm rainfalls in East Africa. The regional study is very useful for hydrologists and engineers involved in determination of peak discharges for bridges and culverts. The research study used the following model to fit a large number of stations data.

$$i = \frac{a}{(b+t)^{n}{}_{c}} \qquad (3.6)$$

Where i = the intensity or rainfall in mm/hr,

t = the duration in hours, a, b and n are constants.

The best fit was found when b equals 0.33 hours and the index n varies from 0.78 to 1.09.

Adopting 24 hour rainfall value as I24

$$i_t = \frac{(b+24)}{(b+t)^{n}{}_{c}} * I_{24} \qquad (3.7)$$

Therefore, in this case, we adopt the value of “b” is 0.33 and “n” equals 0.9 (the average of n is 0.78 to 1.09 for East Africa).

Note:
Refer Rainfall regimes of Ethiopia from Figure 3-2:
The actually intensity value with different dration and return period is attached with Excel Spread Sheet for each rainfall regime.

4 EVAPORATION AND EVAPOTRANSPIRATION ANALYSIS

4.1 DATA SOURCES

I. Ground observed dataset

- Sources: Ethiopia National Meteorological Agency[10]
- Spatial Extent: randomly distributed above 1,200 rainfall gauged stations
- Temporal extent: vary according to the year establishment of a particular station since 1940's
- Time Scale: commonly available with monthly rainfall and daily rainfall with special request while availability of automatic rain gauge rainfall can be accessed for some of the stations with special request
- Quality: observed data is more realistic and good quality data except record gaps in some of the years and inclusion of human error through data processing

II. Blending grid data

- Sources: Ethiopia National Meteorological Agency
- Spatial Extent: Evenly distributed 4km Spatial Resolution grids
- Temporal Extent: available since 1980
- Time Scale: Daily, Monthly and Annually
- Quality: not good as ground observed dataset but better from all other gridded data sets

III. Satellite observed climate datasets

- We have a number of satiates based climate data set, which can be accessed through internet connection

IV. Atmospheric reanalysis data sets (CFSR)

- Sources: National Center for Environmental Prediction (NCEP)[11]
- Climatic data types: precipitation, Minimum Temperature, Maximum Temperature, Dew point Temperature, Relative Humidity,
- Spatial Extent: Evenly distributed 38km Spatial Resolution grids
- Temporal Extent: Starts from 1979 up to date
- Time Scale: Daily and sub daily data with 3, 6, 12 hr data set
- Quality: still not good as ground observed data but much better than pure satellite datasets

Note
All remote sensing based climatic dataset have good coverage and even distribution; however, the data should be to validate with ground observed data set with local scale before use for any project design and analysis. Climate dataset, which related to CROPWAT project is outdated so not recommended for use

[10] www.nma.gov.et

[11] http://rda.ucar.edu/pub/cfsr.html or https://globalweather.tamu.edu/

4.2 OPEN WATER EVAPORATION

Estimation of evaporation from open water, such as lakes and reservoirs, has been the subject of many studies and there are publications dating back to the early 1900s. Because of its nature, evaporation from water surfaces is rarely measured directly, except over relatively small spatial and temporal scales (Jones 1992). A wide variety of methods for estimating open water evaporation has been reported in many literatures and used in practice. They can be categorized into major types of approach which include pan evaporation, mass balance, energy budget models, bulk transfer models, combination models, equilibrium temperature methods and empirical approaches. The selection of the "best" technique to use for a particular location is largely a function of the data availability, type or size of the water body, and the required accuracy of the estimated evaporation.

Properties of the water body affecting evaporation include water depth, thermal stratification, size of surface, rainfall, inflow and outflow, vegetation, turbidity and bottom reflectance and salinity.

The three methods selected based on their data requirement and relative accuracy includes the Penman equations, Meyer's Formula and empirical methods.

Table 4-1: Selected methods for estimating open evaporation

Name	Formula	Required input data
1. Penman's equation (1947)	$E = \frac{1}{\lambda}\left[\frac{\Delta R_n + \rho C_p (e_s - e_a) g_a}{\Delta + \gamma}\right]$	Tmax, Tmin, Wind speed, relative humidity, sunshine hour
2. Meyer's Formula	$E_L = K_M (e_w - e_a)\left(1 + \frac{u_9}{16}\right)$	Tmax, Tmin and wind speed
3. Pan Evaporation	$E_L = K_{pan} E_{pan}$	Pan Evaporation measurement

E = evaporation (kg/m²s)
Δ = Slope of the saturation vapor pressure curve (Pa/ K)
R_n = Net irradiance (W /m^2)
ρ = density of air (kg/ m^3)
c_p = heat capacity of air (J/ kg K)
g_a = momentum surface aerodynamic conductance (m/s)
e_s = Saturation vapor pressure (Pa)
e_a = existing vapor pressure (Pa)
λ = latent heat of vaporization (J/ kg)
γ = psychrometric constant (Pa/ K)
E_L = Lake evaporation (mm/day)
e_w = saturated vapour pressure at the water surface in mm of mercury
e_a = actual vapour pressure of over-lying at air in mm of mercury
u_9 = monthly mean wind velocity in km/hr at 9m above ground
K_m= coefficient accounting for various other factors with a value of 0.36 for deep waters and 0.5 for small, shallow waters
K_{pan} = pan coefficient
E_{pan} = pan Evaporation

4.3 REFERENCE EVAPOTRANSPIRATION (ETO)

The determination of irrigation water requirements requires an accurate estimate of the crop water use rate. Seasonal and annual water use is often required to size irrigated area and to decide on the application type in irrigation. The estimated irrigation water requirement using monthly reference evapotranspiration (ETo) can be good indicator for the water demand of potential irrigated lands. Therefore, ETo should be estimated with different methods. Many methods have been developed to estimate ETo based on climatic factors. The simplest methods generally require the average air temperature. The most complex methods require hourly data on solar radiation, air temperature, wind speed, and vapor pressure. Commonly the methods are categorized in four groups as

- Combination theory methods e.g. Penman–monteith method
- Temperature based methods – e.g Hargreaves method
- Radiation based methods – e.g. Priestley-Taylor method
- Pan evaporation based methods

Selection of appropriate method of computing ETo depends on:

- Type, accuracy, and duration of available climatic data
- Natural pattern of evapotranspiration during the year
- Intended use of the evapotranspiration estimates

The length of time that different types of data are available may dictate the type of method to use for estimating ETo. In most of the case, there are not long period data except temperature, because of more difficulty in measuring them. Therefore, in most locations, we resort to using temperature-based ETo estimating methods or estimate other inputs based on the temperature, latitude and altitude of the area. With these conditions three methods, Penman-Monteith, modified Penman-Monteith and Hargreaves are recommended for ETo estimation in Ethiopian condition.

Table 4-2: Selected ETo estimation methods

Name	Formula	Required input data
1. Penman-Monteith	$ET_o = \dfrac{0.408\Delta(R_n - G) + \gamma \dfrac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34u_2)}$	Tmax, Tmin, Wind speed, relative humidity, sunshine hour
2. Modified Penman-Monteith	$ET_o = \dfrac{0.408\Delta(R_n - G) + \gamma \dfrac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34u_2)}$	Tmax, Tmin, latitude and altitude and others estimated
3. Hargreaves	$ET_o = 0.0023(T_{\max} - T_{\min})^{0.5}(T_m + 17.8)R_a$	Tmax and Tmin

The details on the computation of Evapotranspiration also describe in the Agronomy guideline in relation with crop water analysis.

5 STREAM FLOW ANALYSIS

Stream flow or discharge is the volume of water that moves through a specific point in a stream during a given period of time and is given in units of cubic meter per second (cms). Stream flow varies with the volume of water, precipitation, surface temperature, and other climatic factors. Stream flow is important in irrigation project for determining the design flow through the peak flow analysis and dependable flow estimating the land area to be irrigated.

5.1 DATA SOURCES AND AVAILABILITY

Data on stream flow are obtained by recording the water level at a gauging station on the river and by relating this level to the corresponding rate of flow using rating curve of the station. The precision of water level measurements and the sensitivity of the site to changes in water level when flow rate varies are factors which govern the accuracy and reliability of the data. The Hydrology Directorate of the Ministry of Water, Irrigation and Electricity is responsible for collecting, processing, quality assurance and dissemination of stream flow data in Ethiopia. Currently, there are about 570 operational stream flow gauging stations in the country but their distribution is skewed. Expanding such stations is expensive and technically complex. Therefore, it prefers to estimate stream flows using rainfall-runoff methods.

5.2 PEAK DISCHARGE AND FREQUENCY RELATIONSHIP

Magnitude of peak discharge is a function of its expected frequency of occurrence, which in turn, is related to the magnitude of the potential damage and hazard. Engineers are interested in the development of a flood versus frequency relationship and peak discharges versus the probability of its occurrence or exceedance as well as in the flood volume and time distribution of runoff. Sometimes, there might be a requirement to use flood hydrographs to route floods through drainage or flood storage structures. This is especially important when an embankment is built across a floodplain and flood compensation storage area is required to be provided to compensate for the lost natural floodplain storage area taken up for the road embankment and to mitigate the flood risk to the upstream areas.

5.3 DESIGN FLOOD

Design flood is the flood magnitude, which is expected to occur with a certain return period during the design period of a hydraulic structure. The selection of the return period is genearly made based on safety, econmy, size and category of the scheme, and it should be greater than the design period of the structures. With the national and international expriance, the criteria of design flood and the methods of flood estimation are recommended in this below.

Table 5-1: Recommended return periods for design of hydraulics structures

Types		Return Period (Years)
Field drainages		5
Cross drainage works	Side Ditches	10
	Pipe Culverts	10
	Culverts with 2m<span<6m	25
	Short Span Bridges 6m<span<15m	50
	Medium Span Bridges15m< span<50m	100
	Long Span Bridges spans>50m	200
Diversion weirs		50
Spillways		1,000

5.3.1 Gauged stream flow

Gauged flow data commonly used for computing design flow if the target location coincides with the point of the gauging stations or if it there is possibility to transfer the gauge data to target area with homogenous region. The stream flow data has to be tested for its trend and stationary and homogeneity before we use the flow data to compute design flood

Trend test for stream flow

A steady increase or decrease of the time series characteristics is known as trend. Natural and man-made changes like deforestation, urbanization, large scale landslide, and large changes in watershed conditions are the possible causes for the introduction of trend in stream flow time series data. Test for the presence of trend can be done with turning Point test and Kendal's Rank-Correlation test.

Turning point test

If a value, X_i is either greater or less than both preceding and succeeding values, turning point occurred. The procedure of testing turning point is outlined below.

- Arrange the data in order of their occurrence (time chronological order)
- Apply either of the conditions, Xi-1 < Xi > Xi+1 or Xi-1 > Xi < Xi+1 and count how many turning points are there in the series. Let the total number of turning points be P.
- Expected number of turning points is $E(P) = \frac{2(N-2)}{3}$, where N is the total number of data
- Variance of P is, $Var(P) = \frac{(16N-29)}{90}$
- Expressing P in standard normal form $Z = \frac{\{P - E(P)\}}{\{Var(P)\}^{0.5}}$
- Test it at 5% level of significance, and if Z is less than ±1.96, there is no any trend on the dataset.

Kendal's rank-correlation test

- Arrange the data in order of their occurrence (time chronological order)
- Pick up the first value of the series X1 and compare it with the rest of the series, X2, X3,….Xn. Count the number of times it is greater than the rest, name as P1ex. Repeat it for X2, X3, ….. Xn and name them P2ex, P3ex ….Pnex.
- Find the sum, P= P1ex +P2ex +P3ex ……….Pnex .
- Maximum value of P can be $P_{\max} = \frac{n(n-1)}{2}$, it is for checking the previous analysis, the previous sum should be less or equal to it.
- Expected number of turning points, $E(P) = \frac{n(n-1)}{4}$,
- Kendal's τ is computed as $\tau = \left[\left\{\frac{4P}{n(n-1)}\right\} - 1\right]$, E(τ) should be zero

- Variance of τ, $Var(\tau) = \left[\frac{(2(2n+5))}{9n(n-1)} \right]$
- Standard test for statistics of $Z = \frac{\tau}{\{Var(\tau)\}^{0.5}}$
- Test for the hypothesis at 5% level of significance, if Z is less than ±1.96, there is no any trend on the data set

Note:
- And Excel Add-ins called XLSTAT is available on line that can easily handle both statistical tests
- If there is a trend in the dataset, the trend has to be removed before using for the computation of design flood.

Stream-flow data transfer

There is no research in delineating hydrological homogenous regions in Ethiopia. Most of the researches are limited to defining flow regimes and catchment similarity characterizations, which do not fully support data transferring from gauging stations to ungagged catchments. Therefore, in situations where there are not sufficient rainfall data to use rainfall-runoff relations, it is suggested to use generalized data transfer equation of Admasu Gebeyehu (1986) for Ethiopian basins.

$$Q_u = Q_g \left(A_u / A_g \right)^{0.7} \qquad (5.1)$$

Where: Q_u = mean annual daily maximum flow at ungauged site (m^3/s
A_u= ungauged catchment area (km^2)
Q_g = mean annual daily maximum flow at gauged site (m^3/s),
A_g = gauged catchment area (km^2),

Peak flow Analysis

Once the daily stream flow data is accessed or obtained for the gauge station and transfers to the target ungauged catchment is made, the procedures of frequency analysis used in rainfall analysis is followed based on annual maximum series.

5.3.2 Estimation of design flood using rainfall-runoff relationship

In the absence of gauged stream flows at or near a target site, an accepted practice to estimate peak runoff rates and hydrographs is using rainfall-runoff methods. There are a number of rainfall-runoff methods that can be used for estimation of peak flow but all don't give reasonably accurate estimate for a given catchment. In addition, averaging of results of several methods is not recommended. Therefore, one has to select a method that most expresses the given catchment. Each method has a range of application and limitations, which the engineer should clearly understand prior to using them. The hydrologist or engineer must ensure that the selected hydrologic method is appropriate for target catchment specific conditions and the availability of inputs data to perform the required calculations using the method selected. If possible, the method should be calibrated to local conditions and flood history.

5.3.2.1 Selection of the appropriate method for design flood estimation

An appropriate hydrologic analysis method should be selected for the target site and the hydrologist should consider, at least the following selecting criteria in selecting an appropriate method:

- **Purpose of the design flood:** Selection of the method for estimating design flood depends on type and size of the structures like spillways, bridges, culverts, side ditches and others. Therefore, the. For example, if the design flood is required for design of side ditches or simple culverts, rational method, may be adequate to compute the peak flood. However, if the project site has different detention or retention behaviors, a runoff hydrograph will be required for estimating design flood.
- **Availability of input data to develop the required hydrologic information:** different methods require different input data to relate the rainfall with runoff, therefore selection of method also consider the availability of required data or information.
- **Conditions in the watershed:** It may limit applicability of alternative models. For instance there are ponds, lakes, and depressions in the watershed that affect runoff characteristics through their water storing nature, the rational equation will not be appropriate for such cases.

Methods that can be used in estimating peak flood magnitudes for design of irrigation head works in Ethiopia listed below in Table 5.2.

Table 5-2: Suggested rainfall-runoff methods for estimation of design flood

No	Method	Input data	Maximum Catchment area (km^2)	Preferable* Land use Type to use	Available packages or software's
1.	Rational Method	Catchment area, flow length, average slope, catchment characteristics, rainfall intensity	0.5	Urban	Excel and GIS combination
2.	SCS-CN Method	Catchment area, flow length, length to catchment centroid, max daily rainfall, veg. type, soil cover and synthetic regional unit hydrograph	0.5 - 65	Agricultural	• Excel and GIS • HEC-HMS • SWAT • Others also
3.	Synthetic Unit Hydrograph Method	Catchment area, flow length, length to catchment centroid (center), daily rainfall, veg. type and synthetic regional unit hydrograph	0.5 - 6000	For all	• Excel and GIS • HEC-HMS
4.	Soil Moisture Method**	Catchment Area, Soil Characteristics, Climate data (rainfall, temperature, Humidity, Wind speed, Sunshine), latitude,	All	All	• HEC-HMS • WEAP • 1DMLSS

* The preferable land use type selected based on the original development of the methods, however, it doesn't mean to use it for other land uses.

** The details of the soil moisture methods are not presented here since they have different approach and algorithms for different packages. Therefore, it is recommended to get the detail approach and algorithms for each software packages from their technical document

5.3.2.2 Rational method

The Rational Method is logical, generalized and often reasonable. The Rational Method is most accurate for estimating design storm peak runoff for areas up to 50 hectares (0.5 square km) and presented from the perspective of each of the three “independent” variables; runoff coefficient, rainfall intensity and catchment area. Therefore, the peak flow can generally be expressed as Equation 5.2 with variables in SI units. However, numerical coefficients will be different for other units of these variables.

$$Q_P = 0.00278 * CIA \qquad (5.2)$$

Where Q_p= Peak runoff rate (m3/s) A= Catchment area (ha)
I = Rainfall intensity (mm/hr) C= Runoff coefficient (decimal)

Runoff Coefficient (C)
The run off coefficient (C) can be expressed in two ways, one as scientific parameter that expresses the abstractions (infiltration, interception, detention storage, etc.) and diffusion, the tendency for a hydrograph to spread in time and attenuate in peak as it moves downstream. It is also expressed as empirical parameter that quantify based on best watershed observation. In both perspectives, runoff coefficient can be determined from catchment characteristics, land use/cover, slope and soil type (Tables 5-3 and 5-4).

Table 5-3: Runoff coefficient for rural catchment

Slope factor	Cs	Soil permeability factor	CP	Land use/cover	Cv
< 3.5% Flat	0.05	Well drained soil e.g. sand and gravel	0.05	Dense forest/thick bush	0.05
3.5% - 10% Soft to moderate	0.1	Fair drained soil e.g. sand and gravel with fines	0.1	Sparse forest/dense grass	0.1
10% - 25% Rolling	0.15	Poorly drained soil e.g. silt	0.15	Grassland/scrub	0.15
25% - 45% Hilly	0.2	Impervious soil e.g. clay, organic silts and clay	0.25	Cultivation	0.2
>45% Mountainous	0.25	Rock	0.4	Space grassland	0.25
		Water-logged black cotton soil	0.5	Barren	0.3
C= Cs+Cp+Cv					

Source: Chow, 1964, p. 21-38; ASCE, 1996, p. 590

Table 5-4: Runoff coefficient for urban catchment

Description of Area	Runoff Coefficients	Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95	Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50	Residential: Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75	Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45	Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80	Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25	Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40	Unimproved areas	0.10-0.30
Street : Asphalt	0.70-0.95	Concrete	0.80-0.95
Drives and walks	0.75-0.85	Roofs 0.75-0.95	
Source: Chow, 1964, p. 21-38; HEC No. 19, 1984, ASCE, 1996, p. 590,			

In addition to the catchment characteristics, the runoff coefficient is affected by the return period of the design flood to be estimated. The return period of less than 25 years does not have significant change on the runoff coefficient, but return periods of 25 year and above have effect on the runoff coefficient and this can be expressed with the multiplier presented in Table 5-5.

Table 5-5: T-year multipliers of runoff coefficient (C)

Return Period	< 25	25	50	100
Multiplier	1	1.1	1.2	1.25

Source: Viessman & Lewis (1996), p. 315

To avoid a value of runoff coefficient (C) of more than unit due to multiplier effect, a modification equation is proposed by Chow (1964) and Singh (1988) to adjust the value of runoff coefficient.

$$C = C_{100}\left(\frac{T_r}{100}\right)^{0.075} \qquad (5.3)$$

Where Tr is return period in years, C100 is C for the 100-year event

As many catchments are not of a single land treatment, an area-weighted composite C has to be computed using Equation 5.4 for catchments of multiple land treatments.

$$C_{comp} = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} Ai} \qquad (5.4)$$

Where C_{comp}= Composite C Ai= sub-catchment area i
n= number of sub-catchments Ci=C of sub-catchment i

Rainfall Intensity (I)

As discussed in Chapter 3, the rainfall intensity of a catchment is from the time of concentration (tc) of a catchment and the maximum rainfall for the different return periods.

Time of concentration (tc) is the time required for an entire watershed to contribute to runoff at the point of interest for hydraulic design, which is calculated by the time of travel of runoff or flow from the most hydraulically remote point to the point under investigation. Time of travel is function of length and velocity of flow for a particular watercourse. There may be multiple paths to consider in determining the longest travel time. The hydrologist/engineer must identify the flow path along which the longest travel time is likely to occur. Roussel et al. 2005 recommended the use of Kerby-Kirpich approach for estimating watershed time of concentration. The Kerby-Kirpich approach requires comparatively a few input parameters, straightforward to apply, and produces readily interpretable results. The Kerby-Kirpich approach produces time of concentration by adding the overland flow time (Kerby) and the channel flow time (Kirpich):

$$t_c = T_{ov} + T_{Ch} \qquad (5.5)$$

Where: T_{ov} = overland flow time and T_{ch} = channel flow time

Kerby Method

Kerby method is provided in Equation 5.6 for estimating overland flow time from small catchments where overland flow is an important component of overall travel time.

$$T_{ov} = K(L * N)^{0.467} S^{-0.235} \qquad (5.6)$$

Where: T_{ov} is as defined above and is given in minutes
L = the overland-flow length, in meters
K = units conversion coefficient
K = 1.44 for *SI* units
N = a dimensionless retardance coefficient
S = the slope of terrain conveying the overland flow

Table 5-6: Retardance Coefficients of Kerby Equation

Generalized terrain description	Retardance coefficient (N)	Generalized terrain description	Retardance coefficient (N)
Pavement	0.02	Smooth, bare, packed soil	0.10
Deciduous forest	0.60	Pasture, average grass	0.40
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20	Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

Kirpich Method
For channel-flow, Kirpich equation is:

$$T_{ch} = KL^{0.770} S^{-0.385} \qquad (5.7)$$

Where: T_{ch} is as defined above and is given in minutes
L = the overland-flow length, in meters;
S = slope of main channel (m/m)
K = a unit conversion coefficient (*K* = 0.0195 for *SI* units)

For the existence of a low slope or a transitional slope condition, an adjusted slope should be used in calculating the time of concentration.

<u>Example on computation of peak flood using rational formula</u>
Data is from Shewu Small Scale Irrigation Project, Bench Maji Zone, SNNPR
The drainage area = 21ha more 60% land covered with cultivation
The main flow length = 641.39m Soil Hydrological Group = B

- Upstream Elevation (Elv_{us}) = 1419m Downstream Elevation (Elv_{ds}) = 1368m

Maximum daily rainfall for 25 year return period (P24) = 197.54mm

Computation of Time of Concentration
Slope (S) = (El_{us} – El_{ds})/L = (1419-1368)/641.39 = <u>0.08</u>

For over land flow use Kerby's retardance coefficient, N= 0.2 for cultivated land and K= 1.44 for SI unit.

For the channel flow use the Kirpich 's K= 0.0195 for SI units

$$T_{ov} = K(L * N)^{0.467} S^{-0.235} = 1.44*(641.39*0.2)^{0.467}*(0.08)^{-0.235}) = 25.16\,min$$

$$T_{ch} = KL^{0.770} S^{-0.385} = 0.0195*(641.39)^{0.77}*(0.08)^{-0.385} = 7.49\ min$$

$t_c = T_{ov} + T_{ch}$ = 25.16 + 7.49 = <u>32.65 min</u>

Runoff coefficient

Values for catchment characteristics:

S = 8%; C_T = 0.1; C_s = 0.1 for hydrological soil group B; C_v =0.2 for cultivated land
C= C_T +C_S+C_V = 0.1+0.1+0.2 = 0.4

Rainfall Intensity

24hr maximum precipitation = 197.54mm; it implies I_{24} = 197.54/24 = 8.23mm/hr
Time of concentration = 32.65min (0.544hr), b=0.33 and n=0.9

I_{tc} = (b+24)/(b+tc)^n *I_{24} = (0.33+24)/(0.33+0.544)^0.9 *8.23 = 226.04mm/hr

Peak Runoff rate will then be

Q_p = 0.00278CIA = 0.00278*0.4*226.04mm/hr*21 ha = **5.278 m^3/sec**

5.3.2.3 SCS-CN method

The SCS method is widely used for estimating floods on small to medium-sized ungauged catchments around the world. The method was developed based on 24-hr rainfall and runoff data in USA. Its derivation assumed that no runoff occurs until rainfall equals an initial abstraction, I_a (losses before runoff begins) and satisfies cumulative infiltration, *F* (the actual retention before runoff begins) or water retained in the catchment, excluding *Ia*. The potential retention before runoff begins (S) is the value *F* + I_a *that* would be attained in a very long storm. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or daily storm rainfall. The relationship is given Equation 5.8 below:

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

Where: Q = accumulated direct runoff, mm
P = accumulated rainfall (potential maximum runoff), mm;
I_a = initial abstraction (surface storage, interception, and infiltration prior to runoff), mm;
S = potential maximum retention, mm.

The relationship between I_a and S was developed from experimental data is given as I_a = 0.2STheredfore, Equation 5.8 is given as Equation 5.9 below.

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (5.9)$$

S is related with soil and land cover through the empirical value called Curve Number (CN), the values which ranges 0 - 100. The relationship between S and CN is expressed as in Equation 5.10

$$S = 254\left(\frac{100}{CN} - 1\right) \qquad (5.10)$$

Curve number depends on land use/cover, treatment practices and the soil in the catchment. Land use/cover of the catchment area refers all the cover types, including agricultural and non-agricultural uses, vegetation, water bodies, roads, roofs, etc. The treatment practice address different land practice on the cultivation and non-cultivated process it includes cultivation practice as straight, contour or seeding practice as closed seeded, row and other related practices. Similarly, soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D) as presented in Table 5.6. The determination of runoff curve number has also considered the effects of land-treatment measures in cultivated land use including mechanical practices such as contouring or terracing and management practices such as rotation of crops. Therefore, CN has to be selected based on the combination all aspects. CNs for different land use/cover, hydrologic soul groups and land treatment are given in Tables 5-8 to 5-11.

The CN values give in these tables are based on an average antecedent moisture condition (AMC), i.e., soils that are neither very wet nor very dry when the design storm begins. CNs can be selected only after a field inspection of the catchment area and a review of cover type and soil maps. Table 5.11 gives the antecedent conditions for the three classifications. Empirical relationships have been developed between the CN for average AMC and CNs for dry and we conditions to enable convert the normal CN to wet and dry CNs as shown by Equations 5.11 and 5.12 below.

$$CN_I = \frac{CN_{II}}{2.3 - 0.013CN_{II}} \quad (5.11)$$

$$CN_{III} = \frac{CN_{II}}{0.43 + 0.0057CN_{II}} \quad (5.12)$$

The selection of CN value is always subjective and requires experience and understanding of the catchment. Selection of overly conservative CN's will result in the estimation of excessively high runoff and consequently costly structures while selection of conservatively low values will result in economic loss due to underestimation of design floods. Therefore, care has to be taken in selecting the CN value. This subjectivity of CN selection is the major limitation the method.

Table 5-7: Hydrologic soil group (HSG) classification according to SCS (1972)

Group	Runoff Potential	Description
A	Low	Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.
B	Low -moderate	Soil having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Moderate- high	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.
D	High	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

Table 5-8: Cultivated agricultural land*

Cover description			Curve numbers for Hydrologic soil group			
Cover Type	**Treatment****	**Hydrologic Condition*****	**A**	**B**	**C**	**D**
Fallow	Bare soil	-	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row Crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C & T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T + CR	Poor	65	73	79	81
		Good	61	70	77	80
	Small grain SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T + CR	Poor	60	71	78	81
		Good	58	69	77	80
	Close-seeded SR or broadcast	Poor	66	77	85	89
		Good	58	72	81	85
	Legumes or C Rotation	Poor	64	75	83	85
		Good	55	69	78	83
	Meadow C&T	Poor	63	73	80	83
		Good	51	67	76	80

*Average runoff condition, and Ia = 0.2S.
** Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
***Hydrologic condition is based on a combination of factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or closed-seeded legumes in rotations, (d) percent of residue cover on the land surface (good > 20%), and (e) degree of roughness.
Poor: Factors impair infiltration and tend to increase runoff.
Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 5-9: Other agricultural lands1

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, orange continuous forage for grazing[2]	Poor Fair Good	68 49 39	79 69 61	86 79 74	89 84 80
Meadow-continuous grass, protected from grazing		35	59	72	79
Brush-weed-grass mixture with brush the major element[3]	Poor Fair Good	48 35 30[4]	67 56 48	77 70 65	83 77 73
Woods-grass combination[5]	Poor Fair Good	57 43 32	73 65 58	82 76 72	86 82 79
Woods[6]	Poor Fair Good	45 36 30[4]	66 60 55	77 73 70	83 79 77
Farms—buildings, lanes, driveways, and surrounding lots		59	74	82	86

[1]Average runoff condition, and Ia = 0.2S

[2]Poor: < 50% ground cover or heavily grazed with no mulch, Fair: 50 to 75% ground cover and not heavily grazed, Good >75% ground cover and lightly or only occasionally grazed

[3] Poor: < 50% ground cover, Fair: 50 to 75% ground cover, Good: > 75% ground cover

[4]Actual curve number is less than 30; use CN = 30 for runoff computations.

[5]CNs shown were computed for areas with 50% grass (pasture) cover. Other combinations of conditions may be computed from CNs for woods and pasture.

[6] Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning. , Fair : Woods grazed but not burned, and some forest litter covers the soil. , Good : Woods protected from grazing, litter and brush adequately cover soil.

Table 5-10: Arid and semi-arid rangelands*

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Mixture of grass, weeds, and low-growing brush, with brush the minor element	Poor Fair Good		80 71 62	87 81 74	93 89 85
Mountain brush mixture of small trees and brush	Poor Fair Good		66 48 30	74 57 41	79 63 48
Small trees with grass understory	Poor Fair Good		75 58 41	85 73 61	89 80 71
Brush with grass understory	Poor Fair Good	63 55 49	67 51 35	80 63 47	85 70 55
Desert shrub brush	Poor Fair Good		77 72 68	85 81 79	88 86 84

*Average runoff condition, and Ia=0.2S

**Poor < 30 % ground cover (litter, grass, and brush over story), Fair : 30 to 70 % ground cover Good: > 70 % ground cover

***Curve numbers for Group A have been developed only for desert shrub.

Table 5-11: Runoff curve numbers- urban areas*

Cover description	Curve numbers for hydrologic soil groups				
Cover type and Hydrologic condition	**Average % impervious area****	**A**	**B**	**C**	**D**
Open space (lawns, parks, cemeteries, etc.)***					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50 % to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas: Paved parking lots, roofs, driveways, etc.)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Desert urban areas: Natural desert cover		63	77	85	88
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.05 hectare or less	65	77	85	90	92
0.1 hectare	38	61	75	83	87
0.135 hectare	30	57	72	81	86
0.2 hectare	25	54	70	80	85
0.4 hectare	20	51	68	79	84
0.8 hectare	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

* Average runoff condition, and Ia=0.2S

** The average percent impervious area shown was used to develop the composite CNs. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. If the impervious area is not connected, the SCS method has an adjustment to reduce the effect.

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

Table 5-12: Rainfall ranges for amc, growing and dormant seasons

Antecedent Condition	Conditions Description	Growing Season Five-Day Antecedent Rainfall	Dormant Season Five-Day Antecedent Rainfall
Dry	An optimum Condition of catchment area soils, where soils are dry but not to the wilting point, and when satisfactory plowing or cultivation takes place	Less than 36 mm	Less than 13 mm
Average	The average case for annual floods	36 to 53 mm	13 to 28 mm
Wet	When a heavy rainfall, or light rainfall and low temperatures, have occurred during the five days previous to a given storm	Over 53 mm	Over 28 mm

SCS Peak Discharge Equation

The peak discharge is computed by developing hydrograph using the time conditions and the computed runoff depth (Q) based on the maximum daily rainfall (P) of a given return period. Hydrograph development depends on the catchment area. If the catchment area is less than 10 km^2; we can approximate it with single triangular hydrograph analysis otherwise has to be computed with multiple triangular hydrographs analysis based on rainfall profile.

Single Hydrograph analysis

Once Q is computed from P using SCS-CN equation and the catchment area is less than 10km^2, the peak discharge is approximated with single triangular hydrograph shown in Figure 5.1.

The base of the hydrograph is the time, named as Time of base (Tb) and it is computed from the different time components of the hydrograph as duration of rainfall excess (D), lag time of the peak (La), time of peak (Tp).

Time of the peak is estimated from half of the rainfall excess and large portion of time of concentration as Tp = 0.5D +0.6T_c.

The base length of the hydrograph is given as Tb= 2.67Tp.

From a triangular hydrograph with the assumption that excess rainfall depth equals runoff depth, the peak discharge can be estimated by Equation 5.13.

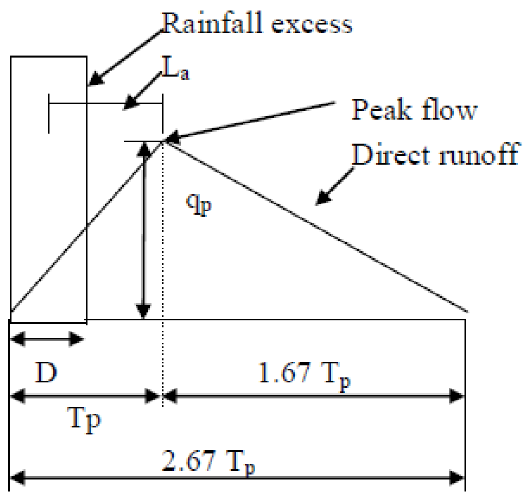


Figure 5-1: SCS triangular hydrograph

$$q_p = \frac{0.208AQ}{T_P} = \frac{0.208AQ}{0.5D + 0.6T_c} \quad (5.13)$$

Where:

q_p = peak discharge (m^3/s)

Q = the excess rainfall depth (mm) or the runoff depth computed with SCS-CN equation;

A = watershed area (km^2)

Tc = time of concentration (hr);

D = duration of excess rainfall (hr), which can be approximated as follows:

$$D \approx \frac{T_c}{6}, \quad if \quad Tc < 3hrs$$

$$D \approx 1hr \quad if \quad 3hrs < Tc < 6hrs$$

$$D \approx 1.5hr \quad if \quad 6hrs < Tc < 9hrs$$

$$D \approx 2hr \quad if \quad Tc > 9hrs$$

Calculation Example
This example is taken from Shewu Small Scale Irrigation Project in Bench Maji Zone of SNNPR. The example is taken for the analysis of peak discharge for the design of cross drainage structure (Shewu_CD_L2) on the main canal.
The drainage area = 298.1ha The main flow length = 3,686.01m

- Upstream Elevation = 1,481m asl Downstream Elevation = 1,359m asl
- Soil Hydrological Group = B Land use condition
- (60% cultivated, 40% wood lands

Computation of Time of Concentration
Slope = (El_{us}– El_{ds})/L = (1481-1359)/3686.01 = <u>0.033</u>

Calculation of T_{ov}

K= 1.44 for SI unit; N= 0.2 for 60% cultivated land; N=0.6 for 40% of woodlands

Weighted N =0.6*0.2 + 0.4*0.6) = 0.36

$T_{ov} = K(L*N)^{0.467}\, S^{-0.235} = 1.44*(3686.01*0.36)^{0.467} *(0.033)^{-0.235}$ = <u>92.23min</u>

Calculation of T_{ch} calculation

Kirpich K= 0.0195 for SI units

$Tch = KL^{0.770}\, S^{-0.385} = 0.0195*(3686.01^{0.770})*(0.033^{-0.385})$ = <u>40.43min</u>

$tc = T_{ov} + T_{ch}$ = 92.23 + 40.43 = <u>132.66min</u> = 2.211hr

Curve Number (CN) and potential maximum retention(S)
For 60% cultivated catchment with 'B' hydrological soil group; CNII = 81

For 40% woodland catchment with 'B' hydrological soil group CNII = 66

Weighted curve number CNII= (0.6*81+0.4*66) = <u>75</u>

Runoff depth and peak discharge
Based on the rainfall frequency analysis, the maximum daily areal rainfall in the catchment with 25 years of return period is **197.54 mm.**
D =tc/6= 2.21/6= 0.37 hrs T_p=D/2 +0.6 tc = 0.37/2 = 0.6*2.21 =1.51 hrs

For Average condition
S= 254*(100/CN_{III} – 1) = 254*(100/75 -1) = <u>84.67mm</u>

Q = (P- 0.2S)^2/(P+0.8S) =(197.54 - 0.2*84.67)^2/(197.54+0.8*84.67) = <u>122.96 mm</u>

$$Q_p = \frac{0.208 * A * Q}{T_p} = \frac{0.208 * 2.981 * 122.96}{1.51} = \underline{\boldsymbol{50.49\ m^3/s}}$$

For saturated condition

$$CN_{III} = \frac{CN_{II}}{0.43 + 0.0057\,CN_{II}} = \frac{75}{0.43 + 0.057 * 75} = 87.23$$

S= 254*(100/CN_{III} – 1) = 254*(100/87.23 -1) = <u>37.18 mm</u>
Q = (P- 0.2S)^2/(P+0.8S) = (197.54 - 0.2*37.18)^2/(197.54+0.8*37.18) = <u>159.01 mm</u>

$$Q_p = \frac{0.208 * A * Q}{T_p} = \frac{0.208 * 2.981 * 159.01}{1.51} = \underline{\boldsymbol{65.29\ m^3/s}}$$

Complex Hydrograph

For a catchment area of over 10km^2, the peak discharge has to be computed with complex/combined triangular hydrograph approach. The complex triangular hydrograph is constructed for six durational storms derived from the 24-hr rainfall profile.

The steps and procedures to construct complex hydrograph and compute peak discharge is described in the following example using data from Shewu weir site .

1. Catchment area = 2,146 ha
2. Weighted curve number for the catchment using the land use, soil and hydrologic condition of the catchment, use weighted CN = 65
3. Compute maximum potential retention (S) between rainfall (P) and direct runoff (Q) using $S = \frac{25400}{CN} - 254$, in this example, the weighted S = 136.8mm
4. Time of concentration (T_c) = 3.26 hr
5. Rainfall excess duration (D): approximated based on the time of concentration of the catchment as
   - If T_c < =3hr, as D= Tc/6
      - If 3 < T_c < =6hr as D=1hr
      - If 6 <T_c<=9hr as D=1.5hr
      - For T_c >9hr as D=2 hr
6. Time of peak (Tp) = 0.5D+0.6Tc = 0.5 *1 + 0.6*3.26 = 2.46hr
7. Time of base (T_b) = 2.67T_p = 2.67*2.46 = 6.56hr
8. 24-hr Design storm (P) for 100 year return period = 224.99mm
9. The peak discharge is computed with six triangular hydrographs that constructed for different excess rainfall in the 24 hr. To construct the six complex triangular hydrograph we use the 24hr rainfall profile chart (Figure 5.2) and point areal to point rainfall ration (Table 5.13). The steps constructed in table 5.14

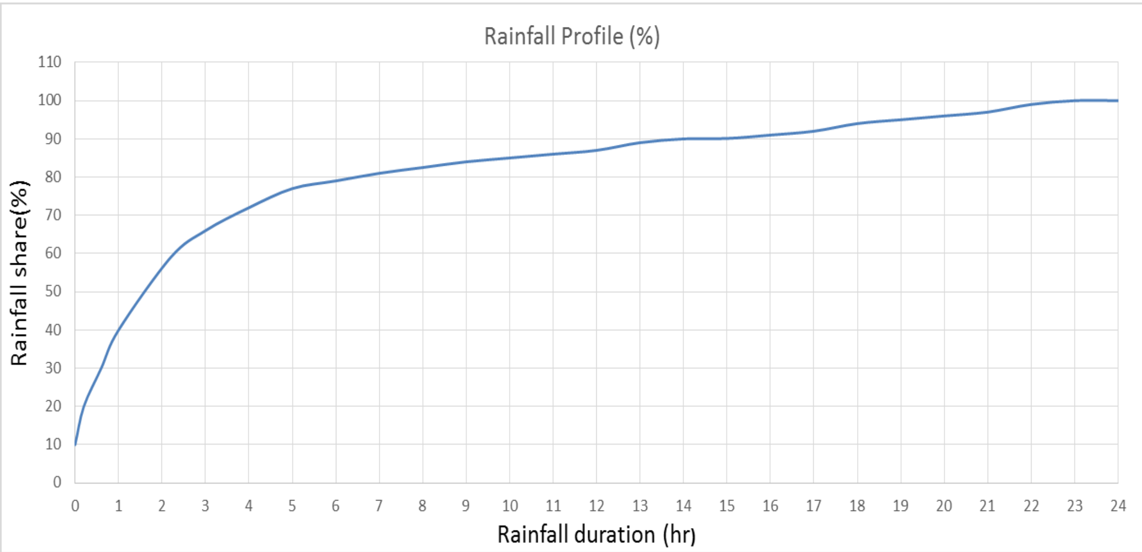


Figure 5-2: Hourly distribution of daily rainfall profile

Table 5-13: Areal to point rainfall ratio (%)

Area	Duration (hr)											
(km2)	0.5	1	2	3	4	5	6	9	12	15	18	21	24
25	88	78	82	85	87	88	88	91	92	93	93	94	94
50	61	71	78	82	84	85	87	89	90	91	92	92	93
75	57	67	75	79	82	84	83	87	89	90	91	91	92
100	54	65	73	78	80	82	83	86	88	89	90	91	91
125	52	63	72	76	79	81	82	85	87	88	89	90	91
150	50	61	70	75	78	80	81	84	86	88	89	89	90
175	48	59	69	74	77	79	81	84	86	87	88	89	90
200	46	58	68	73	76	78	80	83	85	87	88	88	89
225	45	57	57	72	75	77	72	82	85	86	87	88	89
250	44	55	66	71	74	77	78	82	84	86	87	88	88
275	42	54	65	70	74	76	78	81	84	85	86	87	88
300	41	53	54	70	73	75	77	81	83	85	86	87	88
325	40	53	63	58	72	73	77	80	83	84	86	87	87
350	38	52	63	68	72	74	76	80	82	84	85	86	87
375	39	51	62	68	71	74	78	80	82	84	85	86	87
400	38	50	61	67	71	73	75	79	82	83	85	86	87
425	37	50	61	67	70	73	75	79	81	83	84	85	86
450	36	49	60	66	70	72	74	79	81	83	84	85	86
475	36	48	60	66	69	72	74	78	81	83	84	85	86
500	35	48	59	66	69	72	74	78	80	82	84	85	86
525	34	47	59	65	68	71	73	78	80	82	83	85	85
550	34	47	58	64	68	71	73	77	80	82	83	84	85
575	33	46	58	64	68	71	73	77	80	82	83	84	85
600	33	45	57	63	67	71	72	77	79	81	83	84	85
625	32	45	57	63	67	70	72	76	79	81	83	84	85
650	32	45	56	63	67	70	72	76	79	81	82	84	84
675	31	44	56	62	66	69	71	76	79	81	82	83	84
700	31	44	56	62	66	69	71	76	78	80	82	83	84
725	31	43	55	62	66	69	71	75	78	80	82	83	84
750	30	43	55	61	65	68	71	75	78	80	82	83	84

Table 5-14: Steps to construct complex hydrograph

1	2	3	4	5	6	7	8	9	10	11
Duration (hr)	Daily point R.F (mm)	R.F profile (%)	R.F profile (mm)	Areal to point R.F Ratio (%)	Areal to point R.F Ratio(mm)	Incremental R.F (mm)	Descending order (No)	Rearranged order (No)	Rearranged Incremental RF(mm)	cumulative Rainfall (mm)
1	224.99	34.94	78.61	78	61.31	61.31	1	5	20.68	20.68
2		44.66	100.47	82	82.39	21.07	4	3	21.47	42.15
3		54.37	122.33	85	103.98	21.60	2	1	61.31	103.46
4		64.09	144.19	87	125.45	21.47	3	2	21.60	125.06
5		73.80	166.05	88	146.13	20.68	5	4	21.07	146.13
6		83.52	187.91	88	165.36	19.24	6	6	19.24	165.37

Brief descriptions for the columns to construct complex Hydrograph	
1	Fill the six rows with the duration value as D, 2D, 3D, 4D, 5D and 6D
2	Bring the computed design storm with the required recurrence interval (return period)
3	Read the rainfall profile with D, 2D, 3D ….6D from chart presented in Figure 5.2 below
4	Multiply the rainfall profile with the design storm presented in #2 and fill #4
5	Read areal to point ratio for different duration and catchment area from Table 5.14 below
6	Multiply #4 by #5 and fill in #6
7	Calculate incremental rainfall by deducing current areal rainfall from succeeding areal rainfall in #6
8	Assign 1-6 order values with their descending order of the incremental rainfall as listed in #7
9	Rearrange the order value arbitrary but keep the highest as middle of them and check with the cumulative to be a continuous incremental in this Example 5,3,1,2,4,6
10	Fill in the corresponding incremental rainfall value to the rearranged order of #9 from #7
11	Fill in the cumulative rainfall values of #10 by adding the rainfall values in the preceding duration

Continue Table 5-14

	12	13	14	15	16	17	18	19	20
Time (hr)	Direct Runoff (mm)	Incremental Runoff (mm)	Time of beginning	Time of Peak	Time of End	q_p (m3/sec)	Equation for Resign limp	Equation for Recession limp	Construct each hydrograph individual for a given time in between beginning and end time
0 -1	0	0	0	2.46	6.56	0.00	0.2525T	0.992-0.1512T	
1-2	1.44	1.47	1	3.46	7.56	2.67	0.5772T-0.5772	2.6183-0.3463T	
2-3	27.21	25.84	2	4.46	8.56	46.90	10.484T-20.967	53.844-6.2902T	
3-4	40.71	13.18	3	5.46	9.56	23.92	4.4878T-13.463	25.742-2.6827T	
4-5	55.21	14.76	4	6.46	10.56	26.79	4.0691T-16.276	25.782-2.4415T	
5-6	69.32	14.11	5	7.46	11.56	25.61	3.435T-17.175	23.825-2.061	

Brief descriptions for the columns to construct complex Hydrograph	
12	Compute the direct runoff for each cumulative rainfall with weighted curve number(CN) and maximum retention capacity (S) for the catchment
13	Find incremental runoff by reducing the values of #12 by proceeding values
14	Fill in the time of beginning of hydrograph as 0, D, 2D, ……5Dhr
15	Fill in the time to peak as, add tp computed for the catchment on the time of beginning
16	Fill in the time to end as, add tb computed for the catchment on the time of beginning
17	Calculate peak discharge with qp = 0.208AQ/tp, using the tp for each duration;
18	Construct the triangular hydrograph and Drive the linear equation for rising limp for each time duration
19	Construct the triangular hydrograph and Drive the linear equation for recession limp
20	Construct each hydrograph individual for a given time in between beginning and end time

Equation for rising limp

$$\frac{Q_p T}{T_p} \qquad (5.14)$$

Equation for recession limp

$$Q_p - Q_p \frac{(T - T_p)}{(T_b - T_p)} \qquad (5.15)$$

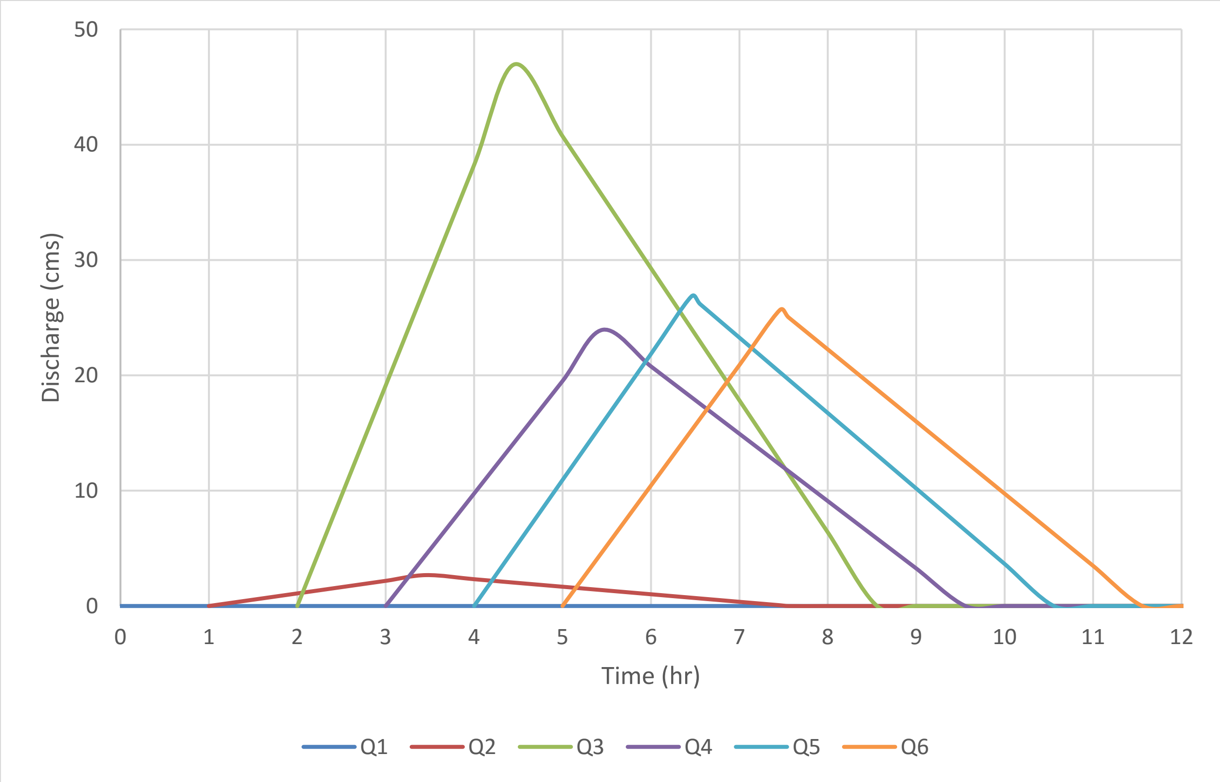


Figure 5-3: Composite Hydrograph developed with complex hydrograph approach

Table 5-15: The Composites the triangular hydrographs to a single hydrograph

Time (hr)	Q1	Q2	Q3	Q4	Q5	Q6	Qt
0.00	0.00						**0.00**
1.00	0.00	0.00					**0.00**
2.00	0.00	1.09	0.00				**1.09**
2.46	0.00	1.59	8.72				**10.31**
3.00	0.00	2.18	19.12	0.00			**21.30**
3.46	0.00	2.68	27.84	4.45			**34.97**
4.00	0.00	2.32	38.25	9.75	0.00		**50.32**
4.46	0.00	2.03	46.97	14.20	4.98		**68.17**
5.00	0.00	1.67	40.74	19.51	10.93	0.00	**72.84**
5.46	0.00	1.37	35.52	23.95	15.91	4.76	**81.51**
6.00	0.00	1.02	29.29	20.78	21.85	10.44	**83.37**
6.46	0.00	0.72	24.06	18.11	26.83	15.21	**84.94**
6.56	0.00	0.65	22.90	17.52	26.17	16.27	**83.51**
7.00	0.00	0.36	17.84	14.94	23.27	20.89	**77.30**
7.46	0.00	0.07	12.61	12.27	20.29	25.65	**70.89**
7.56	0.00	0.00	11.45	11.68	19.63	25.02	**67.77**
8.00	0.00	0.00	6.38	9.10	16.73	22.25	**54.46**
8.56	0.00	0.00	0.00	5.84	13.08	18.76	**37.69**
9.00	0.00	0.00	0.00	3.26	10.19	15.99	**29.44**
9.56	0.00	0.00	0.00	0.00	6.54	12.51	**19.05**
10.00	0.00	0.00	0.00	0.00	3.65	9.74	**13.39**
10.56	0.00	0.00	0.00	0.00	0.00	6.25	**6.25**
11.00	0.00	0.00	0.00	0.00	0.00	3.49	**3.49**
11.56	0.00	0.00	0.00	0.00	0.00	0.00	**0.00**

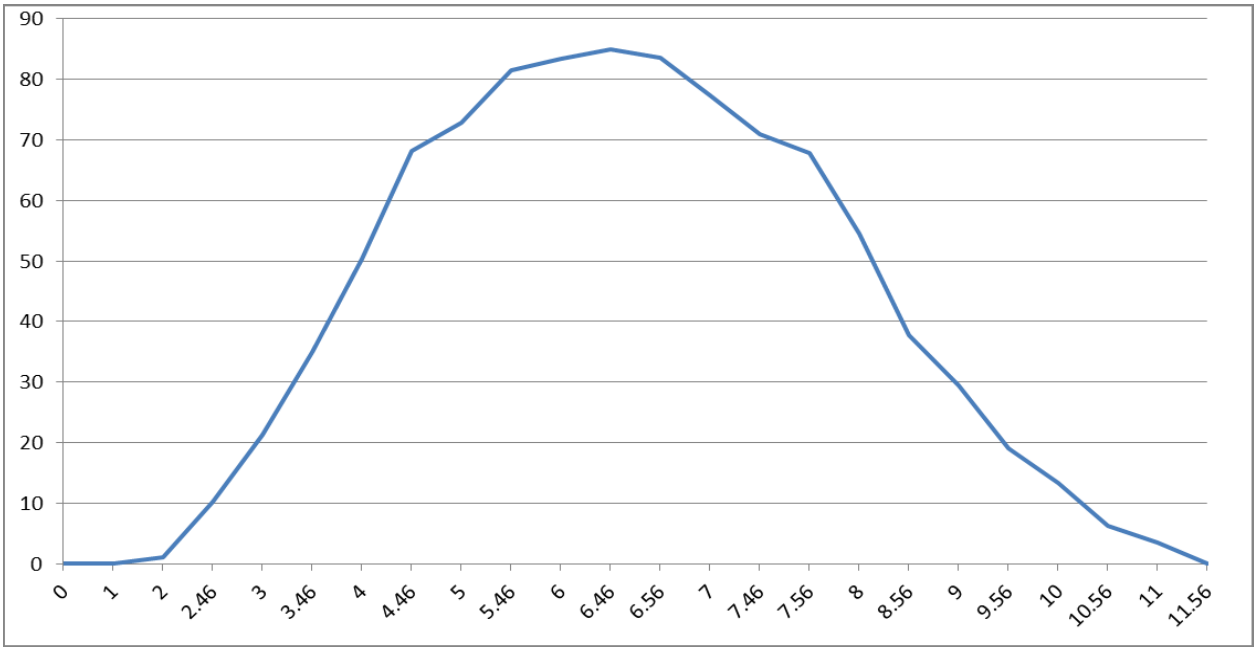


Figure 5-4: Composite hydrograph developed with complex hydrograph approach

5.3.3 Synthetic Unit Hydrograph Method

The derivation of unit hydrograph always depends on the observed rainfall excess hyetograph (ERH) and direct runoff hydrograph (DRH). However, these values are not available for ungauged catchments and this needs the development of an empirical relation that relates the unit hydrograph ordinate with the catchment characteristics. This process is known as synthetic unit hydrograph development. For this, Snyder's approach in combination with other supportive researches conducted in Ethiopia (Mulugteta, 2004) is selected for the derivation of the synthetic unit hydrograph. Using the synthetic unit hydrograph and the rainfall distribution of the daily maximum rainfall for a given return period will help to compute peak discharge of ungauged catchments. Table 5-16 provides the procedures to be followed for deriving synthetic hydrograph.

Table 5-16: Example of developing synthetic unit hydrograph

No	Procedure	Example (Arjo Catchment)	Remark
1	Catchment area (A), km^2	5,488.77	
2	Longest flow length (L) , Km	205.52	
3	Catchment shape factor (flow length from centroid of basin to outlet (Lc), km	111.270	
4	Basin average slope (S), m/m	0.00539	
5	Compute the lag time (T_l) of the basin, which is the time from the midpoint of rainfall excess (Dhr) to the peak of the unit direct runoff hydrograph. $T_l = 0.127 * \left(LL_c / \sqrt{S}\right)^{0.352}$	10.901	
6	Compute duration of excess rainfall (D hrs); $D = \frac{T_l}{5.5}$	1.98 ≈ 2	Adjust to the nearest standard rainfall excess duration (D) (take 2 hrs in this case)
7	Adjust the time of lag with respect to the adjusted excess rainfall duration $T_{la} = T_l + 0.25(D_S - D)$	10.906	Explain Ds

No	Procedure	Example (Arjo Catchment)	Remark
8	Compute time of peak as $T_p = 0.5D + T_{la}$	11.91≈12 hr	Round to whole number as multiple of D
9	Time of base for the unit hydrograph $(T_b) = 5(T_{la} + 0.5D)$	60	
10	Compute the peak discharge (q_p) of the Unit hydrograph in m^3/sec/mm as $q_p = \frac{0.208A}{T_p}$ Where A is drainage area in(km^2), T_p is peak time in hr	95.15m^3/sec/mm	
13	Plot the UH with a smooth line with the three know values, T_p, T_b, and q_p The volume of 1 mm runoff in the catmint area should be equal to the volume of the UH. Therefore, using an iterative process in Excel, adjust the ordinates of the UH until the two volumes are equal. . Read the ordinates of the UH from the iteratively developed UH as given below.		

Time	0	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
Q_{uh}	0	9	18	34	52	76	95.15	86	64	48	38	33	29	26	23	20
Time	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	
Q_{uh}	18	14	13	11	10	9	8	7	6	5	4	3	2	1.18	0	

No	Procedure	Example	Remark
15	Consider the daily rainfall distribution to get the rainfall excess. If the distribution of the rainfall excess duration is different from the duration of the UH, the UH has to be converted to the duration of excess rainfall using S-UH.		
16	The 24hr peak rainfall has to take from the regional IDF of the site or it can be computed based on the frequency analysis.	175.2	In this example, take 100 year return period
17	Compute the 0.5hr MRF using exponential equation α0.5= 1-exp(-125/(Rday +5)).	0.5	

Procedural note for column 18 -22

No	Note
18	The time for the rainfall distribution is taken based on the rainfall duration computed in synthetic UH and it has to be extended up to the time of lag of the synthetic UH
19	The intensity of the rainfall is computed based on the 24hr peak rainfall and 0.5hr MRF is computed in step 16 and 17 using the following intensity exponential equation. $I = \frac{(1 - \exp(2Tc * \ln(1 - \alpha 0.5))) * Rday}{Tc}$
20	The infiltration rate of the catchment is computed based on the Horton equation, with parameters given below for the following soil hydrological groups: HSG / fo (mm/hr) / fc (mm/hr) / K (1/hr) A / 127.00 / 25.40 / 0.20 B / 105.00 / 15.24 / 0.18 C / 81.28 / 12.70 / 0.17 D / 71.12 / 10.16 / 0.16 $f_p = f_c + (f_0 - f_c) e^{-kt}$ In the example catchment, is the dominant hydrological soil group is B for which the Horton equation fp = 15.24 + 89.76$e^{-0.18t}$. Then the infiltration rate at each time is computed as presented above. If a catchment has different hydrological soil groups one has to compute the weighted average values of the parameters based on the area proportion covered by each hydrological soil group.
21	The rate of excess rainfall can then be simply computed by subtracting the infiltration rate from rainfall intensity. (i.e., column 19 – column 20)
22	To get the excess rainfall depth, multiply the excess rainfall rate with the time of excess rainfall, 2hr in the current example,

Computed values with Arjo Example data					
18	19	20	21	22	
Time (hr)	Intensity (mm/hr)	Infiltration Rate (mm/hr)	Excess Rainfall rate (mm/hr)	Excess Rainfall (mm)	
2	0.00	105.00	0.00	0.000	
4	82.14	77.86	4.27	8.546	The only excess rainfall
6	43.63	58.93	0.00	0.000	
8	29.19	45.72	0.00	0.000	
10	21.90	36.51	0.00	0.000	
12	17.52	30.08	0.00	0.000	

Steps 23 -28 computation of stream flow using the synthetic unit hydrograph and Excess rainfall	
23	Time distribution based on the rainfall excess duration up to the base time of the unit Hydrograph (i.e., 60hr , ref. row 9)
24	Use the computed UH with reference in column 14
25	Use the excess rainfall computed in column 22 and multiply the excess rainfall for each UH ordinate. If there is two or more excess rainfall multiply with keeping the lag time.
26	Compute the direct runoff by summing all the ordinate in the same row; for the current example, there is only one time excess rainfall so the sum is the same.
27	Compute the base flow based on the relationship fixed for the river basin using UH sequential approach, for example, the dry flow observed at UH
28	Compute the total discharge by adding the direct runoff and the base flow to get the stream flow with peak rainfall for the given catchment.

Computed values with Arjo Example data					
23	24	25	26	27	28
Time (hr)	UH (m^3/sec/mm)	Excess Rainfall (mm) 8.546	Direct Runoff (m^3/sec)	Base Flow (m^3/sec)	Total Runoff (m^3/sec)
0	0	0.00	0.00	45.12	45.12
2	9	76.91	76.91	54.12	131.03
4	18	153.83	153.83	63.12	216.95
6	34	290.56	290.56	79.12	369.68
8	52	444.39	444.39	97.12	541.51
10	76	649.50	649.50	121.12	770.62
12	95.14	813.15	813.15	140.26	953.33
14	86	734.96	734.96	131.12	866.08
16	64	546.94	546.94	109.12	656.06
18	48	410.21	410.21	93.12	503.33
20	38	324.75	324.75	83.12	407.87
22	33	282.02	282.02	78.12	360.14
24	29	247.83	247.83	74.12	321.95
26	26	222.20	222.20	71.12	293.32
28	23	196.56	196.56	68.12	264.68
30	20	170.92	170.92	65.12	236.04
32	18	153.83	153.83	63.12	216.95
34	14	119.64	119.64	59.12	178.76
36	13	111.10	111.10	58.12	169.22

Computed values with Arjo Example data					
23	24	25	26	27	28
Time (hr)	UH (m^3/sec/mm)	Excess Rainfall (mm) 8.546	Direct Runoff (m^3/sec)	Base Flow (m^3/sec)	Total Runoff (m^3/sec)
38	11	94.01	94.01	56.12	150.13
40	10	85.46	85.46	55.12	140.58
42	9	76.91	76.91	54.12	131.03
44	8	68.37	68.37	53.12	121.49
46	7	59.82	59.82	52.12	111.94
48	6	51.28	51.28	51.12	102.40
50	5	42.73	42.73	50.12	92.85
52	4	34.18	34.18	49.12	83.30
54	3	25.64	25.64	48.12	73.76
56	2	17.09	17.09	47.12	64.21
58	1.189	10.08	10.08	46.31	56.47
60	0	0.00	0.00	45.12	45.12
The peak discharge is 953.41m3/sec and this can be the design discharge					

5.4 DEPENDABLE FLOW

Dependable flow is defined as the flow of stream flow with different probability of exceedence (60-90%) in a flow duration curve under a quasi-natural flow condition. A-thumb-of-rule 75%, 90% and 100% dependable flow for irrigation, power supply and drinking water supply respectively. The dependable flow can be approximated by the computed value from monthly simulated discharge as a first order approximation. A popular method of studying the dependability of stream flow is through flow duration curves which can be regarded as a standard reporting output from hydrological data processing. A flow-duration curve is a plot of discharge against the percentage of time the flow was equaled or exceeded. This may also be referred to as a cumulative discharge frequency curve and it is usually applied to daily mean discharges.

5.4.1 Understanding of flow-duration curve

Characteristics of the flow duration curve

- The virgin-flow duration curve when plotted on a log probability paper follows as a straight line at least over the central region. From this property, various coefficients expressing the variability of the flow in a stream can be developed for the description and comparison of different streams.
- The presence of a reservoir in a stream considerably modifies the virgin-flow duration curve depending on the nature of flow regulation.
- The chronological sequence of occurrence of the flow is masked in the flow-duration curve. A discharge of say 1,000 m^3/s in a stream will have the same percentage Pp whether it has occurred in January or June. This is a serious handicap and must be kept in mind while interpreting a flow- duration curve.
- The slope of a flow duration curve depends on the interval of data selected. For example, a daily stream flow data gives a steeper curve than a curve based on monthly data for the same stream.
- For a flow-duration curve plotted on a log-log paper, a steep slope of the curve indicates a stream with a highly variable discharge whereas a flat slope indicates a slow response of the catchment to the rainfall and small variability.

- At the lower end of the curve, a flat portion indicates considerable base flow while a flat curve on the upper portion is typical of river basins having large flood plains and large snowfall during a wet season.

Flow-duration curves find considerable use in water-resources planning and development activities. Some of the important uses are:

- I Evaluating various dependable flows in the planning of water-resources engineering projects;
- Evaluating the characteristics of the hydropower potential of a river;
- Design of drainage systems and flood-control studies;
- In Computing sediment and dissolved solids load of a stream; and
- Comparing adjacent catchments with a view to extend the stream flow data

5.4.2 Flow Duration Curve for Gauged Stream

The first step in developing flow duration curve is collecting all available stream flow data. This will then be followed by developing the flow duration curve with different approach and cases.

Case 1: flow duration curves from daily flow data

Steps

i. Choose a constant width class interval ***(ci)*** that can *result in* about 25 to 30 classes with the ranges of the available data sets
ii. Develop the classes from upper to lower class ranges for each class and arrange them in their descending order
iii. Count total number of days in each class equal and greater than the lower bound
iv. Cumulate the number of days in each class interval to get the number of days above the lower limit of each class interval.
v. Compute the probabilities of exceedence dividing the quantities obtained from step (iv) by the total number of days in the record (for example, 365 if one year record is considered for the construction of flow duration curve).
vi. Multiply the probabilities of exceedence obtained from step (v) by 100 to get percentage exceedence.
vii. Plot the probabilities of exceedence in percentage against the corresponding lower bound of class interval on linear graph paper. Sometimes, the flow duration curve better approximates to a straight line if lognormal probability paper is used in place of linear graph paper.

Example

A daily flow data from Mike River at Mike village gauging station from 2 Jan, 1985 to 23 Dec, 2010 with a total of 7054 data set is used to develop flow duration curve at the gauged station. The data set has a maximum of 139.68m^3/sec and a minimum of 0 m^3/sec discharge. With class interval of 5, 28 classes are constructed.

Table 5-17: Example for Flow duration curve using gauged data with daily data set

Upper	Lower	Number of days in the class	Cumulative (m)	Probability (m/(N+1)*100
140	135	1	1	0.014
135	130	0	1	0.014
130	125	0	1	0.014
125	120	1	2	0.028
120	115	1	3	0.043
115	110	1	4	0.057
110	105	1	5	0.071

Upper	Lower	Number of days in the class	Cumulative (m)	Probability (m/(N+1)*100
105	100	2	7	0.099
100	95	2	9	0.128
95	90	0	9	0.128
90	85	6	15	0.213
85	80	4	19	0.269
80	75	7	26	0.369
75	70	10	36	0.510
70	65	10	46	0.652
65	60	30	76	1.077
60	55	32	108	1.531
55	50	42	150	2.126
50	45	53	203	2.877
45	40	71	274	3.884
40	35	86	360	5.103
35	30	113	473	6.704
30	25	171	644	9.128
25	20	296	940	13.324
20	15	449	1389	19.688
15	10	517	1906	27.016
10	5	799	2705	38.342
5	0	4349	7054	99.986

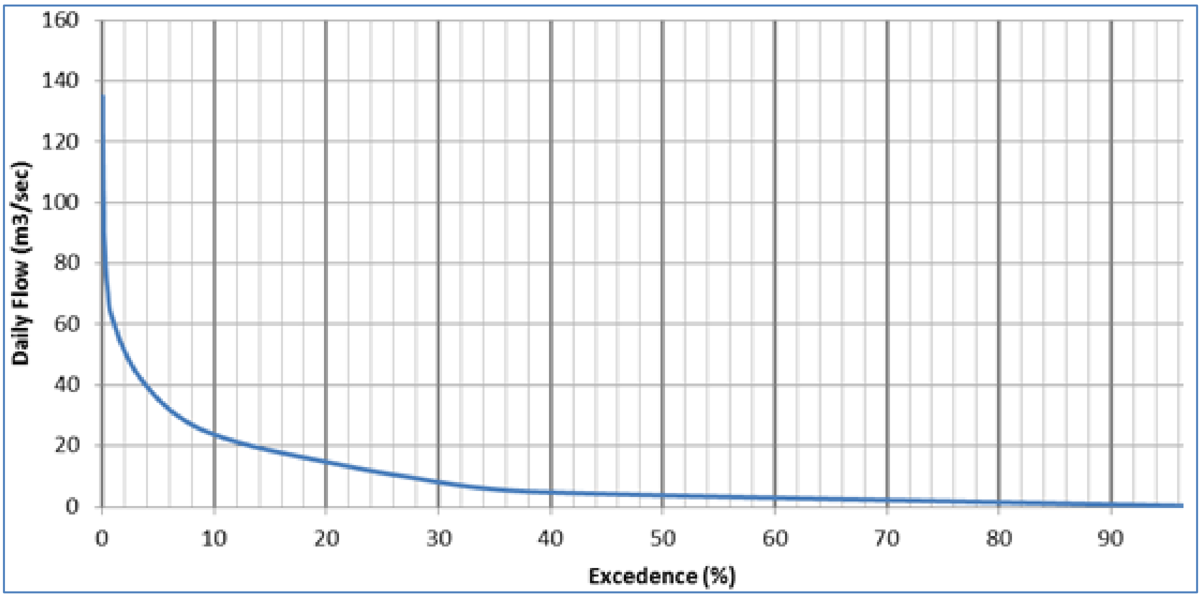


Figure 5-5: Flow duration curve of Meki River at Meki village Gauging station in normal graph

With this analysis, the following dependable flows can be read from the curve:

70% dependable daily flow = 2.433 75% dependable daily flow = 2.028
80% dependable daily flow = 1.623 90% dependable daily flow = 0.812
95% dependable daily flow = 0.406 99% dependable daily flow = 0.082

Case 2: flow duration curves from monthly or any other duration larger than 'daily' flow

i. Arrange the flow data in descending order for each month or required duration
ii. Rank them with, ***m* = 1** for the highest flow values to **m=*N*** for the lowest value (or variant).
iii. Assign the probability of exceedences to each data item obtained from step (i) using the Weibull plotting position formula

$$P = \frac{m}{N+1} \times 100 \qquad (5.16)$$

Note: If the flow duration curve is required to be linearized on normal probability paper or lognormal probability paper, the probability of exceedences may be assigned using *Blom*b *plotting position formula*

$$P = \frac{m-0.375}{N+0.250} \times 100 \qquad (5.17)$$

iv. Plot the ranked flow values against the probabilities of exceedence (computed using Weibull plotting position) on linear graph paper to get the flow duration curve. Use normal probability paper if the required dependable flow (or probability of exceedence) is to be extrapolated.

Note:
In order to linearize the flow duration curve, the probabilities of exceedence may be computed using *Blom*b *plotting position.*

For this analysis, monthly data for 30 years become reasonable for the computation of the dependable flow; however at the time of data shortage, it is possible to compute using 20 years data as minimum data year requirement

Example
The mean monthly flow of Gilgel Abay gauge station at Merawi was collected from the Hydrology Directorate of MoWIE; The monthly flow is arranged in descending order for each month and assigned with the rank starting from 1 to N=51. The probabilities of exceedences are then computed with Weibull plotting position.

Table 5-18: Example for flow duration curve using gauged data with monthly data set

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	m	P
1960	17.92	15.53	11.72	10.96	22.74	37.47	122.80	181.30	734.00	70.00	35.20	42.20	1	0.02
1961	15.99	12.51	11.34	10.59	10.59	29.57	93.90	179.56	161.83	66.96	29.75	26.33	2	0.04
1962	14.63	12.11	10.96	10.59	10.59	28.85	90.43	169.77	146.80	50.02	25.10	17.92	3	0.06
1963	14.19	11.34	10.59	10.23	9.87	28.25	84.00	169.50	135.20	44.70	22.74	17.42	4	0.08
1964	13.76	11.34	10.23	9.18	9.87	26.33	82.48	168.00	121.69	39.80	22.74	16.46	5	0.10
1965	10.96	10.96	10.23	8.52	9.18	23.36	81.88	168.00	121.40	39.02	21.61	16.46	6	0.12
1966	10.59	9.18	8.21	7.89	7.89	20.63	80.56	160.70	104.47	38.21	21.61	15.99	7	0.13
1967	8.72	5.67	3.97	2.94	3.29	16.69	77.20	160.70	104.40	37.47	21.40	15.99	8	0.15
1968	6.98	5.37	3.88	2.78	3.29	16.21	74.20	158.90	99.10	37.43	21.06	14.50	9	0.17
1969	6.96	4.53	3.68	2.78	2.94	14.80	73.40	153.50	97.80	34.47	20.28	13.69	10	0.19
1971	6.40	4.53	3.68	2.78	2.94	14.48	71.36	151.70	97.58	34.41	19.73	11.14	11	0.21
1972	6.30	4.29	3.60	2.63	2.94	13.76	69.00	151.14	93.69	33.03	18.75	10.59	12	0.23
1973	6.12	4.29	3.40	2.50	2.78	12.92	67.90	150.40	92.60	33.02	17.65	9.86	13	0.25
1974	5.97	4.29	3.27	2.49	2.78	12.51	66.96	150.00	92.60	33.02	16.78	9.52	14	0.27

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	m	P
1975	5.71	4.28	3.21	2.46	2.78	12.18	66.28	145.99	92.60	32.36	16.21	9.18	15	0.29
1976	5.67	4.08	3.21	2.46	2.78	12.11	65.88	139.80	87.60	32.25	15.73	9.09	16	0.31
1977	5.67	4.08	3.11	2.39	2.63	11.37	65.80	138.20	85.57	31.55	14.80	7.90	17	0.33
1978	5.46	4.04	3.11	2.39	2.55	10.60	64.28	137.98	83.10	30.92	14.80	7.89	18	0.35
1979	5.21	3.97	3.06	2.35	2.55	10.26	61.69	135.60	82.48	30.86	14.38	7.64	19	0.37
1980	5.21	3.97	3.00	2.35	2.49	9.18	60.80	135.60	81.60	30.24	14.15	7.59	20	0.38
1981	5.04	3.71	2.97	2.35	2.46	9.18	58.80	134.09	80.40	29.75	13.76	7.30	21	0.40
1982	5.04	3.68	2.94	2.35	2.35	9.18	58.80	134.00	79.07	27.56	13.45	7.30	22	0.42
1983	4.79	3.68	2.88	2.28	2.28	8.72	58.64	132.40	78.28	27.52	12.60	7.30	23	0.44
1984	4.74	3.65	2.82	2.21	2.26	8.52	57.65	129.50	75.60	26.93	12.30	7.30	24	0.46
1985	4.51	3.65	2.78	2.12	2.21	7.30	56.80	129.20	74.50	26.31	12.18	7.20	25	0.48
1986	4.51	3.48	2.78	2.05	2.21	6.75	56.80	125.75	73.73	25.53	11.37	6.75	26	0.50
1987	4.29	3.29	2.63	1.93	2.21	6.68	54.72	124.40	73.40	24.51	11.37	6.68	27	0.52
1988	4.29	3.11	2.63	1.93	2.07	6.12	54.12	121.69	72.49	23.36	11.14	6.23	28	0.54
1989	4.29	3.11	2.49	1.79	2.07	5.71	52.54	118.00	72.49	23.13	10.98	6.23	29	0.56
1990	4.08	3.11	2.49	1.79	2.05	5.21	52.05	114.90	71.36	22.80	10.98	6.23	30	0.58
1991	4.08	3.11	2.35	1.65	1.93	4.69	50.96	114.16	70.14	21.96	10.60	6.12	31	0.60
1992	3.88	2.94	2.35	1.65	1.93	4.53	47.36	113.40	69.00	21.70	10.59	5.81	32	0.62
1993	3.88	2.78	2.21	1.65	1.93	4.51	46.70	100.40	65.80	21.60	10.23	5.71	33	0.63
1994	3.68	2.78	2.21	1.65	1.65	4.08	44.70	97.80	63.80	21.16	10.23	5.71	34	0.65
1995	3.60	2.74	2.07	1.65	1.65	3.68	44.70	95.90	62.40	20.85	9.87	5.71	35	0.67
1996	3.48	2.63	2.07	1.61	1.65	3.65	43.80	94.60	60.80	20.51	9.87	5.71	36	0.69
1997	3.48	2.49	1.93	1.61	1.65	3.48	43.80	94.57	58.64	19.60	9.86	5.46	37	0.71
1998	3.48	2.49	1.93	1.51	1.61	3.26	43.40	93.90	57.80	18.61	9.52	5.42	38	0.73
1999	3.48	2.46	1.65	1.39	1.61	3.25	42.24	93.90	56.77	18.61	9.18	5.40	39	0.75
2000	3.48	2.21	1.65	1.26	1.61	3.11	42.24	90.65	56.66	18.25	8.85	5.37	40	0.77
2001	3.11	2.21	1.65	1.24	1.51	2.97	42.24	90.65	55.80	18.13	8.52	4.98	41	0.79
2002	3.11	2.11	1.59	1.23	1.51	2.97	35.90	89.36	54.72	17.42	8.20	4.97	42	0.81
2003	3.08	1.98	1.51	1.15	1.49	2.82	34.47	87.60	53.80	17.17	8.20	4.97	43	0.83
2004	2.94	1.72	1.49	1.06	1.49	2.78	32.96	82.80	53.25	17.08	7.89	4.74	44	0.85
2005	2.73	1.65	1.26	1.01	1.47	2.78	32.96	82.20	51.90	15.81	7.59	4.69	45	0.87
2006	2.57	1.65	1.15	1.01	1.39	2.27	31.55	80.67	50.10	14.80	7.29	4.51	46	0.88
2007	2.49	1.59	1.15	0.91	1.06	2.21	31.42	68.05	47.36	14.63	7.02	4.29	47	0.90
2008	2.26	1.47	1.12	0.72	0.64	2.07	29.51	67.90	46.50	14.63	6.23	4.29	48	0.92
2009	1.98	1.26	1.01	0.64	0.64	1.84	26.93	65.89	42.24	14.34	6.17	3.44	49	0.94
2010	1.84	1.15	0.96	0.58	0.58	1.63	22.80	60.80	42.20	13.45	6.17	3.44	50	0.96
2011	1.72	1.06	0.88	0.51	0.51	1.36	21.70	60.66	41.40	12.18	5.71	2.90	51	0.98

Table 5-19: Finally the monthly different dependable flows for Giligel Abay at Merawi Station

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
60%	4.08	3.11	2.35	1.65	1.93	4.69	50.96	114.16	70.14	21.96	10.60	6.12
70%	3.48	2.63	2.07	1.61	1.65	3.65	43.80	94.60	60.80	20.51	9.87	5.71
80%	3.11	2.11	1.59	1.23	1.51	2.97	35.90	89.36	54.72	17.42	8.20	4.97
90%	2.49	1.59	1.15	0.91	1.06	2.21	31.42	68.05	47.36	14.63	7.02	4.29

5.4.3 Flow duration curve for ungauged stream

If there is no gauged stream flow data on the targeted point, the issue has to be addressed in different ways.

Case 1: There is no observed stream flow data on the site but there is stream flow data with similar stream flow data in the homogeneous region

(i) Transfer the daily or monthly flow data according to the data availability from the gauged site to ungauged site. One can use the gauged and ungagged data transfer relation proposed by Admasu Gebeyehu (1986)

$$Q_u = Q_g \left(A_u / A_g\right)^{0.7} \tag{5.18}$$

(ii) Develop the flow duration curve using the procedure stated in Case 1 or Case 2 for gauged site according to the transferred data type.

Case 2: If neither gauged data nor transferrable data is available one has to resort to the following model based steps

i. Develop the rainfall-runoff relationship using any possible available hydrological models (HEC-HMS, SWAT, WEAP, HBV etc….) for the existing site for the specific duration, analyzing the available rainfall-runoff records of concurrent periods.

Note: hydrological model is used, continuous rainfall-runoff methods have to be selected.

ii. Develop the flow duration curve using the procedure described the section 5.4.2.

Case 3: *Use regionalized flow duration curve*

Regionalization of flow duration curve (FDC) is a tool for flow estimation at ungauged sites. Various researchers have developed different methods for regionalization of flow in the past. For the estimation of regional FDC, physiographic parameters play an important role. The flow at geologically similar ungauged sites can be predicted by parametric method using regression techniques. In the absence of these parameters, other methods like dimensionless flow and drainage area methods can be used. Although, there is an uncertainty associated with the such estimation methods, they are useful for areas where there is no sufficient stream flow and rainfall data for developing flow duration curves using gauged flow or hydrological modeled flow.

In this regionalized flow duration curve, one should have a tested mean annual rainfall, drainage area and stream length based relations for the development of Q70 and Q80 with different catchment area ranges.

Table 5-20: Regionalized FDC adopted from (Tsedey T. and Zelalem H. 2007)

Catchment Area range	Regional FDC relationships
Less than 3000km^2	Q_{70} = 0.001681MARF + 0.00238A -2.36144
	Q_{80} = 0.0012MARF + 0.00169A – 1.69041
	Q_{mean} = 76.56419Q_{70} + 28.42441
	Q_{mean} = 10.67323Q_{80} + 29.4947
3000 – 10000 km^2	Q_{70} = 0.00171MARF + 0.00251A -2.58325
	Q_{80} = 0.00122MARF + 0.00176A – 1.84521
	Q_{mean} = 60.9579Q_{70} + 46.82427
	Q_{mean} = 87.10567Q_{80} + 46.6525
MARF = Mean Annual Rainfall (mm); A = Catchment area of the basin (km^2)	

Table 5-21: Flow regime based FDC adopted from (Berhanu B. et.al, 2015)

Flow Regime	Linear Relationship	R-Square
Ephemeral streams	$Q10 = -3.7621 + 0.3614L - 0.0077\ A$	0.642
	$Q20 = -1.4761 + 0.1540\ L - 0.0042A$	0.533
	$Q33 = -0.3934 + 0.0500\ L - 0.0014\ A$	0.640
	$Q66 = -0.0988 + 0.0122\ L - 0.0003\ A$	0.624
	$Q80 = -0.0833 + 0.0084\ L - 0.0002\ A$	0.605
	$Q90 = -0.0465 + 0.0049\ L - 0.0002\ A$	0.570
Intermittent streams	$Q10 = 10.0137 - 0.0870\ L + 0.0138\ A$	0.913
	$Q20 = 3.0111 + 0.0294\ L + 0.0045\ A$	0.805
	$Q33 = 1.2064 + 0.0114\ L + 0.0017\ A$	0.738
	$Q66 = 0.5749 - 0.0061\ L + 0.0005\ A$	0.864
	$Q80 = 0.3164 - 0.0033\ L + 0.0002\ A$	0.836
	$Q90 = 0.1823 - 0.0007\ L + 0.0001\ A$	0.665
Perennial streams	$Q10 = 13.756 - 0.2223\ L + 0.0274\ A$	0.976
	$Q20 = 6.7423 + 0.0916\ L + 0.0111\ A$	0.915
	$Q33 = 5.5108 + 0.0552\ L + 0.0053A$	0.858
	$Q66 = 1.2939 - 0.0036L + 0.0018\ A$	0.959
	$Q80 = 0.6057 + 0.0062\ L + 0.0009\ A$	0.952
	$Q90 = 0.4870 - 0.0021L + 0.0006\ A$	0.959
L = Length of the longest river in the catchment (km); A = Catchment area of the basin (km^2)		

Note:
These regional flow duration computation methods should only be used when there is no any observed or transferrable stream flow data set In addition, these regional equations should be used to estimate the capacity of the stream as input for site identification before going to feasibility studies.

6 SPRING FLOW ANALYSIS

6.1 BASIC CONCEPT

Springs are scattered throughout hillsides, most of which are the products of perched water tables within the limestone. In order to sustain spring flow through the dry season, rain must infiltrate and percolate into the karst. Outflow of springs for one or more days after precipitation in a karst region can be assumed to occur from upstream aquifers along the underground flow path to the spring. This type of flow is known variously as base flow, drought flow, or low flow (Brutsaert W and Lopez JP, 1998).

6.2 SPRING DISCHARGES MEASUREMENT METHODS

1. **Bucket/stopwatch.** Try to concentrate the flow in one channel/drain and collect it in a bucket or container. The volume of the bucket divided by the filling time is the discharge. One might apply a correction factor if it is difficult to concentrate the entire flow. Repeat the measurement several times and calculate the average, excluding the extremes. Preferably, the 'constructions that were made for easier measurement' should remain, to facilitate later measurement and to improve the comparability.

2. **Velocity/area**. Try to find a downstream stretch with smooth flow and semi-constant cross section (or make one). Measure the time it takes a leave (or other floating material) to pass a certain distance at the heart of the flow. Distance divided by time gives velocity. The velocity at the surface is to be corrected to get the mean velocity in the central cross section. The correction factors for stream velocity for float method with different character of stream bed is given as, 0.85 for rectangular, smooth concrete channel; 0.75 for large and slow clear stream; 0.65 for small regular streams; 0.45 for shallow turbulent stream; 0.25 for very shallow rocky stream (Harvey, 1993). Repeat this at least 4 times and determine the average. Determine the average area of cross section, excluding areas with stagnant water (width time's average depth). Finally calculate the yield as *Yield = Area times Velocity.*

3. Spring outflow analysis without Measurement:

The procedure developed by Brutsaert and Nieber (1977) can be used for stream flow recession analysis, when one is not able to measure it. As declining groundwater reservoirs control both stream base flow recession and upland spring outflow recession, the method is equally valid for both situations. One advantage of this method is that it is independent of the ambiguity inherent in identifying when drought flow starts. This drought flow analysis is based on the Boussinesq equation (Boussinesq, 1903 & 1904), which describes flow in unconfined aquifers.

$$\frac{dQ}{dt} = -aQ^b \qquad (6.1)$$

Where "Q" is the recession flow [L^3/T], "t" is time, and "a" and "b" are constants. The coefficient "a" can be directly related to the groundwater reservoir characteristics; "b" is an exponent whose value depends on the time scale, The negative sign show the flow direction as decline (outflow). There are three theoretical solutions of the Boussinesq equation that have the general form of a power function (Brutsaert and Nieber, 1977):

1. Short-time flows generally have a higher Q than long time flows. Brutsaert and Lopez (1998) showed the following solution for short-time flow:

$$a = \frac{1.13}{kfD^3 L^2}, \qquad b = 3 \qquad (6.2)$$

Where K is the hydraulic conductivity [L/T], f is the drainable porosity, D is the aquifer thickness [L], and L is the total length of upstream channel intercepting groundwater flow [L].

2. One long-time flow

$$a = \frac{4.8K^{0.5}DL^2}{fA^{1.5}}, \qquad b = 1.5 \qquad (6.3)$$

Where A is the upland drainage area

3. Another long-time flow that is frequently used is the linear reservoir solution

$$a = \frac{0.35\pi^2 KDL^2}{fA^2}, \qquad b = 1 \qquad (6.4)$$

Table 6-1: Procedure with Example for spring flow analysis

	Procedure	Example (Bereda Lencha)	Remark
1	**Hydraulic conductivity (k)** accessed from different studies	0.0508	Ref. Berhanu et.al, (2013)
2	**Drainable Porosity (f)** accessed from geological investigation of the site and also textural based studies	0.04	
3	**Aquifer Thickness (D):**accessed from geological investigation of the site (m)	30	Check the thickness characteristics if there is full or perched clay pan
4	**Upstream channel Drainage Area (A):** compute using GIS as catchment delineation techniques (m^2)	210000000	
5	**Upland Recession flow Length (L):** compute it from catchment analysis (m)	47312	
6	**Recession flow (Q):** compute it using one of catchment runoff methods, example SCS-CN (m3/sec)	0.48	Section 5.2 stream flow analysis
7	**Boussinesq equation coefficients** $a = \frac{4.8K^{0.5}DL^2}{fA^{1.5}}, \quad b = 1.5$	a =0.597 b=1.500	
8	Yield (Low flow) (m3/sec) $\frac{dQ}{dt} = -aQ^b$	0.198	

7 DRAINAGE MODULE

7.1 BASIC CONCEPT

Drainage is the removal and disposal of excess water from a field. Drainage flow is the volume (or discharge) of water diverted or removed by collector and drainage network from a reclaimed area for a certain period of time. Drainage flow depends on soil permeability, slope of drainage area, depth of laying collector, drainage network, groundwater recharging conditions, specific rate of drainage flow (drainage module), and other factors. Drainage flow module is a quantitative characteristic of the groundwater flow from a unit of a drained land (ha). Drainage flow modulus (q_d) is measured in l/s per hectare. Drainage flow modulus is used to determine the design discharge of drains and collectors provided that optimal ameliorative regime is maintained. Determining the correct or a reasonable design flow for any drainage structure is critically important, both for the structure to perform properly and to prevent failures of structures. A reasonable design flow is commonly estimated based on a storm event of a certain recurrence frequency (return interval). Therefore, cross drainage facilities and field drainage network should be designed for recurrence interval of 25 year and 5 year respectively.

7.2 CROSS DRAINAGE

In an irrigation project, when the network of canals are provided, then these canals may have to cross natural drainage lines like rivers/ streams gullies, at different points within the irrigated area. So, suitable structures must be provided at the crossing points for proper flow in the canal and disposal in the drainage systems. These structures are known as cross-drainage works. The design of these structures needs reasonable estimation of peak flows from a catchment with the outlet at cross structure points.

Note: For the peak flow analysis for design of cross drainage structures, use the methods describe in Chapter 5 in this guideline.

7.3 FIELD DRAINAGE MODULE

Field drainage structures are designed to dispose excess water from a command area. In most cases, the common sources of the excess water is precipitation, over-irrigation and extra water needed for flushing away of salts from the root zone. The amount of water in the field drain can be estimated using different methods and set as drainage module.

The watershed characteristics particularly the storm characteristics have significant role in the determination of drainage coefficient. The drainage coefficient for the given irrigated field can be computed using a regression equation called the Cypress Creek equation (NRCS, 1998) that correlate the drainage module (Eq 7.1) with the 24hr maximum rainfall of 5 years return period.

$$q = 0.21 + 0.00744\,P_{24} \qquad (7.1)$$

Where q = drainage coefficient related to the drainage area and the magnitude of the storm (cubic meters per second per square kilometer)

P_{24}= the 24 hr. maximum rainfall (mm) with 5 years of return period preferred

Example

Suppose the 24hr Max rainfall of Shewu small scale irrigation site in Bench Majj Zone is 162.83mm for 5 years return period, compute the drainage module.

Therefore, the drainage module for this site is computed as

q= 0.21 +0.00744 (162.83) = 1.421m^3/sec/km^2 Or q=1.421m^3/sec/km^2 * 1000/100 = 14.l/sec/has

8 SEDIMENT ANALYSIS AND SEDIMENTATION

8.1 INTRODUCTION

Sedimentation is a global issue where land-use change has resulted in excess sediment being delivered to and deposited on the beds of streams, rivers, estuaries and reservoirs. Excess sediment directly affects the health of a waterway, decreasing life-supporting capacity of the reservoirs. Deposited fine sediment occurs naturally in the beds of rivers and streams. It usually enters a stream either because of terrestrial weathering processes, or bank erosion and in-stream fluvial processes. Sediment particles are transported and deposited in streams and receiving waters, such as lakes, estuaries and reservoirs, as the result of flowing water. Because sediment is naturally transported longitudinally through a river network, its state at any given point will be influenced by climate, geology, topography and current velocity. Human activities can impact on this natural sediment cycle by accelerating the delivery of sediment to streams and increasing the quantity of smaller particle sizes.

Although, Ethiopia has long experience on erosion quantification and soil and water conservation practices to minimize sedimentation, there are well tested techniques to quantify the magnitude of sediments in a given flow and its distribution in the bed of rivers and reservoirs to enable design different water hydraulic structures. Therefore, simplified methods of estimating (predicting) the magnitude of erosion and deposition are suggested with the improvement of computing input parameters more accurately using the GIS support. In addition, the simplified methods given in this guideline are proposed based on local experience, and their application for specific local situations have been checked through detailed investigation and adjusted through practices.

8.2 SEDIMENT YIELD

Sediment yield is the fraction of material eroded from a catchment and entering into a storage reservoir or a diversion pool. It is commonly expressed in units of weight, volume, or uniformly eroded depth of soils. Sediment yield decreases from arid to humid regions and it can be estimated by one of the following methods:

a) Measuring the sediment discharge of the streams at the inlet of reservoirs or pools,
b) Surveying of sediment deposits in reservoirs and measuring sediment discharge at the reservoir outlet,
c) Estimating wash load from catchment erosion and bed material load by sediment transport equations,
d) Adopting sediment yield of other places.

8.2.1 Measurement of sediment load in stream flow

Sediment yield can be obtained directly by measuring the sediment load of a stream at the inlet of a reservoir. The total sediment load of a stream is composed of wash load and bed material load and measured in respect of the volume of the total flow. Wash load consists of fine particles, and depends mainly upon supply from the source area. It moves entirely in suspension form. Bed load consists of fine as well as coarse or heavy particles eroded from the bed and banks of the stream channel or rolling materials from slopes. It is controlled by the transport capacity of the stream which depends upon the bed material composition and the relevant hydraulic parameters. Bed material load may move either as temporarily suspended load or as bed load. It is recommended to have sediment load measurement at least one season with 50-60 sample data depending on project duration.

a) Selection of sampling site:

The following general requirements shall be fulfilled in selecting a sampling site for measurement of sediment.

- The site should be located in a straight reach for over five channel widths) both upstream and downstream from the measuring location.
- It should be located in a stable section (no erosion or deposition) to take highest discharge within banks.
- It should have uniform bed slope
- The water flow should have sufficient depth with respect to the dimensions of the sampling equipment (minimum one meter).
- The site should be accessible and clear of natural and/or artificial obstacles (frees, bridge piers, etc.).
- It should have well defined geometrical dimensions (local depth, width and position).

b) Measurement of suspended sediment discharge

Standard methods of measurement require the entire cross-section to be subdivided into a number of vertical sections. Normally, the number of subdivisions is in the order of 5 to 7. Sediment discharge passing through each section is obtained by taking measurements along tile vertical within the portion of section it represents for measurement of stream flow as well as sediment load. However, for small scale irrigation systems small streams are usually developed to supply the required amount of water, and adoption of the standard methods of sediment measurement will not be technically and economically feasible.

Selection of measuring vertical: For streams of SSIS where the strength of flow at the measuring section is highly variable in the section, tile measuring section should be divided into a minimum of three subsections – one containing the main current and the two tile containing the secondary currents on either side. Measuring verticals for each subsection may be taken at the middle of the subsection where velocity and depth of flow are also recorded instantaneously. If the pattern of the flow is symmetrical about the middle of the channel, only measuring vertical for the main current and for one of tile secondary currents can be considered. For streams which have no significant difference in strength of the flow on the measuring cross-section, the transverse distribution of sediment is expected to be fairly uniform, and one measurement of vertical located at the middle of the section can be taken to be representative.

Sampling by selecting sampling points:-The commonly used standard methods consider a minimum number of three points on a vertical in a stream of ordinary size. Simplified methods are usually adopted for lessening tile work involved in taking and processing samples. The simplest of all the commonly used simplified methods is to take sample at one point at a relative depth (ratio of depth of sampling point to stream depth of 0.5 or 0.6). Time integrated sample should be taken at this point using a sampler that can be easily handled.

Sampling by depth integration: Sediment samples can be also collected by depth integration by round trips of lifting and lowering the sampler or only by a single trip from the surface to the bottom or from the bottom to the surface of the stream. The volume of sample at each point shall be proportional to the local velocity. This can be achieved if the intake velocity of the nozzle is approximately equal to the local velocity of the stream. The sample bottle shall not be allowed to fill completely.

c) Computation of sediment discharge

For point samples:

Sediment discharge per unit width: $q_s = C_{si}V_i d$

Where: C_{si} = the sediment concentration at the measuring point as determined *in* the laboratory; g/1 or kg/m^3;

V_i= the velocity at the measuring point recorded simultaneously as the sediment water sample is being taken; m/s; and

d = depth of flow; m.

Sediment discharge of the entire cross-section, $Q = \sum_{i=1}^{N} q_{si} b_j$

Where j = No of the sampling Vertical;

q_{si} = sediment discharge per unit width at the j sampling vertical

bj = channel width represented by J the vertical

For depth integrated samples:

Sediment discharge in the vertical, $q_s = C_{sq}q$

Where C_{sq} = average sediment concentration in the vertical; g/1 or kg/m;

q = discharge per unit width, m^3/s/m

Sediment discharge of the entire cross - section is computed by the same method used for point samples.

d) Bed Load Estimation

It is customarily assumed that the total sediment load consists of suspended and bed load. Furthermore, certain relationships show the existing bed load in relation to suspended load and, hence, for such small scale reservoirs 15% up to 20% is considered to account for the bed load as percentage of the suspended load

e) Grain Size Analysis of Sediment

Grain size distributions of the sediment samples should be determined for a better understanding of the sediment transport of various size groups. Standard procedures of sieve analysis for coarse grains and hydrometric analysis for fine grains of sediment have to be employed.

f) Frequency and Timing of Sampling

Timing and frequency of sampling depends on the runoff characteristics of the catchment. For many streams, an average of 70 to 90% of the annual sediment load is carried down the river during the flood season. During rainy seasons, sampling of sediment should be made at two hour intervals or at lesser intervals based on occurrence of flood to obtain a reliable estimate of sediment concentration. During the rest of the year, sampling frequency can be reduced to daily or even weekly sampling.

8.2.2 Estimation of sediment using reservoir capacity survey

Reservoir Capacity Survey is a direct measurement carried out periodically to assess the volume of deposit along with its location in the reservoirs. This method, however, indicates the total sediment yield of the entire catchment but not for sub-catchments for which sediment yield is required as sediment sampling of the different streams is necessary. The main survey methods are the contour method and the range line method. The contour method of survey is generally applicable for all types of reservoir shapes and this guideline recommends it to be used. In contour method, a contour map of the reservoir with suitable scale and contour interval is prepared, from which the capacity of the reservoir at the time of survey is computed. The difference in capacity between two surveys indicates the loss of capacity due to sediment deposition during the intervening period.

Computational procedure for capacity survey

The contour map of the reservoir is prepared and the successive areas enclosed by the contours are computed using any spatial analysis tools (ArcGIS, Global mapper, AutoCAD 3D, etc). Starting from the lowest contour, a submergence area-elevation curve is first prepared. From this curve the area between different elevations can be obtained. The capacity can then be computed using either Contour area interval method, or Modified prismoidal formula

Contour area interval method

The capacity between two successive elevations/contours can be obtained arithmetically by taking the average submergence area and multiplying by the unit height.

Modified prismoidal formula

The capacity between the successive unit elevations can also be estimated by the modified prisrnoidal formula:

$$V_s = \frac{H}{3}\left(A + 4B + C\right) - Vy \qquad (8.1)$$

Where V_s = capacity between middle and top contours;

H = contour interval between A and B;

A = area of the bottom contour;

B = area of the middle contour;

C = area of the top contour; and

V_y = capacity between middle and bottom contours previously determined.

The capacity below the lowest contour may be computed by the contour area interval method which provides Vy. After finding the volume below the lowest contour, this formula can be used progressively for each succeeding higher contour. The cumulative value of the capacity between the successive unit elevations will give the new capacity of reservoir at different elevations. The difference between the old capacity and the new capacity at any elevation will provide the accumulation of silt deposit between the surveys.

8.2.3 Estimation of wash load from catchment erosion

8.2.3.1 Estimation of Catchment Erosion

Soil erosion takes place mainly in cultivated lands, sparsely vegetated non-cultivated lands, grazing lands and road construction sites. The amount of eroded soil in a catchment can be fairly estimated by delineating the different sources of erosion and identifying their relative importance in contributing to the total erosion. Usually cultivated lands form the major source of sediment as cultivation is the main disturbing activity in a catchment. Soil erosion from the sources takes place as sheet-rill erosion, gully erosion and stream bank erosion. Erosion in cultivated lands takes place mainly as sheet-rill erosion and forms the major contribution to the soil eroded in a catchment. The contribution from other types of erosion is relatively low compared to that from sheet-rill erosion. The rate at which soil is eroded from a given area is expressed in volume or weight units per unit area and time. Usually, it is given as m^3/km^2/year or tons/ ha/ year.

I Sheet-Rill Erosion

Revised Universal Soil Loss Equation (RUSLE) computes average annual erosion from field slopes in tons/ha/year (Renard, 1997)

$$A = R*K*(LS)*C*P \qquad (8.2)$$

Where A = Average Annual Soil Loss (tons/ha/year)

R = Rainfall-Runoff Erosivity factor ($MJ*mm*ha^{-1}*hr^{-1}$) **K =** Soil Erodibility Factor

LS = Slope Length-Steepness Factor **C =** Land Cover-Management Factor

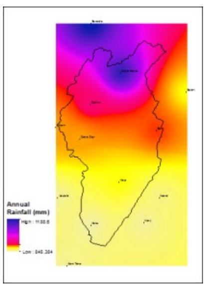
P = Conservation Practice

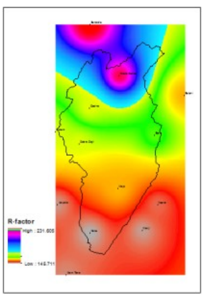
R factor: The R-Factor is the rainfall and runoff factor spatially distributed by geographic location, as the greater intensity and duration of the rainstorm has higher the erosion potential. There was a well-known empirical relationship between R-factor and annual precipitation proposed by Hurni (1978) but it was found to give overestimates the R-factor. Therefore, empirical equation developed by Kurt Cooper (2011) is evaluated and recommend adopting for Ethiopian condition (Berhanu B. et.al, 2015), which use the mean annual rainfall (P) in mm as input.

$$R = 0.01523P^{1.36}, \qquad (8.3)$$

Example with Mojo catchment

- Define the target watershed and available rainfall stations with mean annual rainfall within and at the surrounding of the catchment. Moji Watershed with 12 rainfall stations of mean annual data is used in this example.
- Interpolate areal rainfall using IDW/Co-Krkining or Thesisen Polygon. Here, IDW is used to interpolate the annual rainfall of the stations.
- Compute R-factor using the above formula





K-factor: The K-Factor is the average soil loss in tons per unit area for a particular soil in cultivated, continuous fallow with an arbitrarily selected slope length of 22.1 m and slope steepness of 9%. Soil-texture is the principal factor affecting K, but structure, organic matter and permeability also contribute. Therefore, Williams (1995) developed an equation for k-factor based on the different soil erodoblity factors of soil textures, which also is adopted by Berhanu etal. (2013) for the development of Ethiopian geo-soil database.

$$K_{USLE} = f_{csand} * f_{cl-si} * f_{orgc} * f_{hisand} \tag{8.4}$$

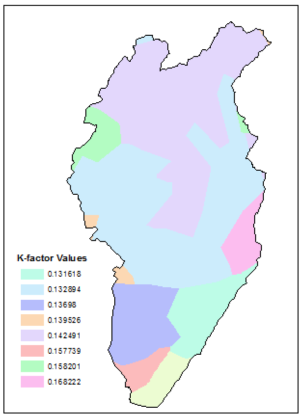
Where

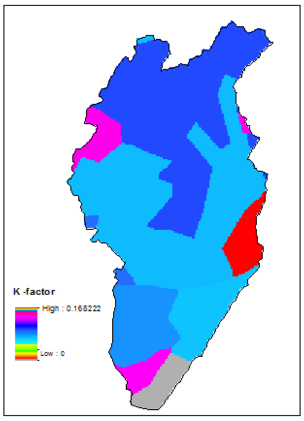
- f_{csand} is a factor that gives a low soil erodibility value for soils with high coarse-sand contents and high values with less sand,
- f_{ci-si} is a factor that gives a low soil erodibility value for soils with high clay to silt ratios,
- f_{orgc} is a factor that reduces the soil erodibility for soils with high organic carbon content,
- f_{hisand} is a factor that reduces the soil erodibility for soils with extremely high sand contents.

The computed k-factor for the whole country is available in the Etho-geosoil database (Berhanu, et.al, 2013)

Example with Mojo catchment

- Access the Eth-geoSoil data base and clip the data with the target catchment,
- Convert the data set to raster format





LS-Factor: The LS-Factor represents a ratio of soil loss under given conditions to that at a site with the "standard" slope steepness of 9% and slope length of 22.1 m.

The topographic calculations for the RUSLE are shown separately in the following Equations

$$L = \left(\frac{\lambda}{22.1}\right)^m$$

Where, L is the slope length factor,

λ is the horizontal plot length, and

m is a variable exponent calculated from the ratio of rill-to-inter rill erosion, as described in the following equations.

$$S = 10.8 \sin\theta + 0.03, \qquad \text{slopegradient} \leq 9\%$$
$$S = 16.8 \sin\theta - 0.50, \qquad \text{slope gradient} > 9\%$$

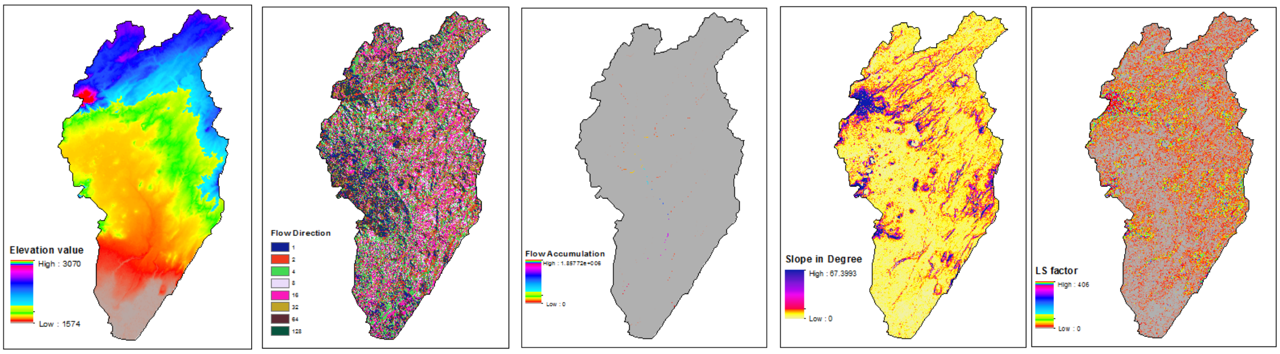
Where, S is the slope factor, and θ is the slope angle.

Depending on the measured slope gradient, a different equation for S must be used. Choosing S allows the RUSLE to be more finely tuned for different terrains. This is important because the topographic factor (and the RUSLE entirely) is very sensitive to the slope factor (S). The RUSLE method of calculating L and S terms are not directly applicable to the out-of-box functionality of ArcMap. However, there are programmatic methods for calculating the L and S factors from the empirical models in Equations.

Example with Mojo catchment

- Access the national DEM dataset and extract the DEM for the target catchment
- Fill the DEM to correct errors in the DEM sets
- Use the Fill DEM compute the flow direction and flow accumulation consecutively
- Compute the Slope in Degree
- Compute the LS factor copy the following equation in raster calculator

Power(flowacc*[cellresolution]/22.1,0.4)*Power(Sin(sloperasterdeg*0.01745)/0.09, 1.4)*1.4



C-factor: - The C-Factor is used to determine the relative effectiveness of soil and crop management systems in terms of preventing soil loss. It is a ratio comparing the soil loss from land under a specific crop and management system.

Important Note: The C factor resulting from this calculation is a generalized C factor value for a specific crop that does not account for crop rotations.

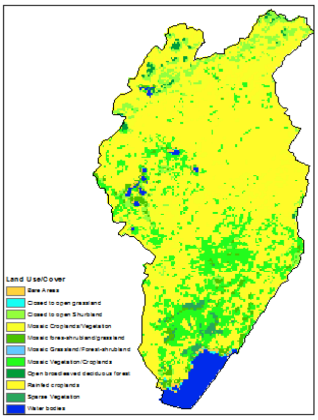
Table 8-1: C-factor per different crop type

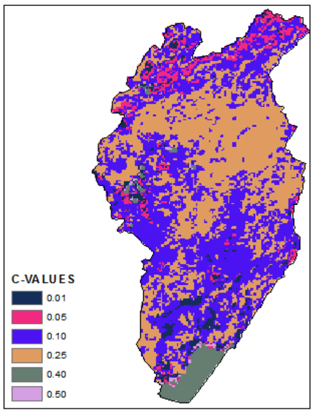
Cover type	C-value	Cover type	C-value
Bad land hard	0.050	Sorghum-maize	0.10
Bad land soft	0.400	Cereals, pulses	0.15
Dense grass	0.010	*Teff*	0.25
Degraded grass	0.050	Fallow hard	0.05
Dense forest	0.001	Fallow ploughed	0.60
Other forest (with modest ground cover)	0.010	Continuous fallow (without cover)	1.00

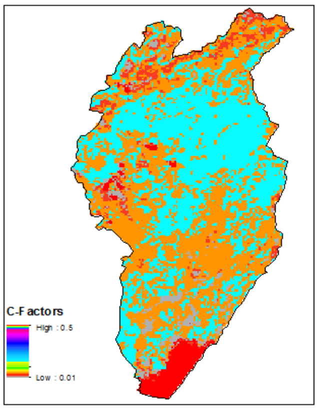
Source Hurni (1988)

Example with Mojo catchment

- Access the land use dataset from recognized data sources
- Assign the crop factor value according to the table using conditional function in GIS
- convert it to raster with C-factor value







P-Factor: - The P-Factor is known as the support practice factor. It reflects the effects of practices that will reduce the amount and rate of the water runoff and thus reduce the amount of erosion. The P factor represents the ratio of soil loss by a support practice to that of straight-row farming up and down the slope.

$$P = Pc \times Ps \times Pt$$

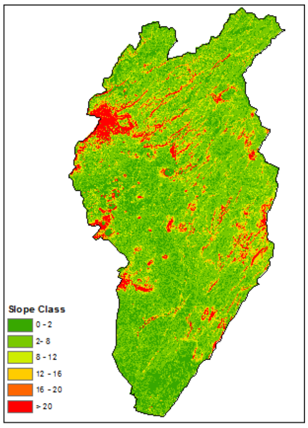
Table 8-2: P-factor with different land management options

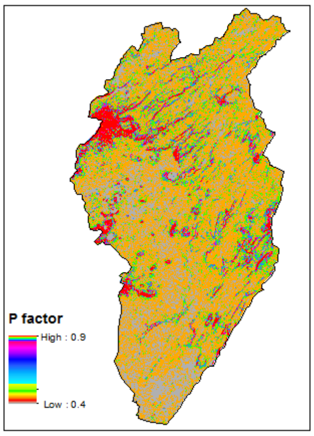
Land slope %	Contour factor	Strip crop factor	Terrace factor	
			Closed outlet	Open outlet
1-2	0.4	0.2	0.5	0.7
3-8	0.5	0.25	0.6	0.8
9-12	0.6	0.3	0.7	0.8
13-16	0.7	0.35	0.8	0.9
17-20	0.8	0.4	0.9	0.9
21-25	0.9	0.45	1.0	1.0

Sources: Wischmeier and Smith (1978) and USDA-ARS (1997)

Example with Mojo catchment

- Add DEM and compute slope in Percent
- Reclassify the slope according to the p-factor table
- Select the treatment type and use the conditional equation for the selected treatment
- Here use contour factor (Pc)





Finally multiply all the factor raster in raster calculator to get the annual soil loss rate with ton/ha/yr.

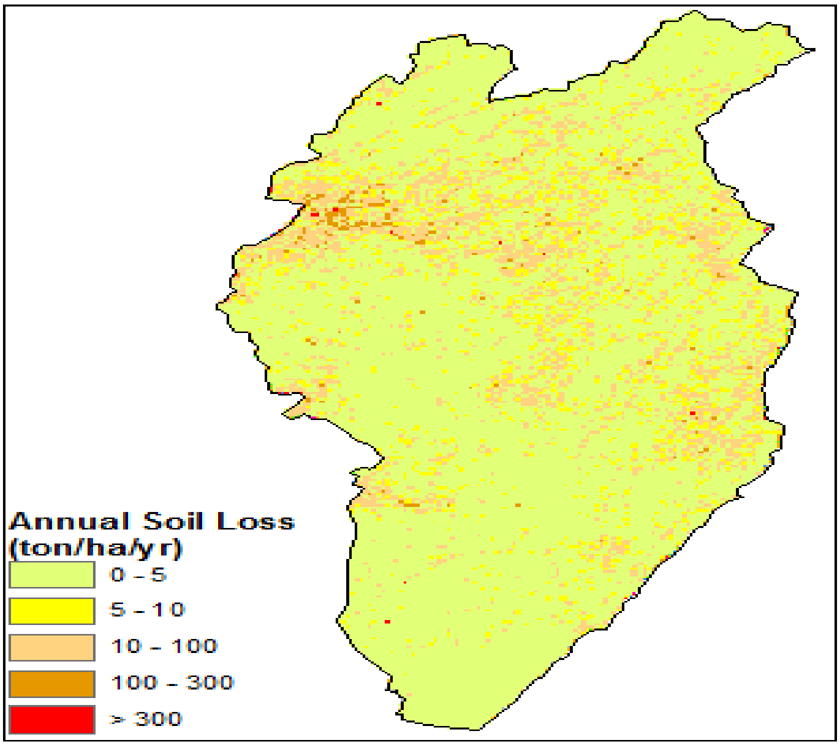


Figure 8-1: Annual soil loss for Mojo catchment

II. Gully Erosion

Gully erosion is usually less than sheet-rill erosion. The percentage of gully erosion to sheet-rill erosion (or total erosion) is a useful recommendation leading to the estimation of total erosion. The progress of gully erosion can be estimated from time sequence comparisons of surveyed cross-section. When the development of gully erosion is being studied, measurements of the horizontal spread of the gully and the vertical changes are needed. One method is to put in a line of pegs at a fixed distance from the gully; another is to lay out a rectangular grid of pegs. In either case measurements from the datum are repeated at regular intervals to establish the rate at which the edge of the gully is moving. For this type of survey a photographic record is also useful, and quantitative estimates can be made provided the photographs are taken accurately. Markers should be established so that subsequent photographs can be taken from precisely the same point and direction, and the photographs must include some means of measuring the scale. The percentage of gully erosion to sheet erosion can be estimated by a wise judgment by using the observed gully expansion rate and the proportion of gullied area to the total catchment area.

Channel Erosion

Erosion of stream bed and banks are obtained from time sequence comparisons of surveyed cross sections from maps and aerial photographs and from historical records. If data is not available consider 10 -20% of the wash load

8.2.3.2 Sediment delivery

All the eroded material is not delivered to the reservoirs. The percentage of sediment delivered from the erosion source to a stream is affected by size and texture of erodible material, climate, land use, local environment and general physiographic position. The sediment entering a reservoir from a catchment is expressed by sediment delivery ratio (SDR), where

SDR = SY/A; Where SY = the sediment yield at the measuring point (inlet into reservoir); tons/ha/yr, and A = the total erosion (soil loss) from the catchment; tons /ha/yr.

The sediment delivery ratio value in a given watershed indicates the integrated capability of a catchment for storing and transporting the eroded soil. It compensates for areas of sediment deposition that become increasingly important with increasing catchment area and therefore, determines the relative significance of sediment sources and their delivery (Lu H, et.al 2003). It is affected by many highly variable physical characteristics of a watershed such as drainage area, slope, relief-length ratio, runoff-rainfall factors, land use land cover and sediment particle size (Benedict MM, and Andreas K, 2006). Numerous Sediment Delivery Ratio (SDR) relationships have been developed based on combinations of the variable physical characteristics of a watershed. There two sediment delivery ratio (SDR) relations based on the drainage area and main channel slope are suggested to use. Although both were developed outside of Ethiopia, both have adaptation and customization with the Ethiopia case with different studies.

$$SDR = 0.37\,A^{-0.125}$$
$$SDR = 0.627\,S^{0.403}$$

Where, Where, SDR = sediment delivery ratio,

A = catchment area in Km^2 and

S = main channel slope in m/m

Example

Based on Drainage Area

Drainage area of the mojo catchment in the above example = $2075km^2$

Therefore the sediment deliver ratio (SED) is simple computed as

$= 0.37A^{-0.125} = 0.37*(2075)^{-0.125} = 0.014$

Average annual soil loss rate of the catchment (A) = 6.78ton/ha/yr

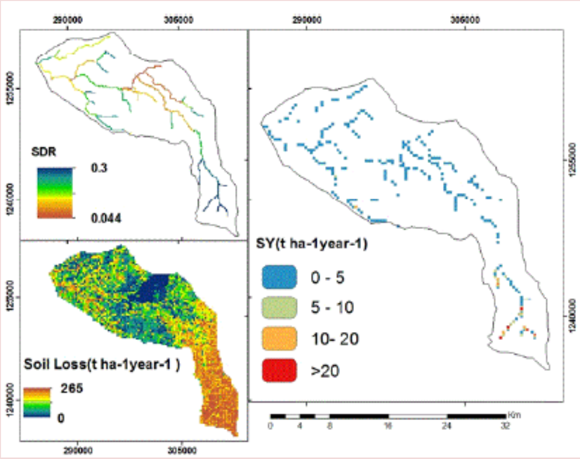
SY= SEDSRD*A = 6.78*0.014 = 0.095ton/ha/yr

Mean annual sediment inter in koka reservoir from mojo catchment (SY ton/yr)

= 0.095*2075*100

= 19712.5 ton/yr inter to the reservoir

Based on main Channel Slope (adopted from Gelagay HS (2016))



8.2.4 Amount and distribution of sediment deposit in reservoirs

8.2.4.1 Amount of sediment deposit in reservoirs

The capacity of a reservoir to capture some part or all of the sediment entering the pool is known as trap efficiency; i.e,

$$TrapEfficency = \frac{Sediment Depostad}{Sediment Inflow}$$

The Brune (1953) curve relates the long term average trap efficiency of storage reservoirs with the ratio of reservoir capacity to average annual inflow. Through long time research Jothiprakash and Garg (2008) modified the Brune curves that it considers the age of the reservoirs. Here also suggested to use this relationship to determine the sediment deposit in reservoirs.

$$T_{eT} = \frac{\left(\dfrac{C_T}{I_T}\right)}{\left[0.00025 + 0.01\left(\dfrac{C_T}{I_T}\right) - 0.0000045\, a_T \sqrt{\dfrac{C_T}{I_T}}\right]}$$

Where T_{eT} is the trap efficiency of the reservoir at time 'T', C_T, is the capacity of the reservoir at time 'T', I_T, is the inflow during the time 'T' and a_T, is the age of the reservoir in years at 'T' from its inception.

Once the trap efficiency computed based on the above relation the deposited sediment at the year ‘T’ is simple computed as

Sediment deposited = Trap efficiency X sediment inflow

8.2.4.2 Distribution of sediment deposit in reservoirs

For small reservoirs, all the trapped sediment should be assumed to be deposited in the deep area (dead storage area) of the reservoir, i.e., starting from the deepest section the sediment gradually accumulates in the reservoir encroaching the dead storage area. For small reservoir, it is reasonable to consider a life span of 20 to 25 years. The volume of sediment deposited *in* the reservoir, in the manner recommended above during the life of the reservoir, gives dead storage capacity. However, a maximum limit of dead storage capacity of 20% of the total reservoir capacity should be adopted.

9 RESERVOIR ROUTING

Water storage reservoirs may be created by constructing a dam across a river, along with suitable appurtenant structures. Routing of reservoir is essential to study the relation between flood discharge, reservoirs capacity and spillway size in order to design, construct and optimally operated reservoirs. Fundamentally, a reservoir serves to store water and the size of the reservoir is governed by the volume of the water that must be stored, which in turn is affected by the variability of the inflow available for the reservoir.

Flow routing is a mathematical procedure for predicting the changing magnitude, speed, and shape of a flood wave as a function of time at one or more points along a watercourse (waterway or channel). The routing of flood through the reservoir and the spillway is done by solving the continuity of flow within reservoir, which may simply be stated as:

Inflow to reservoir - Outflow to reservoir = increase in the storage of the reservoir. That is,

$$(I - O) * \Delta t = \Delta S \qquad (9.1)$$

Where, I is the inflow discharge (m3/s),

O is the Outflow discharge (m3/s),

Δs is the in storage volume (m3)

Δt (h).time interval

If consider different inflow, outflow and storage volume at the beginning and end of the time interval, it is possible to rewrite the continuity equation as:

$$\Delta t\left(\frac{I_1 + I_2}{2}\right) - \Delta t\left(\frac{O_1 + O_2}{2}\right) = S_2 - S_1 \qquad (9.2)$$

Suppose, the following values are known:

I_1 = Inflow (m^3/s) at the beginning of the time interval

O_1= Outflow (m^3/s) at the beginning of the time interval

S1= Total storage volume of the reservoir (m^3) at the beginning of the time interval

I_2 = Inflow (m^3/s) At the end of the time interval

And the unknown values are

S_2= Total storage volume of the reservoir (m3/s) at the end of the interval

O_2= Outflow (m3/s) at the end of the time interval

Then, the continuity equation rewrite as the known items in one side and unknown in the other side as

$$\Delta t\left(\frac{I_1 + I_2}{2}\right) + \left(S_1 - \frac{\Delta t Q_1}{2}\right) = S_2 - \frac{\Delta t Q_2}{2} \qquad (9.3)$$

This equation use to route the reservoir as modified-Pul's routing methods. The procedure describe with the example below

Example of Reservoir Routing using the case of Arjo-Dedessa Reservoir

Step 1: organize the elevation-capacity-discharge relation, which is the major input for reservoir routing. Table 9.1 present it for the case of Arjo-Dedessa reservoir

Table 9-1: Elevation-Storage-discharge relation for Arjo-Dedessa reservoir

Elevation (m)	1313	1314	1315	1316	1317	1318	1319	1320
S (mcm)	0.00	0.26	0.58	2.65	6.57	10.72	16.64	24.74
Q(m3/sec)	0.00	34.12	59.09	76.29	90.27	102.35	113.16	123.01
Elevation (m)	1321	1322	1323	1324	1325	1326	1327	1328
S (mcm)	34.20	45.83	59.57	74.91	91.78	110.27	132.36	158.23
Q(m3/sec)	132.14	140.67	148.72	156.35	163.63	170.59	177.28	183.73
Elevation (m)	1329	1330	1331	1332	1333	1334	1335	1336
S (mcm)	186.07	215.85	247.84	283.45	322.92	365.83	411.98	460.72
Q(m3/sec)	189.96	195.99	201.85	207.53	213.07	218.46	223.73	228.87

Step 2: select the time interval for routing it has to depend on the inflow data interval and to be a multiple of the time for maximum inflow; for the case of Arjo-Dedessa select 5hr

Step 3: prepare the working table using the elevation-storage-discharge relation table and the selected time interval by computing S+QΔt/2; for Arjo-Dedessa presented in Table 9.2

Table 9-2: Working table for Arjo-Dedessa reservoir

Elevation (m)	1313	1314	1315	1316	1317	1318	1319	1320
S (mcm)	0	0.26	0.575	2.653	6.574	10.72	16.64	24.74
Q(m3/sec)	0	34.12	59.09	76.29	90.27	102.4	113.2	123
S+QΔt/2	0	0.523	1.03	3.24	7.268	11.5	17.51	25.69
Elevation (m)	1321	1322	1323	1324	1325	1326	1327	1328
S (mcm)	34.2	45.83	59.57	74.91	91.78	110.3	132.4	158.2
Q(m3/sec)	132.1	140.7	148.7	156.3	163.6	170.6	177.3	183.7
S+QΔt/2	35.21	46.91	60.71	76.11	93.04	111.6	133.7	159.6
Elevation (m)	1329	1330	1331	1332	1333	1334	1335	1336
S (mcm)	186.1	215.9	247.8	283.4	322.9	365.8	412	460.7
Q(m3/sec)	190	196	201.8	207.5	213.1	218.5	223.7	228.9
S+QΔt/2	187.5	217.4	249.4	285	324.6	367.5	413.7	462.5

Step 4: Develop routing table and compute the unknown values (S_2 and Q_2) using the inflow, the working table as presented in Table 9.3.

Table 9-3: Routing table of Arjo-Dedessa Reservoir

	Hr.	Inflow (m^3/sec)	av. I (m^3/sec)	av.IΔt (MCM)	S-QΔt/2 (MCM)	S+QΔt/2 (MCM)	Elev. (m)	S (MCM)	Q (m^3/sec)
1	0	150					1313	0.00	0.00
2	5	283	216.5	3.897	0.00	3.90	1316.16	3.29	78.57
3	10	440	361.5	6.507	2.59	9.09	1317.43	8.36	95.48
4	15	550	495.0	8.910	7.50	16.41	1318.82	15.55	111.18
5	20	675	612.5	11.025	14.55	25.58	1319.99	24.63	122.88
6	25	800	737.5	13.275	23.53	36.80	1321.14	35.78	133.30
7	30	835	817.5	14.715	34.58	49.29	1322.17	48.20	142.06
8	35	871	853.0	15.354	46.92	62.28	1323.10	61.13	149.49
9	40	800	835.5	15.039	59.78	74.82	1323.92	73.62	155.71
10	45	720	760.0	13.680	72.22	85.90	1324.58	84.67	160.56
11	50	630	675.0	12.150	83.22	95.37	1325.13	94.11	164.50

	Hr.	Inflow (m^3/sec)	av. I (m^3/sec)	av.IΔt (MCM)	S-QΔt/2 (MCM)	S+QΔt/2 (MCM)	Elev. (m)	S (MCM)	Q (m^3/sec)
12	55	580	605.0	10.890	92.63	103.52	1325.56	102.23	167.56
13	60	530	555.0	9.990	100.72	110.71	1325.95	109.40	170.26
14	65	480	505.0	9.090	107.87	116.96	1326.24	115.63	172.22
15	70	440	460.0	8.280	114.08	122.36	1326.49	121.03	173.85
16	75	400	420.0	7.560	119.46	127.02	1326.70	125.67	175.26
17	80	380	390.0	7.020	124.10	131.12	1326.88	129.76	176.50
18	85	360	370.0	6.660	128.17	134.83	1327.04	133.46	177.56
19	90	340	350.0	6.300	131.87	138.17	1327.17	136.79	178.39
20	95	320	330.0	5.940	135.19	141.13	1327.29	139.75	179.13
21	100	300	310.0	5.580	138.14	143.72	1327.39	142.34	179.77
22	105	294	297.0	5.346	140.72	146.07	1327.48	144.68	180.35
23	110	288	291.0	5.238	143.06	148.29	1327.56	146.90	180.91
24	115	282	285.0	5.130	145.27	150.40	1327.64	149.01	181.43
25	120	274	278.0	5.004	147.38	152.38	1327.72	150.98	181.92
26	125	266	270.0	4.860	149.34	154.20	1327.79	152.80	182.38
27	130	260	263.0	4.734	151.16	155.89	1327.86	154.49	182.80
28	135	254	257.0	4.626	152.84	157.47	1327.92	156.06	183.19
29	140	247	250.5	4.509	154.41	158.92	1327.97	157.51	183.55
30	145	245	246.0	4.428	155.86	160.28	1328.02	158.87	183.87
31	150	240	242.5	4.365	157.22	161.58	1328.07	160.16	184.16
32	155	237	238.5	4.293	158.51	162.80	1328.11	161.38	184.44
33	160	236	236.5	4.257	159.72	163.98	1328.16	162.56	184.70
34	165	234	235.0	4.230	160.90	165.13	1328.20	163.70	184.96
35	170	232	233.0	4.194	162.04	166.23	1328.24	164.81	185.20
36	175	230	231.0	4.158	163.14	167.30	1328.27	165.87	185.44
37	180	228	229.0	4.122	164.20	168.33	1328.31	166.90	185.67
38	185	226	227.0	4.086	165.23	169.31	1328.35	167.88	185.89
39	190	224	225.0	4.050	166.21	170.26	1328.38	168.83	186.10
40	195	222	223.0	4.014	167.15	171.17	1328.41	169.74	186.31
41	200	220	221.0	3.978	168.06	172.04	1328.44	170.60	186.50
42	205	214	217.0	3.906	168.92	172.83	1328.47	171.39	186.68
43	210	208	211.0	3.798	169.71	173.51	1328.50	172.08	186.83
44	215	206	207.0	3.726	170.39	174.12	1328.52	172.68	186.97
45	220	196	201.0	3.618	171.00	174.62	1328.54	173.18	187.08
46	225	190	193.0	3.474	171.50	174.97	1328.55	173.53	187.16
47	230	184	187.0	3.366	171.85	175.21	1328.56	173.77	187.21
48	235	180	182.0	3.276	172.09	175.36	1328.56	173.92	187.24
49	240	172	176.0	3.168	172.24	175.41	1328.57	173.97	187.25
50	245	166	169.0	3.042	172.28	175.32	1328.56	173.88	187.23
51	250	160	163.0	2.934	172.20	175.13	1328.56	173.69	187.19
52	255	152	156.0	2.808	172.01	174.82	1328.54	173.38	187.12
53	260	145	148.5	2.673	171.69	174.37	1328.53	172.93	187.02
54	265	142	143.5	2.583	171.24	173.83	1328.51	172.39	186.90
55	270	140	141.0	2.538	170.71	173.25	1328.49	171.81	186.77
56	275	130	135.0	2.430	170.13	172.56	1328.46	171.12	186.62
57	280	122	126.0	2.268	169.44	171.71	1328.43	170.28	186.43
58	285	114	118.0	2.124	168.60	170.72	1328.40	169.29	186.21
59	290	108	111.0	1.998	167.62	169.61	1328.36	168.18	185.96
60	295	100	104.0	1.872	166.51	168.38	1328.31	166.95	185.68

10 WATER BALANCE AND UPSTREAM-DOWNSTREAM RELATIONSHIPS

10.1 GENERAL

With regard to hydrology, as surface waters flow from headwaters (upstream) to lower-elevation floodplains (downstream), it creates linkages between upstream and downstream areas. The linkages are relative to the physical hydrologic system in nature. Understanding of this upstream-downstream linkage in hydrological processes is essential for water resources planning in river basins. The activities in upstream areas have both beneficial and adverse effects on downstream communities. Good watershed management practices provide better opportunities to downstream communities, for example, a clean and sustainable water supply for irrigation. However, bad watershed management practices do not only degrade upstream environmental conditions, but also adversely affect the opportunities in downstream areas. Hence, as the 'opportunities' and 'threats' flow from upstream to downstream areas, the users in downstream areas often have great 'concerns' about upstream land-use and water management practices (Santosh, 2012).

A point in a mountain (e.g. irrigation land) is considered as downstream from the upstream catchment area which provides water to the irrigation land. However, both catchment and irrigation land could be upstream relative to the area below which receives water after irrigation. Therefore, upstream and downstream relationships can be seen throughout a river basin at different scales. As a common tool, these upstream –downstream relationships can be evaluated using water balance equations at catchment and scheme levels. The water balance of a given irrigated area is special tools that confirm the upstream-downstream relationship of the irrigation sites, which is based on the continuity, momentum, and energy equations for various hydrologic processes (Chow et al. 1988).

Equation of continuity

The integral equation of continuity follows from Reynolds theorem and is the basis for the water balance equation. By the law of conservation of mass, *dm/dt=0* because mass cannot be created or destroyed, which is the basis for the water budget concept, widely used in the field of hydrology and rewritten as

$$\frac{ds}{dt} + O(t) - I(t) = 0 \qquad (10.1)$$

$$I(t) - O(t) = \frac{\Delta S}{\Delta t} \qquad (10.2)$$

Where I = the inflow in [L^3/T],

O = the outflow in [L^3/T], and

ΔS/Δt = the rate of change in storage over a finite time step in [L^3/T] of the considered control volume in the system.

The equation holds true for a specific period of time and may be applied to any given system provided that the boundaries are well defined.

10.2 WATER BALANCE AT CATCHMENT SCALE

The water balance in this scale is often applied to a diversion/storage work. The input equals the precipitation *P* while the output comprises the evapotranspiration *ET* and the river discharge *Q* at the outlet of the catchment. Hence, the water balance may be written as

$$P - ET - Q = 0$$

Where *ΔS* is the change of storage over the time step *Δt.*

The water balance in this scale is computed in annual time step considering the change in storage of the catchment to be zero. Therefore, the water balance equation is articulated as.

$$P - ET - Q = \frac{\Delta S}{\Delta t} \qquad (9.3)$$

Example
The mean annual rainfall and evapotranspiration for Shew SSI catchment is computed as 1,739mm and 1,337mm respectively. Therefore, the surface flow in the catchment is computed as (Q= P-ET; Q= 1739 – 1337= 402mm). If this computed annual surface flow is multiplied by the catchment area (2147ha), it gives 8.631MCM runoff volume or 0.274m3/sec average flow rate which is much greater than the annual designed diverted water at weir site (0.61MCM). Therefore, the catchment can provide sufficient water for the proposed irrigated command area and downstream uses in the stream condition.

10.3 WATER BALANCE AT SCHEME LEVEL

Similarly, the water balance analysis can be done for the irrigation scheme to confirm the availability of water for the command area and release for downstream users with 70% dependable monthly flow in the stream system. The diverted water compares with the gross irrigation requirement as allocated water flow. The return flow will account for the downstream users. If diverted water covers the gross demand and deliver significant return flow for the downstream users, the irrigation system is accepted without negative impact in the downstream.

Describe the terms

- **Available water:** the flow water in the sources (Stream or spring or others sources) in other words it is the dependable flow of the stream.
- **Diverted Water:** the water captured with the structure that diverted to the main canal.
- **Actual Consumed:** the water demand of the target crop type which computed as CROPWAT computation as net Irrigation requirements
- **Return flow:** the part of the diverted water that return back to the system through drainage system of the command area ***(i.e, it simply computed as subtraction of actual consumed from diverted water)***
- **Downstream flow:** It is the part of the stream flow which will available for different uses downstream of the target scheme. ***(i.e., it computed as Available Water – Diverted Water + Return flow)***

Table 10-1: Example for Scheme water balance (lt/sec)

	Available water (Dependable flow)	Diverted water	Actual Consumed	Return flow	Downstream flow
Jan	100	54.2	51	3.2	49
Feb	70	60	60	0	10
Mar	90	40.6	31	9.6	59
Apr	210	13.5	7	6.5	203
May	370	0	0	0	370
Jun	920	0	0	0	920
Jul	1960	1.7	0	1.7	1960
Aug	1810	0	0	0	1810
Sep	1660	0	0	0	1660
Oct	730	3.4	2	1.4	728
Nov	310	13.5	7	6.5	303
Dec	180	45.7	24	21.7	156

10.4 ENVIRONMENTAL FLOW REQUIREMENT

10.4.1 Environmental flow assessment

Environmental flow Assessment (EFA) is used to identify the reliance of different ecosystems on the different components of flows and their sensitivity to changes in these components. EFA provides the scientific basis for understanding the relationship between the different flow components and ecosystem responses. But deciding on how much and at what time(s) water should be allocated to the environment at either the river basin or project level is a decision that can only be taken in the context of all the demands on the water resource. There is no absolute quantity and timing of flows that are required for the environment or for that matter for any other use. Instead, a social choice has to be made about what uses are important, to what degree they need to be addressed, and which ecosystem services need to be preserved (and to what degree) to meet society's objectives for a particular water resource. This choice will then determine the flows that are needed to deliver those services. For example, society may decide to increase irrigated agriculture using a particular groundwater resource—at the expense of some groundwater-dependent wetlands that rely on high water tables—because the net societal benefits are greater when irrigated agriculture is increased and wetlands are decreased.

These choices have always been made in water resources planning and management. The contribution of environmental flows is that the EFA makes explicit the consequences of different choices on aquatic ecosystems and communities that depend on those ecosystem services and so leads to a more informed decision-making process. It enhances equity and sustainability in the decision-making process

10.4.2 Estimation of environmental water use

Environmental water uses may include water stored in impoundments and released for environmental purposes (held environmental water). Environmental water usage includes watering of natural or artificial wetlands, artificial lakes intended to create wildlife habitat, fish ladders, and water releases from reservoirs timed to help fish spawn, or to restore more natural flow regimes. Environmental water requirements of an aquatic ecosystem are defined as the quality and quantity of water for protection of the structure and functioning of an ecosystem and its dependent species in order to ensure ecologically sustainable development and utilization of water resources

Total annual environmental water requirement of an aquatic ecosystem in this pilot assessment was made of two components: environmental low flow requirement (LFR) and environmental high flow requirement (HFR). The LFR indicates the basic requirements of aquatic life throughout the year. The HFR is important for river channel maintenance, wetland flooding, fish and other aquatic species, and riparian vegetation to flow variability. The existing experience in this method for environmental flow estimation is quantify with the sum of HFR and LFR, which accounted as 20% of the mean annual flow (MAF) of the targeted river (Hughes & Münster 2000).

Example

The mean annual flow for Shew SSI catchment is computed as 0.274m3/sec or 8.63MCM as presented above; therefore, the Environmental requirement of this project computed as 20% of this average flow, which is 0.055m3/sec or 1.73MCM if we add the diverted flow requirement at weir site (0.61MCM), the total water requirement become 2.34MCM, which is fully covered by the average flow of the river and it has a relief for downstream use also.

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