MODELLING URBAN STORM WATER FLOODING IN HOSSANA TOWN: THE CASE OF ADDIS SUBCITY

MSc THESIS

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HAWASSA, ETHIOPIA

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MODELLING URBAN STORM WATER FLOODING IN HOSSANA TOWN: THE CASE OF ADDIS SUBCITY

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A THESIS SUBMITTED TO DEPARTMENT OF WATER RESOURCEAND IRRIGATION ENGINEERING, FACULTY OF BIO-SYSTEMS AND WATER RESOURCES ENGINEERING, INSTITUTE OF TECHNOLOGY, SCHOOL OF GRADUATE STUDY, HAWASSA UNIVERSITY, HAWASSA, ETHIOPIA

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS MASTER OF DEGREE IN IRRIGATION AND DRAINAGE ENGINEERING

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ADVISORS' APPROVAL SHEET

As thesis research advisor, this is to certify that the thesis entitled "Modeling Urban Storm Water Flooding in Hossana Town: The case of Addis sub-city" submitted in partial fulfillment of the requirement for the degree of master of science in irrigation and drainage engineering department and it is a record of original research carried out by Abera Mecha Sewore.

I, therefore, recommend that it is acceptable as fulfilling the thesis requirements.

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EXAMINERS' APPROVAL SHEET

We, the undersigned, members of the Board of Examiners of the final open defense by Abera Mecha Sewore have read and evaluated his thesis entitled "**MODELLING URBAN STORM WATER FLOODING IN HOSSANA TOWN: THE CASE OF ADDIS SUBCITY**", and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for Master of degree in irrigation and drainage Engineering.

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STATEMENT OF THE AUTHOR

I declare that the work presented here is my own original work and all contributions have been acknowledged properly. In addition, this thesis has not been previously submitted and presented to any other for a degree or diploma award.

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ABSTRACT

Urban flooding is a condition where storm water cannot enter a storm drain or it is beyond the capacity of the urban drainage system. Such urban flooding is mainly caused by inefficient urban drainage systems, and it cause damage to public and private buildings and disrupts public life. Therefore, modeling urban floods were very essential to mitigate flooding related problems. The main objective of this research was modeling urban storm water flooding in Hossana town specifically in Addis sub-city. There were many tools for urban storm flood modeling, but out of limited free tools, Storm water management model (SWMM) has a good record of applicability in many locations elsewhere. Various rainfall and spatial data were collected to model urban floods. Daily maximum rainfall data from Hossana station for 28 years (1990-2017) was used to identify parent probability distribution and enabled to develop the Intensity duration frequency curve (IDF) for study area. SWMM was implemented to simulate the flood depth in the drainage channels and Arc GIS is used to delineate the extent of the flood contributing area. The hydrology and hydraulic routing of the watershed was estimated using the SWMM at the total of 1-hour time of concentration daily time series data of different year return periods. The inputs of the models (nodes, conduits and sub- catchment area property) were assigned with the help of Auto-CAD, Arc-GIS 10, Google earth pro, and field survey. The result obtained from the model showed that (39%), (41%) drainage manholes encountered flooding for 10 and 25 years return periods respectively. Hence, approximately (40%) of the drainage system of the city previously designed was inadequate to discharge the system runoff to final point. Discharge of the catchment was also determined by using Soil conservation service (SCS) curve method and rational method that was used to relate and check the SWMM model output value. In general, the results of this study will be useful for further design and planning of the sub cities in the Hossana town. Moreover, further research on urban flooding which accounts two-dimensional flow is recommended to account the flood extent and risks associated with it.

Key words*:-urban flooding, sub-catchments, SWMM, ArcGIS, Storm Drainage system, IDF curve*

AVERVIATIONS

ASC	Arada sub-catchment
CCSC	City center sub-catchment
DEM	Digital Elevation Model service
DHI	Danish Hydrology Institute
DTM	Digital Terrain Model
EPASWMM	Environmental Protection Agency Storm Water Management Model
ERA	Ethiopian Road Authority
FAA	Federal Aviation Administration
FUPCoB	Federal Urban Planning Coordinating Bureau
GIS	Geographical Information System
GPS	Geographical positioning system
HEC	Hydrological Engineering Center
IDF	Intensity-Duration-Frequency
NMA	National meteorological agency
LSC	Lucid sub-catchment
SCS	Soil Conservation Service
UDFCD	Urban drainage and flood control district
US NRCS	United States Natural Resources Conservation Service
US SCS	United States Soil Conservation Service

1. INTRODUCTION

1.1 Background

Floods are natural events that have been integral part of human history which occurs along the rivers, streams, lakes, agricultural lands, plain area, towns and rural areas and an area covered with trees and garland where the stormwater capacity is beyond the soil infiltration capacity (Linmei, 2004; Lei *et al*., 2014). However, flooding in the urban area is a condition where stormwater or wastewater cannot enter a storm drain or beyond the capacity of the urban storm drainage system (Thuy, 2009). Furthermore, flooding in the urban area mostly occurs with undersized drainage channels provision and lack appropriate management of existing drainage system (Mamak *et al*., 2013). According to Justin, (2016) flooding in urban areas in most cities of developing countries of the world was common to complain about issues raised from urban society in every heavy rainfall season. It is difficult to solve the flooding problem of the urban catchment, because of an urban watershed immediate change characteristics unless future climatic and rainfall variation was considered in the design of the drainage and stormwater removal system.

Flooding in urban areas may cause public and private property damage and even human death. Furthermore, street flooding may interfere with traffic and in some cases, may danger to pedestrian safety. The increase of urbanization cause unexpected increase of impervious area in the urban catchment, which leads to continuous change in rainfall trends and runoff amount; i.e. maximize the amount of runoff and decrease the time of concentration (Jian, 2009). Urban stormwater drainage systems have existed as an essential city infrastructure to collect and convey stormwater away from urban areas to rivers and lakes. Because of improper design and inadequate use of society, in many cities in Ethiopia, the deposition of silts and aggregate in drainage systems blocks the direction of flow and diverts the flow into streets and buildings occur during the rainy season. As a result, urban runoff is generally high to control runoff at source and dispose of an outlet (Dagnachew, 2011). Many towns in developing countries in the world the main cause for urban flooding was inadequate drainage system capacity. Hence, Drainage problems in urban areas introduce mainly flooding, deterioration of roads, land degradation, sedimentation and waterlogging (FUPCoB, 2008). It has been observed that inadequate integration between the road and urban storm drains, misplaced and albescence of curb inlets, unconscious way of waste disposal placement and undersized drainage system was the main cause for urban flooding in Hossana town, especially in Addis Sub city. Previously there were not any specific researches' conducted in line to the topic what this researcher had addressed the main problems and attempt to solve the above-mentioned challenges by modeling the urban drainage system for different design period to resist the climatic and rainfall factors. The main design function of stormwater drainage systems (storm drain piping, ditches, and channels) is to provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without flooding, surcharging inlets and overflows; otherwise it causes surface flooding, land degradation and erosion (ERA Drainage Design Manual, 2013).

In general, evaluation of the hydraulic performance of urban stormwater drainage system channels capabilities, the capacity of designed nodes(junctions) had been checked and evaluated for different year return design periods by using both Manning roughness channel equations and Environmental protection agency, stormwater management model (EPA SWMM) hydraulic (Extran block analysis). It has been observed that modeling urban flood by using computer analysis programs like EPA SWMM was very essential to evaluate the discharge carrying capacity of storm drain and to predict impact flooding into the surrounding environment that helps to understand the restriction in the urban drainage system and to estimate the existent of flooding (Rossman, 2016). The estimation of flood discharge was beneficial to save human lives and protecting public and private property. To overcome the problem of urban flooding, engineering prediction is essential to control the peak flow and the maximum depth allocation. Several mathematical models used to generate or simulate the dynamics of rainfall runoff and flood modeling. For this research SWMM with the help of geographic information system (GIS) was be implemented to simulate flooding on nodes or 1D view of the flood depth in the drainage system, to locate flooding nodes and to estimate the extent of flood inundated area.

1.2 Statement of the Problem

The widespread phenomena of unplanned urbanization and climatic change impact in many cities of Ethiopia lead for unexpected flooding in urban areas, due to the increased impervious area due to pavement roads, and buildings (Justin, 2016). The storm drainage system in the urban area is mainly provided to handle peak flow resulting from heavy rainfall occurred in design return period equal or greater to their design year and the system are expected to function smoothly in handling flow along or across their alignment without any cause of overland flowing and beyond capacity flooding.

However, most designed storm drainage systems in Hossana town specifically in Addis sub city was exposed to overland flow and flooding after heavy rainfall. This is might be due to undersized or insufficient drain capacity of the storm drainage canals. Besides, mismanagement of waste disposal and sediment into the drainage canal further aggravate to decrease the capacity of the canals. As a result, the designed storm drainage structures failed to deliver the runoff adequately to the desired outfall sites. Furthermore, problems that observed in the city is that; inadequate dimensions and placement of the curb inlets to covey the initial overland flow from roads entering into the drainage canals and lack of interconnection of drainage network further exacerbating the flooding problem in the town. Investigating the adequacy of the storm drainage system for future and modeling stormwater flood was very essential from the perspective of public safety and safeguarding material property. However, evaluation of the existing stormwater drainage system problem in Hossana town was not yet explored. Thus, evaluation of the existing drainage system for its hydraulic performance and identifying the flood extent for various return periods is essential to set strategies for safeguarding the public from flooding hazard and helps to keep the environment as well.

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1.3 Objectives of the study

1.3.1 General Objective

The General objective of the study was to model urban storm water flooding in Hossana town the case of the Addis sub - city.

1.3.2 The Specific Objectives:

- To develop the intensity duration frequency (IDF) curves for Hossana town.
- To evaluate the hydraulic performance of the existing drainage system.
- To delineate runoff contributing areas and estimate runoff for the catchments.
- To produce 1D view flood maps in channels and locate flooding nodes various year return period by using Storm Water Management Model (SWMM).

1.4 Research questions

- What would be the IDF curve of Hossana town?
- Does the 10 year return period intensity produce an overflow in the drainage system?
- Which part of the sub-city is more exposed to flooding?
- How can 1D view map of urban drainage channels produced through SWMM?

1.5 Significance of study

- To simplify rainfall intensity estimation procedures at the time of catchment discharge determination period.
- To plan and design, urban drainage system with consideration of urbanization increase
- Providing efficient urban floods disaster control strategy and mechanisms to reduce flooding problems.
- To create a flood free environment and keep the beauty of the urban catchment.
- To increase understanding of flooding problems, floodable zones and protect the drainage system of the town from the overflow in the surrounding environment by providing flood preventing structure and designing adequate drainage infrastructure.

1.6 Scope of this study

This study specifically focused on modeling and evaluating the existing design of the storm drain size of urban storm drainage systems to solve the urban flooding problem. The work was geographically limited to Hossana town, Addis sub city most floods prone areas. The study was included the delineation catchments contributing runoff, frequency analysis of twenty eight years daily maximum rainfall data to produce an IDF curve for study area and estimates peak discharges for the different year return period by using rational, Soil conservation service (SCS), and by using the SWMM model for the catchment. Finally, this study ends by mapping 1D view of channels flooding in flood prone area and locating floodable nodes on the catchment.

2. LITERATURE REVIEW

2.1 Overview of urban drainage system

Historical accounts show that the objectives of the urban drainage systems were to collect rainwater, prevent flooding, and convey wastes within it. At the time, planning and designing were limited; but few numerical standards were existed for urban drainage system evaluation. However, the engineering calculations and mathematical modeling were not used for design, drainage system at that time. Despite the lack of optimization and the use of trial-and-error construction methods, numerous ancient urban drainage systems could not function successful (Bulter and Davies, 2004). Urban drainage systems were firmly established as a vital public work system in the early parts of the twentieth century. Engineers and designers continued to improve design concepts and methods. During the second half of the twentieth century regulatory elements were spread in the United States, Europe, and other locations addressing urban drainage issues. Computer modeling tools, advanced the methods used to design and analyze urban drainage systems. Regulations, monitoring, computer modeling, and environmental concerns have altered the perspective of urban drainage from a public health and nuisance flooding concern during the first half of the twentieth century into a public health and nuisance flooding with additional concerns for ecosystem protection and urban sustainability (Zhou, 2012).

Communities worldwide are yet searching for innovative techniques to capture, detain, and use rainwater within the watershed instead of constructing massive drainage structures. Many communities are developing watershed wide storm water quality management plans to meet the dual objectives of flood prevention and water quality control. Urban drainage has indeed expanded significantly during the past few decades beyond a technical challenge to drain the urban area rapidly to include the consideration of social, economic, political, environmental, and regulatory factors (Bulter and Davies, 2004).

2.2 Urban drainage system and problems

2.2.1 Urban drainage system

Urban drainage systems are one part of urban infrastructure element which is mainly used to collect, store, convey, and treat runoff in urban areas. These facilities normally include detention and retention facilities, streets, storm sewers, inlets, open channels, and special structures such as manholes and the energy dissipates (FUPCoB, 2008). Full urban drainage systems are most necessary in developed urban areas because of the interaction between human activity and the natural water circulation. The way of conveying drainage systems to outlet points can be completely artificial or combination of man-made and natural watercourses (Linmei, 2003). According to Karolina, (2007) an efficient storm drainage system is essential to avoid the impact of flooding on life and property. According to ERA Drainage manual, (2013) urban drainage system should manage rains of return periods of at least 10 years return period rainfall without cause of flooding.

2.2.2 Problems of urban drainage system

Problems that connected with the urban drainage system can arise from different causes (Karolina, 2007).

- **The drainage system design**, the life length of different components in the system can differ a lot, and storm water pipes, especially in the old parts of a city center can be very old. The design criteria and urbanization have probably changed a bit since the first pipes placed in the ground, which might decrease to convey unexpected events, e.g. heavy precipitation of the coming return period.
- **Cross-connections** in the pipe system (storm, and drainage pipes) can be a problem due to a large variety of trash materials and in the system (treatment will be more difficult), and the damage to property the substances and water flows can cause.
- **Damage, roots, and sediments** in pipes decrease the flow capacity of the pipes (storm and drainage pipes) which can cause damage to the infrastructure and property during rainfall events.

- **Infiltration** into storm drain channels causes cracks in drainage systems. Infiltration can affect the treatment processes and failure drainage infrastructure.

2.3 Components of urban storm drainage systems.

The major components of urban drainage systems are catchment and sub catchment, nodes, conduit, links and outlet (Linmei, 2003; Rossman, 2016).

Catchment and sub catchment: - A catchment is the area collecting water from nearby higher terrain surface, which is delineated by help of digital elevation modem (DEM) with Arc GIS software and topographic contour maps. A catchment is usually described by its parameters (catchment area, the percentage of impervious area, average slope, the longest flow length, width, and approximate shape).

Nodes: - are junctions to link the storm drains. They also provide storm water transition between surface and subsurface systems. Typical examples of nodes are manholes. Manholes should be provided at intersections of storm water drains, junctions between different size of storm water drains, where a storm water drain changes direction or gradient.

Conduit: - is an artificial or natural channel, usually a closed structure such as a pipe or open channel.

Links: - transport flow in the system, and are often open channels or closed sewers with regular or irregular cross sections. Links usually contains both nodes and conduits.

Outlet (outfalls): - is the most downstream component of the urban drainage or the flow removal point, which discharges the drainage system flows of water from the system to receiving waters (Linmei, 2003; Rossman, 2016).

2.3.1 Major and Minor Drainage Systems

According to Bulter and Davies, (2004) many urban areas, drainage is based on a completely artificial system of sewers, pipes and structures that collect and dispose of both waste and storm water. A complete storm drainage system design includes consideration of both major and minor drainage systems. The minor system, sometimes referred to as the "Convenience" system, consists of the components that have been historically considered as part of the storm drainage system which used to convey wastewater (Bulter and Davies, 2004). As Ethiopian context, the minor drainage system considered locally, but the major drainage system provides overland relief for storm water flows exceeding the capacity of the minor system.

The major system is composed of pathways that are provided knowingly or unknowingly for the runoff to flow to natural or man-made receiving channels such as streams, or rivers. The designer should give attention on the flow pathways and related depths and velocities of the major system under less frequent or check storm conditions (typically, above 25-year event is used as the check storm for cross drainage structures) (ERA Drainage manual, 2013).

2.4 Storm water management

Storm water management practices were properly selected to mitigate the adverse hydrologic and hydraulic impacts caused by drainage facilities, thereby protecting downstream areas from flooding, soil erosion and water quality degradation (Kokeb, 2015). Poor storm water management will increase flooding problems in downstream channels (Annette, 2010).

Storm water management makes the natural environment, be functional, safe, and aesthetically acceptable. Several alternatives may be possible to manage storm water and provide water quality. These are:-

- ✓ Careful modeling and planning can produce optimum results.
- ✓ Re vegetation with native, non- invasive grasses, shrubs and possibly trees may be required to achieve compatibility with the surrounding environment (Brown et *al.*, 2013).

The storm water results from all kinds of precipitation (rainfall, snowmelt) and comprises the water flowing on the surface. Therefore, the characteristics of both the rainfall and the catchment area were important factors in the storm water management and modeling (Butler and Davies, 2004).

2.4.1 Effect of urbanization on urban drainage system

The increase of urbanization is greatly affects the disturbing balance of runoff generation process due to replacement of large pervious areas of land by impervious areas. As a result, more flash floods can be experienced in urban areas as compare to rural areas (Nilmini, 2003).

Urban expansions are modifications to the rural or natural areas. As a result, the population growth and infrastructure development and it causes direct and indirect impacts to the area and its surroundings, such as vegetation destruction, floods and climate change (Boulomytis *et al*., 2018). According to Kokeb, (2015) the evaluation of urban drainage system is related more to urban development and the increment of floods derived area. In urbanized areas the paved and impervious surfaces hinder the capacity of the soil to absorb water. Consequently, the velocity of the runoff is increased and leading to sharp peak discharges and greater amount of water in the surface. The knowledge of the contribution to urban runoff from pervious and impervious areas essential to model, manage, and design, urban storm water drainage system (Butler and Davies, 2004).

2.5 Urban flooding

According to Thuy, (2009) urban flooding occurs when channels were filled with water beyond their capacity and results in excess water pouring out of the channel into the adjacent floodplain. However, some human activities such as, increasing human settlements and economic assets in floodplains and the reduction of the natural water retention by land use and climate change contribute to an increase in the likelihood and adverse impacts of urban flood events. To identify critical urban flood prone areas such as zones with high water depths and flow velocities; hydrologic and hydraulic modeling be very essential (Behzad, 2018).

2.5.1 Types and causes flooding

Types of floods

Flash Floods: - is a very direct response to rainfall with a very high intensity or sudden massive melting of snow. The area of inundated by flash flood is relatively small compared to other types of floods. The amount of water that covers the land is usually not very large, but is so concentrated on a small area and travelling with high speed of the water. As a result, flash floods were very dangerous which cause several damages in a short period of time (Melese, 2014).

Coastal Floods: Type of flood when the coast is flooded by the sea. The cause of such a surge is a severe storm, wind pushes, and high waves around the coast (Melese, 2014).

Riverine floods: Rainfall over an extended period and an extended area can cause major rivers to overflow their banks. Downstream areas affected, even when they did not receive much rain themselves. River flooding happens when extreme rainfall affects a river basin (Linmei, 2003).

Fluvial floods: Rise from the incapacity of watercourses, such as, rivers and streams to convey catchment runoff (Linmei, 2003).

Pluvial floods: are due to overland flow, blocked or overloaded drains, sewers, and broken water mains (Linmei, 2003).

Groundwater flooding: caused due to the rise of the groundwater table resulting in flooding of basements and filling up of the sewers and drains (Deepak, 2009).

Causes of urban flooding

The causes of floods can be broadly divided into physical, such as natural forces (climate factors), and human influences such as deforestation and urban development's. The most common causes of floods are climate related, mostly high rainfall (Thuy, 2009). Some factors that tend to increase the urban flooding risk are:

- Ageing drainage system
- Increase of urbanization
- Increase in paving
- Insufficient drainage capacity

In general, most cities in Ethiopia urban floods occur due to insufficient drainage facilities to overcome design period peak flow. Furthermore, the main cause floods are heavy rainfall, tropical storms, snow or ice melt, dam break, mudslide (Anteneh, 2015).

2.6 Urban flood modeling

Urban flood modeling has been one of the key tasks of governments, municipalities and social institutions as well as city planners, designers and engineers. There are different models have been used to simulate different flooding situations, which range from single process-based models, such as various loss models, rainfall runoff model, channel or pipe flow model, to integrated modeling package that combines several single-processed models (James, 2011). Most urban flooding models make significant roles in flood forecasting, protection and mitigation (Linmei, 2003). Depending on rainfall-runoff analysis urban flooding modeled by:

- Hydrological analysis based on physically based parameters solving rainfall-runoff analysis analytically. This method is complicated given the time, cost and effort required to carry out these types of studies and these studies play an important role in understanding the processes governing the physical movement of water varying with time (Deepak, 2009).

- Hydrodynamic rainfall-runoff analysis: This method is mainly computer based programming model which is software related. Computer based rainfall-runoff model predicts the hydrological response (runoff) to a certain input (precipitation), usually as a function of time (Henri, 2013).

2.6.1 Hydrological Modeling for Determining Peak runoff

Rational Method: - Provides peak runoff rates for small urban and rural catchment areas, less than 50 hectares, but modified rational method used for size of drainage area greater than 50 hectares. It also best suited to urban storm drain systems and rural ditches. It used with mistakes if the time of concentration exceeds 30 minutes. The peak flow from a drainage basin an area is a function of the basin's photographic properties such as size, shape, slope, soil type, land use, as well as climatological factors such as selected rainfall intensities (ERA Drainage manual, 2013).

US (NRCS) or SCS Method: -Techniques developed by the United State Natural Resources Conservation Service (U.S.NRCS), formerly the U.S. Soil Conservation Service (SCS) for calculating rates of runoff require the same basic data as the rational method. The basis of the SCS method is a series of unit discharge hydrographs expressed in cubic meters of discharge per second per square kilometer of watershed per millimeter of runoff (Brown *et al.*, 2013).
The SCS Curve number method utilized as an alternative to the rational method. The SCS Peak Flow method utilized for larger drainage areas (up to 800 hectares). Like the Rational method, use of this method should be limited to basins with relatively homogeneous curve numbers and an overall curve number greater than 40 (SUDAS, 2013). Two types of hydrographs used in the NRCS procedure, unit hydrographs and dimensionless hydrographs. A unit hydrograph represents the time distribution of flow resulting from 1mm or (1in) of direct runoff occurring over the watershed at a specified time. A dimensionless hydrograph represents the composite of many unit hydrographs. The dimensionless unit hydrograph is plotted in non-dimensional units of time versus time to peak and discharge at any time versus peak discharge (NRCS, 2010).

According to Jenny, (2013) NRCS method is based on a 24-hour storm event, which has a certain storm distribution. Several techniques have been developed and are currently available to engineers for the estimation of runoff volume and peak discharge using the NRCS methodology. Some of the more commonly used methods to determine runoff for catchment are:-

- NRCS Technical Release 55 (TR-55 and TR-20);
- Unit Hydrograph Methods - for drainage areas greater than 50 hectares;
- Watershed Regression Equations - for all routine designs at sites where applicable;
- Log Pearson III Analyses - preferable for all routine designs provided there is at least 10 years of continuous or synthesized record for 10-year discharge estimates and 25 years for 100-year discharge estimate (NRCS, 2010).

2.6.2 Overview of Hydrodynamic peak discharge determination

According to Emilie, (2011) Modeling floodplains in urban areas are more complex than in underdeveloped areas. Many cities are also located in coastal areas and are therefore at risk from flooding from extreme storm events in addition to typical rainfalls. A hydrodynamic model for an urban system needs to be able to handle these various complexities in order to model the area accurately (Umamahesh *et al*., 2016). Until recent times, hydrodynamic modeling of urban systems involved solving a one dimensional (1D) network of channels, pipes and nodes. However, with the development of high resolution, gridded topographical data, the application of two-dimensional (2D) modeling methods is becoming more popular (Jonathan, 2017). 2D flood models can be more accurate than 1D-flood models because data can be input in a gridded format. The major downside to using a 2D model is the increased computational cost and unavailability to free charge required compared to the traditional1D approach (Deepak, 2009).

Common 1D software for modeling floodplain inundation and/or urban pipe systems includes SWMM 5.1, HMS, HEC-RAS, and MIKE11 (HEC, 2010). 2D models linked models include TELEMAC-2D, LISFLOOD-FP, XPSWMM-2D (TUFLOW), and 1D/2D models are ISIS, XPSWMM MIKE FLOOD (DHI user manuals, 2011). The most commonly used hydrodynamic (computer based) hydrologic and hydraulic models are:

HEC-HMS: (Hydrologic Engineering Center Hydrologic Modeling System) is an open source software for the modeling of the rainfall-runoff process developed by the U.S. Army Corps of Engineering´s. The HMS is intended to simulate the precipitation-runoff processes of dendritic watershed system and its design allows applicability in a wide range of geographic areas for solving diverse problems, including large river basin water supply and flood hydrology, and small urban or natural watershed runoff (HEC, User manual, 2015). HEC HMS is also used to methods to analyze and compute urban flood damage by using HEC-Geo HMS, and GIS as a preprocessor can accelerate the building of an HMS which is to simulate the rainfall-runoff process of watershed systems (Aysha and Tanim, 2016).

MIKE 11 is developed by the Danish Hydraulic Institute (DHI) water and environment used to simulate surface-runoff flow, sediment transport and water quality in rivers, flood plains, channels. It designed for flood risk analysis and mapping, design of flood alleviation system, real-time flood forecasting, hydraulic analysis and design of structures (DHI user manuals, 2011).

MOUSE: - (Modeling of urban sewers) is a hydrologic-hydraulic model applicable only for modeling of urban catchments. This model used extensively in sewerage design in Australia compared to the design of storm water drainage networks (DHI user manual, 2011). The hydrologic part of the model deals with simulation of runoff using two methods.

Simple method based on time-area diagram and a **complex** method based on kinematic wave theory and the continuity equation. The hydraulic part of the model simulates flow routing in closed conduits or open channels. Three options are available in MOUSE to compute depth and velocity of flow (DHI user manual, 2011).

- The kinematic wave method, which is mostly, applied to part full flow conditions.
- The diffusive wave method, which considers backwater and surcharge in the systems.
- The dynamic wave method, which provides a full hydrodynamic solution.

MOUSE, is like SWMM, which is well suited for analyzing the hydraulic performance of complex looped sewer systems, including overflows, storage basins and pumping stations (Linmei, 2004).

HEC-RAS is a hydraulic modeling software developed by the U.S. Army Corps of Engineer´s Hydrologic Engineering Center; which designed to perform one dimensional steady or gradually unsteady flow hydraulic calculations for networks of natural and artificial channels (HEC, 2015; Jeremiah, 2011).

HEC-Geo RAS is an extension to work under Arc View GIS environment. The software is capable of performing one-dimensional (1-D) steady and unsteady-flow simulations and comprises a graphical user interface, separate hydraulic analysis components, data storage and management capabilities as well as graphics and reporting facilities (Jeremiah, 2011).

SWMM: the association of American engineers for the United State Environmental Protection Agency developed Storm water management model (SWMM). EPA). It used in the United State for the design of storm water drainage systems and incorporated in regional water quality management planning. In addition, it applied globally for storm water planning, design and rehabilitation purposes (Reza, 2016). SWMM is a widely accepted model and is currently applicable in Ethiopia for planning, analysis and design related to water systems in urban areas (Getachew *et al*., 2015).

SWMM takes the rainfall and catchment characteristics, determines the quantity and quality of runoff, routes the runoff through a combined or separate sewer system and identifies the effluent impact on receiving waters (Rossman, 2016). Thus, it is a mathematical model capable of representing urban storm runoff, including sewage storage and treatment and combined sewer overflow Phenomenon. However, improvements have to be added on SWMM in order to simulate urban flooding. The SWMM 5.1 can provide an integrated environment for editing, study area, input data, running simulations and viewing the results in a variety of forms (time series graphs and tables, profile plots, scatter plots and color-coded maps). The data required for modeling include rainfall, topography, hydraulic (drain dimensions), soil type of infiltration rate, agricultural land use and hydrologic properties (Rossman, 2016).

2.7 Model Selection

As stated in Deepak, (2009) the choice of modeling method will depend on the purpose, the spatial scale of the problem, the level of detail required, data and software availability and the time and cost constraints for the flooding problem was being investigated. A particular modeling method will have an advantage in terms of performance, data requirement and accuracy for a particular case. SWMM 5.1.03. is selected for this study due to the following reasons; SWMM 5.1.03 is mainly used for:-

- Design and sizing of drainage system components for flood control;
- Flood plain mapping of channel systems;
- Designing control strategies for minimizing storm drain overflows;
- Used to generate both hydrological and hydraulic characteristics of the catchment;
- Its simplicity and easy to understand;
- There have been previous studies carried out in the relative geographical area using the same software;
- Its availability related graphical user interface free of charge (Rossman, 2016).

According to Rossman, (2016) SWMM 5.1.03 was limited in 1D storm drain analysis and it can't simulate floods along streets. Hence, additional modeling tools be developed, or improvement must be made on the SWMM 5.1.03 models for simulating storm plain flooding. According to Daniel *et al*., (2016) SWMM was designed to go one-step further, since it combined a GIS-based selection system with 1D/2D hydrological modeling software for the location and assessment of different urban drainage systems. However, SWMM by itself solvesonly1D Shallow Water Equations (1D-SWE) in both storms drains and surface systems as a set of links and nodes (Kourtis *et al*., 2017). Therefore, SWMM have an ability to simulate flood with in channels and nodes.

2.7.1 Modeling Using EPA SWMM 5.1

SWMM model is designed to simulate the runoff of a drainage basin for any pre-described rainfall pattern. Total watershed divided into a finite number of smaller units or sub-catchments that described by their hydraulic or geometrical properties (Rossman, 2016). The model has mainly three blocks first one is Runoff block (surface flow), second is a Transport block (quantity and quality of flow) and a third is an extra block (hydraulic operation). Extran block has completed the hydraulic calculations for overland flows, in channels, and in pipes. It performs dynamic wave routing and provides all the depth, velocity and energy grade line information.

The delineations were based upon the topography of the study area, utilizing Digital Elevation Model (DEM), following existing drainage system and present land use pattern. Secondly, infiltration losses were calculated by the inbuilt model in SWMM by using the SCS curve number method. The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage or treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps (Rossman, 2016).

2.7.2 Flow Routing

Flow routing within a conduit link in SWMM governed by the conservation of mass and Momentum equations for gradually varied, unsteady flow (i.e. the SaintVenant flow equations). The SWMM user has a choice on the level of sophistication used to solve these equations: (Rossman, 2016). According to Rossman, (2016) SWMM 5.1.03 flow routing algorithms divided into three

Steady Flow

- Simple hydrography translation
- Applicable only to branched networks

Kinematic Wave

- Gravity force balanced by friction force
- Attenuated and delayed outflow due to channel storage
- Applicable only to branched networks

Dynamic Wave

- Solves full St. Venanteqautions
- Accounts for channel storage, backwater effects, pressurized flow, and reverse flow

- Applicable to any network layout
- Requires a smaller time step

2.8 GIS application in urban flooding

Geographical information system (GIS) is a computer based information system that enables capturing, modeling, manipulating, retrieval, analysis and presentation of geographically referenced data (Linmei, 2004). The use of GIS in urban storm water models has been limited due to the need for large, detailed and expensive databases that are usually difficult to assemble. In addition, the fact that computer models used in storm water modeling were not capable of to be integrated with GIS (Micheal, 2017). Amongst the various techniques available for assessing flood risk, the ones using GIS are most appropriate. GIS is used to construct and update hydraulic models that are capable of predicting one-dimensional and two-dimensional flooding (Shirish *et al*., 2010).

GIS as a modern technology has several techniques and tools that can be used for effective flood modeling and mapping(vahdetti *et al*., 2016).The development of GIS functionalities for hydraulic and hydrological models made it possible to identify areas that are at risk of flooding in a particular earth's surface area (Ismail, 2016). GIS differs from CAD and other graphical computer applications in that all spatial data geographically referenced to a map projection in an earth coordinate system (Efrain, 2005).

2.8.1 Digital Elevation Model (DEM)

DEM reflects the topographical condition of the study area that used as the basic data for the urban flood inundation model and was one of flood risk indices (Xu *et al*., 2018). DEM is an ordered array of numbers that represents the spatial distribution of elevations above an arbitrary datum in a landscape. It generally consists of elevations sampled at discrete points, or contour lines. DEM is an essential tool for presenting and analyzing the features of topography (Mark *et al*., 2004).

Watershed and Stream Network Delineation: - Every hydrological modeling starts from delineating streams and watersheds, and obtaining watershed properties such as area, slope, flow length, flow direction, stream network. Traditionally, it was done manually by using contour maps.

Catchment Area Delineation: -is depends on topographical and drainage network data. In urban areas, catchments must be further divided into sub-catchments connected to the appropriate location in the local drainage channel (Mark *et al*., 2004).

Stream Network Verification: -After the drainage, lines of the project area are determined by running the terrain processor model they have to be verified against existing natural streams. The verification is required because there may be some inconsistencies between the actual streams and streams obtained from terrain processing due to inaccuracy of digital elevation model data (Mudher and Abdulla, 2011).

2.9 GIS integration with SWMM model

SWMM and Arc View was not found to be a user-friendly environment to perform storm water flood modeling easily as other storm water flood models. Hence, further techniques must be introduced to interrelate SWMM with GIS software (Rossman, 2016).Many storm water modeling functions can be facilitated by GIS, tools that work within a widely used GIS can be a valuable component of a modeler's toolbox. The scripts thus focus on viewing input and output files for SWMM's Runoff and Extran modules. Linking Extran to a GIS is easy, as an Extran, input file describes a link-node conduit network. However, linking the Runoff input is too complex (Shamsi *et al*., 2005).

2.10 Hydraulics of Storm Drainage Systems

2.10.1 Flow Type Assumptions

The engineering calculations used in the modeling and evaluation of drainage systems are, to a large extent, applications of the fundamental physical laws of conservation of mass, energy and momentum. Flow in drainage systems can be piped flow running under pressure and open channel flow, characterized by water conveyed by gravity with a free surface at atmospheric pressure. However, when it comes to storm drainage systems the flow is a combination of both of them, resulting in a part- full pipe flow (ERA Drainage manual, 2013). The most basic principle of the hydraulics of pipelines is the conservation of mass, which assumes that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment was assumed constant with respect to time and distance. However, in actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform (Chow, 1988).

2.10.2 Flood frequency Analysis

The determination of flood flows at different recurrence interval is a common problem in engineering hydrology. The standard procedure to determine probabilities of flood flows consists of fitting the observed stream flow record to specific probability distributions. However, this procedure only works for basins that have long enough stream flow records to warrant statistical analysis. Continuous hydrologic simulation is a valuable tool to determine flood frequencies in un-gauged watershed and in gauged watersheds that have short stream flow records (Brown et *al*., 2013).

2.10.3 Hydraulics of drainage system

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance or manning’s roughness coefficient. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's Equation(chow, 1998).

Manning Equation: - is used to determine the channel capacity discharge and that is essential to examine the existing drainage channel to sustaining coming discharge without causing any flooding problems to the environment. This method is mainly dependent on the roughness characteristics of the channel system (Brown *et al*., 2013).

Manning’s Roughness Coefficient: In applying the Manning equation, the greatest difficulty lies in the determination of the roughness coefficient, n; there is no exact method of selecting the n value. Selecting a value of n actually means to estimate the resistance to flow in a given channel, which is really a matter of intangibles (NRCS, 2010). Surface Roughness, Vegetation cover, Channel Irregularity, Channel Alignment, Silting and Scouring Obstruction are the main factors that affect Manning's roughness coefficient magnitude.

3. MATERIALS AND METHODS

3.1 Description of study area

3.1.1 Location

Hossana town is located southwest of Addis Ababa at a distance of 232 Km and from Hawassa, which is Regional capital 175Km. The absolute geographic location of Hossana town is from $7^{0}30`00''$to $7^{0}35'00''$North latitude and from $37^{0}35'00''$ to $37^{0}59'00''$East longitudes. Hossana town is the administrative and commercial center of the Hadiya zone. The town got its new administrative structure in 2004 E.C consisting of three sub-cities; namely Addis sub-city, Gofer Meda sub-city and Sech-Duna sub-city; and eight cables. The Addis sub city was one of the sub-city of the of Hossana town which was mainly exposed for flooding problem, because of its location, landscape, and existing drainage system inadequacy (Hossana Town Municipality, 2017).

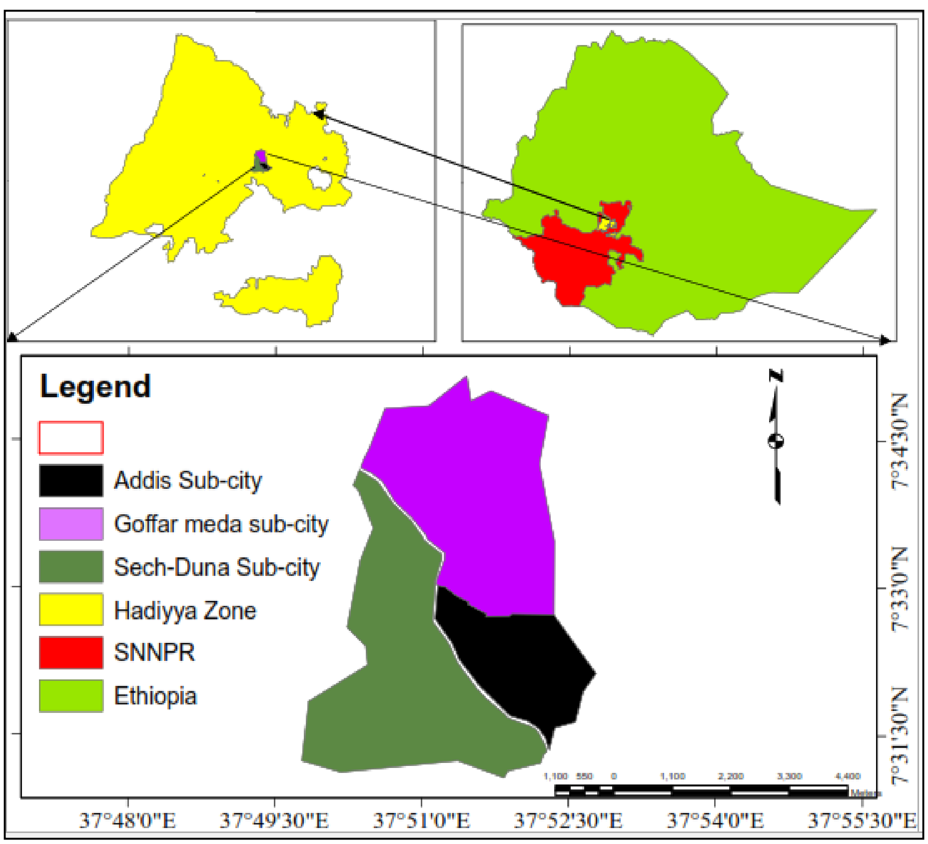

Figure 3.1: Location Map of the study area.

3.1.2 Topography

Hossana town is topographically located in mostly steep slope or vertical landscape orientation and incorporated with several ups and downs, hills and plains. The elevation the town ranges from 2,365m around Queen Eleni referral Hospital, and 2,210 m around Olola site bridge. From site survey and field investigation, most part of the runoff contributing area is lay between an elevation of (2,290 - 2345) m and slope of (5%-7%) (Field survey and Hossana town municipality, 2017).

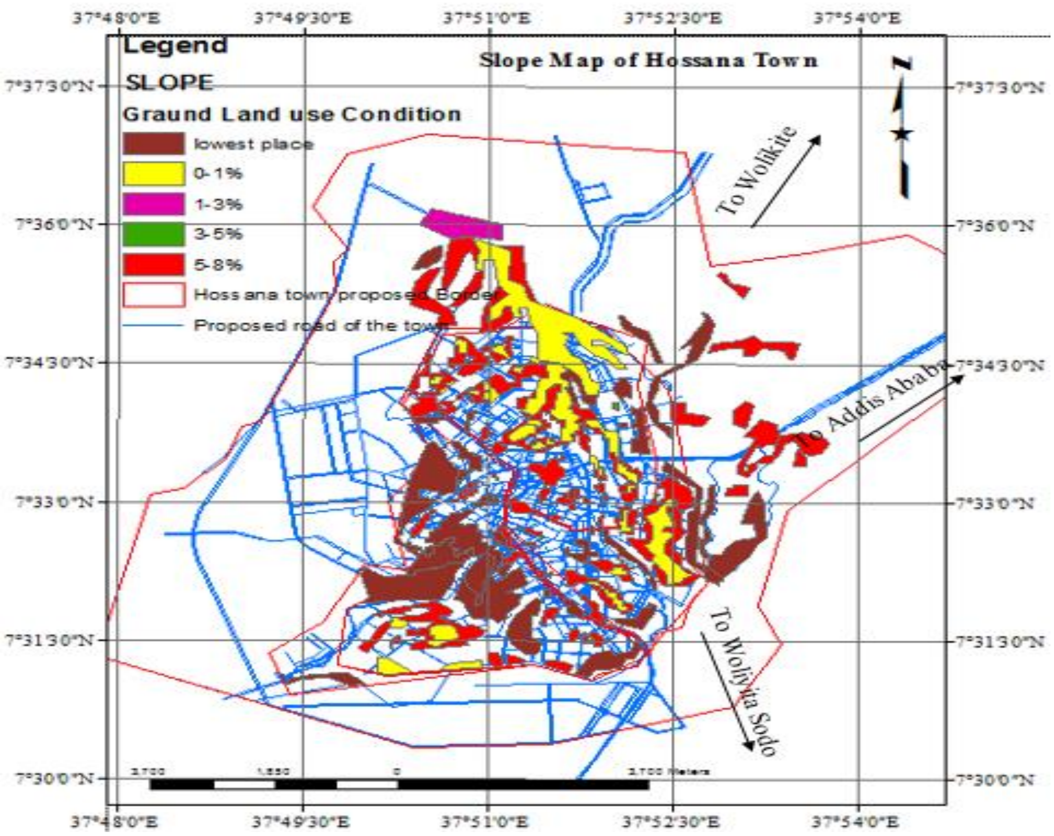

Figure 3.2: Slope Map of the Hossana town (Source: -Hossana town Municipality, 2017).

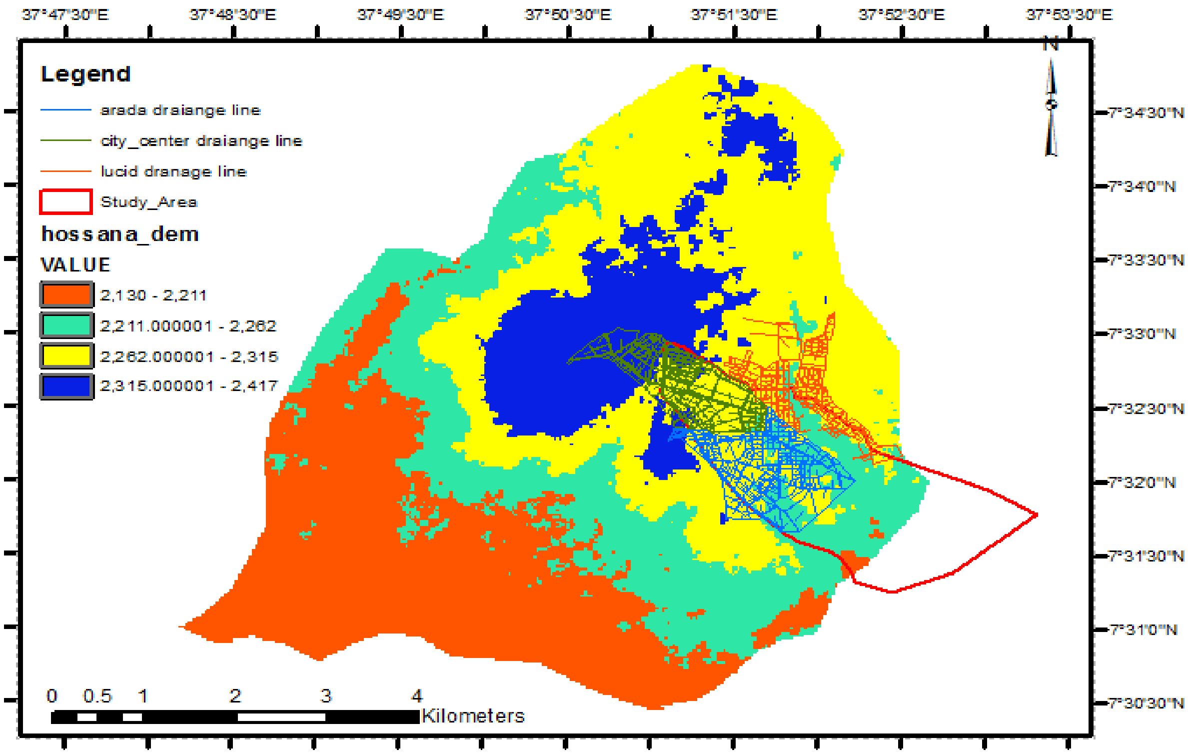

Figure 3.3: Topographic map of Hossana town (source: -https://earthdata.nasa.gov and GIS analysis of field surveyed data).

3.2 Climatic Condition

Depending upon altitude Hossana town is mainly characterized by highland ‘'Weynadega‟ climatic conditions. The mean maximum and minimum monthly temperature vary from of 24.5^{0}C to 10.8^{0}C respectively based on methodological recordings of with periods of (1990-2017). The monthly average rainfall of the town is shown below in figure 3.4 for rainfall season (1990-2017). Depending upon rainfall characteristics, the town has four distinct seasons, I. e. ‘‘Tsadayi’’(September to November) which provide little rain for the town ‘'Bega' (Dec, Jan, Feb) provide a limited rainfall for some parts of the town, ‘'Belg‟ (March, April, and May) of the medium rainfall seasons. Seasonal maximum rainfall was received in "kiremt "that occurred in June, July, and August.

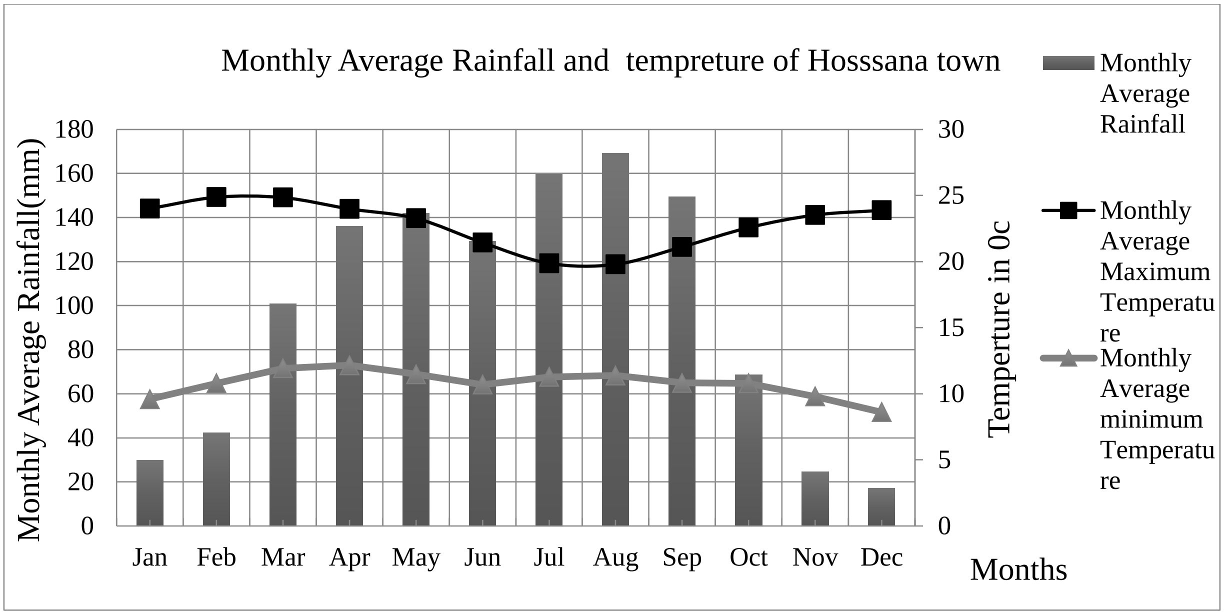

Figure 3.4: Monthly average rainfall and average temperature of Hossana town. (Source: NMA 1990-2017).

3.3 Land use land cover

Land use and soil type has a direct impact on the flood amount, speed and potential to create damage that is why the study gives notice for land use and land cover of the catchment.

The study area is relatively small; field survey and help of Google earth pro did categorization of different land use. The land use land cover for the catchment area contains Mixed Residences, Government compounds, colleges, Asphalt roads, Cobblestone roads, Commercial Areas, school, Health Center, playgrounds, Agricultural lands and buffer zone. An area for each category of parcel land use was obtained with the help of Auto CAD and Arc-GIS (Hossana town Municipality, 2017).

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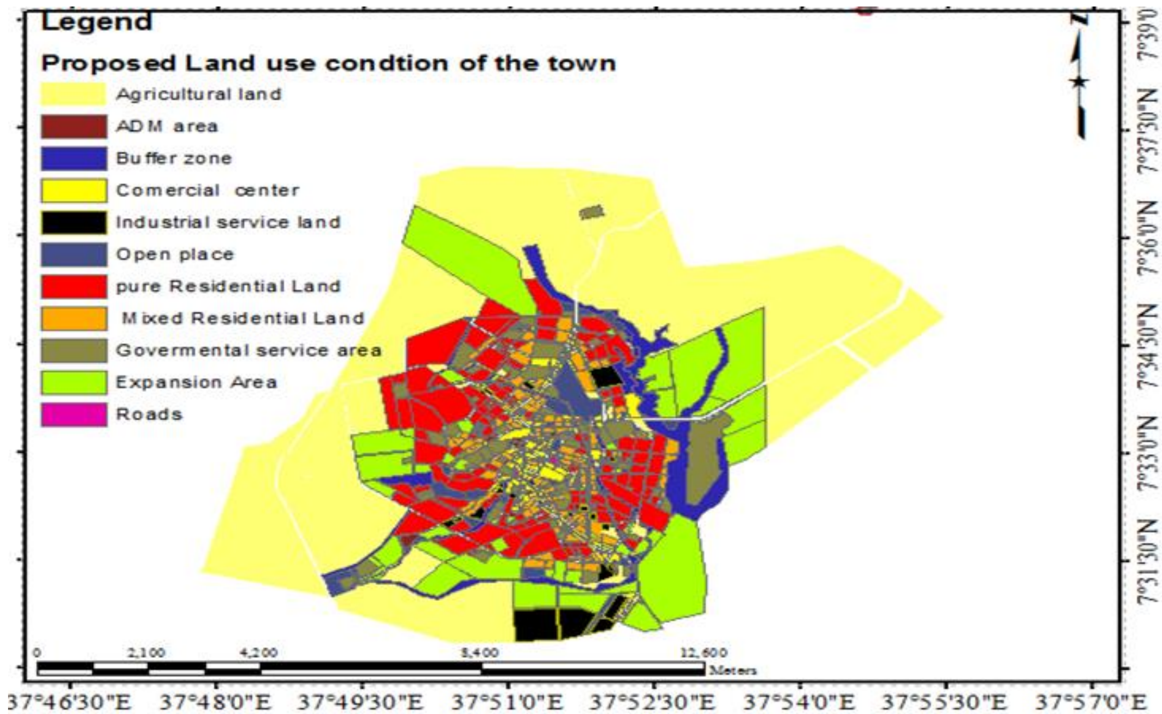


Figure 3.5: Land use land cover map of the Hossana town (Source: -Field survey and Hossana town municipality GIS analysis).

3.4 Materials and software's

The materials and software's that used to perform this research and that helps to attain the overall researcher's objectives are:-

- **Arc-GIS:** used to delineate the catchment area and used convert the Digital Elevation Model (DEM that was the main input parameter for Arc GIS, as supportive inputs for other hydrological flood modeling software.
- **Google Earth pro:** to verify watershed and helps to observe catchment properties of the study area
- **Digital camera:** use to capture necessary photo's at the time of site investigation
- **Hand Held GPS:** used to collect coordinates of the stations at each selective area of the study, like junctions, links, and nodes of the drainage system of the town.
- **Tape Meter:** helps to measure the existing drainage structures cross sections like openings, depths, width, slope, and the length of conduits.

- **Auto CAD:** is used to draw cross section dimensions of the drainage channels which is used to prepare the main input for EPA SWMM and GIS.
- **Leveling instrument:** used to measure the elevation difference between the upstream and downstream sections.


- **Excel-Sheet and Calculator**: - Excel is very essential Microsoft computer program, which used to evaluate and arrange numerical data in well manner. Also used to calculate repeated data in short period of time.
- **Easyfit5.6**. professional software. Used to determine statistical analysis of rainfall data and used to select best-fit distribution method.

SWMM V.5.1.03: is a freely available software is used to design and analyze urban stormwater drainage systems, and used to simulate urban flooding in the conduit

3.5 Methods of Data analysis

The methodological approaches employed in this study was provided in figure 3.6.

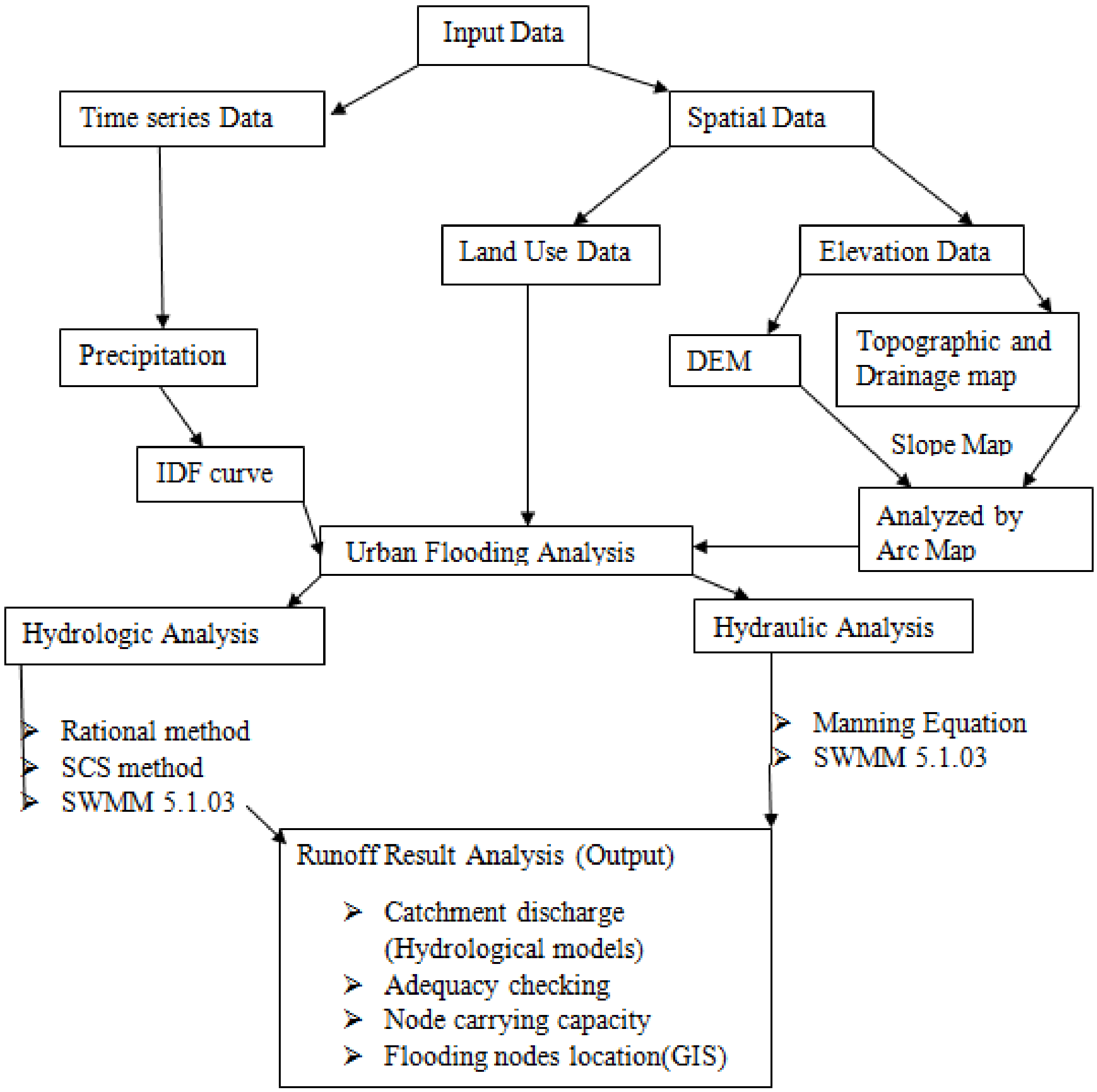


Figure 3.6: Methodological framework representation

3.6 Intensity Duration frequency (IDF) curve

Data used in preparation of IDF curve are mainly precipitation data

I. **Precipitation data**: - is the type of time series data arranged or ordered in hourly data, monthly data, yearly data (Justin, 2016). However, for this research daily arranged 28 years (1990-2017) rainfall data was used for analysis and design rainfall depth and to develop intensity duration curve. However, the study area has only one rain gauge station to record rainfall data. Hence, further rain-gauge stations were used to check the quality and consistency of the recorded data.

II. Estimating missing rainfall data: Due to the absence of the recorder or instrumental failure, rainfall data record occasionally is incomplete. In such a case one can estimate the missing data by using the nearest station rainfall data by using different missed data estimation mechanism. For this research, a normal ratio method is used to estimate missing rainfall data for Hossana town. According to the normal ratio method, the missing data PX can be calculated as equation 3.1 (Sabarmaniya,2008).

$$Px = \frac{1}{3}\left(P_1\frac{N_x}{N_1} + P_2\frac{N_x}{N_2} + P_3\frac{N_x}{N_3}\right) \qquad 3.1$$

Where P_x -missing daily rainfall data of Hossana

P_1, P_2 and P_3 – rainfall data at nearest different station (daily rainfall data Angacha, Fonko and Gimbichu respectively).

N_x - mean annual rainfall at Hosseana station.

N_1, N_2, and N_3- mean annual rainfall at different nearest stations (Angecha, Fonko, and Gimbichu respectively).

The three nearest stations were selected to analyze the missed data of the study site. The data obtained national meteorological agency shows (1990-1997) there is not missed rainfall data at the study site. Therefore, other station data were used to fill missing rainfall data for remaining years (1997-2017). Figure 3.7: show that annual rainfall of the selected three stations to fill missing data for the study site station.

Table 3.1: Rainfall station around Hossana rainfall station

N0.	Rainfall stations	Distance from main station Hossana (Km)	Direction	AARF (mm)
1	Angacha (N1)	28	South east	1260
Hossana station (Nx)				1195
2	Fonko (N2)	25	North east	1180
3	Gimbichu (N3)	32	North west	980

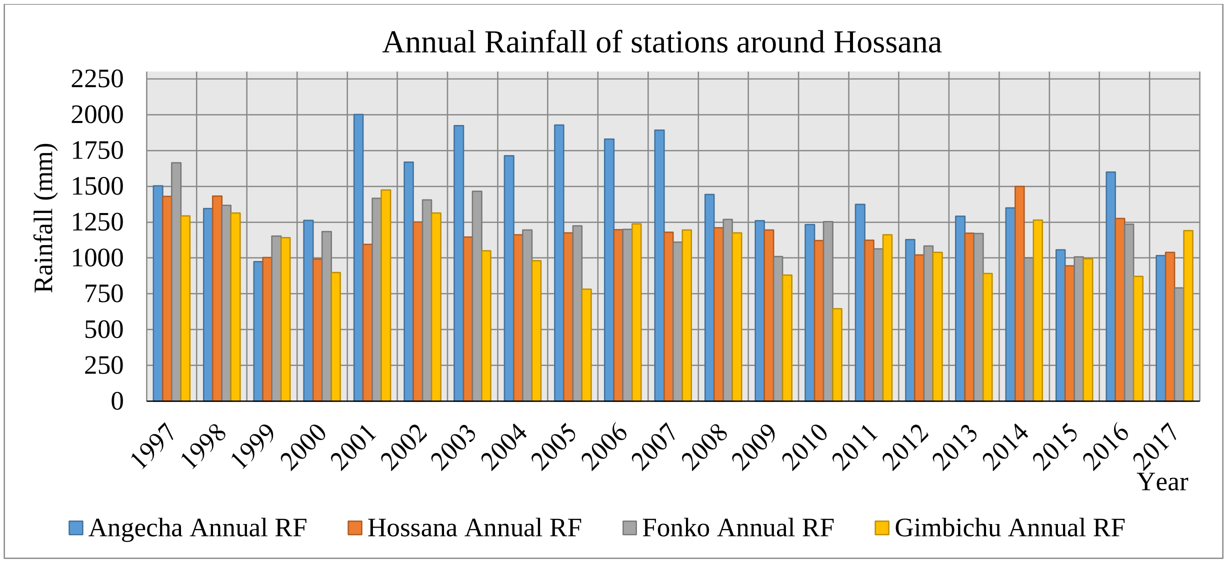


Figure 3.7: Annual Rainfall for stations near to study site (Source: NMA, 2017**)**

III. Rain fall data consistency checking

Checking rainfall consistency is used to check the gap between recorded data of the station with selected rainfall stations. If the conditions relevant to the recording of a rain gauge station have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. The checking for the inconsistency of a record is done by double mass curve technique (Garg, 2005). The double mass curve is a graphical method for identifying and adjusting inconsistency in a station record by comparing the accumulated totals of the gauge in question with corresponding totals for a representative group of the nearby gauge. If a decided change in the regime of the curve is observed it should be corrected by using equation 3.2 corresponding totals for a representative group of the nearby gauge.

$$P_m = \frac{M_c}{M_a} * p_x \qquad \text{......3. 2}$$

Where: P_m is corrected precipitation at any time

P_x is original recorded precipitation at time period,

M_c is corrected slope of the double mass curve and

M_a is original slope of the double mass curve.

For this study, annual rainfall data consistency record was checked by double mass curve technique as shown in figure 3.8: which is shows rainfall data for all stations was good consistence.

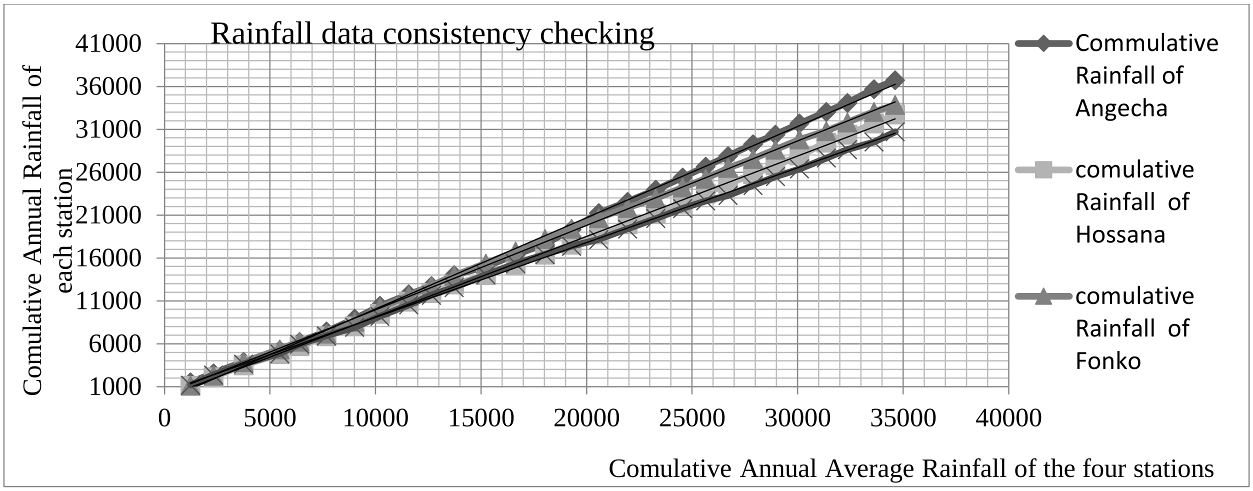


Figure 3.8: Rainfall data consistency checking by using double mass curve

3.6.1 Rainfall frequency analysis

The objective of frequency analysis of hydrologic data is to relate the magnitude of extreme events to their frequency of occurrence through the use of probability distributions (Brown *et al*., 2013). Since rainfall data of previous years was use to design rainfall frequency analysis. The study area was found in the "B_2" rainfall region according to ERA drainage manual, (2013) shown in figure 3.9. Filled rainfall data used to develop Intensity duration frequency (IDF) curve by selecting maximum daily rainfall.

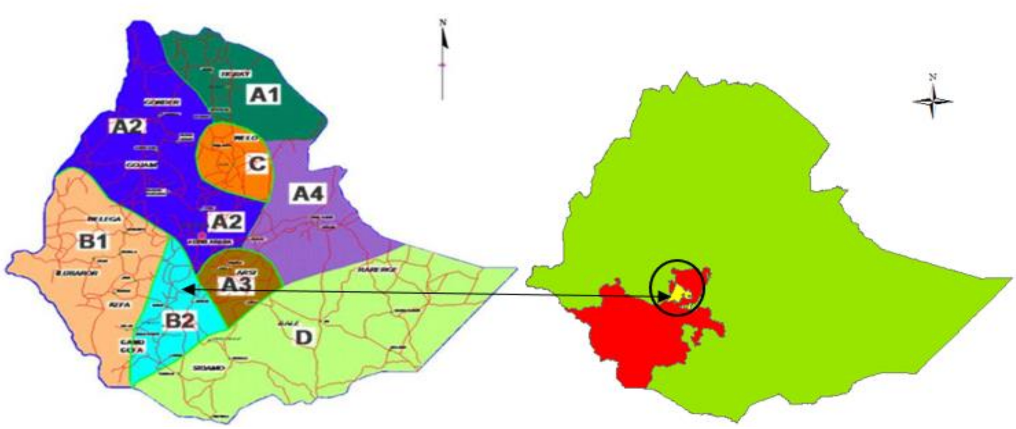


Figure 3. 9. Location map rainfall region "B2" (Source: - ERA Drainage manual, 2013)

IDF curve for Hossana town is not yet prepared. Hence, the preparation of IDF curve is essential to determine the peak discharge of the study area rather using ERA prepared IDF curve for divisional rainfall regions. Therefore, the measured rainfall data distributed in a good manner to determine peak extreme value. Currently, the estimation of extreme rainfall data was accomplished based on statistical frequency analysis of maximum daily precipitation records where available sample data could be used to calculate the parameters of a selected frequency distribution (Olofintoye *et al*., 2009).

The best-fitted distribution used to estimate rainfall event magnitudes corresponding to return periods greater than or less than those of the recorded events do.

A. Goodness of fit test

The goodness of fit (GOF) tests measures the compatibility of a random sample with a theoretical probability distribution function. Several goodness of-fit tests exist in determining which distribution is the best model to describe the daily rainfall. Among of those the most commonly used are: (Oseni, 2013).

1. Chi-Squared Test (χ^2) This test simply compares how well theoretical distribution fits the empirical distribution (PDF). The statistic is defined as

$$x^2 = \sum_i^k 1 \left\{\frac{Oi-Ei}{Ei}\right\}^2 \text{--3. 3.1}$$

Where Oi = observed frequency for bin i

Ei = the expected frequency for bin

K = the number of classes

Ei = F(X2) – F(X1)

Where:- F is the Continuous distribution function (CDF) of the probability distribution being tested and X_1 and X_2 are the lower and upper limit for (i-k)(Oseni, 2013)..

2. Kolmogorov – Smirnov Test (K-S)

This test statistic is used to decide if a sample comes from a hypothesized continuous probability distribution function (PDF). The test is defined as, for a random variable X and samples ($X_1, X_2, \ldots, X_n$) the empirical CDF of X.

Fn(x) is given as

$Fn\ (X)$ = 1/[$Number of observations \leq X$]……………………………………………...3.3.2(a)

The Kolmogorov-Smirnov statistic (Dn) is based on the largest vertical difference between F(x) and $Fn\ (x)$. It is defined as

$Dn = x^{sup}|(x) - F(x)|$…………………….……………………………………………..3.3.2(b)

When comparing different distribution, lower statistics means better fit (Oseni, 2013).

3. Anderson-Darling (A-D) Test

In cases with relatively large extremes, it may be expected the A-D test to be more suitable to select the best-fitted model to data maxima. The A-D test statistic, the quadratic class of the EDF test statistic, is expressed as A as follows:

$A^2 = \sum [(2i - 1) \ln F_X(x_i) + \ln[1 - F_X(x_{n+1}\text{-}i)]/n] - n]$…………………..……………………...3.3.3

Where :$F_x(x_i)$ is the Continuous distribution factors(C DF) of the proposed distribution at x_i, for i = 1, 2,... n. The observed data must be arranged in increasing order, as $X_1 < X_2 < .X_n$ (Ashraful *et al.*, 2018).

In all three tests, a parameter or statistic unique to each method is calculated for the required distribution types and these distributions are ranked based on their parameter values. The best-fit PDF was selected by using Easyfit5.6 professional software. But from different probability distribution method; according to Ashraful *et al.*, (2018) Log Pearson type III and Gumbel's Extreme Value type I Probability distribution frequency methods are most common to analysis hydrological data

B. Design rainfall for shorter duration

The available precipitation data is used to determine the rainfall frequencies for various rainfall durations. Such frequency data for storms of various durations can be represented by Intensity-frequency duration (IDF) curve. i.e An average intensity is ratio of Rainfall to Duration in mm/hr (Subarmaniya, 2008).

Design and analysis of drainage structures require rainfall intensity relationship of shorter duration. Rainfall data for shorter duration is unavailable, appropriate IDF derivation for shorter duration is required. ERA drainage manual, (2013) suggests the following equation to develop IDF curve

$R_{Rt} = \left(\frac{t}{24}\left(\frac{(b+24)^n}{(b+t)^n}\right)\right)$ Or $R_{Rt} = [\frac{R_t}{R_{24}}]$……………………………...…………………...3.4

By re-arranging the above equation $R_t = \left(\frac{t}{24}\left(\frac{(b+24)^n}{(b+t)^n}\right)\right) * R_{24}$..…………………..………..3.4(a)

$I_t = \frac{R_t}{t} = \left[\frac{R_{24} \times (b+24)^n}{24 \times (b+t)^n}\right]$…………………………………….....................................3.4(b)

Where: R_{Rt} = Rainfall depth ratio

R_t = Rainfall depth in a given duration t

R_{24} = 24hr rainfall depth

t=time in hour

n=0.92 (0.78≤n≤1.09)

b=0.3(recommended for east Africa gauges by ERA Drainage manual, 2013).

Where I_t = is intensity in mm/hr at duration time t and $R_{24}$is 24 hour rainfall in mm.

The average values of short duration rainfall depth was analyzed by best probability distribution methods to develop IDF curve study area.

C. Probability Distribution methods

1. Gumbel extreme value type 1 Method

Chow, (1988) has shown that hydrological studies can be expressed by the following equation of the hydrologic frequency analysis

The rainfall (P_T) corresponding of a given return period (T) using the Gumbel's Distribution is given by:

$P_T = \sigma + K.S$……………………………………………………………………….……...3. 5

Where σ = Average Annual Daily Maximum Rainfall

S = Standard Deviation of Annual Daily Maximum Rainfall

K = Frequency Factor given by: $K = -\frac{\sqrt{6}}{\pi}\left[0.5772 + \ln\left(\ln\left\{\frac{T}{T-1}\right\}\right)\right]$ or $K_T = \frac{Y_T - Y_n}{S_n}$

Where: Y_n and S_n are constants obtained from Gumbel distribution table depending up on sample size.

$Y_T = -\ln\{\ln\left[\frac{T}{T-1}\right]\}$ Where: T is return period in year and Y_T is frequency factor constant.

Then, the intensity of rainfall (I_T) is obtained for the return period T from the below equation:

$I_T = \frac{P_T}{T}$…………………………………………………………………………..…3. 6

Where T is duration in hour (Antigha and Ogarekpe, 2013).

2. Log-person type III method.

For this distribution, the first step is to take the logarithms of the hydrologic data, $y = \log_{10} x$. The mean y, standard deviation S_y, and coefficient of skewness(C_s) are calculated for the logarithms of the data. The frequency factor K_T depends on the return period T and the coefficient of skewness C_s. (Chow,1988)

$K_T = z + (z^2 - 1)\, k + \frac{1}{3}(z^3 - 6z)k^2 - (z^2 - l)k^3 + zk^4 + \frac{1}{3}k^5$ ……………………………………..3. 7

where $k = \frac{C_s}{6} = N \frac{\sum(Y - y\prime^3)}{(N-1)(N-2)(\sigma y)^3} * \frac{1}{6}$ and $C_{S=}$ is also obtained from provided table depending up on exceedance probability and return period (T) (Chow, 1988) read from provided table (See Appendix).

$y_T = y + K_T S_y$….or X_T= antlog(Y_T)…………………………………………………….……..3. 8

Then finally $I_T = \frac{X_T}{T}$

By using equations, IDF curve for study site was developed by using selected or best-fit probability density frequency (PDF) distribution method.

Summary of steps used to prepare design rainfall depth and IDF curve are:.

- Checking the quality and consistency of meteorological data(Rainfall data)
- Design rainfall analysis (frequency analysis by using different statistical parameter; ''Easyfit5.1 professional software'' then check the coefficient "Goodness of fit " for each formula and select better value with consideration
- Develop rainfall depth and IDF curve for 10,25,50 and 100 year return period
- Then check the difference between study site IDF curve and ERA IDF curve for study site (rainfall region ''B_2'').

3.7 Delineating Runoff contributing area

The data to be used to delineate runoff contributing area are

- Digital Elevation Model(DEM)
- Drainage map of the town and field surveyed data

Digital elevation model (DEM) is the main type of spatial data that was used to delineate the watershed of study area. DEM for Hossana town was accessed from (USGS earth explore Aster DEM online source) with some adjustment by Arc-Map spatial analyst tool as showed figure 3.10. But DEM for study site (Addis sub-city) was developed from contour map of the sub-city.

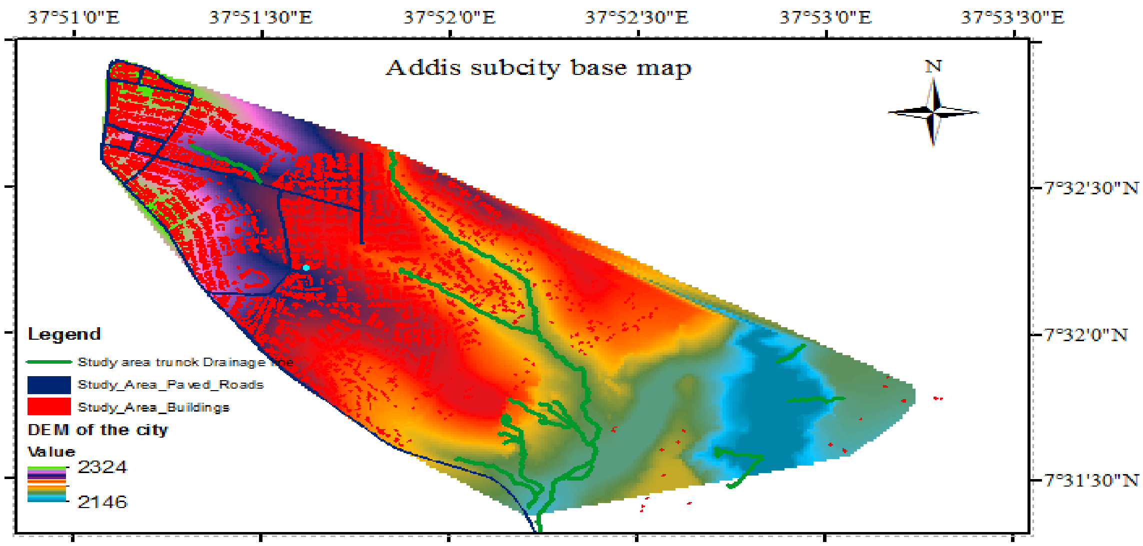


Figure 3.10: DEM containing Base map of Addis sub-city

3.7.1 Catchment area Delineation

Watershed and Stream Network Delineation: The steps to delineate watershed are: preparing DEM data or import raw data, make raw DEM depression less, determine flow direction, computer flow accumulation grid, define stream, segmentation of steam, catchment grid delineation, catchment polygon processing and drainage line processing (Linmei, 2004).The above nine steps are of great importance for hydrological and hydraulic analyses of the natural waterway (See figure 3.11).

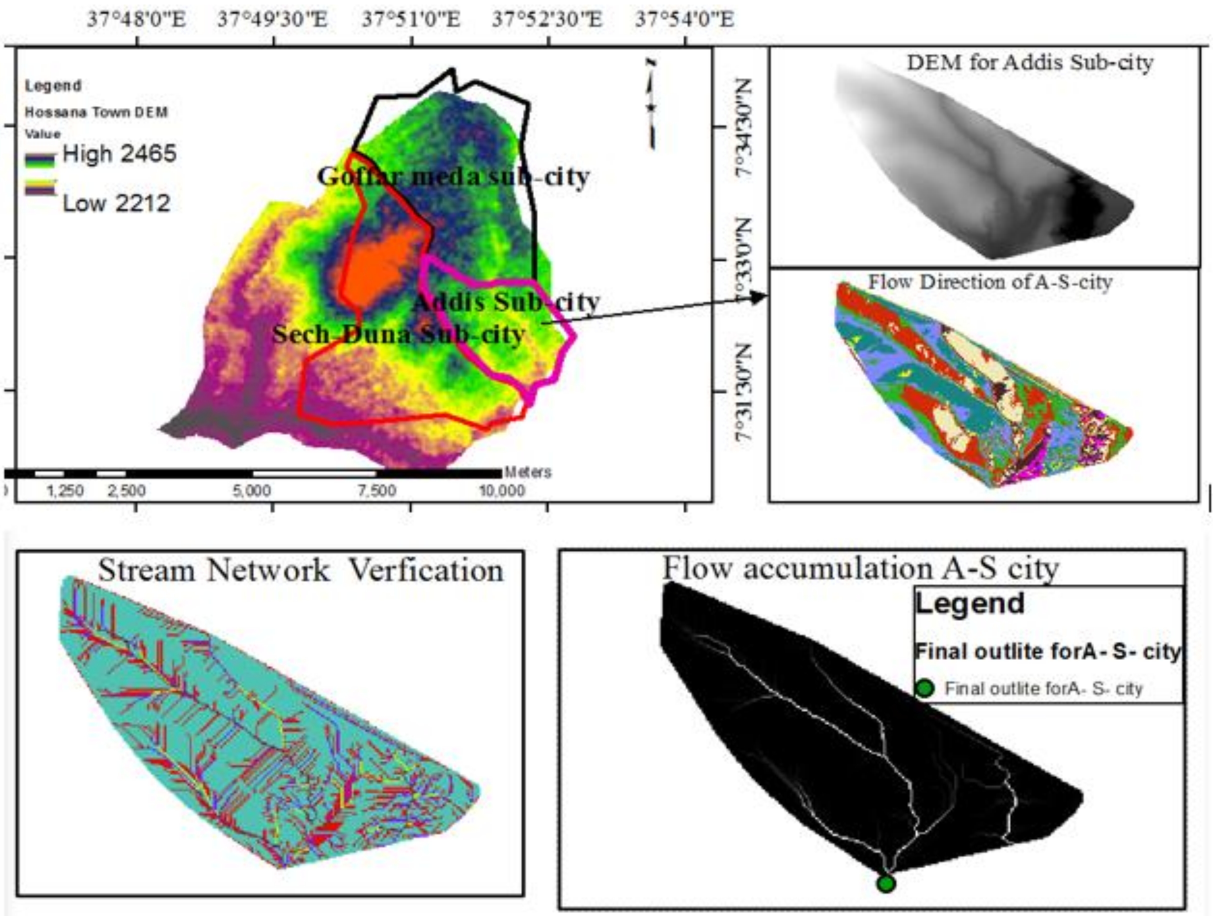


Figure 3.11: Watershed Delineation and stream network verification procedures(Source: Arc GIS analysis)

In generally, catchment area delineation was be taken in to account based on its the class of study area (Hossana town), the flow direction of the catchment, and elevation data used for modeling that obtained from field survey and structural plan map of the Hossana town.

Auto CAD had been very essential application software to obtain area, length, and conduit cross sections of the catchment. In addition, it used to prepare the inputs for Arc GIS and SWMM. The properties of nodes, outlets, and sub-catchment, catchments, and boarders were surveyed from the field and it was used to analysis of the catchment flow direction and drainage system.

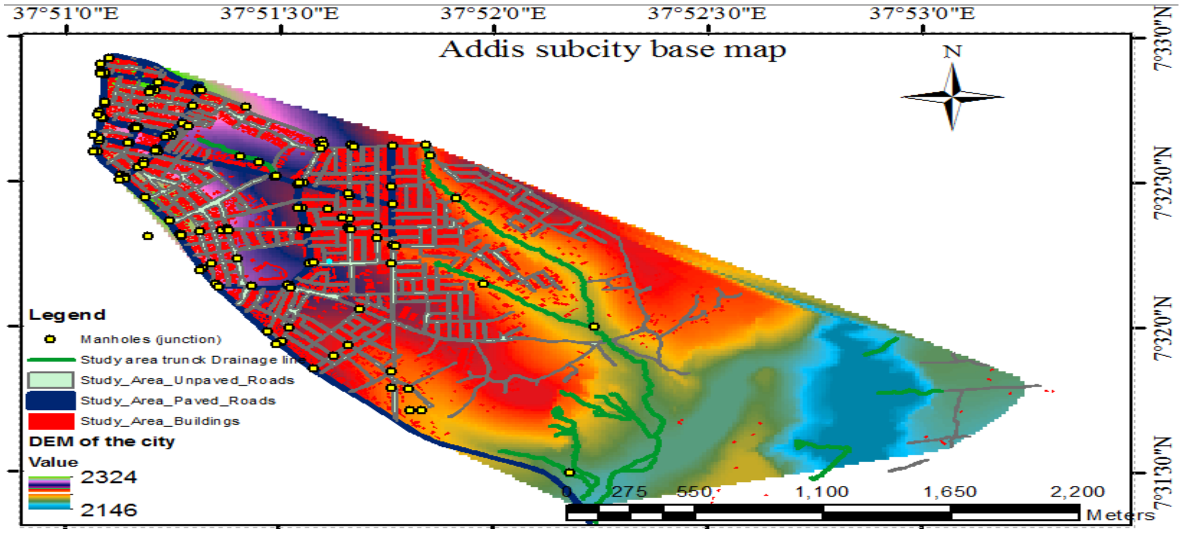


Figure 3.12: Base map and junctions' location of study site (Addis sub-city).

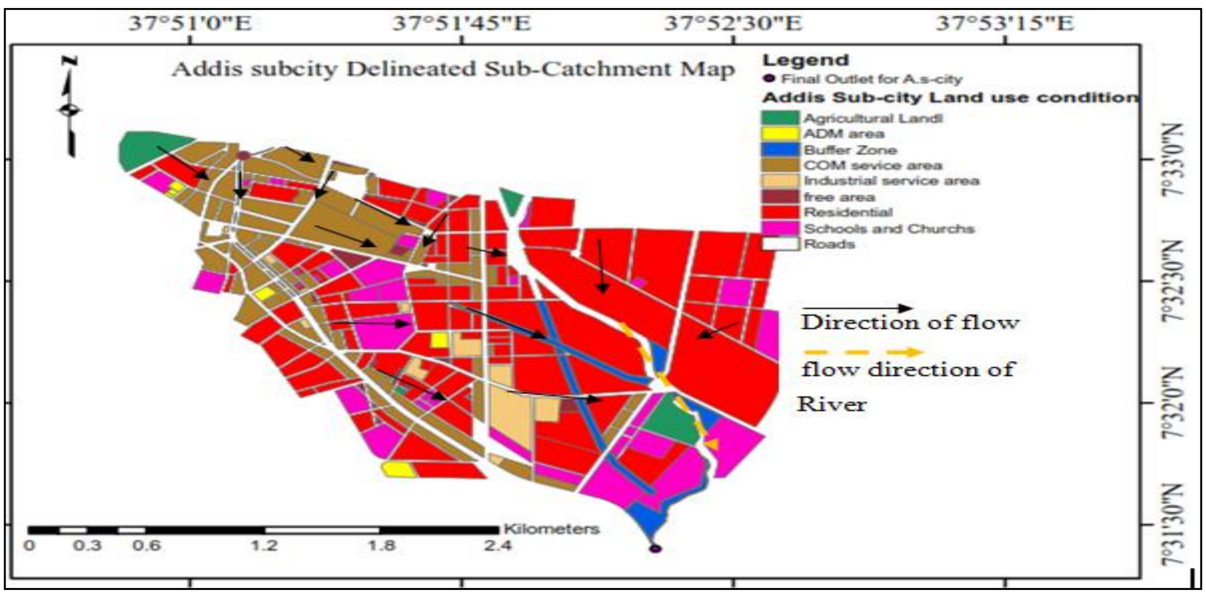


Figure 3.13: Delineated map of Addis sub city watersheds based on class of study area.

ii. Flow Direction Map

Depending up on flow direction and slope of drainage line study area delineated into three main watershed area and each watershed have again contain sub-catchment depending up on the entrance nodes and drainage line availability as shown figure 3.15 and figure 3.16.

1. City center watershed flow from northeast to south west direction which contains seventeen sub catchment, fifty junction points, one outlet point and 49 conduits to discharge the system flow (see figure 3.15b).

2. Arada watershed flows from north east to southeast direction and it contains four sub catchment and twenty one junctions, three outlets and 19 conduits. This catchment was more affected by flash floods due to its steep slope (see figure 3.16c).

3. Lucid watershed flows from northwest to south east direction and it contains five sub-catchment and three junction manholes, one outlets and 3 conduits. This catchments had not adequate drainage facility to convey discharge into Aberariver(outlet for lucid catchment) compared to other watersheds (see figure 3.16d).

Figure 3.14: Addis sub-city City center watershed Google earth image shows the location of sub-catchments and flow direction (Source: - Google earth pro SWMM analysis).

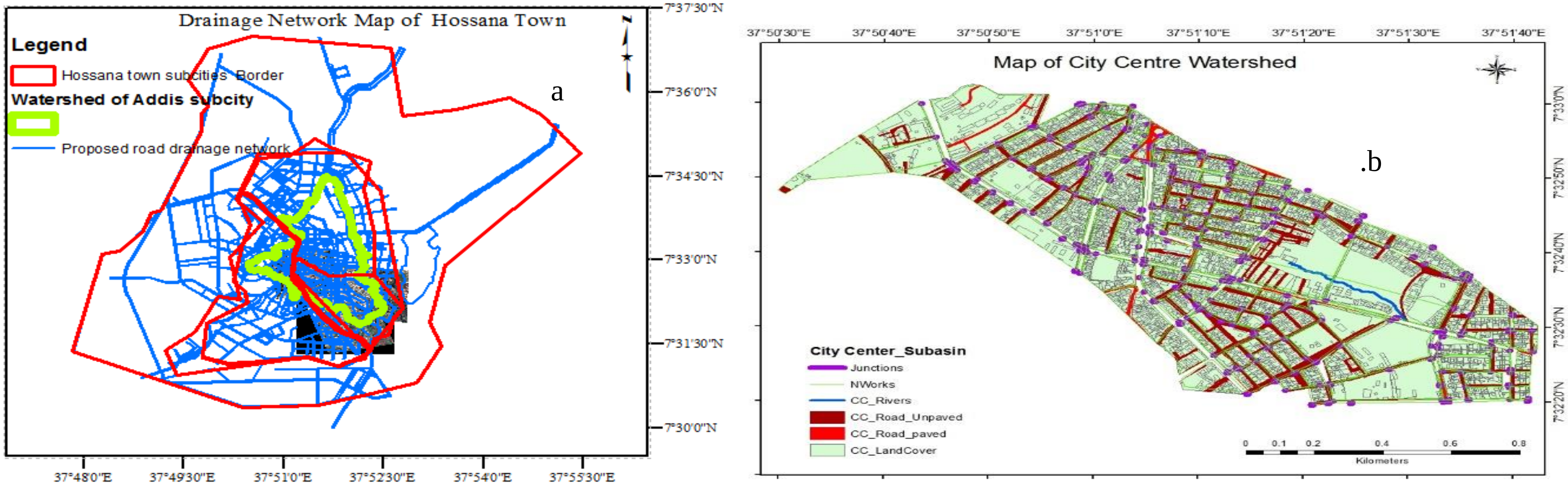


Figure 3.15:a) Delineated watershed of Addis subcity b) city center watershed drainage map

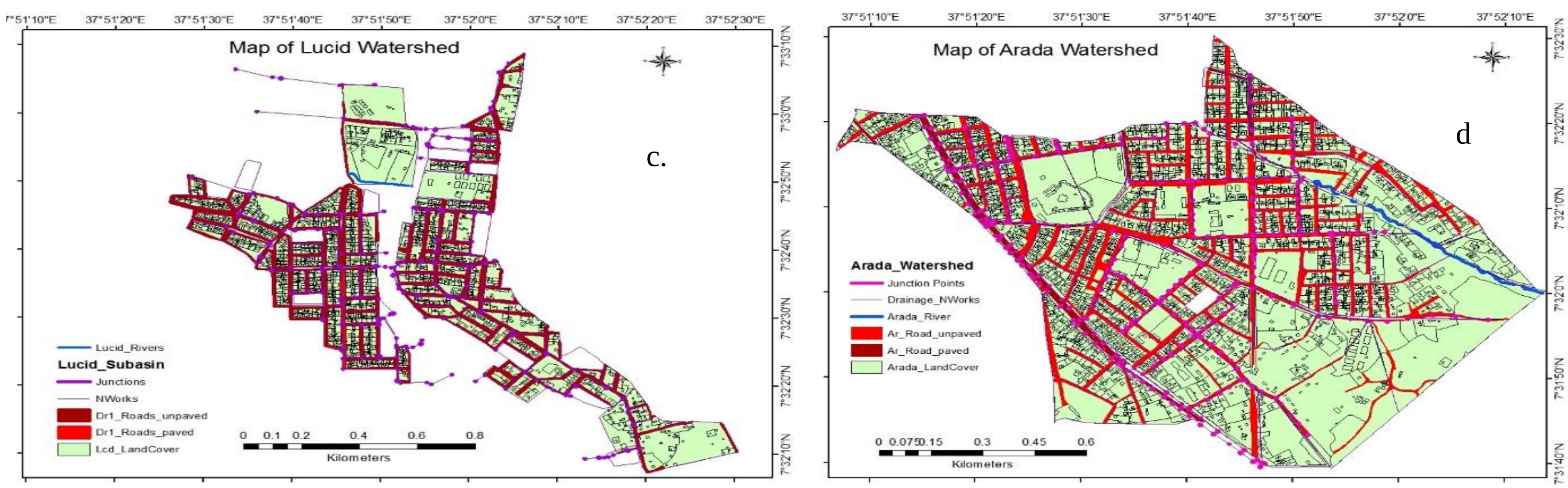


Figure 3.16: c) Arada watershed drainage map and d) Lucid watershed drainage map of Addis sub-city (Source: Hossana city Municipality**)**

3.8 Adequacy of drainage system

3.8.1 Hydrological modeling uses to Estimate peak Discharge

a) Rational Method

The idea behind the rational method is that rainfall intensity (I) which continues indefinitely, the runoff at the outlet of a catchment would increase until the time of concentration (t_c), when the whole catchment is contributing flows to the outlet. This takes the assumption that peak runoff doesn't result from more intense rainfall of shorter duration(UDFCD, 2018). The peak runoff in rational method is given by

Q_p =0.00278 CIA C_f……………………………………………….....................................3.9

Where: Q_P = Peak runoff in is in m^3/s

I = is the average intensity of rainfall in mm/hr for a period of time equal to the time of concentration (t_c) for the drainage area.

A = the area in hectares contributing runoff to the point of design point of the catchment.

C= the coefficient of runoff representing the ratio of peak runoff rate "Q_p" to average rainfall intensity rate "i" for a specified area "A".

C_f= frequency factor (Given in table 3.2)\

Table 3.2: Runoff Frequency Factors for Rational Formula (C_f) (source ERA,2013).

Frequency Factors for Rational Formula (Cf)	
Recurrence interval (years)	Cf
5	1
10	1
25	1.15
50	1.2
100	1.25

Table 3.3: Road drainage system design return periods (Source: ERA drainage manual, 2013).

Location	Frequency (years)
Major system - includes all above and below ground components	50 or 100 years
Minor system components	
Cross drainage excluding fords	50
Diversion channels	50
Road surface drainage including intersections	10
Bridge deck drainage	10
Sediment basins	2
Road surface drainage of pavement	1
Water quality treatment devices	1

The procedures in rational method to determine peak discharge are:-

1. Collect the necessary information for each sub area or catchment such as; drainage area, land use condition, Soil types (its permeability/highly permeable or impermeable),distance from the farthest point of the drainage area to the point of discharge and difference in elevation from the farthest point of the drainage area to the point of discharge.

2. Determine the time of concentration depending on catchment property (length, slope, runoff coefficient and catchment area)

3. Determine the rainfall intensity for the selected recurrence intervals/return period

4. Select the factor of frequency (C_f)

5. Select the appropriate runoff coefficient(C)

6. Compute the design flow (Q_p= 0.00278CIA*C_f)

1. Runoff Coefficient Determination

The runoff coefficient is the most important variable in the rational method of rainfall to runoff transformation. The study area constitutes different land use types ranging from free open area to high-density residential parcels. A weight-age method is employed to obtain the representative runoff coefficient i.e. the individual areas multiplied by their specific runoff coefficient and their values added together and divided by the cumulative area. The following table 3.4.is used to assign runoff coefficient to study area (UDFCD, 2018).

$$C = \sum \frac{CiAi}{Ai} = \frac{\sum CiAi}{AT} \quad \text{……3.9 (a)}$$

Where C is weighted runoff coefficient,

Ci is runoff coefficient for sub area Ai, A_T Total area

Urban drainage and flood control district (UDFCD) develop runoff coefficient for urban and sub urban environment.

Table 3.4: Runoff coefficient for urban and suburban areas (Source: UDFCD, 2018).

Type of Surface	Runoff Coefficient (C)	C_i
Rural Areas		
Concrete, or Hot Mix Asphalt pavement	0.95 - 0.98	0.97
Drives and walks	0.75-0.85	0.8
Roofs	0.75-0.95	0.85
Steep grasses areas (1:2, vert.:horiz.)	0.6 - 0.7	0.65
Turf meadows	0.1 - 0.4	0.25
Forested areas	0.1 - 0.3	0.20
Cultivated fields	0.2 - 0.4	0.30
Urban/Suburban Areas		
Flat residential, @ 30% of area impervious	0.40	0.40
Flat residential, @ 60% of area impervious	0.55	0.55
Moderately steep residential, @ 50% of area impervious	0.65	0.65
Moderately steep built up area, @ 70% of area impervious	0.80	0.80
Flat commercial, @ 90% of area impervious	0.82	0.82

2. Time of Concentration (T_c) Determination

Time of concentration (Tc) is the time required for runoff to travel from the hydraulically remotest point in the watershed to the outlet. Therefore, the time of concentration is the time for a drop of water to flow from the most remote point in the watershed to the point of interest (ERA drainage manual, 2013). Time of concentration was mainly depend upon the slope, the character of land cover, the rainfall pattern of an area and the flow length. Many empirical equations are available for calculating the time of concentration for a selected watershed. For this thesis, time of concentration was calculated according to the UDFCD Drainage manual, (2018) by dividing the waterway into overland flow length and channelized flow lengths, according to the channel characteristics. For urban areas (more than 20 percent impervious ratio), the time of concentration consists of an initial time or overland flow time plus the channelized flow travel time through the storm drain, paved gutter, roadside ditch, and channel conduit (UDFCD drainage manual, 2018). For non-urban areas, the time of concentration consists of an overland flow time, ti, plus the time of travel in a defined drainage path, such as a swale, channel, or stream.

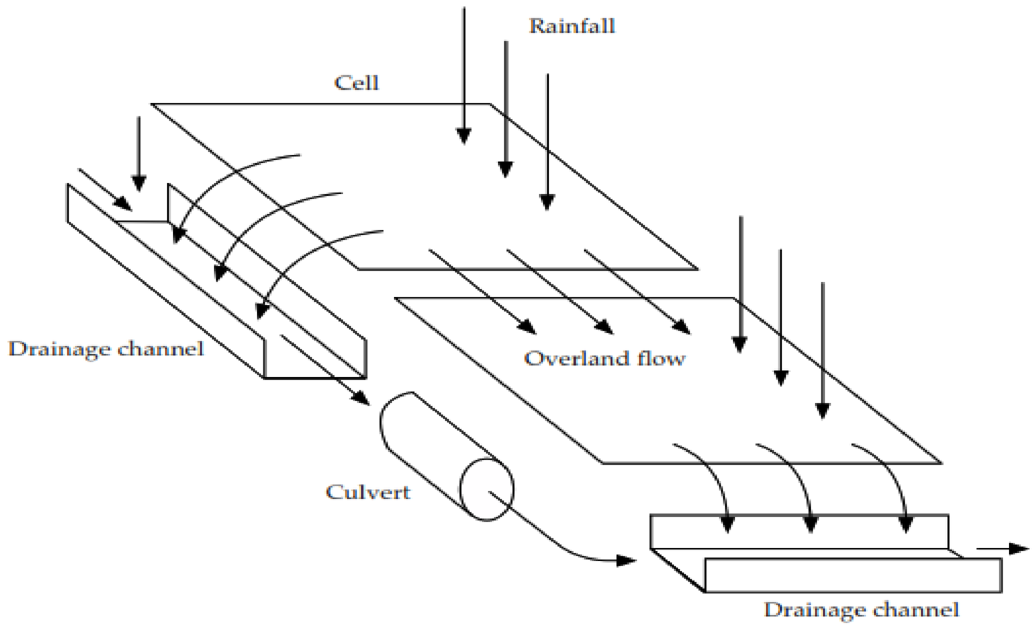


Figure3.17: Channel and over land flow determination in urban catchment (Source: Elmira 2016).

According to UDFCD Drainage manual, (2018) time of concentration for both urban and non-urban areas computed using equation 3.4

Tc=ti+tch……………………………………………………………………………………………………3.9(b)

Where: t_c = computed time of concentration (minutes)

t_i = overland (initial) flow time (minutes)

t_{ch}= channelized flow time (minutes).

Initial or Overland Flow Time: The initial or overland flow time be calculated by using Federal Aviation Administration (FAA) Equation as follows. Initial or overland flow time, on the other hand, will vary with surface slope, depression storage, surface cover, antecedent rainfall, and infiltration capacity of the soil, as well as distance of surface flow (NRCS, 2010).

$t_i = \frac{3.26*(1.1-C)\sqrt{Li}}{So^{0.33}}$………..……………………………………………………………..3.9(b-1)

Where: t_i = overland (initial) flow time (minutes) see table 3.5

C = runoff coefficient (from table 3.4)

L_i = length of overland flow (m)

S_o = average slope along the overland flow path (m/m).

Equation 3.9(b-1) is adequate for distances up to 100m in urban areas and 175m in rural areas. However, in urban it may less than 100m due to effective man-made drainage systems that collect and convey runoff.

Channelized Flow Time: The channelized flow time (travel time) is calculated using the hydraulic properties of the conveyance element. The channelized flow time, T_{ch}, is estimated by dividing the length of conveyance by the velocity.

$$T_{ch} = \frac{Lch}{60*Vt} \qquad \text{...3.9 (b-2)}$$

V_t= Q/A for channel flow ……..…………………………………………….........3.9(b-2.1)

Where: t_{ch}= channelized flow time (travel time, min)

L_{ch} = channel waterway length (m)

S_o = waterway slope (m/m)

V_t = travel time velocity (m/sec)

A =sub catchment area (m^2or hectare)

Q= Discharge of open channel (m^3/sec)

The total time of concentration, tc, is the sum of the initial (overland) flow time, t_i, and the channelized flow (T_{ch}).

Minimum Time of Concentration

Use a minimum t_c value of 5 minutes for urbanized areas and a minimum tc value of 10 minutes for areas that are not considered urban catchment (UDFCD Drainage manual, 2018). However, according to ERA drainage manual, (2013) minimum time of concentration taken 15 minute when calculations result was lesser time of concentration.

Table 3.5: Initial and Channel Time of concentration computed values

Sub catchment ID	weighted runoff coefficient	Average slope area	Longest Channel follow Length (m)	over land flow length (m) L(ov)	velocity of channel	over land time of concentration (Tov)	channel flow time concentration Tch	Total time of concentration	Allowable Tc According to ERA
CCS-1	0.63	0.08	582.30	132.00	5.43	15.92	1.79	17.71	17.71
CCS-2	0.78	0.04	804.50	188.20	7.78	16.64	1.72	18.36	18.36
CCS-3	0.74	0.09	305.60	82.00	8.51	9.61	0.60	10.21	15
CCS-4	0.79	0.09	300.00	135.00	7.92	10.31	0.63	10.94	15
CCS-5	0.83	0.08	382.40	99.00	3.76	7.89	1.69	9.58	15
CCS-6	0.85	0.07	345.60	163.00	3.58	9.75	1.61	11.36	15
CCS-7	0.81	0.04	287.70	95.00	5.70	10.75	0.84	11.59	15
CCS-8	0.76	0.02	448.20	98.00	5.38	15.12	1.39	16.51	16.51
CCS-9	0.78	0.07	311.64	90.00	7.03	9.24	0.74	9.98	15
CCS-10	0.84	0.02	382.50	90.00	6.08	11.93	1.05	12.98	15
CCS-11	0.84	0.05	335.40	122.00	8.53	9.80	0.66	10.45	15
CCS-12	0.79	0.08	133.20	104.00	6.37	9.57	0.35	9.92	15
CCS-13	0.75	0.07	635.00	278.00	5.66	18.23	1.87	20.10	20.10
CCS-14	0.62	0.08	595.50	135.00	5.40	17.01	1.84	18.85	18.85
CCS-15	0.77	0.03	398.30	95.00	8.58	13.65	0.77	14.42	15
CCS-16	0.65	0.04	746.60	150.00	2.34	20.60	5.33	25.93	25.93
CCS-17	0.70	0.05	864.00	85.00	2.34	13.19	6.17	19.35	19.35
ASC-1	0.63	0.04	834.40	178.00	6.02	24.37	2.31	26.68	26.68
ASC-2	0.78	0.04	421.00	130.00	8.58	13.06	0.82	13.88	15
ASC-3	0.64	0.03	531.00	130.00	4.20	20.54	2.11	22.64	22.64
ASC-4	0.53	0.04	830.50	120.00	6.16	23.87	2.25	26.11	26.11
LSC-1	0.65	0.04	709.00	226.00	6.16	24.64	1.92	26.56	26.56
LSC-2	0.80	0.07	360.00	180.00	4.06	12.57	1.48	14.04	15

3. Rainfall intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for a selected return period that is based on a duration equal to the time of concentration (T_c). Once a particular return period has been selected for design and a time of concentration has been calculated for the drainage area, the rainfall intensity can be determined from intensity-duration-frequency curves provided for an area.

b). SCS method to determination peak Discharge

The SCS runoff equation is a method of estimating direct runoff from 24-hour or 1-day storm rainfall and it is potentially more accurate than the rational method (ERA Drainage manual, 2013). The output of SCS method (rainfall, runoff, and Intensity) is used as input for SWMM. For this research 24-hour storm rainfall within 1-hour time of concentration used to compute the rainfall-runoff process by the model.

Procedures to determine Peak discharge by SCS (NRCS) are:

- Runoff contributing area determination:- is determined with the help of Arc GIS software. DEM of catchment was main input for determination of area and the slope.
- Longest length: is also extracted from watershed delineated from DEM
- Time of concentration is computed in by provided equation
- Rainfall excess duration $D=T_c/6$
- Determine time to peak, lag time and base time

$T_p= 0.5D+0.6T_c$,$T_l=0.6T_c$and $T_b= 2.67T_p$

- Peak rate of discharge created by 1mm rainfall excess on whole of the catchment $q_p=(0.21*A)/t_p$where A is area in Km^2
- Prepare Incremental Rainfall Distribution (See Appendix table 7)

Area to point ratio and Rainfall profile in percentage (read from Appendix figure 9)

- Calculate Excess Rainfall and weighed Curve number by using equation 3.10.

According to Banasik *et al.*, (2014) Peak runoff estimated as

$$Q = \frac{(p - I_a)^2}{((P - I_a) + S)} \quad \text{……3. 10}$$

An empirical relationship used in the NRCS method for estimating $I_a$is:

I_a= 0.2S and by substituting into equation 3.10 peak runoff estimated as

$$Q = \left(\frac{(P\text{-}0.2S)^2}{P+0.8S}\right) \text{………for } P \geq 0.2S \text{……3. 10(a)}$$

Where Q = runoff (mm)

P = rainfall (mm)

. S = potential maximum retention after runoff begins (mm)

I_a= initial abstraction including surface storage, interception, evaporation, and infiltration prior to runoff (mm)

$S = \frac{25400}{\text{CNc}} - 254$ ……………………………………………………………………...3.10(b)

CNc=$CN_P + \frac{P_{im}}{100}(98 - CN_P)$…………………………………………………….……3.10(c)

Where: -CNc = composite runoff curve number

CNp = pervious runoff curve number (SUDAS, 2013) (see table 3.6)

- Developing Hydrograph of Direct Runoff from the Drainage Area by using the following equation 3.11

$q_p = \frac{0.21*A*Q_a}{T_p}$…………………….……………………………………………….3. 11

Where: Qa = effective runoff (excess) from equation 3.10

A = watershed area (square Km) and T_p is Time of peak in (hr)

- Next, set up the unit hydrograph ordinates t/Tp and q/qp. The time increment (t) for unit hydrograph ordinates must be the same duration as the period of effective rainfall selected for the rainfall ordinates (D) (San Diego County Hydrology Manual, 2003)

Finally by using triangular hydrograph for each interval of time on can determine the peak discharge (See Appendix table7)

The combined effect of land use land covers and conservation practices on the rainfall runoff relationship represented by runoff curve number. The major factor that determine CN are the hydrologic soil group (HSG), cover type, treatment, hydrologic soil condition and antecedent moisture condition (Banasik *et al*., 2014)

Soil Data

Knowing the hydrological soil group was used to calculate discharge by using Soil Conservation Service (SCS) methods. According to data obtained from Ministry of water, Irrigation and electricity (MoWRE)the most dominant soil types are of catchment, pellicVertisols (96.6%), Eutricnitisols (0.43%) orthicsolonchaks (2.97%).

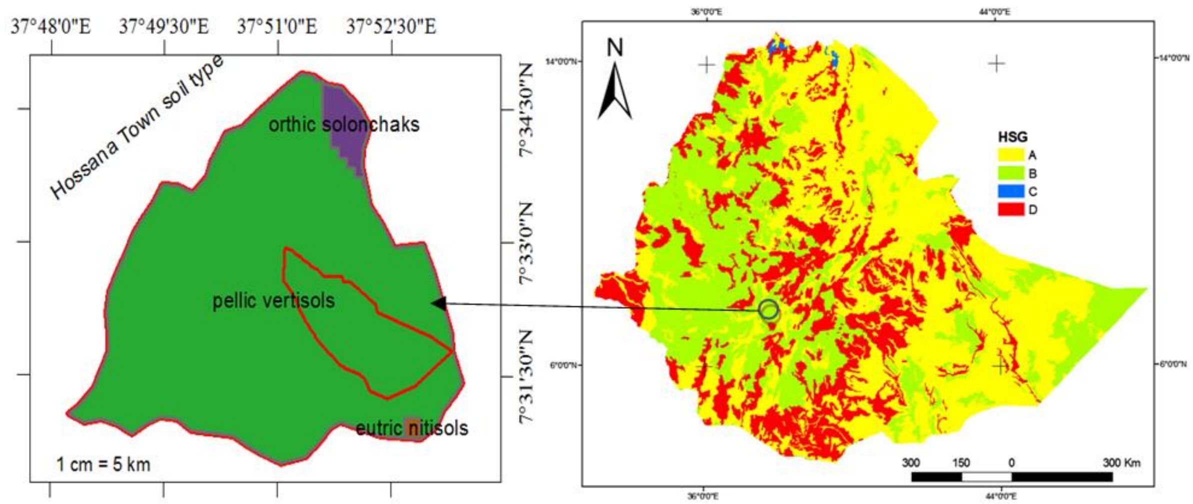


Figure 3.18:Hossana hydrological soil group map (source:-Belete *et al* and MoWR).

However, according to Belete *et al.*, (2013) soil group for study area categorized into hydrological soil group “B” and this soil groups have moderate infiltration rates when thoroughly wetted, consisting chiefly, and moderate rate of water transmission. Curve number adjustment for watershed soil moisture status are made according to three classes of antecedent moisture conditions (AMC) namely, the below average (AMC I), average (AMC II) and above average (AMC III).

Table 3.6:.Runoff curve number for selected agricultural, suburban, and urban land use (antecedent moisture condition II, I_a=0.2S).

AMC	Total 5 day antecedent rainfall (mm)	
	Dormant season	Growing season
I	Less than 12	Less than 36
II	12 to 28	36 to 53
III	Over 28	Over 53

Land use Description		Hydrologic Soil Group			
		A	B	C	D
Fair condition: grass cover on 50% to 75% of the area		49	69	79	84
Commercial and business area (85% impervious)		89	92	94	95
Industrial districts (72% impervious)		81	88	91	93
Residential					
Average lot size	Average % impervious				
1/8 acre or less	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
Paved parking lot, roofs, driveways, etc		98	98	98	98
Streets and roads:					
Paved with curbs and storm sewers		98	98	98	98
Gravel		76	85	89	91
dirt		72	82	87	89

3.8.2 Hydraulic Analysis

Manning's Equation: can be used for uniform flow in a pipe, and stream channel, but the Manning's roughness coefficient needs to be considered variable, dependent upon the depth of flow. The Manning's equation issued for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for the flow of a specified depth in a pipe or open channel of known diameter and or stream channel cross-section. Manning's equation is applicable for a constant flow rate of water through a channel with a constant slope, size and shape, and roughness.

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \qquad \text{--} 3.\ 12$$

Where, Q = the volumetric flow rate passing through the channel reach in m^3/sec.

A = the cross-sectional area of flow normal to the flow direction in m^2

S = the bottom slope of the channel in m/m (dimensionless).

n = a dimensionless empirical constant called the Manning roughness coefficient.

R = the hydraulic radius = A/P.

P = the wetted perimeter of the cross-sectional area of flow in m.

Manning uniform flow channel equation was mainly dependent on channel cross section and the manning roughness coefficient (Chow, 1988).

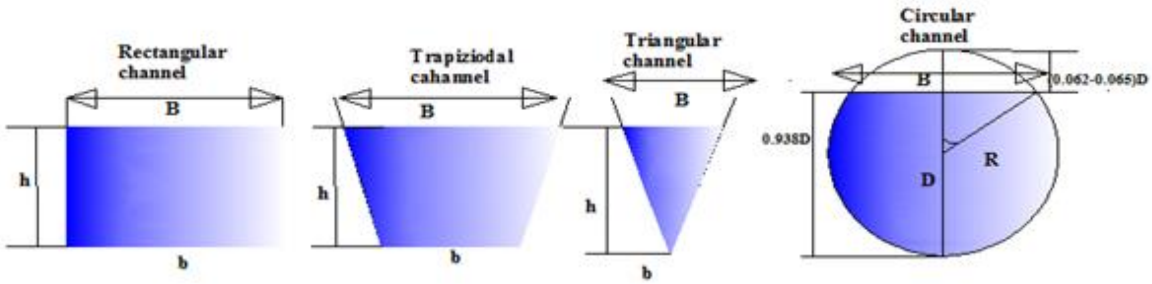


Table 3.7: Geometric property of different channel sections

Shape	Rectangular	Trapezoidal	triangular	Sem-circular
Flow area	bh	(b+mh)*h	mh2	1/8(θ-sin θ)D^2
Wetted perimeter	b+2h	b+2h$\sqrt{1+m^2}$	2h$\sqrt{1+m^2}$	1/2 θD
Hydraulic radius	$\frac{bh}{b+2h}$	$\frac{(b+mh)h}{b+2h\sqrt{1+m^2}}$	$\frac{mh}{2\sqrt{1+m^2}}$	$\frac{1}{4}\left[1-\frac{\sin\theta}{\theta}\right]*D$
Top width	b	b+mh	2mh	$\left(\sin\frac{\theta}{2}\right)*\mathrm{D}$
Hydraulic depth	h	$\frac{(b+mh)h}{b+2mh}$	½*h	$\frac{\theta-sin\theta}{sin\theta/2}*\frac{D}{8}$

3.9 Modeling Using EPA SWMM 5.1

3.9.1 Model Description

The runoff component of SWMM operates on a collection of sub catchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage or treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each sub catchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps (Rossman, 2016).

3.9.2 Flow Routing

Flow routing within a conduit link in SWMM is governed by the conservation of mass and Momentum equations for gradually varied, unsteady flow (i.e. the Saint Venant flow equations). The SWMM user has a choice on the level of sophistication used to solve these equations: (Rossman, 2016).

According to Rossman, (2016) SWMM 5.1.03 flow routing algorithms isdivided into three

Steady Flow

- simple hydrograph translation
- applicable only to branched networks

Kinematic Wave

- gravity force balanced by friction force
- attenuated and delayed outflow due to channel storage
- applicable only to branched networks

Dynamic Wave

- solves full St. Venanteqautions
- accounts for channel storage, backwater effects, pressurized flow, and reverse flow
- applicable to any network layout
- requires smaller time step

Governing Equations of the model

The conservation of mass and momentum for unsteady free surface flow through a channel or pipe are known as the St. Venant equations and can be expressed as:

Continuity Equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial t} = 0 \qquad \text{3. 13}$$

Momentum equation

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial H}{\partial x} + gASf = 0 \qquad \text{3. 14}$$

Combine equation

$$\frac{\partial Q}{\partial t} = 2U\frac{\partial A}{\partial t} + U^2\frac{\partial A}{\partial x} - gA\frac{\partial H}{\partial x} - gASf \qquad \text{3.15}$$

Where X = distance (m) ; t = time (sec) A = flow cross-sectional area (m^2)

Q = flow rate (CMS)

H = hydraulic head of water in the conduit or the sum of conduit invert elevation and conduit water depth(m)

S_f= friction slope (head loss per unit length) $S_f = (n)^2 \frac{Q|U|}{AR^{4/3}}$

g = acceleration of gravity (m/sec^2)

n = the Manning roughness coefficient ($sec/m^{1/3}$)

R = the hydraulic radius of the flow cross-section (m)

U = flow velocity, equal to Q/A (m/sec)

The derivation of these equations the assumptions on which they are based are:

- flow is one dimensional
- pressure is hydrostatic
- the cosine of the channel bed slope angle is close to unity (Rossman, 2016)

3.9.3 SWMM model input data and parameters

1. **Rain Gages** supply precipitation data for one or more sub-catchment areas in a study region. The rainfall data can either a user-defined time series or come from an external file. It can be also cumulative volume rain depth or rainfall intensity.

Table 3.8: Rainfall intensity hyetograph for differentyear return period by using SCS method.

Time of beginning (hr)	10 year RT Intensity (mm/hr)	25 year RT Intensity (mm/hr	50 year RT Intensity mm/hr)	100 year RT Intensity (mm/hr)
00	0.0	0.0	0.0	0.0
0.083	16.50	20.06	21.13	23.60
0.167	22.55	27.40	28.88	32.26
0.25	23.20	28.19	29.71	33.18
0.333	23.71	28.82	30.37	33.93
0.417	24.62	29.92	31.53	35.23
0.500	33.52	40.74	42.93	47.96
0.583	85.10	103.42	108.98	121.75
0.667	40.73	49.50	52.17	58.30
0.750	25.40	30.87	32.53	36.34
0.833	23.84	28.98	30.54	34.11
0.917	23.45	28.51	30.04	33.56
1.000	22.81	27.72	29.21	32.63

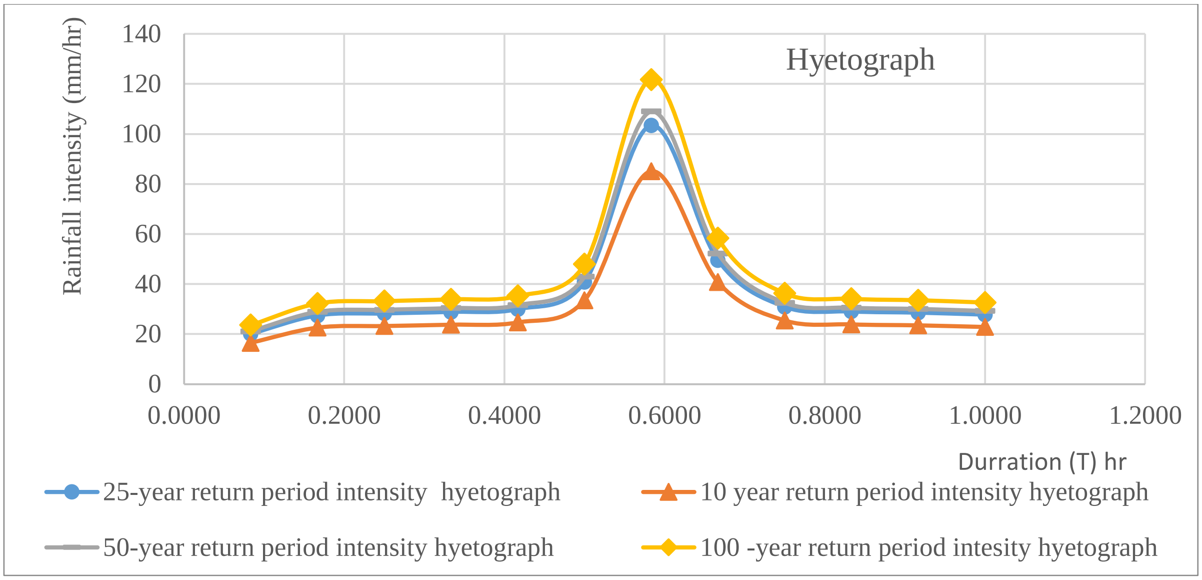


Figure3.19: Effective rainfall intensity for different year return period by using SCS method.

2. Sub catchment are hydrologic units of land whose topography and drainage system elements direct surface runoff to a single discharge point and impervious properties of the catchment. It composes mainly waterway width, average slope, and catchment property as input for study site(Wanniarachchi *et al*., 2012).Sub-catchment area, width, overland length and channel length were computed by Arc-GIS extension tool and Auto-CAD see table 3.12.

2.1. Imperviousness:-The percent imperviousness of a sub catchment is another parameter that can, in principle be measured accurately from aerial photos or land use maps. In practice, unless impervious layers are included in a GIS representation of the basin, such work tends to be tedious, and it is common to make careful measurements for only a few representative areas and extrapolate to the rest. Runoff volume and flow rates are strongly sensitive to estimates of imperviousness; hence, care should be taken in imperviousness estimates. (Rossman, 2016).

One approach to estimating impervious area across large areas with multiple land uses is to associate a percent impervious area with each category of land use. The imperviousness was calculated for each sub-catchment according to the percentage of different surfaces; see Equation below (Rossman, 2016).

$$\varphi = \frac{A1 * \varphi 1 + A2 * \varphi 2 + \cdots + An * \varphi n}{A1 + A2 + \cdots + An} \quad \ldots\ldots 3.16$$

Where φ =imperviousness of the whole sub-catchment, φi=imperviousness of each type of surface, Ai =area of each surface

Table 3.9: Percentage Imperviousness of land use land cover of the catchment

Land Use condition	Percent Impervious Area
Commercial	56
Concrete and asphalt surface	80
Gravel path with undeveloped parts of soil	20
Gravel road, Sharply slope mountainous area no vegetation cover	40
High density residential	51
Industrial	76
Institutional	34
Low density residential	19
Medium density residential	38
Open Urban Land	11
Paved surface with grave joint	70
Roofs	90

2.2. Width:-The sub-watershed width is an estimate of the average distance that water has to travel before reaching a channel (Jing, 2015). This parameter is important in the modeling of flood peaks and for calibration. There are different approaches was used to estimating the sub-watershed width, but for regular shaped urban catchment width of water way was computed by dividing catchment area with overland flow (Rosman, 2016).

2.3. Infiltration:- Green-Ampt equation infiltration equation is used for this study to because this method received considerable attention to calculate infiltration urban catchments soils in recent years (Rosman, 2016). Infiltration parameters of different soils were given.

Table 3.10: Infiltration capacity parameters for different types of soils (Source: Rossman,2016)

Soil Class	Porosity, φ	Effective Porosity, φe	Wetting Front Suction Head ψs (mm)	Hydraulic Conductivity, Ks (mm/hr)
Sand	(0.374–0.500)	(0.354–0.480)	(8.85–250)	118.5
Loamy sand	(0.363–0.506)	(0.329–0.473)	13.5-275	29
Sandy loam	(0.351–0.555)	(0.283–0.541)	25.4-425	10.5
Loam	(0.375–0.551)	(0.334–0.534)	13-585	3.25
Silt loam	(0.420–0.582)	(0.394–0.578)	28-940	6.5
Sandy-clay-loam	(0.332–0.464)	(0.235–0.425)	43-1000	1.5
Clay loam	(0.409–0.519)	(0.279–0.501)	47-875	1
Silty clay-loam	(0.418–0.524)	(0.347–0.517)	55-1000	1
Sandy clay	(0.370–0.490)	(0.207–0.435)	40-1300	0.5
Silty clay	(0.425–0.533)	(0.334–0.512)	60-1300	0.5
Clay	(0.427–0.523)	(0.269–0.501)	60-1500	0.25

3**. Junctions** are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connection fittings. The main properties of junctions that surveyed from the field are; co-ordinates, invert elevation, and maximum depth of channel. See table 3.13.

4**. Conduits** are pipes or channels that move water from one node to another in the conveyance system. Shape, length, and cross-section property of the conduit are the main inputs for the model that surveyed from the field. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries. See table 3.14.

5**. Outfalls or outlets** are terminal nodes of the drainage system used to define final downstream boundaries under dynamic wave flow routing.

In general, the total catchment divided into the three sub catchments and their property were identified first by field survey, study site investigation, and based on map of the town

Table 3.11: Summary of sources for SWMM inputs parameters

Input parameters of model	Source
Assigned rain-gage Rainfall data	RF stations
Outlet node and sub-catchment property	Field survey
assigned land use conditions	Google earth
tributary surface area	Arc-GIS
Imperviousness ratio %	user manual
Slope of the catchments	Field survey
characteristic width of overland flow (m)	Field survey
Manning's n for overland flow on both pervious and (n)	Field survey
Impervious areas (%)	M-Excel
depression storage in both pervious and impervious areas	Default
percent of impervious area with no depression storage	Default

Figure 3.20: surveying the elevation and location junctions by using Theodolite

Figure 3.21: Surveying Nodes dimensions and channel cross sections by using tape meter

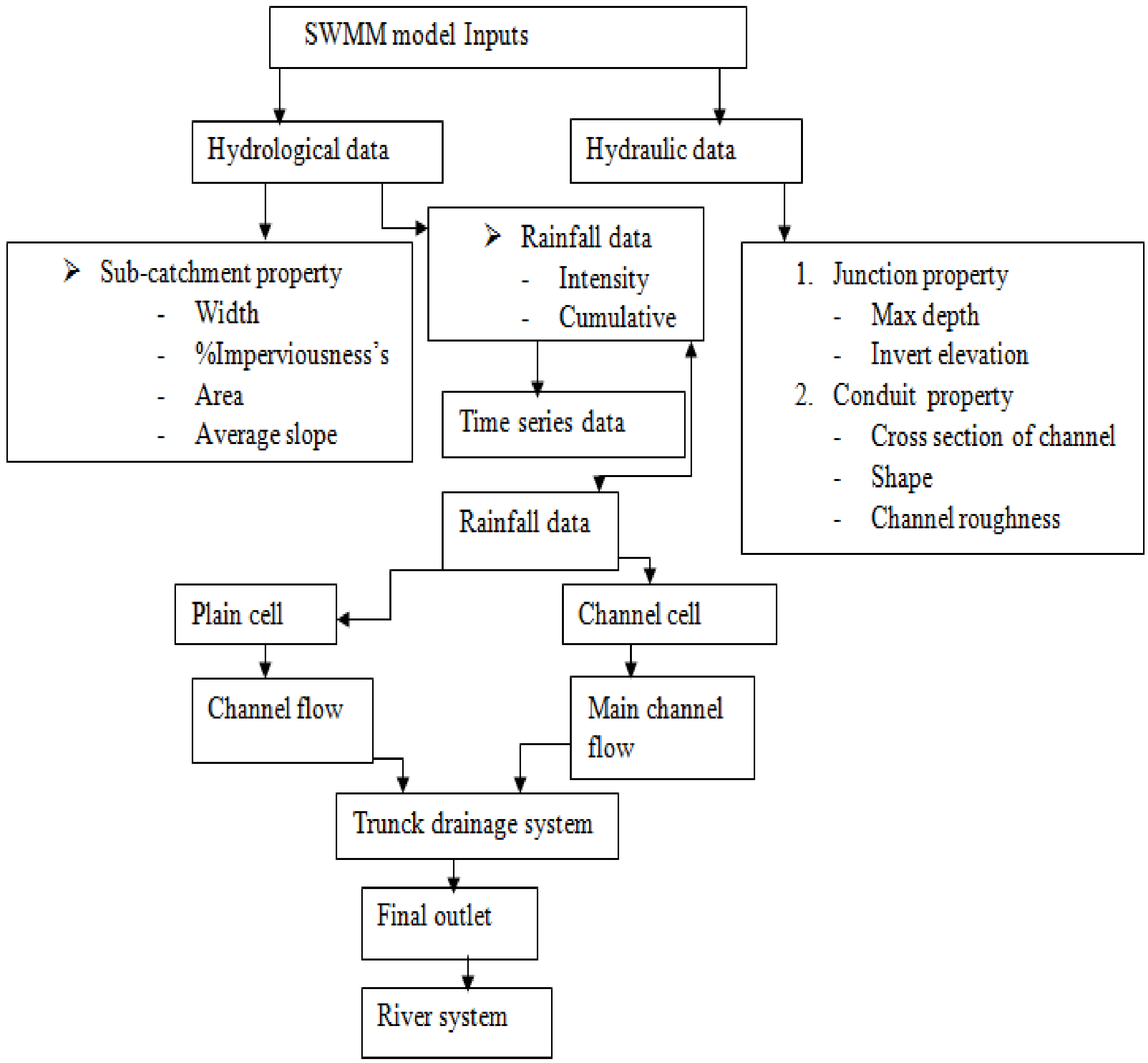


Figure 3.22: Discharge flow chart in EPA SWMM model.

Table 3.12: Sub-catchment property inputs parameters

Addis sub-city catchment property							
Sub catchment ID	city center (CC) sub catchment property						
	Area (ha)	sub catchment width (m)	overland flow length (m)	Imperviousness	Slope	Outlet	Roughness
SC. CC-1	17.49	1325.26	132	53.75	0.083	J-1	0.012
SC. CC-2	37.76	1464	258	50.35	0.039	J-2	0.013
SC. CC-3	5.768	703.42	82	55.07	0.085	J-3	0.011
SC. CC-4	5.41	400.71	135	55.25	0.09	J-4	0.013
SC. CC-5	2.61	263.722	99	57.88	0.08	J-6	0.012
SC. CC-6	5.56	341.33	163	70.28	0.070	J-46	0.0118
SC. CC-7	3.12	328.67	95	56.65	0.04	J-12	0.014
SC. CC-8	6.32	645.40	98	58.58	0.022	J-24	0.011
SC. CC-9	3.617	401.94	90	63.8	0.073	J-52	0.0115
SC. CC-10	3.3	365.961	90	58.97	0.018	J-56	0.0114
SC. CC-11	7.89	646.73	122	51.61	0.051	J-54	0.0117
SC. CC-12	3.82	368.12	104	61.09	0.076	J-16	0.018
SC. CC-13	12.36	444.77	278	52.26	0.072	J-50	0.0115
SC. CC-14	18.48	1369.57	135	41.18	0.076	J-37	0.0112
SC. CC-15	18.07	1184.32	152	28.35	0.027	J-34	0.012
SC. CC-16	24.01	1600.72	150	52.79	0.04	J-50	0.013
SC. CC-17	10.73	1262.57	85	54.73	0.046	J-38	0.011

Table 3.13: Junctionof the city center watershed (Source: field survey)

ID	Easting (lat)	Northing(long)	Invert elv-n	max D
J-16	373526	833906	2295	0.6d
J-57	373555	833551	2295	0.4
J-15	373770	833817	2293	1
J-5	373142	834238	2325	0.2
J-65	373246	834007	2317	0.4
J-61	373278	834399	2319	0.4
J-64	373267	834136	2307	0.9
J-29	373296	834464	2307	0.4
J-6	373273	834000	2317	0.4
J-58	373286	834114	2307	0
J-63	373299	834215	2308	0.2
J-52	373427	834065	2300	0.7
J-55	373356	834356	2330	0.7
J-7	373392	833960	2324	0.2
j-53	373314	834494	2326	0
j-46	373526	834341	2318	0.7
J-47	374280	833932	2285	0.2

ID	Easting (lat)	Northing(long)	Invert elv-n	max D
J-1	372618	83464	2365	0.4
J-2	372848	834475	2347	0.2
J-3	372890	834418	2349	0.2
J-4	372952	834547	2348	0.2
J-9	373125	834242	2322	0.2
J-10	373221	834415	2332	0.2
J-8	373104	834029	2324	0.6
J-66	373302	833836	2309	0.4
J-12	373271	833974	2316	0.9d
J-67	373373	833740	2308	0.4
J-54	373397	833945	2306	0.7
J-25	373525	833531	2323	0.4
J-17	373384	833744	2320	0.4
J-13	373437	833807	2318	0.4

Table 3.14: Conduit property of city center watershed (source: field survey)

Conduit ID	Links ID		Length(m)	conduit depth	Conduit width	shape
	From	to				
CC-1	J1	J8	804.5	0.4	0.6	Trapezoidal
cc-2	J2	J8	582.3	0.2	0.4	Trapezoidal
CC-3	J9	J8	192	0.2	0.4	rectangular
CC-4	J3	J8	305.6	0.2	0.4	rectangular
CC-5	J10	J9	202.6	0.2	0.4	rectangular
CC-6	J4	J10	300	0.2	0.4	rectangular
CC-7	J5	J64	142	0.1	0.2	rectangular
CC-8	J61	J64	233.12	0.4	0.6	Trapezoidal
CC-9	J8	J12	190.3	0.4	0.6	Trapezoidal
CC-10	J29	J64	124	0.4	0.6	Trapezoidal
CC-11	J65	J64	382.4	0.4	0.6	Trapezoidal
CC-14	J53	J46	332	0.4	0.2	rectangular
CC-15	J55	J24	427	0.4	0.2	rectangular
CC-16	J29	J24	448.2	0.6	0.4	rectangular
CC-17	J47	J24	311.64	0.4	0.6	Trapezoidal
CC-18	J63	J52	382.5	0.8	0.6	rectangular
CC-19	J58	J52	298.82	0.8	0.6	rectangular
CC-20	J6	J64	109.35	0.6	0.4	Trapezoidal
CC-21	J7	J56	335.4	0.2	0.4	rectangular
CC-22	J64	J56	255.4	0.7	0.5	rectangular
CC-23	J66	J12	176.36	0.4	0.6	Trapezoidal
CC-24	J17	J16	263.63	0.4	0.6	Trapezoidal
CC-25	J12	J54	88	0.4	0.6	Trapezoidal
CC-26	J57	J15	133.2	0.2	0.3	Trapezoidal
CC-27	J12	J54	221	0.4	0.2	rectangular
CC-28	J15	J16	635	0.7	0.5	rectangular
CC-29	J54	J16	313.2	0.7	0.5	rectangular
CC-30	J53	J14	264.9	0.1	0.1	rectangular
CC-31	J51	J23	864	0.1	0.2	rectangular
CC-32	J56	J50	746.6	0.7	0.5	rectangular
CC-34	J15	J50	123.85	1.2	1	rectangular
CC-33	J50	J30	398.3	1	1	rectangular
CC-35	J51	J49	243.3	0.2	0.3	rectangular
CC-36	J25	J17	438	0.4	0.6	Trapezoidal
CC-37	J14	J50	414	0.2	0.4	rectangular
CC-38	J18	J37	595.5	0.2	0.4	rectangular
CC-39	J30	J37	150	1.2	1	rectangular

4. RESULTS AND DISCUSSIONS

4. 1 Intensity Duration Frequency (IDF) Curve

4.1.1 Rainfall data analysis

The result obtained from Easyfit5.6 professional software show that, the Gumbel extreme value type I was selected as best –fit probability density function (PDF) to compute maximum design discharge.

Table 4.1: Best-fit PDF selection by using Easyfit5.6. Professional software.

Goodness fit test methods	Gumbel extreme value type 1 (G)			Average rank		Log person type 3(LP)		
	p-value	statics	rank			p-value	statics	Rank
Chi-square	0.4876	1.43	23			0.45394	1.5796	28
Anderson-Darling	-	0.3355	10			-	0.32776	9
Kolmogorov-Smirnov	0.8284	0.112596	11			0.83906	0.11157	10
Conclusion of test	Chosen as best-fit PDF by comparing obtained values			1	2	Best-fit, but little less than G for runoff computation		

The obtained value was calculated using daily maximum rainfall of Hossana town to determine design rainfall for upcoming floods. The computed values were show that, no far difference was observed between Log Pearson type III distribution and Gumbel extreme value type 1-distribution methods. According to Ashraful *et al*., (2018) Gumbel extreme value type1was practically most preferable to determine maximum floods. Also the calculated values in table 4.2 show that Gumbel extreme value type1 gives maximum design rainfall than Log person type III distribution methods. To reduce the problems associated to flooding, maximum design rainfall Gumbel extreme value type1was selected to prepare IDF curve and to determine maximum discharge for study area.

Table 4.2: Comparison of Log Pearson type III and Gumbel maximum value distribution design maximum rainfall of Hossana town.

Log person type III		Gumbel extreme value type1 distribution	
Return period(T)	Rain fall(mm)	Return period(T)	Rain fall(mm)
2	49	2	50
5	62	5	64
10	70	10	74
15	74	15	79
25	81	25	86
50	90	50	95
100	98	100	103
200	107	200	112

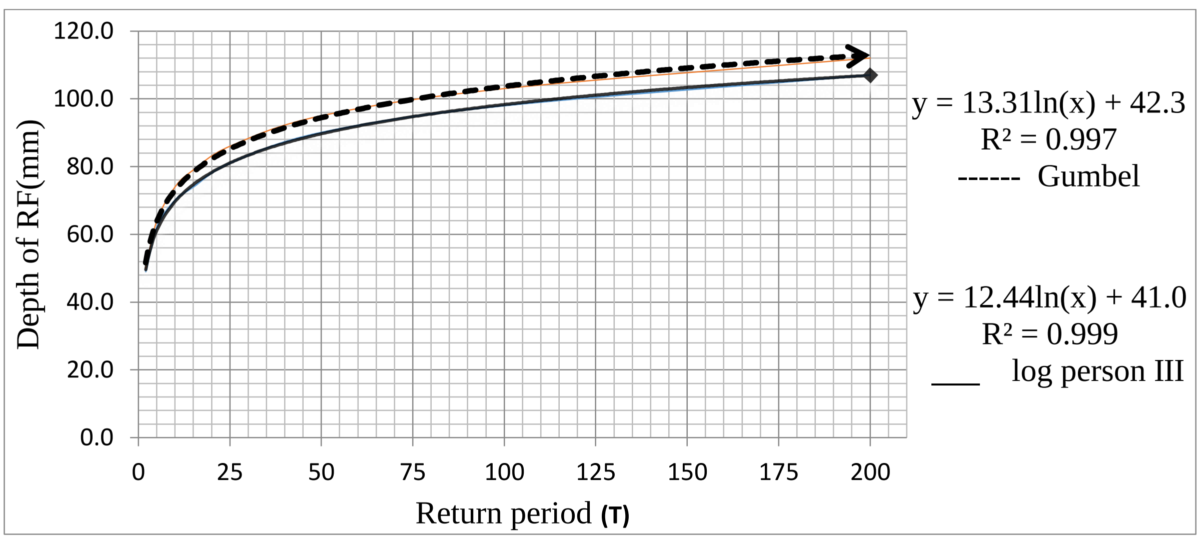


Figure 4.1: Comparison of Log Pearson type III and Gumbel extreme value distribution maximum daily rainfall depth (mm**).**

The design rainfall depth calculated for study area was compared to ERA recommended rainfall depth for study site region ("B_2") to examine the existing drainage system hydraulic performance of the town. Table 4.3. show that the rainfall depth series calculated for study site was less ERA calculated rainfall depth series (the detail was shown Appendix table 4).

Table 4.3: Comparison of rainfall depth ERA and Hossana town rainfall depth (Source: ERA, Drainage manual, 2013 and Own computation)

Return periods in years	2	5	10	25	50	100	200
RR "B_2" X_T ERA (RF depth)	55.26	69.95	79.68	92.03	101.29	110.61	120.07
Study site RF X_T(Hossana)	50	64	74	86	95	103	112

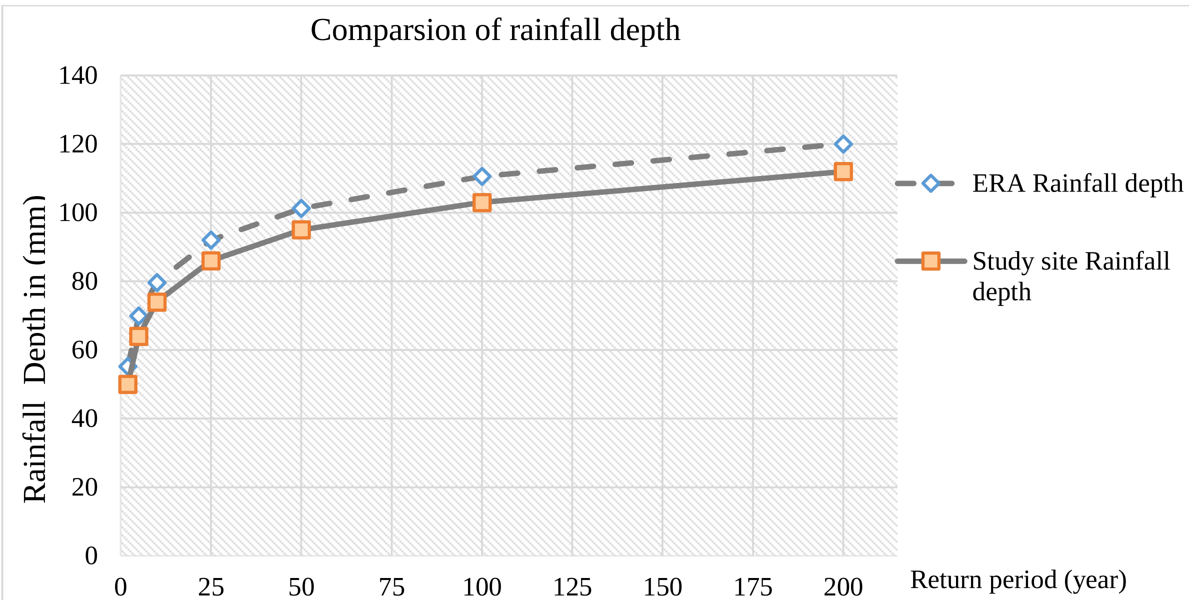


Figure4.2**:** Comparison of rainfall depth ERA with Hossana town rainfall depth.

4.1.2 Short Duration Design Rainfall

According to ERA, drainage manual design rainfall can be calculated according to equation 3.3. for different time interval. However, for this study design rainfall was calculated for 0.25hr, 0.5hr, 0.75hr, 1hr, 1.5hr, 2hr, 2.5hr and 3hr as shown table 4.4.

Table 4.4: Short Duration Rainfall depth for different time interval

Year	A.A. max daily RF	15min	30min	45min	60min	90min	120min	150min	180min
1990	62.3	21.2	30.0	35.0	38.4	42.7	45.4	47.4	48.9
1991	61.2	20.8	29.5	34.4	37.7	41.9	44.6	46.5	48.0
1992	39.4	13.4	19.0	22.2	24.3	27.0	28.7	30.0	30.9
1993	63.9	21.7	30.8	35.9	39.4	43.8	46.6	48.6	50.1
1994	37.9	12.9	18.3	21.3	23.4	26.0	27.6	28.8	29.7
1995	51.3	17.4	24.7	28.9	31.6	35.1	37.4	39.0	40.2
1996	42.3	14.4	20.4	23.8	26.1	29.0	30.8	32.2	33.2
1997	94.1	32.0	45.3	52.9	58.0	64.5	68.6	71.6	73.8
1998	37.1	12.6	17.9	20.9	22.9	25.4	27.0	28.2	29.1
1999	35.4	12.0	17.0	19.9	21.8	24.3	25.8	26.9	27.8
2000	40.6	13.8	19.6	22.8	25.0	27.8	29.6	30.9	31.9
2001	50.4	17.1	24.3	28.3	31.1	34.5	36.7	38.3	39.5
2002	67.3	22.9	32.4	37.9	41.5	46.1	49.1	51.2	52.8
2003	49.7	16.9	23.9	28.0	30.6	34.1	36.2	37.8	39.0
2004	39.4	13.4	19.0	22.2	24.3	27.0	28.7	30.0	30.9
2005	59.0	20.1	28.4	33.2	36.4	40.4	43.0	44.9	46.3
2006	73.9	25.1	35.6	41.6	45.5	50.6	53.9	56.2	58.0
2007	47.8	16.2	23.0	26.9	29.5	32.8	34.9	36.4	37.5
2008	52.2	17.7	25.1	29.4	32.2	35.8	38.1	39.7	41.0
2009	46.9	15.9	22.6	26.4	28.9	32.1	34.2	35.7	36.8
2010	33.5	11.4	16.1	18.8	20.6	23.0	24.4	25.5	26.3
2011	37.2	12.6	17.9	20.9	22.9	25.5	27.1	28.3	29.2
2012	45.0	15.3	21.7	25.3	27.7	30.8	32.8	34.2	35.3
2013	40.0	13.6	19.3	22.5	24.6	27.4	29.2	30.4	31.4
2014	66.8	22.7	32.2	37.6	41.2	45.8	48.7	50.8	52.4
2015	54.6	18.6	26.3	30.7	33.6	37.4	39.8	41.5	42.8
2016	62.8	21.3	30.2	35.3	38.7	43.0	45.8	47.8	49.3
2017	59.0	20.1	28.4	33.2	36.4	40.4	43.0	44.9	46.3
Mean	51.8	17.6	25.0	29.1	31.9	35.5	37.8	39.4	40.7
ST.DEV	14.0	4.8	6.7	7.9	8.6	9.6	10.2	10.7	11.0
skew coefficient	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

The IDF curve for study area was developed for 28years (1990-2017) daily rainfall data for gauge located Hossana town by using Gumbel extreme value type I distribution methods shown as follows (Appendix table 1-3for detail).

The intensity of the rainfall for design purposes is obtained from the intensity-duration frequency (IDF) relationship for the sub catchment by taking minimum time of concentration 15 minute recommended by ERA (ERA drainage manual, 2013).

Table 4.5: Duration of Rainfall against their corresponding Average rainfall Intensities

	Rain fall Intensities in mm/hr.				
Duration in minute	5 year	10 year	25 year	50 year	100 year
15	87.0992	100.03	107.33	116.38	140.533
30	61.7126	70.883	76.057	82.47	99.5987
45	48.0508	55.188	59.214	64.206	77.5366
60	39.4911	45.367	48.683	52.791	63.7663
90	29.261	33.609	36.06	39.102	47.2212
120	23.3539	26.822	28.778	31.203	37.6805
150	19.4877	22.382	24.014	26.038	31.4429
180	16.757	19.247	20.651	22.392	27.0424

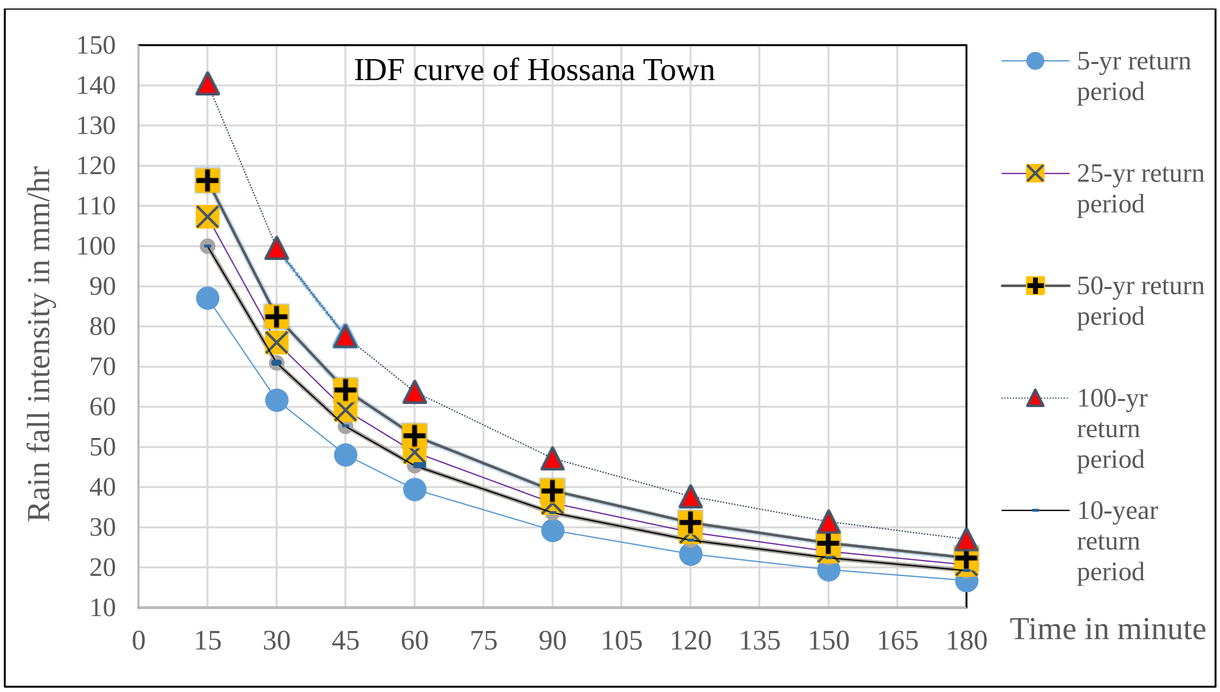


Figure 4.3: IDF curve for Study site (Hossana town).

IDF curve of previously have been developed by ERA that presented in ERA Drainage Design Manual, (2013) for region"B_2"compared with prepared IDF curve for this study to check the accuracy and adequacy of the drainage system of the town. The result was tabulated in table 4.6 and the detail was presented in Appendix in table1,table 2, and table3.

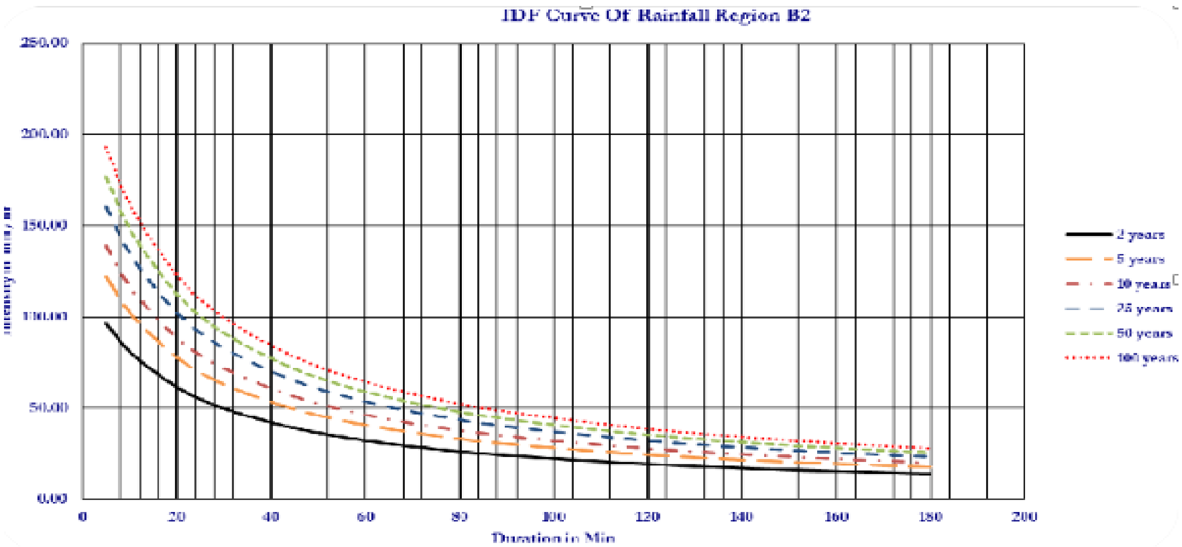


Figure 4.4: ERA previously prepared IDF curve for region “ B_2”

Table 4.6: Comparison of IDF curve of study area with ERA pre prepared IDF curve for study region.

Return periods in (yrs)	Duration in minute									
	15		30		60		90		120	
	Study site IDF	ERA IDF	Study site IDF	ERA IDF	Study site IDF	ERA IDF	Study site IDF	ERA IDF	Study site IDF	ERA IDF
5	87.1	82	61.7	54	39.5	46	29.3	30	23.4	26
10	100.0	99	70.9	62	45.7	49	33.6	35	26.8	32
25	107.3	118	76.0	74	48.7	54	36.0	39	28.8	34
50	116.4	125	82.5	80	52.8	57	39.2	43	31.2	36
100	140.5	132	99.6	85	63.7	62	47.2	46	37.7	38

The result in table 4.6 and figure 4.4Vs figure 4.3show that there was not a big difference between ERA previously prepared IDF curve and IDF curve prepared study site for this thesis work. However, to be in behalf of accurate side the IDF curve prepared for study site was used to determine peak discharges.

4.2 Computed discharge by using rational method

4.2.1 Runoff coefficient and imperviousness

The runoff coefficient and imperviousness was directly depend on land use land cover of the catchment. The weighted runoff coefficient (C_w) and weighed imperviousness (I_w) for different land use type resulted in table 4.7 in order to estimate peak discharge of the sub catchments (see table 4.8). However, weighed imperviousness (I_w) was used as SWMM hydrological input. Delineated area for each catchment, Time of concentration in chapter 3 and runoff coefficient table 4.7 were used to determine the rational peak discharge.

Table 4.7: Sample of runoff coefficient an imperviousness ratio for selected sub catchments in different land use conditions.

Polygon Id	Area each segments (m^2)	Land use condition	Name of catchment	Total area of catchment (ha)	C_i for each segments	I_i for each segments	Weighed runoff coefficient	Weighed imperviousus ratio
48	16043	bar land	City center sub-catchment -15 (CSC15)	24.01	0.65	11	0.65	28.45%
14	100198	bar land			0.65	11		
51, 72	35041.6	Mosque			0.8	11		
56,25	62405	residential			0.55	51		
As-15	11000.3	asphalt road			0.97	80		
Gr-15	15420.7	gravel road			0.5	70		
93	11712	residential	City center sub-catchment-17 (CSC17)	18.07	0.8	51	0.77	53%
80	16493	residential			0.8	51		
101,104,110	22751.6	residential			0.8	51		
67,35,32	92795	residential			0.8	51		
65	19892	residential			0.8	51		
Gr-17	17081.9	gravel road			0.5	70		
23,84,123	37079	Residential	Arada sub-catchmnt-1 (ASC-1)	19.3	0.8	51	0.61	43%
23, 36	79627	Church & school			0.5	11		
As-A1	23160	asphalt			0.97	80		
Gr-A1	42980.	gravel			0.5	70		
Cs-A1	10133.	cobble tone			0.5	70		

Table 4.8**:** Peak discharges of each sub-catchments by using rational method

	1	2	3-10	3-25	3-50	3-100	4	1*3-10*4*k
Sub-catchments		Time of concentration	Intensity 10-yr RT	Intensity 25-yr RT	Intensity 50-yr RT	Intensity 100-yr RT	Average runoff coefficient	Q for 10 year RT
ID	area (ha)							
CCS-1	17.49	17.71	90.57	96.75	104.41	124.87	0.63	2.76
CCS-2	37.76	18.36	87.54	93.43	100.72	120.20	0.78	7.15
CCS-3	5.77	15	100.03	107.33	116.38	140.53	0.74	1.18
CCS-4	5.41	15	100.03	107.33	116.38	140.53	0.79	1.184
CCS-5	2.61	15	100.03	107.33	116.38	140.53	0.83	0.60
CCS-6	5.56	15	100.03	107.33	116.38	140.53	0.85	1.32
CCS-7	3.12	15	100.03	107.33	116.38	140.53	0.81	0.70
CCS-8	6.32	16.51	94.52	101.16	109.38	131.34	0.76	1.27
CCS-9	3.62	15	100.03	107.33	116.38	140.53	0.78	0.79
CCS-10	3.29	15	100.03	107.33	116.38	140.53	0.84	0.77
CCS-11	7.89	15	100.03	107.33	116.38	140.53	0.84	1.85
CCS-12	3.83	15	100.03	107.33	116.38	140.53	0.79	0.84
CCS-13	12.36	20.10	83.86	89.31	96.06	114.09	0.75	2.15
CCS-14	18.49	18.85	87.20	93.00	100.20	119.42	0.62	2.76
CCS-15	18.07	15	100.03	107.33	116.38	140.53	0.77	3.88
CCS-16	24.01	25.93	71.28	75.50	80.74	94.73	0.65	3.09
CCS-17	10.73	19.35	85.94	91.61	98.64	117.42	0.70	1.79
ASC-1	21.54	26.68	69.90	74.00	79.08	92.65	0.63	2.65
ASC-2	17.50	15	100.03	107.33	116.38	140.53	0.78	3.81
ASC-3	18.67	22.64	77.86	82.70	88.70	104.74	0.64	2.60
ASC-4	82.33	26.11	70.95	75.15	80.34	94.23	0.53	8.57
LSC-1	19.91	26.56	70.31	74.44	79.56	93.24	0.65	2.52
LSC-2	21.53	15	100.03	107.33	116.38	140.53	0.80	4.78
LSC-3	22.67	28.60	66.81	70.63	75.38	88.05	0.60	2.53
LSC-4	29.29	26.21	70.84	75.02	80.20	94.03	0.60	3.46
LSC-5	37.66	36.52	56.56	59.56	63.28	73.21	0.60	3.57

Note:-k is Conversion factor = 0.00278, R- rational method and Q is discharge in m^3/sec

4.3 Peak discharge computing by using SCS curve number method

This method is used to determine the catchment discharge by assuming unit hydrograph consideration and mainly used to prepare rainfall data series for different year return period. The computed SCS peaks were used to check and relate runoff capacity of the catchments with other methods. See Appendix in table7. The catchment discharge for different year return periods by SCS methods was resulted in table 4.9.

Table 4.9: Simulated discharge by using SCS curve method.

	SCS Method Discharge for different year return period								
	Discharge in m^3/sec				Discharge in m^3/sec				
Sub-catch ID	QSC-10 yr	QSC-25 yr	QSC-50 yr	QSC-100 yr	Sub-catch ID	QSC-10 yr	QSC-25 yr	QSC-50 yr	QSC-100 yr
CC-SC-1	2.6	3.39	3.62	4.14	CC-SC-14	2.74	3.58	3.82	4.37
CC-SC-2	5.69	7.31	7.81	8.93	CC-SC-15	2.73	3.5	3.74	4.28
CC-SC-3	0.87	1.12	1.19	1.36	CC-SC-16	3.63	4.65	4.96	5.68
CC-SC-4	0.82	1.05	1.12	1.28	CC-SC-17	1.62	2.08	2.22	2.54
CC-SC-5	0.39	0.51	0.54	0.62	ASC-1	3.25	4.17	4.45	5.1
CC-SC-6	0.84	1.08	1.15	1.32	ASC-2	2.64	3.39	3.62	4.14
CC-SC-7	0.47	0.6	0.65	0.74	ASC-3	2.82	3.62	3.82	4.42
CC-SC-8	0.95	1.23	1.31	1.5	ASC-4	12.42	15.95	17.02	19.48
CC-SC-9	0.54	0.7	0.75	0.86	LSC-1	3.01	3.86	4.12	4.71
CC-SC-10	0.49	0.64	0.68	0.78	LSC-2	3.25	4.17	4.45	5.09
CC-SC-11	1.19	1.53	1.63	1.87	LSC-3	3.42	4.39	4.69	5.36
CC-SC-12	0.58	0.74	0.79	0.91	LSC-4	4.41	5.67	6.05	6.93
CC-SC-13	1.87	2.4	2.56	2.93	LSC-5	5.69	7.3	7.78	8.91

Note: - CC-SC –city center sub-catchment QSC –discharge SCS method
ASC – Arada sub-catchment LSC –Lucid sub-catchment.

4.4 Adequacy of existing Drainage system

The results of the hydraulic design indicate that majority of the existing drainage canals capacity obtained by manning equation are not sufficiently carried the runoff during rainfall events for 10 years of rainfall intensity. The key problem areas (sub-catchments), which are shown in figure 4.5, are typical example whereby most of drainage canals are inadequate to discharge catchment flow to the system without cause of flooding. The adequacy of existing drainage system with estimated design discharge sub-catchment was discussedin figure 4.5. Even though some drainage canals (CCSC-9, CCSC-14 and ASC-2) are enough to carry the rainfall intensity of within 10 years of return periods.

In some areas apart from the drainage capacity, lack of good management activities of drainage system could also play a decisive role incause of flooding problem. (See figure 4.6. and Appendix site photos).

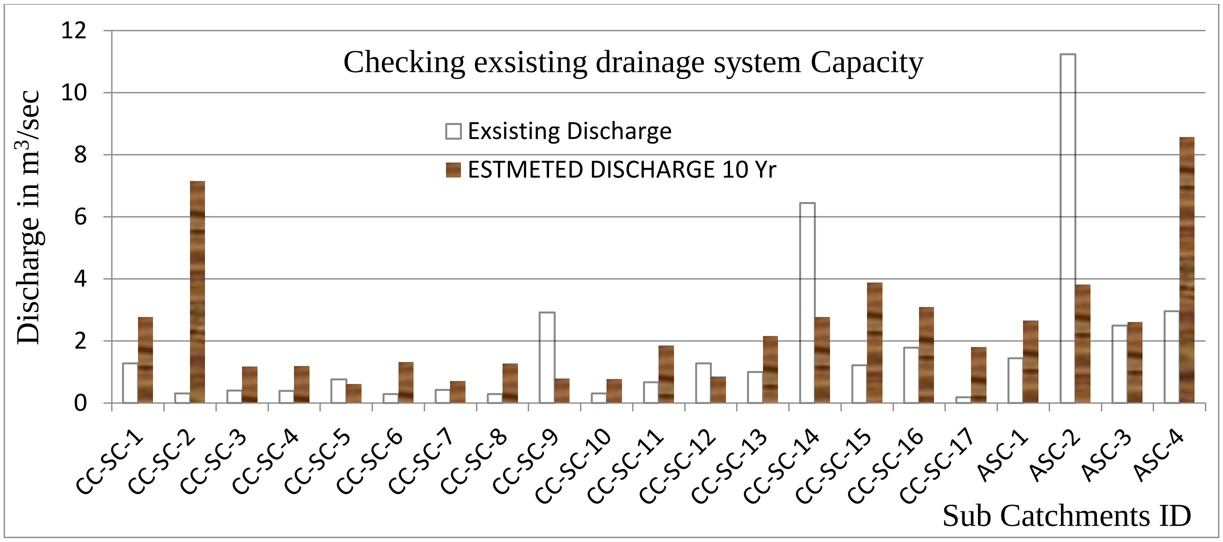


Figure 4.5**:** Checking the existing drainage system adequacy with estimated discharge by rational method for 10yr return period.

Figure 4.6: Associated drainage problems in the field (Source: field survey)

4.5 Catchment Peak discharge by using SWMM 5.1.03.

SWMM5.1.03 in this study was used to calculate the peak discharge for catchment, and to evaluate hydraulic performance of the catchment channels. In addition, the outputs of SWMM simulated result was checked by SCS curve number method and rational method. The simulated results for all sub-catchments, intake manholes (junctions), and conduits result were presented in table 4.10, table 4.11 and table 4.12 for 10-year return period, at 1-hour time of concentration with 5-minutetime interval rainfall time series data.

Table 4.10: Peak Runoff for each sub-catchment SWMM and its comparison Rational & SCS

Sub catchment ID	Q(R)10Y (m^3/sec)	Q(SWMM) 10Y(m^3/sec)	Q(SCS) 10 Year (m^3/sec)	% error (S-R) difference	% error (S-Sc) difference
CCSC-1	2.76	2.71	2.6	-1.82	4.05
CCSC-2	7.15	7.11	5.69	-0.53	19.97
CCSC-3	1.18	1.21	0.87	-2.6	28.10
CCSC-4	1.184	1.09	0.82	7.91	24.77
CCSC-5	0.60	0.56	0.39	7.20	30.36
CCSC-6	1.32	1.14	0.84	13.73	26.32
CCSC-7	0.69	0.62	0.47	11.24	24.19
CCSC-8	1.27	1.2	0.95	5.45	20.83
CCSC-9	0.79	0.77	0.54	2.15	29.87
CCSC-10	0.77	0.7	0.49	9.162	30.00
CCSC-11	1.85	1.71	1.19	7.53	30.41
CCSC-12	0.84	0.8	0.58	4.85	27.50
CCSC-13	2.15	2.02	1.87	5.96	7.43
CCSC-14	2.76	2.92	2.74	-5.67	6.16
CCSC-15	3.87	3.72	2.73	4.00	26.61
CCSC-16	3.09	3.16	3.63	-2.12	-14.87
CCSC-17	1.79	1.85	1.62	-3.13	12.43
ASC-1	2.65	2.85	3.25	-7.59	-14.04
ASC-2	3.81	3.72	2.64	2.48	29.03
ASC-3	2.60	2.72	2.82	-4.548	-3.68
ASC-4	8.57	8.6	12.42	-0.36	-44.42
LSC-1	2.525	2.51	3.01	0.48	-19.92
LSC-2	4.78	4.67	3.25	2.33	30.41
LSC-3	2.52	2.53	3.42	-0.2050	-35.18
LSC-4	3.46	3.46	4.41	0.0165	-27.46
LSC-5	3.569	3.65	5.69	-2.272	-55.89
Total discharge	68.58	68.26	68.93	0.46	-0.98

The detail for different year return periods in Appendix in table 8.

The result obtained in table 4.10 show that SWMM result was more related with rational method result than SCS curve method result. SWMM model could not simulate the surface runoff directly, but by calculating the overflow water quantity to determine the inundation condition indirectly, the inundation index determined by SWMM is not so precise (Lie *et al*., 2015). As result, rational and SCS curve number, method was used to recheck simulated results by SWMM.

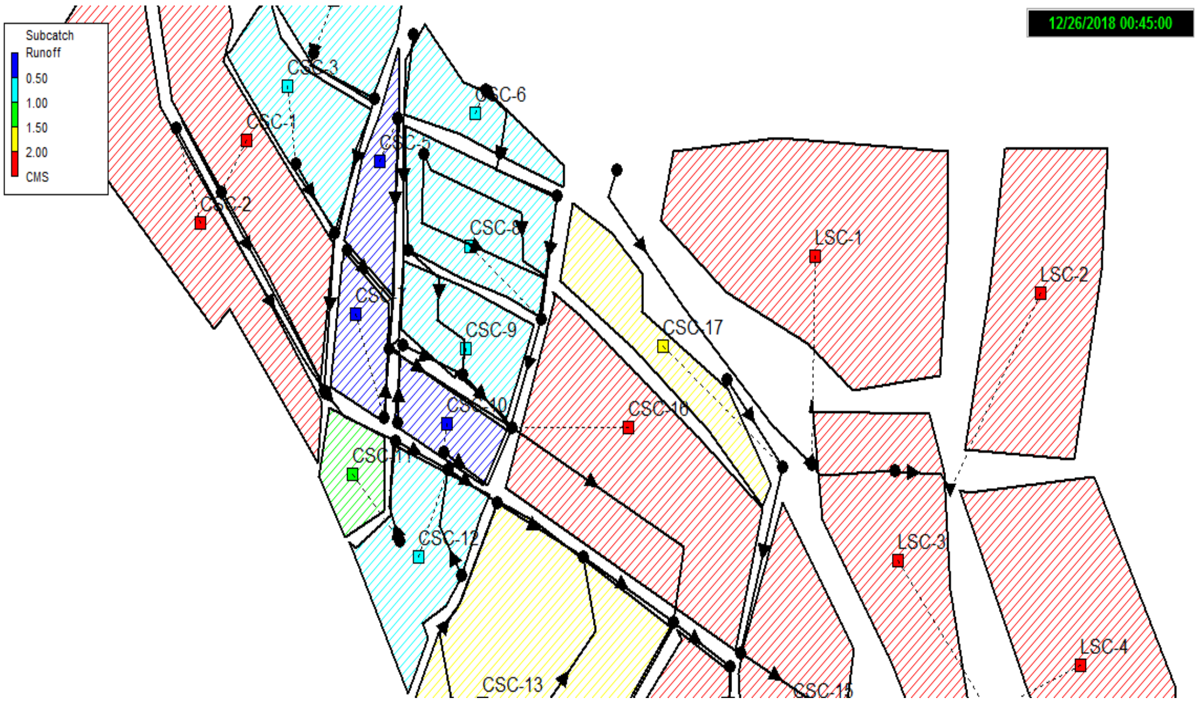


Figure 4.7: sub-catchment location and Catchment Runoff characteristics for 10 year return period at time of 45-minutetime series data.

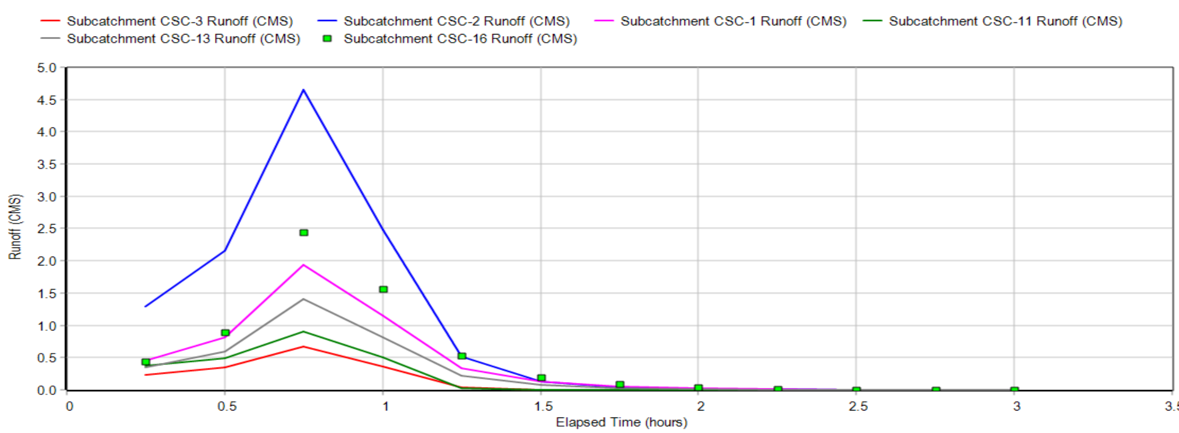


Figure 4.8: Runoff hydrographs for selected sub-catchments by SWMM.

a.

4.6 Outline of the storm water drainage systems conduits and Junctions

4.6.1 Conduits discharge carrying capacity

The cross-sectional shapes of conduits surveyed were mostly rectangular and trapezoidal. But circular and irregular natural cross-section shapes are also supported, as user-defined closed shapes in SWMM. Based on the field collected data and site investigation the drainage system of town, the discharge carrying capacity of conduits were evaluated by using SWMM model and Manning equation that was resulted in table 4.11.andTable4.12

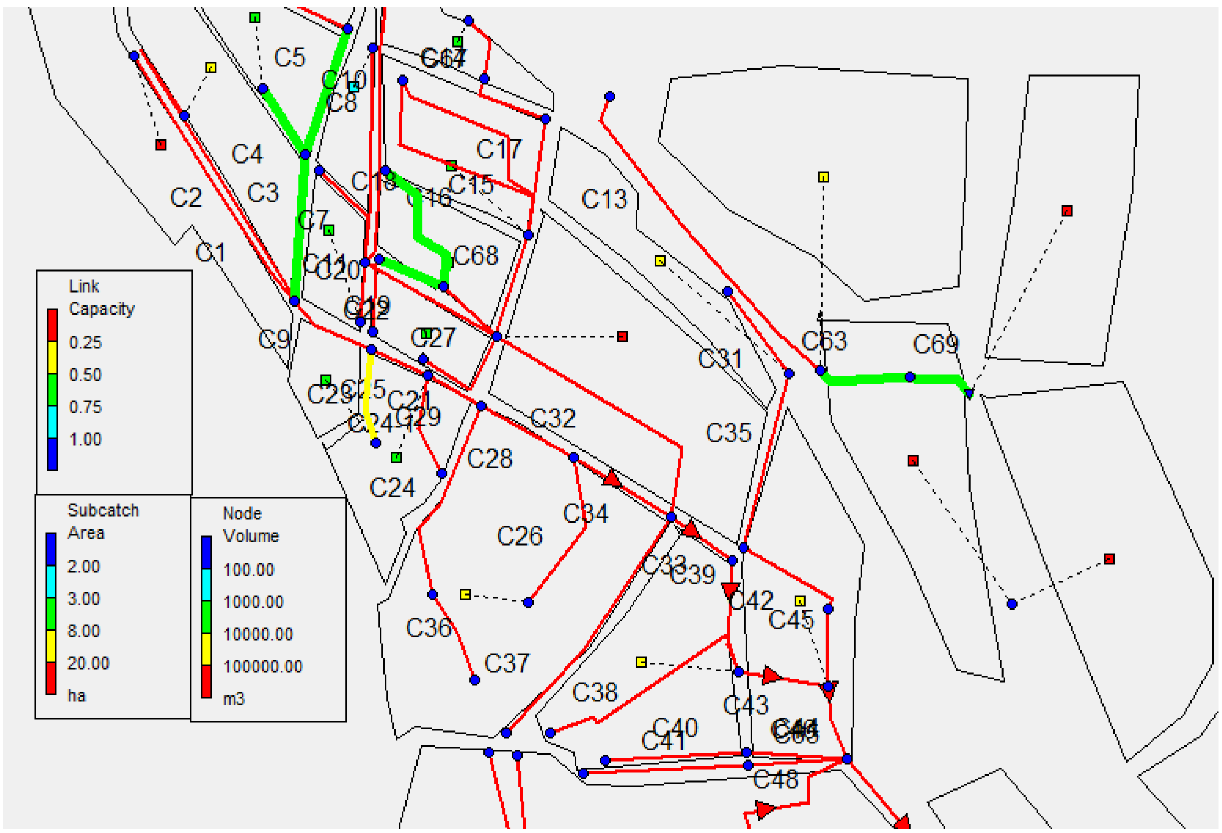
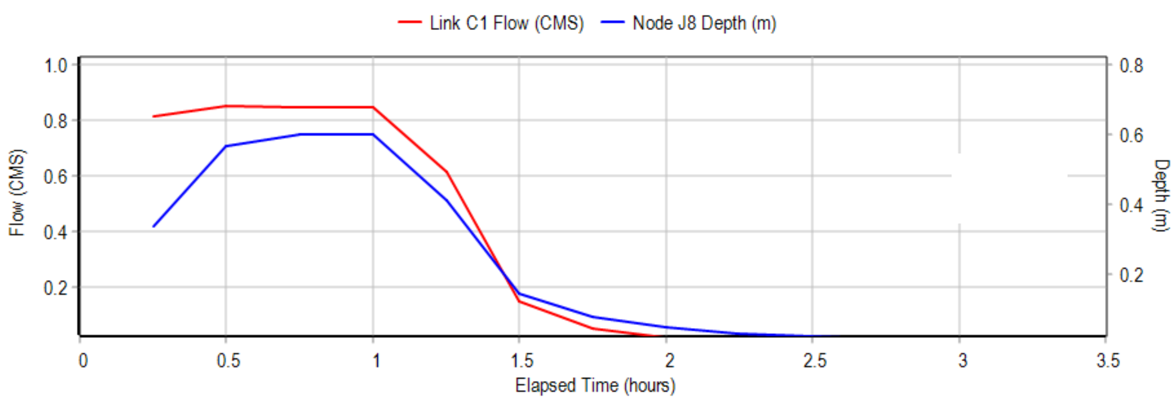


Figure 4.9: Conduits flow carrying capacity of city center watershed by EPA SWMM model

The properties of the open channels and pipeline network was collected, including the surface and bottom elevation, maximum water depth of the junctions, the length, the shape, the diameter and slope of the storm drain conduits.

Table 4.11: Peak discharges of Conduits by Manning Equation and SWMM for selected sub-catchment.

Con duit ID	Dept h(m)	Widt h (m)	Slop e	shape	velocity (m/sec)	Area(m^2)	Discharge by SWMM Q(m/sec)	Discharge (M) Q(m/sec)
C-1	0.4	0.6	0.05	trapezoidal	6.38	0.2	1.28	1.276
C-2	0.2	0.4	0.04	rectangular	3.91	0.08	1.112	0.3133
C-3	0.2	0.4	0.04	trapezoidal	3.87	0.07	0.5662	0.271
C-4	0.2	0.4	0.06	rectangular	5.00	0.08	0.362	0.400
C-5	0.2	0.4	0.02	trapezoidal	2.73	0.07	0.133	0.19
C-6	0.2	0.4	0.08	trapezoidal	5.64	0.07	0.182	0.395
C-7	0.1	0.2	0.01	rectangular	1.655	0.02	0	0.033
C-8	0.4	0.6	0.02	trapezoidal	3.84	0.2	0.678	0.768
C-9	0.4	0.6	0.04	trapezoidal	5.69	0.2	1.1055	1.14
C-10	0.4	0.6	0.04	trapezoidal	5.76	0.2	0.9975	1.152



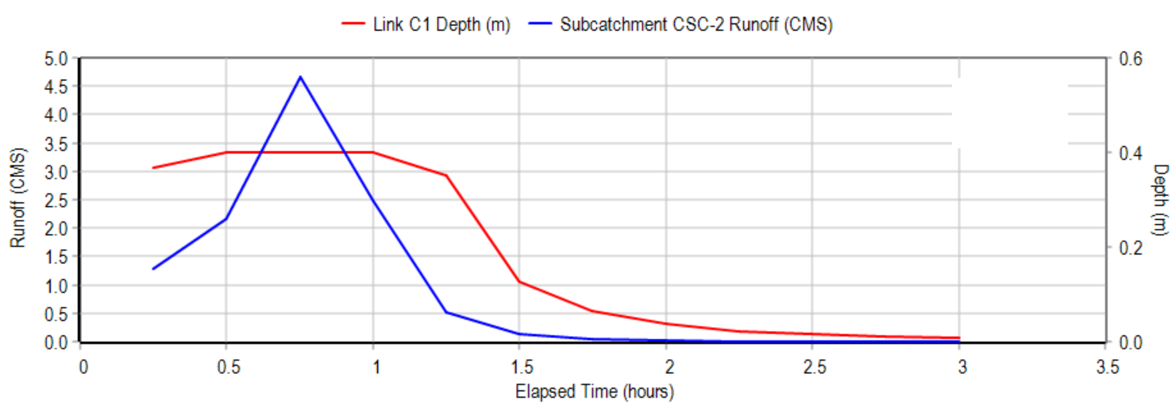


Figure 4.10: Flow and conduit carrying capacity for selected drainage channels.

Table 4.12: Hydraulic analysis of existing drainage system conduit capacity by EPA SWMM and Manning equations.

Conduit ID	Max depth conduit by survey(m)	Max Discharge Q(m/sec) by Manning	max channel cheking velocity (Manning)	Max depth of conduit by SWMM (m)	Max Discharge Q(m/sec) by SWMM	max channel cheking velocity (SWMM)	Is the conduit floods	remark
C1	0.4	1.276	6.382	1	1.218	6.09	YES	-
C2	0.2	0.3	3.9	1	1.1	5.6	YES	care must be taken
C3	0.2	0.27	3.87	1	0.18	2.28	YES	-
C4	0.2	0.4	5.001	1	0.588	7.35	YES	care must be taken
C5	0.2	0.19	2.74	1	0.13	3.32	YES	-
C6	0.2	0.3951	5.6442	1	0.362	4.52	YES	-
C7	0.1	0.0331	1.6554	0.5	0	0	YES	-
C8	0.4	0.77	3.84	0.79	0.55	3.61	YES	-
C9	0.4	1.139504	5.697519	0.78	0.941	6.37	YES	-
C10	0.4	1.152	5.762	0.5	0	0	YES	-
C11	0.6	0.4256	2.8373	0.88	0.592	3.46	YES	care must be taken
C12	0.9	2.436	4.512	0.54	0.07	1.67	NO	-
C13	0.4	0.5951	2.9754	0.5	0	0	YES	-
C14	0.4	0.286	3.58	0.75	0.186	3.14	YES	-
C15	0.4	0.2864	3.5805	0.42	0	0	YES	-
C16	0.6	1.292	5.3835	0.28	0	0	NO	-
C17	0.4	1.406135	7.030673	0.58	0.186	3	YES	-
C18	0.8	2.91609	6.07518	0.55	0.306	3.48	NO	-
C19	0.8	4.0935	8.5282	0.53	0.346	3.18	NO	-
C20	0.6	0.1	0.9	0.4	0	0	NO	-
C21	0.2	0.3134	3.9172	1	0.44	5.51	YES	care must be taken
C22	0.7	1.303	3.7229	1	0.974	4.87	YES	-
C23	0.4	0.6653	3.3266	0.78	0.445	3	YES	-
C24	0.4	1.214715	6.073576	0.5	0	0	YES	-
C25	0.4	1.274004	6.37002	0.5	0.495	4.16	YES	-
C26	0.4	1.00168	5.008399	0.74	0.095	2.38	YES	-
C27	0.4	0.313777	3.92221	0.53	0.118	2.08	YES	-
C28	0.7	1.9815	5.6613	0.6	0.749	3.54	NO	-
C29	0.7	2.41583	6.90237	0.85	1.231	4.51	YES	-
C30	0.3	0.0701	2.3353	0.5	0	0	YES	-
C31	0.1	0.01975	1.97509	0.5	0	0	YES	-
C32	0.7	1.784	5.098	0.99	1.543	4.7	YES	-
C33	1	7.57	7.57	0.42	2.387	4.77	NO	-
C34	1	7.57	7.57	0.35	0.84	2.45	NO	-
C35	0.2	0.18	3.06	1	0.09	3.22	YES	-
C36	0.4	0.941	4.705	0	0	0	NO	-
C37	0.2	0.432	5.3994	0.5	0	0	YES	-
C38	0.2	0.35	4.38	0.5	0	0	YES	-
C39	0.6	0.7288	3.03668	0.7	2.387	6.68	yes	care must be taken
C40	0.6	1.15	4.8	0	0	0	NO	-
C41	0.6	1.152339	4.801411	0	0	0	NO	-
C42	0.6	1.262	5.26	0.21	0.092	3.58	NO	-
C43	1.2	6.44645	5.37204	0.53	5.253	8.46	NO	-
C44	0.2	0.1854	2.3174	0.5	0	0	YES	-
C45	0.7	1.2188	3.4824	0.53	0.092	1.76	NO	-
C46	2	20.603	8.5845	0.63	8.997	8.88	NO	-
C48	0.2	0.1854	2.3174	1	1.528	6.37	YES	care must be taken
C49	1.2	11.237	7.8034	1	9.177	6.37	NO	-
C50	0.2	0.336	4.201	1	0.266	4.11	YES	-
C51	0.6	0.728803	3.036679	0.5	0	0	NO	-
C52	0.4	1.016	5.082	0.5	0	0	YES	-
C53	0.4	1.016	5.082	0.5	0	0	YES	-
C54	1	4.1043	6.8405	0	0	0	NO	-
C55	0.4	1.1524	5.7619	0	0	0	NO	-
C56	0.4	1.15238	5.7619	0	0	0	NO	-
C57	0.4	0.708295	3.541473	0.5	0	0	YES	-
C58	0.8	2.958659	6.163873	0	0	0	NO	-
C60	0.8	2.500523	5.209424	0	0	0	NO	-
C61	0.8	2.958659	6.163873	0.69	0.147	3.73	NO	-
C62	0.6	0.973904	4.057933	1	7.434	4.42	YES	care must be taken
C63	0.6	0.974	4.058	1	0.337	4.21	YES	-
C64	0.3	0.1028	0.8566	0.08	0.079	2.04	NO	-
C67	0.2	0.198379	3.306309	0.23	0.186	2.28	YES	-
C65	0.2	0.047	0.79	0.5	0	0	YES	-
C68	0.4	0.24294	1.21472	0.92	1.308	7.22	YES	care must be taken
C69	0.5	3.142452	15.71226	0.5	0	0	NO	-

Simulated result by SWMM model show that in table 4.12 60% of the conduits was not carry the coming discharge for designed year return periods, because of maximum depth limitation and 15% incapacity of channel cross-section to carry designed discharge with it. As result, flooding on the town occur not only undersized drainage system only.

4.6.2 Junctions Hydraulic property

Junctions can be modeled as rectangular, trapezoidal, and circular cross-sections. The field-surveyed data show that the junction property was obtained mostly rectangular and circular.

By using 3-hour duration rainfall intensity in 5-minutetime interval time series rainfall data, the SWMM simulated result shows total volume of flooding and hours of flooding for flooded manholes for 25 years return periods was shown in table 4.13 below. According to ERA drainage manual, (2013) the drainage system designed for only 10 and checked for 25 year return period's rainfall intensity. Flooding nodes for 10 and 25-year return period was computed in detail in (Appendix table 5)

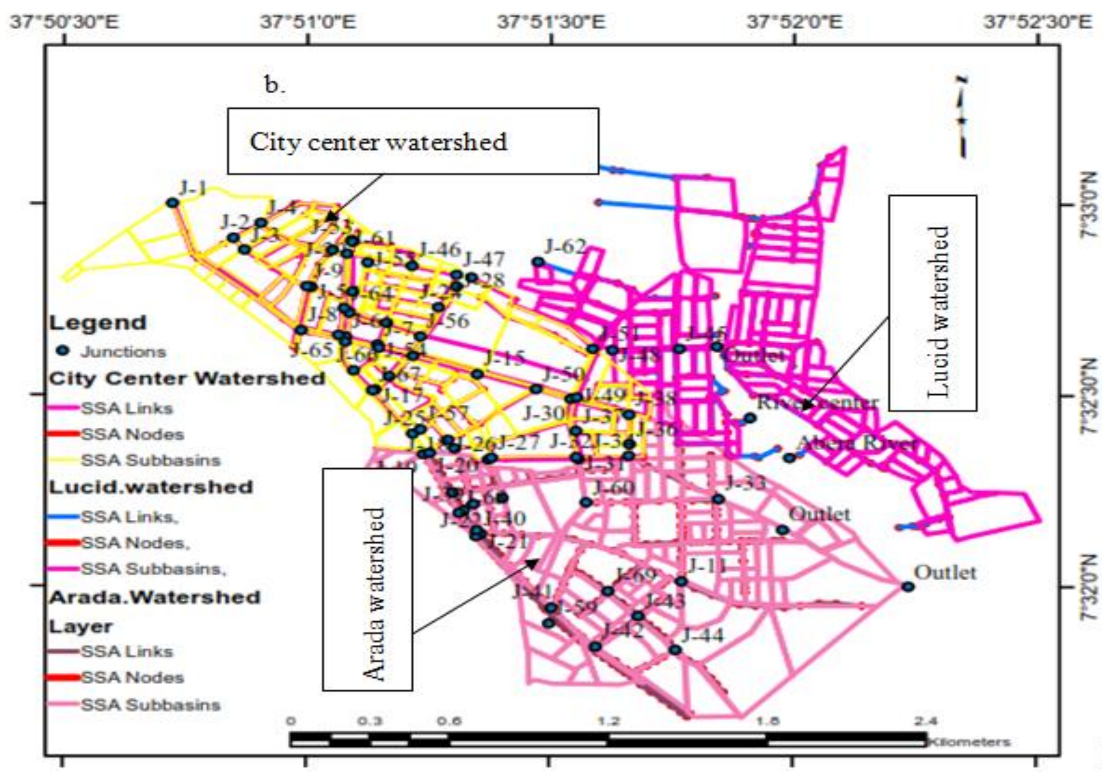


Figure 4.11: Flooding node location Map (Auto CAD 2007 analysis)

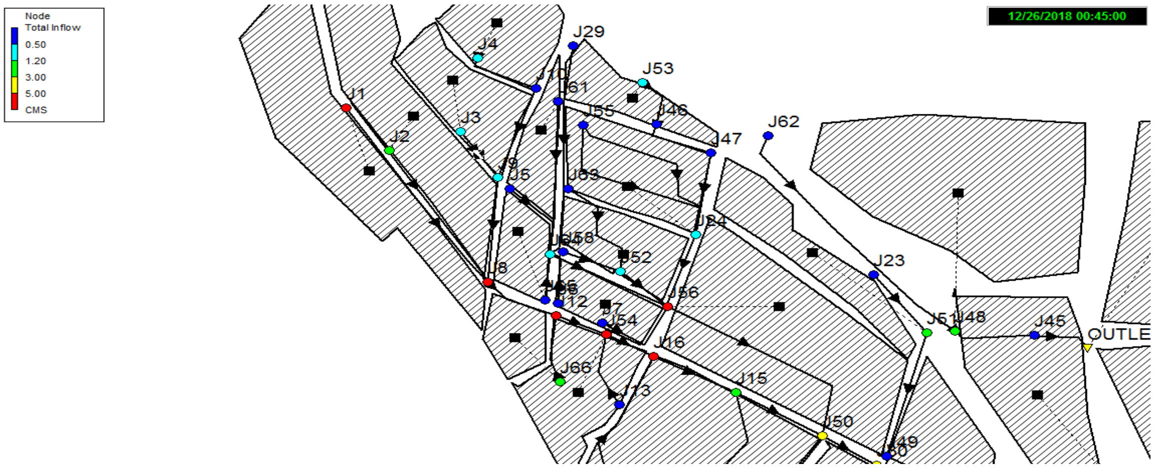


Figure 4.12: Nodes location and inflow capacity of nodes (Source: SWMM analysis)

Table 4.13: Flooding nodes and location of sub-catchments flooding occur

flooding Node Q_{25} Year RT	Hours Flooded	Max discharge cms	Total flood volume 10^6ltr	Location of catchment Flooding occurring
J1	0.94	7.655	7.95	CCSC-2
J2	0.59	2.313	1.603	CCSC-1
J3	0.23	0.898	0.333	CCSC-3
J9	5.84	0.903	3.172	Interaction node
J4	0.67	0.982	0.576	CCSC-4
J10	0.94	0.229	0.708	Interaction node
J8	0.99	1.042	3.245	Interaction node
J64	0.05	0.393	0.038	Interaction node
J53	0.94	1.217	1.191	CCSC-6
J63	5.25	0.378	0.453	Interaction node
J56	0.9	5.682	6.952	Interaction node
J16	0.63	0.67	0.451	Interaction node
J66	5.86	2.617	5.094	CCSC-11
J58	5.48	0.428	0.51	Interaction node
J7	0.16	0.425	0.122	CCSC-10
J57	1.33	2.441	3.459	CCSC-12
J34	0.08	2.928	0.442	Outlet for city center
J51	1.23	2.242	3.055	CCSC-17
J35	0.47	1.555	0.855	ASC-1
J40	5.86	11.043	23.887	Interaction node
J69	5.87	0.062	0.454	Transmission node(AC-1)
J60	1.07	4.335	5.099	Interaction node
J11	1.54	2.758	4.436	Interaction node
J33	0.32	2.144	1.841	Final outlet (ac-1,and cc)
J48	1.17	2.879	4.1	Transmission node
J45	5.77	0.338	1.857	LSC-1
River A	5.92	7.707	15.451	River Junction point

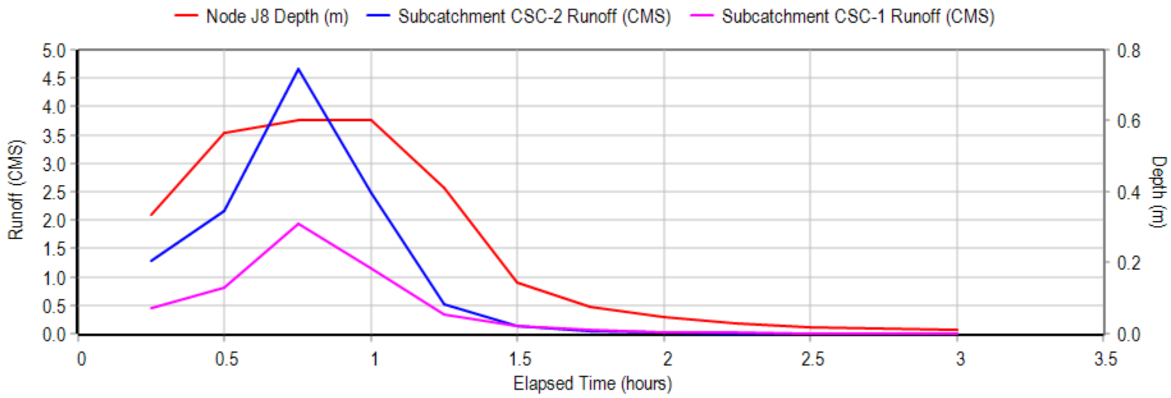


Figure 4.13: Nodes flow carrying capacity for selected catchment discharge of 10yr return period

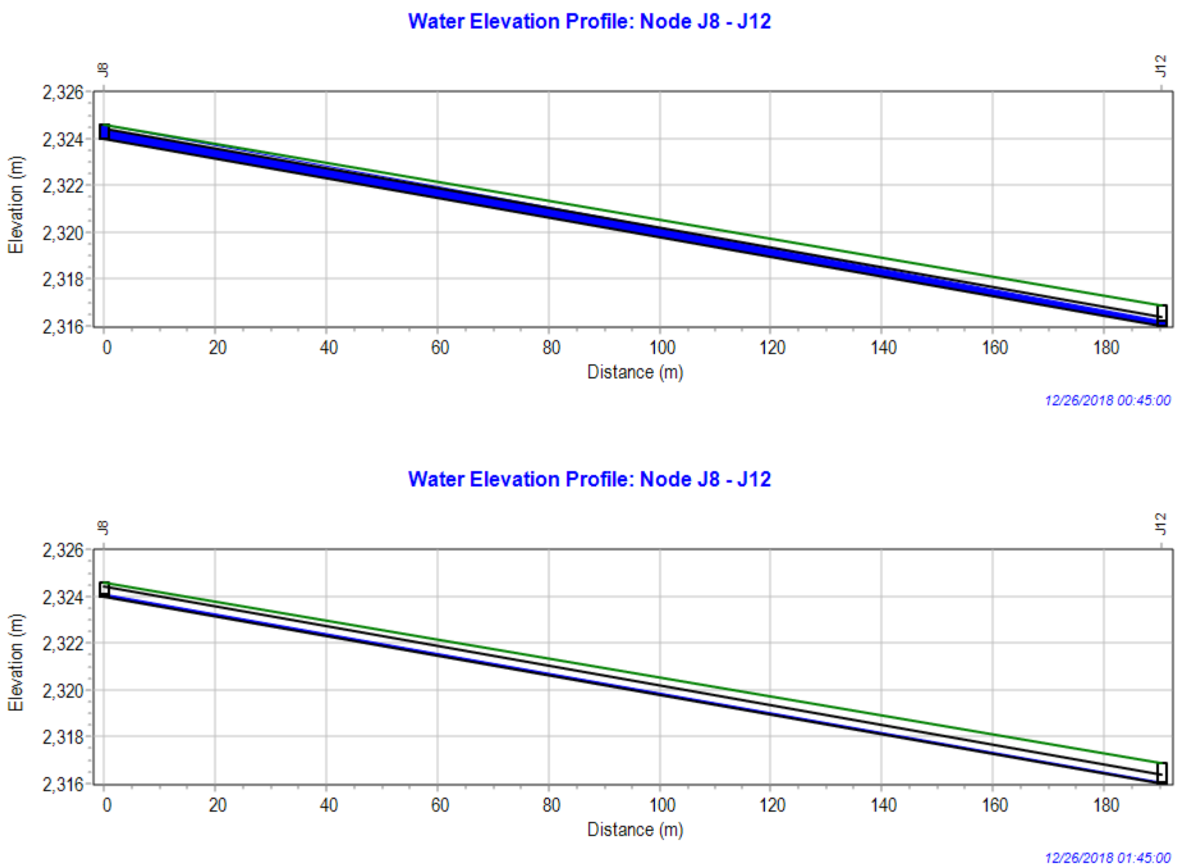


Figure 4.14:1D view of flooding nodes for10 year return period of rainfall data series in different time interval.

The model output tabulated above table 4.13 for nodes location maped in figure 4.11; show that 27 nodes from total of 72 junctions (nodes)were surcharged (flooded) for discharge of 10 year return period. The flooding nodes percentage for three watershed was shown as in table 4.14.

Table 4.14: Average flooding nodes of the sub-catchments for 10yr and 25yr RT rainfall.

Watershed location	Total No. of nodes	No. Nodes flooding (10yr T)	No.of nodes flooding(25yr T)	% flooding nodes	
				10yr RT	25yr RT
City center	49	17	18	34.6	36.73
Arada	19	8	9	42.10	47.36
Lucid	5	2	2	40	40
Total		27	29	38.9	41.4

Depending up on simulated result by model in table 4.14 shows that 42.3% flooding nodes were found in Arada watershed. Hence, Arada watershed was more senesentive to flooding.

4.7 Nodes Flood Inundation Mapping and Visualization.

Hydraulic like SWMM models have been used for predicting flooding in urban drainage system. Performance of storm network model is verified in the areas of interest with the aid of flow survey and rainfall data both collected simultaneously for short duration. SWMM 5.1is utilized to simulate the behavior of the flow in the main channel.

The flow in a flooded conduit system is complex, and the computations in the present urban flood model are based on an implicit finite difference scheme, adopted from the St Venant equations (Rosman, 2016). Flood modeling in GIS is used to produce flood vulnerability maps for flood planning and analysis. To make a significant advance on this approach, and to model the dynamics of flooding in urban areas reliably, high-resolution data on terrain model are needed (Ibrahim, 2017). Due to absence of high-resolution DEM data and the model incapability and limit of essential data, the researcher cannot model floods in 2dimensional (2D) flood extent or on surface flow. But 1dimensional (1D) view as figure 4.14 and flooding nodes location with help of Arc GIS 10 was shown figure 4.15 and figure 4.16 for 10 and 25 year return periods respectively.

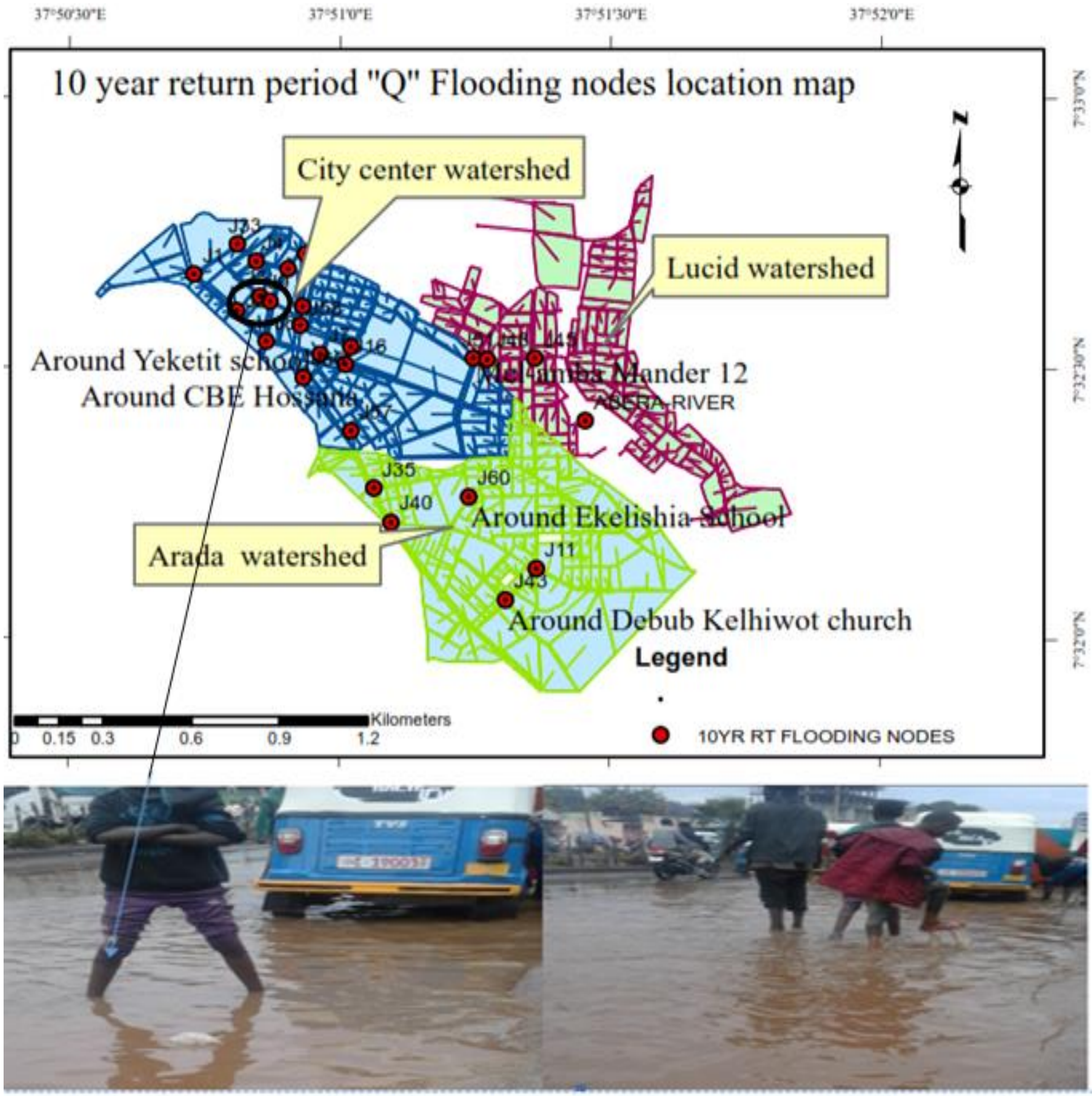


Figure 4.15:10 year return period flooding nodes locations

Floods from manholes easily spread to the adjacent sub-catchments and thus, surface flooding may spread to plain. Figure. 4.15and figure 4.16.show that a sample of flooding nodes for 10 and 25 year return periods respectively. The main output of the SWMM model flooding nodes where use in order to simulate storage of water next to the street system.

SWMM model used to determine flooding on nodes and Arc GIS provides information on where water from overflow will run to and be accumulated (Aysha and Tanim,2016).

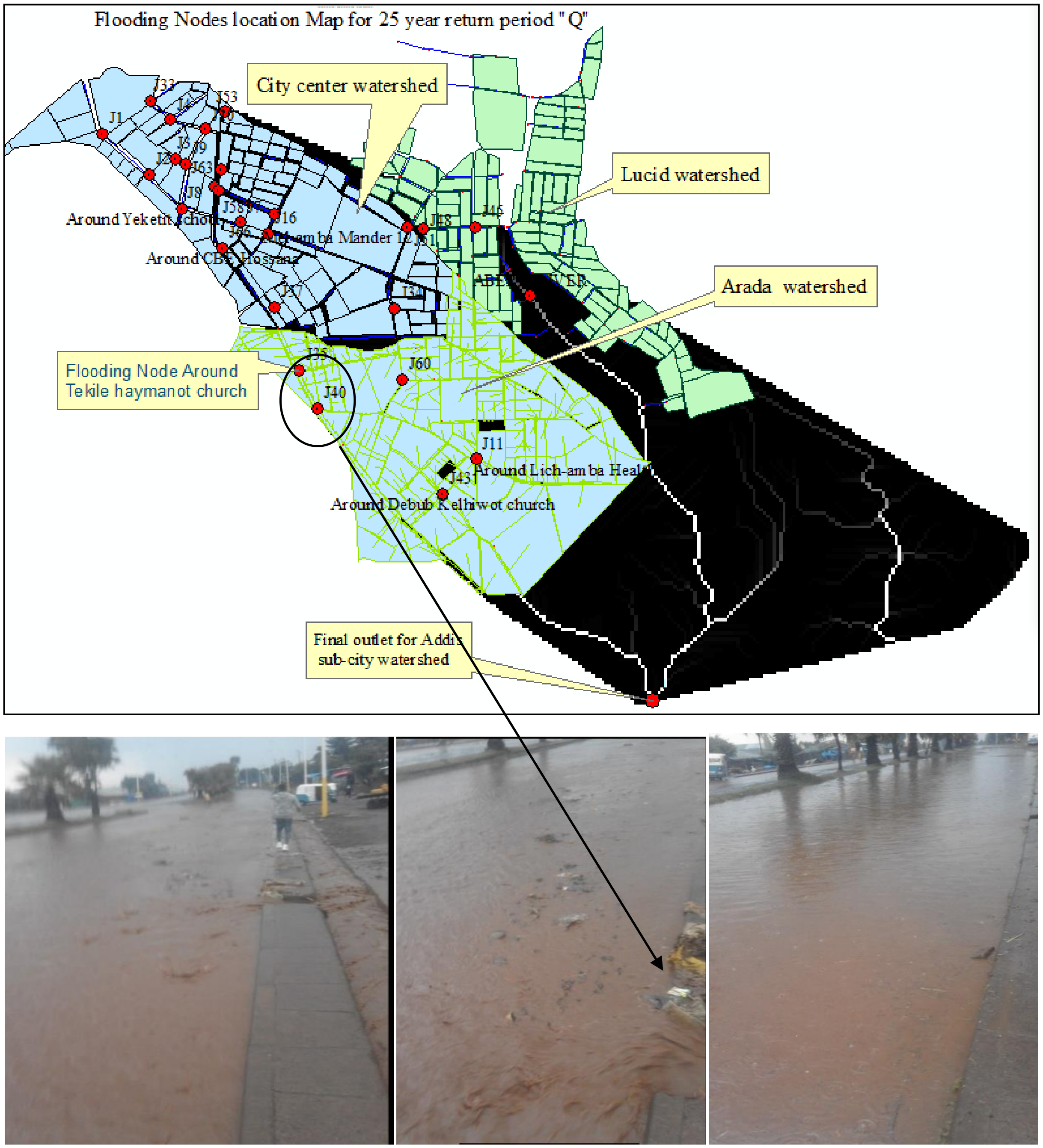


Figure 4.16:25 year return period flooding nodes locations map

By observing occurred floods depth, one can find out the actual amount of water depth return periods that can cause flooding in an area by help of statistical distribution analysis of rainfall design depth discussed in section 4.1.1. In general, by analyzing and modeling of the catchment, it can be easy to understand the hydrological behavior of an area.

Table 4.15. Flood risk of the sub-catchments for 10-year return period of rainfall data series

Location of area	Total area (km^2)	Flooding area in (km^2)	%floodingnode, and Risk	Associated problems for floods
City center	1.86	0.645	34.6	Drainage incapacity
Arada(lichamba)	1.46	0.6132	42	Lack of management and drainage inadequacy
Lucid	1.35	0.54	40	Lack of drainage system Infrastructures.
Total	4.67	1.8	39.8	All above problems

5. CONCLUSIONS AND RECOMANDATIONS

5.1 Conclusions

According to this thesis, the Stormwater management model (SWMM5.1.03), one of the widely used urban flood planning and management models, used to simulate the urban flooding of Hossana town in Addis sub-city based on Dynamic wave routing and Modified Green Ampt infiltration approaches were applied to analyze flow routing through the catchment. ArcGIS 10, Auto Cad 2007, and Google earth pro software's were used to prepare the inputs for the hydrological and hydraulic model. Gumbel extreme value type I distribution method was used to analyze rainfall data of 28 years (1990- 2017) for 10, 25, 50,100 years return period to determine design rainfall depth and to develop IDF curve for the study area. The study was conducted by dividing the total catchments into 3 watersheds (City center, Arada, Lucid), 26 sub-catchments, 72 junction, 70 conduits, and 3 outfalls.

The peak flow determined by using the rational method and SCS runoff curve number method used to check the simulated result by the model. The SWMM simulated result shows that 32.9% of City center, 42%Arada, and 40% Lucid watershed nodes were expose flooding for 10-year return peaks of 33.55m3/sec, 17.9m3/sec, and 16.8m3/sec respectively. Currently, the constructed drainage system was inadequate to cover the peak flow of the 10-year return period within it. This is because of the undersized drainage system, the rapid increase of urbanization and the development of silt from different sources and household waste. The results also shown in the SWMM model is capable for urban flood forecasting in 1D view, as it has no surface runoff (2D) routing, the urban flooding could not be seen precisely, but flood inundated nodes in the study was located by the help of ArcGIS10. Generally, it can be concluded that the main problem of flooding of the study area was found to be inadequate capacity of existing drainage system, insufficient drainage structures provision, lack of proper interconnection between the road inlet and lack of good management and maintenance.

5.2 Recommendations

- ✓ IDF curve prepared to Hossana town gave better and accurate result than ERA regional rainfall IDF curve. Hence, designers and urban planners better to use IDF curve of the study area for the catchment discharge computation period.
- ✓ Minimum time of concentration limit by ERA was better to decreased 5 minute to determine maximum peak intensity which recommended in Brown *et al.,(*2013)
- ✓ According to the obtained results, Addis sub- city, the main flood problem in the study area was insufficient capacity of drainage system. Therefore, existing drainage system of the Hossana town was revisited and the future drainage system be designed in consideration of the urbanization growth and climatic change.
- ✓ Developing the skill of SWMM software was better for planning and design of hydrologic and hydraulic analysis of urban drainage systems in urban areas, and for monitoring the drainage infrastructures.
- ✓ Drainage system of the town is open, which was exposed to disposal of solid waste. This may blocks flow of water and increase flooding problem during rainy season. Therefore, concern organizations better to prepare waste management rules and regulation for waste managing awareness concerning the impact of disposing solid wastes on drainage system for town publicity.
- ✓ ERA Drainage manual, (2013) recommends urban drainage design period to 10 years return period peak only. But, from this thesis result, its better urban drainage design return period to be increased 25-year.
- ✓ Finally, this research geographically limited Hossana town, Addis sub –city. It will be better further researches on urban flooding which accounts 2D view flood extent and problems associated it is recommended for whole town.

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7. APPENDIXES.

Table 1: Maximum annually 24 hour rainfall data for Hossana town

STEP1 Calculation of annually daily RF- data						Sample N	28
year	A daily maximum RF	year	A.daily max RF with De	Rank (m)	P=m/(n+1)	Tr=1/P	
1990	62.3	1997	94.1	1	0.03448	29.000	
1991	61.2	2006	73.9	2	0.06897	14.500	
1992	39.4	2002	67.3	3	0.10345	9.667	
1993	63.9	2014	66.8	4	0.13793	7.250	
1994	37.9	1993	63.9	5	0.17241	5.800	
1995	51.3	2016	62.8	6	0.20690	4.833	
1996	42.3	1990	62.3	7	0.24138	4.143	
1997	94.1	1991	61.2	8	0.27586	3.625	
1998	37.1	2017	59.0	9	0.31034	3.222	
1999	35.4	2005	59.0	10	0.34483	2.900	
2000	40.6	2015	54.6	11	0.37931	2.636	
2001	50.4	2008	52.2	12	0.41379	2.417	
2002	67.3	1995	51.3	13	0.44828	2.231	
2003	49.7	2001	50.4	14	0.48276	2.071	
2004	39.4	2003	49.7	15	0.51724	1.933	
2005	59	2007	47.8	16	0.55172	1.813	
2006	73.9	2009	46.9	17	0.58621	1.706	
2007	47.8	2012	45.0	18	0.62069	1.611	
2008	52.2	1996	42.3	19	0.65517	1.526	
2009	46.9	2000	40.6	20	0.68966	1.450	
2010	33.5	2013	40.0	21	0.72414	1.381	
2011	37.2	2004	39.4	22	0.75862	1.318	
2012	45	1992	39.4	23	0.79310	1.261	
2013	40	1994	37.6	24	0.82759	1.208	
2014	66.8	2011	37.2	25	0.86207	1.160	
2015	54.6	1998	37.1	26	0.89655	1.115	
2016	62.8	1999	35.4	27	0.93103	1.074	
2017	59	2010	33.5	28	0.96552	1.036	

Table 2: Rainfall intensity for short duration of time.

	3. corrosponding intensity for given rain fall is computed as follows **I** (mm/hr)								
15min	30min	45min	60 min	90min	120min	150min	180min	210min	240min
84.70341	60.0056	46.72404	38.38899	28.45666	22.71146	18.95174	16.29303	14.3098	12.77155
83.20785	58.94611	45.89906	37.71118	27.95421	22.31045	18.61712	16.00535	14.05714	12.54605
53.56845	37.94897	29.5494	24.27811	17.99667	14.36326	11.98553	10.3041	9.04986	8.077031
86.87878	61.54668	47.92402	39.3749	29.18749	23.29474	19.43847	16.71147	14.67731	13.09955
51.52904	36.50421	28.42442	23.35382	17.31151	13.81644	11.52923	9.911809	8.705322	7.76953
69.74775	49.41071	38.47421	31.61084	23.43221	18.70141	15.60553	13.41625	11.78319	10.51654
57.51131	40.74217	31.72435	26.06508	19.32129	15.42046	12.86772	11.06252	9.715966	8.671534
127.9389	90.63446	70.57356	57.98401	42.98188	34.30414	28.62535	24.60953	21.614	19.29058
50.44136	35.73367	27.82443	22.86086	16.9461	13.5248	11.28587	9.702589	8.521568	7.60553
48.13003	34.09628	26.54946	21.81333	16.16959	12.90506	10.76873	9.257996	8.131092	7.257028
55.19998	39.10477	30.44938	25.01754	18.54479	14.80072	12.35057	10.61793	9.32549	8.323032
68.52411	48.54386	37.79923	31.05626	23.02112	18.37331	15.33175	13.18088	11.57647	10.33204
91.50144	64.82146	50.47397	41.46997	30.7405	24.5342	20.47275	17.60065	15.45826	13.79655
67.57238	47.86964	37.27424	30.62493	22.70138	18.11813	15.11881	12.99781	11.41569	10.18854
53.56845	37.94897	29.5494	24.27811	17.99667	14.36326	11.98553	10.3041	9.04986	8.077031
80.21671	56.82713	44.24909	36.35554	26.94932	21.50844	17.94788	15.42999	13.55182	12.09505
100.4748	71.17839	55.42387	45.53686	33.75517	26.94023	22.48048	19.32672	16.97423	15.14956
64.98913	46.03961	35.84927	29.45415	21.83352	17.42548	14.54082	12.50091	10.97927	9.799038
70.9714	50.27757	39.1492	32.16541	23.8433	19.0295	15.87931	13.65162	11.98992	10.70104
63.76549	45.17276	35.17428	28.89958	21.42243	17.09739	14.26704	12.26554	10.77255	9.614537
45.54678	32.26625	25.12449	20.64256	15.30173	12.21242	10.19075	8.761097	7.694678	6.867527
50.57732	35.82999	27.89943	22.92248	16.99178	13.56125	11.31629	9.728741	8.544538	7.62603
61.18224	43.34273	33.74931	27.72881	20.55457	16.40474	13.68906	11.76864	10.33613	9.225036
54.38421	38.52687	29.99939	24.64783	18.27073	14.58199	12.16805	10.46101	9.187675	8.200032
90.82164	64.33987	50.09897	41.16187	30.51211	24.35193	20.32065	17.46989	15.34342	13.69405
74.23445	52.58918	40.94916	33.64428	24.93954	19.90442	16.60939	14.27928	12.54118	11.19304
85.38321	60.48719	47.09904	38.69709	28.68504	22.89373	19.10384	16.42379	14.42465	12.87405
80.21671	56.82713	44.24909	36.35554	26.94932	21.50844	17.94788	15.42999	13.55182	12.09505

Table 3: Rainfall frequency Determination by gumbel extreme value type 1 PDF distribution method

3. probabilty distribution selection to calculate return period rainfall depth.

1. gumbel probabilty distribution method

$$P_T = \sigma + K_T$$

$\sigma =$ is Avarege of given daily maximum for each duration

KT= is frequancy factor expresed as

$$K_T = \frac{Yt - \bar{Y}n}{\bar{S}n}$$

T Frequancy or return perion of maximum event

Y'n and Sn are read from table

$$Yt = -\ln\{\ln\left(\frac{T}{(T-1)}\right)\}$$

sample siz	28	2
		5
		10
		15
		25
yn	0.5343	50
sn	1.1047	100
		200

Rain fall depth in mm

n in minute	mean rainfa	standard (	2 year	5 year	10 year	15 year	25 year	50 year	100 year	200
			0.3665	1.49994	2.25037	2.6738	3.198534	3.901939	4.600149	5.295812
			-0.1519	0.8741197	1.55342	1.9367	2.411726	3.048464	3.680501	4.310231
15min	17.614	4.76	16.891	21.77481	25.0083	26.833	29.09382	32.12469	35.13318	38.1307
30min	24.956	6.75	23.931	30.856308	35.4416	38.029	41.23515	45.53313	49.79938	54.05006
45min	29.15	7.88	27.953	36.038063	41.391	44.411	48.1544	53.1719	58.15235	63.11462
60min	31.93	8.65	30.616	39.491135	45.3671	48.682	52.79143	58.29922	63.76633	69.2135
90min	35.5	9.6	34.042	43.891549	50.4129	54.092	58.65257	64.76526	70.83281	76.87822
120min	37.783	10.21	36.232	46.707762	53.6435	57.557	62.40673	68.90782	75.36091	81.79046
150min	39.41	10.65	37.792	48.719374	55.954	60.036	65.09489	71.87615	78.60733	85.31396
180min	40.66	10.995	38.99	50.270946	57.7399	61.954	67.17693	74.17787	81.12711	88.05099

Rain fall Intensities in mm/hr

Duration	2 year	5 year	10 year	15 year	25 year	50 year	100 year
15	19.040	87.0992	100.03	100.03319	107.33	116.38	140.5327
30	13.500	61.7126	70.883	70.883222	76.0572	82.47	99.59876
45	10.507	48.0508	55.188	55.187973	59.2147	64.206	77.53646
60	8.650	39.4911	45.367	45.367116	48.6823	52.791	63.76633
90	6.400	29.261	33.609	33.608579	36.0614	39.102	47.22187
120	5.105	23.3539	26.822	26.821729	28.7783	31.203	37.68046
150	4.260	19.4877	22.382	22.381586	24.0143	26.038	31.44293
180	3.665	16.757	19.247	19.246632	20.6513	22.392	27.04237

Tr	aveg. Value	KT	Stand.Dv	in fall(mm)	PT
2	51.821	-0.152	14.014	50	
5		0.874		64	
10		1.553		74	
15		1.937		79	
25		2.411		86	
50		3.048		95	
100		3.680		103	
200		4.310		112	

	Coefficient of skew, Cs	Recurrence interval T in years					
		2	10	25	50	100	200
x1	0.4	-0.066	1.317	1.88	2.261	2.615	2.949
x	X=0.3973	-0.0597	1.3168	1.879	2.259	2.613	2.946
xo	0.3	-0.05	1.309	1.849	2.21	2.544	2.856

L

by using lagrangian linear interpolation relation ship

Recurrence interval T in year	Kz
2	-0.0597
10	1.3168
25	1.879
50	2.259
100	2.613

$\dot{Y}$ = 1.7002170

$$Y_T = \bar{Y} + Ky * \boldsymbol{\sigma y}$$

Figure 1: (ERA, Rainfall region Division for determination IDF curve (source, ERA Drainage manual, 2013)

Table 4: ERA rainfall depth for division region (source ERA drainage manual, 2013)

	Return Period in years							
	2	5	10	25	50	100	200	500
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67 .66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

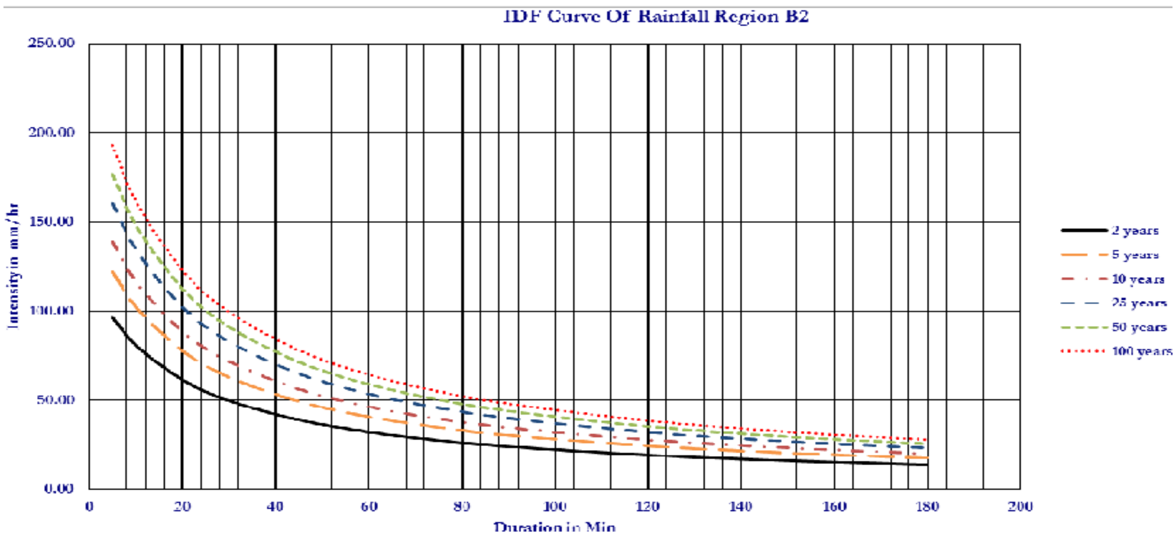

Figure 2: IDF curve for rainfall region “B” (source, ERA Drainage manual, 2013)

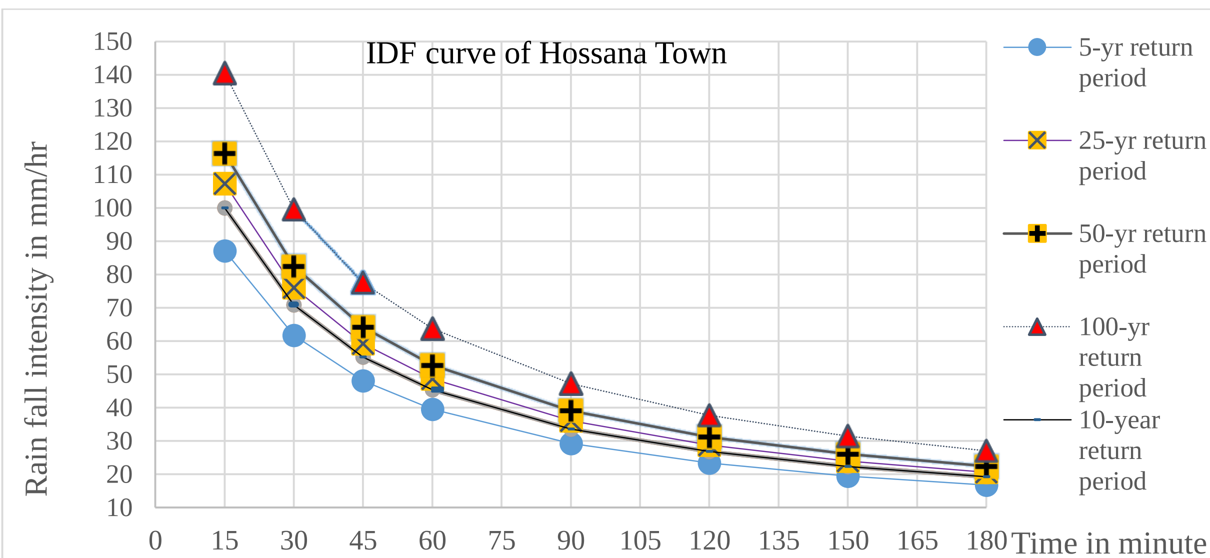

Figure 3: Hossana town IDF curve

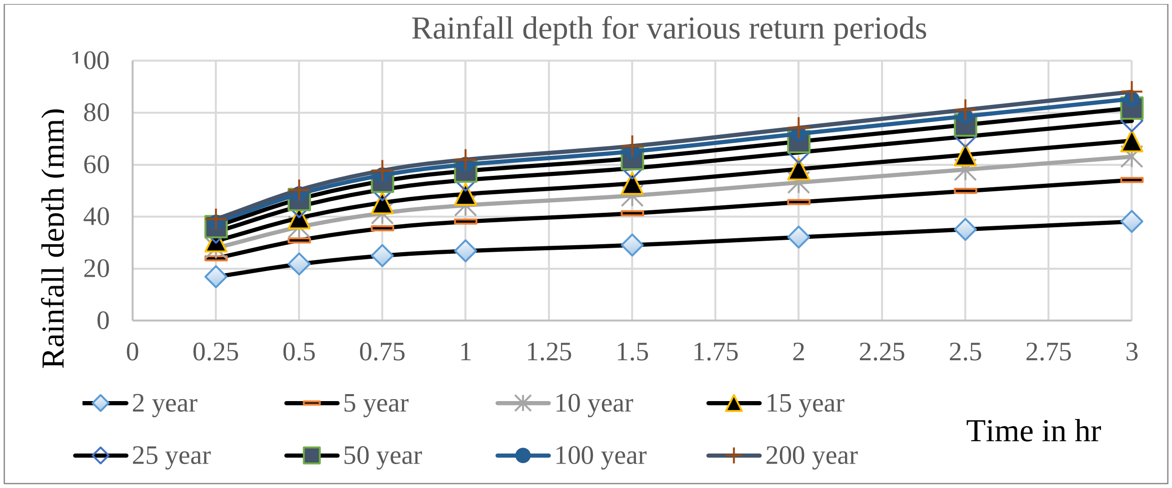

Table 5: Floodingnodes for 10 and 25 year return periods Rainfall data (SWMM Hydraulic analysis)

Flooding Node		Hours Flooded		Max Rate 10 yr (Cms)		Hours of maxi- Flooding		Total flood volume10^6 ltr	
10yr	25yr	10yr	25yr	10yr	25yr	10year		10year	25 year
J1	J1	0.9	0.94	5.88	7.655	0:40	0:40	5.657	7.95
J2	J2	0.46	0.59	1.595	2.313	0:40	0:40	0.856	1.603
J3	J3	0.18	0.23	0.615	0.898	0:40	0:40	0.188	0.333
J9	J9	2.83	5.84	0.903	0.903	0:41	0:39	2.907	3.172
J4	J4	0.39	0.67	0.721	0.982	0:40	0:40	0.335	0.576
J10	J10	0.9	0.94	0.229	0.229	0:37	0:33	0.613	0.708
J8	J8	0.98	0.99	1.207	1.042	0:41	0:57	3.497	3.245
	J64		*0.05*		*0.393*		*0:41*		*0.038*
J53	J53	0.9	0.94	0.949	1.217	0:40	0:40	0.862	1.191
J63	J63	2.63	5.25	0.306	0.378	0:40	0:40	0.359	0.453
J56	J56	0.81	0.9	4.289	5.682	0:40	0:40	4.557	6.952
J16	J16	0.35	0.63	0.482	0.67	0:41	0:41	0.242	0.451
J66	J66	2.86	5.86	2.147	2.617	0:40	0:40	4.203	5.094
J58	J58	2.63	5.48	0.346	0.428	0:40	0:40	0.404	0.51
J7	J7	0.12	0.16	0.261	0.425	0:40	0:40	0.057	0.122
J57	J57	1.31	1.33	1.919	2.441	0:40	0:40	2.725	3.459
	J34		*0.08*		*2.928*		*0:40*		*0.442*
J51	J51	1.18	1.23	1.758	2.242	0:40	0:40	2.371	3.055
J35	J35	0.19	0.47	0.877	1.555	0:40	0:40	0.309	0.855
J40	J40	2.85	5.86	8.589	11.043	0:40	0:40	18.725	23.887
	J69		*5.87*		*0.062*		*0:57*		*0.454*
J43	J43	1.95	2.34	0.069	0.72	1:02	0.57	0.269	0.31
J60	J60	1.05	1.07	3.461	4.335	0:40	0:40	3.971	5.099
J11	J11	1.46	1.54	2.137	2.758	0:40	0:40	3.367	4.436
J33	J33	0.2	0.32	2.026	2.144	0:43	0:45	1.175	1.841
J48	J48	1.09	1.17	2.169	2.879	0:40	0:40	2.952	4.1
J45	J45	2.75	5.77	0.337	0.338	0:40	0:14	1.757	1.857
A-RIVER		2.92	5.92	5.983	7.707	0:40	0:40	12.208	15.451

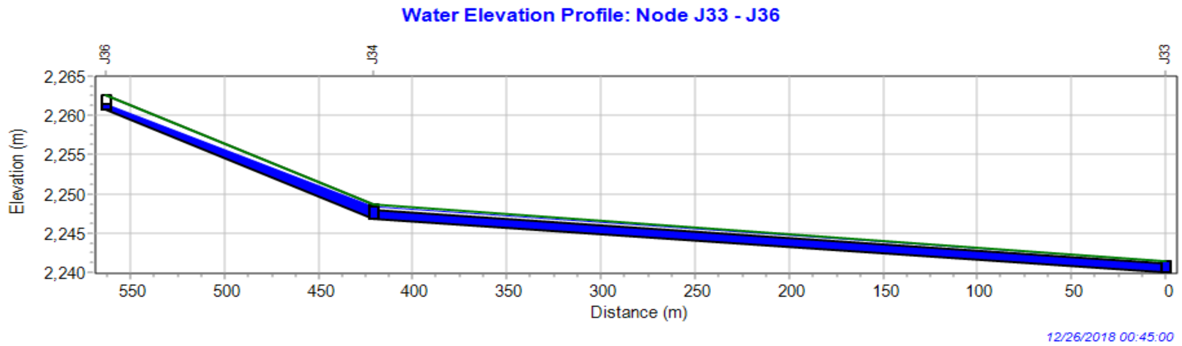

Figure 4: Water depth profile by SWMM for 10 year return period.

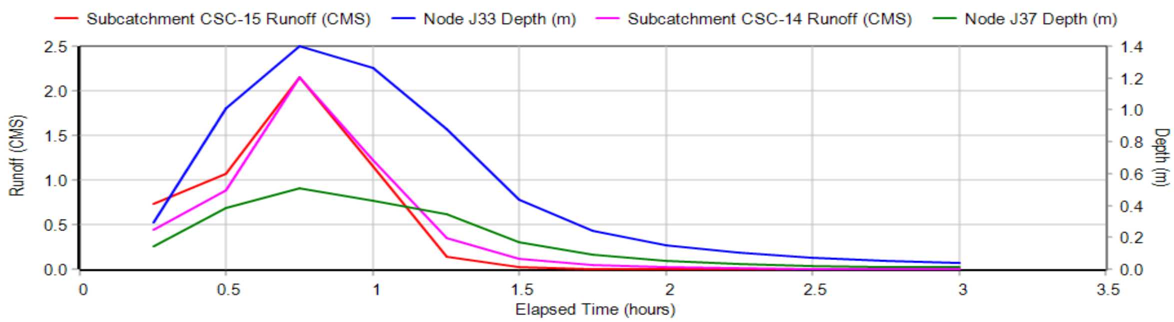

Figure 5: Node discharge carrying capacity (CSC-15 with J33 and CSC-14 with J37)

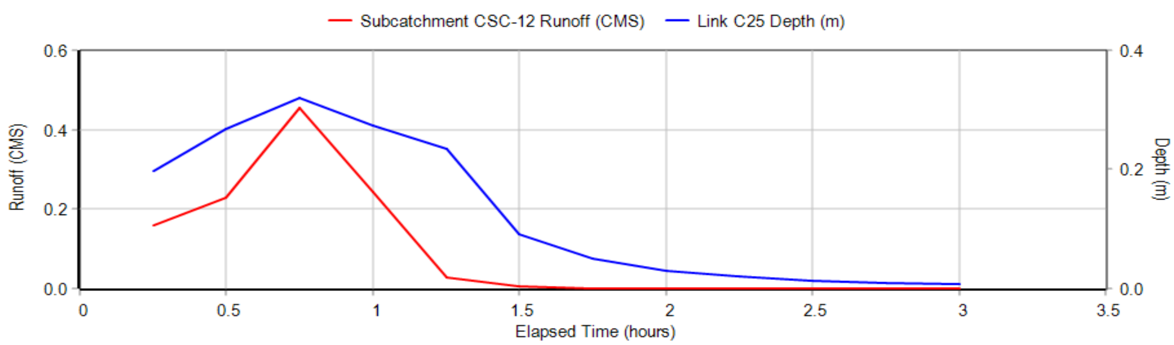

Figure 6: Conduit sub-catchment discharge carrying capacity

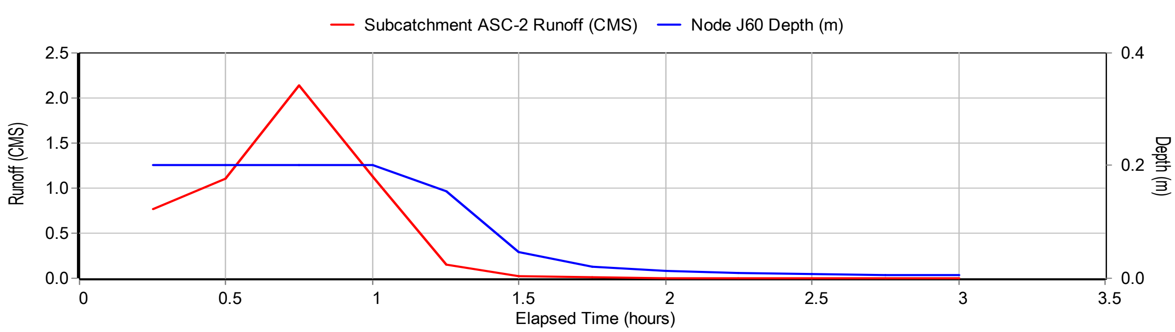

Figure 7: Nodes depth discharge carrying capacity for10 year floods

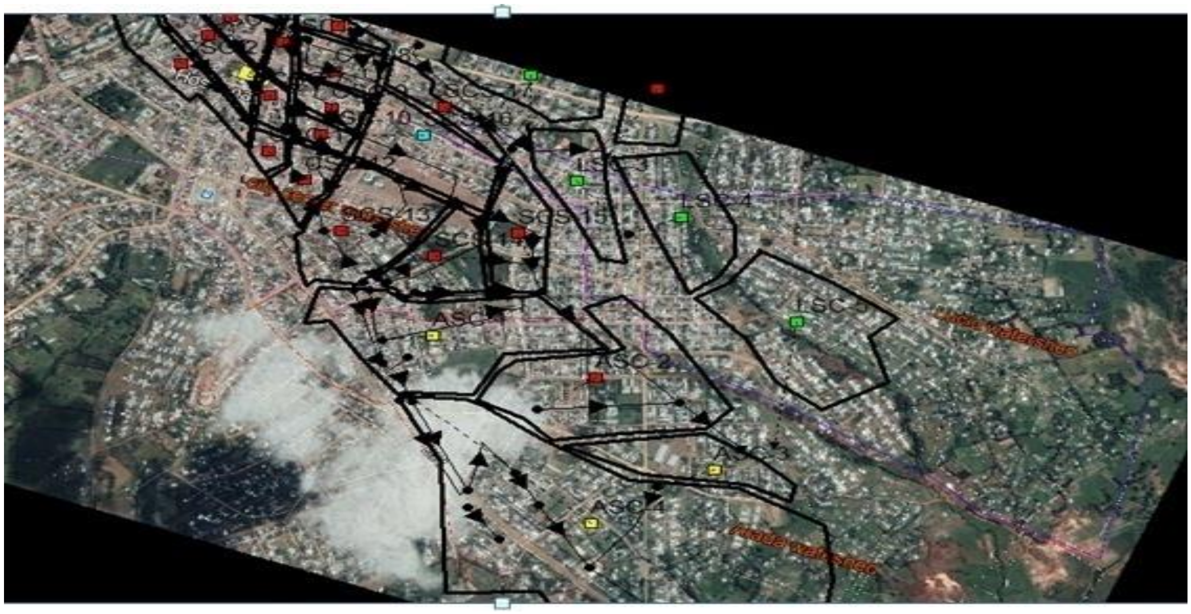

Figure 8: Sub-catchment location and flow direction map lay out (SWMM analysis)

Table 6: Catchment runoff coefficient and Imperviousness determination methods as Sample

Name polygons	Ai(m2)	Land use condition	Name of catchment	Area of subcatchment	water width	runoff coefficient	Impervious ratio	Average runoff coefficient	Average Imperviousness
cc1	66044.14	Public Blg				0.65	38		
cc2	8496.5	gravel				0.5	70		
cc3	20827.4	asphalt				0.97	80		
cc4	3919.027	cobel stone	**CCSC-1**	**17.49349722**	**1325.265**	0.5	70	**0.62778536**	**53.75286**
cc5	6713.135	resd				0.8	51		
cc6	46264.66	mixed bulg				0.55	51		
cc7	9836.578	cobel stone				0.5	70		
cc8	12833.53	gravel				0.5	70		
cc-13	69001.18	mixed resd				0.8	38		
cc-14	66012.28	school				0.55	34		
cc-15	179611.2	resd				0.8	51		
cc-16	5031.248	gravel	**CCSC-2**	**37.76077995**	**2006.418**	0.5	70	**0.77838862**	**50.35641**
cc-17	57951.88	asphalt				0.97	80		
cc-32	32747.34	resd bulg				0.8	51		
cc-34	12573.72	mixed bulg	**CCSC-3**	**5.768115324**	**703.4287**	0.8	51	**0.73571511**	**55.07138**
cc-36	5435.135	cobel stone				0.5	70		
cc-38	6924.955	cobel stone				0.5	70		
cc-53	23381.06	mixed bulg				0.8	51		
cc-54	5923.406	resd				0.8	51		
cc-55	18493.48	comercial	**CCSC-4**	**5.40977631**	**400.7242**	0.82	56	**0.78744079**	**55.25249**
cc-56	4511.2	cobel stone				0.5	70		
cc-57	1788.619	asphalt				0.97	80		
cc-42	7009.922	comercial				0.82	56		
cc-45	17051.11	comercial	**CCSC-5**	**2.61083011**	**263.7202**	0.82	56	**0.83176218**	**57.88195**
cc-46	2047.271	asphalt				0.97	80		
cc-23	9118.655	comercial				0.82	56		
cc-24	10316.1	comercial	**CCSC-6**	**5.563755049**	**341.3347**	0.82	56	**0.85474373**	**70.27912**
cc-25	28761.6	asphalt				0.97	80		
cc-26	7441.2	cobble stone				0.5	70		

Note (CCSC- city center sub-catchment, LSC lucid sub-catchment)

Table 7: Peak Discharge by Soil Conservation Service (SCS) Curve number method

12	13	14	15	16	17	18	19	20	21	22	23	24	25	
Duration	Daily point rainfall for 10yrs Rt	Rainfall profile in percentage	Rainfall magnitude	Areal to point rainfall ratio	Areal rainfall	Incremental rainfall	order	Rearrang	Rearranged	Cumulative	Times of incremental hydrograph			
hr	mm	%	mm	%	mm	mm		No.	mm	mm	T-beginning(hr)	T-peak(hr)	Tend(hr)	Intesity (mm
0-0.08333		10	7.2	98.5	7.092	7.092	1	2	1.3752	1.3752	0.08333333	0.683	2.67	16.502466
0.08333-0.167		12	8.64	98	8.4672	1.3752	2	12	1.8792	3.2544	0.16666667	0.8496667	2.8366667	22.55049
0.1667-0.25	72	15	10.8	98	10.584	2.1168	3	9	1.9332	5.1876	0.25000001	0.933	2.92	23.198493
0.25-0.333		18	12.96	97	12.5712	1.9872	4	7	1.9764	7.164	0.33333335	1.0163334	3.0033334	23.716895
0.333-0.4167		22	15.84	97	15.3648	2.7936	5	10	2.052	9.216	0.41666669	1.0996667	3.0866667	24.624098
0.4167-0.5		27	19.44	96.5	18.7596	3.3948	6	5	2.7936	12.0096	0.50000003	1.183	3.17	33.523334
0.5-0.5833		30	21.6	96	20.736	1.9764	7	1	7.092	19.1016	0.58333337	1.2663334	3.2533334	85.10434
0.5833-0.667		33	23.76	95.5	22.6908	1.9548	8	6	3.3948	22.4964	0.66666671	1.3496667	3.3366667	40.737763
0.667-0.75		36	25.92	95	24.624	1.9332	9	3	2.1168	24.6132	0.75000005	1.4330001	3.4200001	25.401702
0.75-0.833		39	28.08	95	26.676	2.052	10	4	1.9872	26.6004	0.83333339	1.5163334	3.5033334	23.846495
0.833-0.9167		42	30.24	94.5	28.5768	1.9008	11	8	1.9548	28.5552	0.91666673	1.5996667	3.5866667	23.457694
0.9167-1		45	32.4	94	30.456	1.8792	12	11	1.9008	30.456	1	1.683	3.67	22.809691

26	27	28		29	30	31	
Land use type	Area ratio	Hydrologic condition		Curve No.	Weighted CN	Sum weighted CN	
asphalt	0.1	Good (Hydrologic soil gr		98	9.8	AMC	CN
gravel	0.12	Good (Hydrologic soil gr		85	10.2	II	87.56
commericial	0.18	Soil group B)		92	16.56	III	94.2
Residential area with avg	0.6	Siol group B	85		51	$S = \frac{25400}{CN} - 254$	
	Weighted CN				87.56	S	15.64

Time(hr)	P (mm)	Q (mm)	Time(hr)	P (mm)	Q (mm)
0.083	1.3752	0.2212331	0.75	24.6132	12.433975
0.17	3.2544	0.0010134	0.83	26.6004	14.086417
0.25	5.1876	0.2396637	0.91	28.5552	15.743525
0.33	7.164	0.8278764	1.00	30.456	17.380832
0.42	9.216	1.7058056			
0.50	12.0096	3.2168708			
0.58	19.1016	8.0710801			
0.66	22.4964	10.715569			

Time	H1	H2	H3	H4	H5	H6	H7	H8	H9	H10	H11	H12	SUM	
0.08	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00		
0.17	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
0.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	
0.33	0.00	0.00	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.09	
0.42	0.00	0.00	0.18	0.22	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.40	
0.68	0.00	0.00	0.47	0.93	1.06	1.25	0.00	0.00	0.00	0.00	0.00	0.00	3.71	
1.00	0.00	0.00	0.81	1.78	2.32	3.42	9.17	0.00	0.00	0.00	0.00	0.00	17.50	
0.85	0.00	0.00	0.74	1.38	1.72	2.40	5.86	2.19	0.00	0.00	0.00	0.00	14.29	
0.93	0.00	0.00	0.74	1.82	2.05	2.97	7.69	3.19	1.43	0.00	0.00	0.00	19.89	
1.02	0.00	0.00	0.71	1.82	2.72	3.54	9.53	4.19	2.07	1.37	0.00	0.00	25.95	
1.10	0.00	0.00	0.68	1.74	2.72	4.68	11.36	5.19	2.72	2.00	1.37	0.00	32.47	
1.68	0.00	0.00	0.46	1.21	1.92	3.50	15.03	12.18	7.27	6.36	5.76	5.07	58.76	
2.67	0.00	0.00	0.09	0.31	0.57	1.18	4.41	8.19	14.96	13.76	13.17	12.40	69.03	maax discharge of the given chachment
2.84	0.00	0.00	0.03	0.15	0.34	0.78	3.15	2.060355	5.32	15.01	14.42	13.63	54.91	
2.92	0.00	0.00	0.00	0.08	0.23	0.59	2.52	1.716962	1.34	5.12	15.05	14.25	40.89	
3.00	0.00	0.00	-0.03	0.00	0.11	0.39	1.89	1.37357	1.12	1.287438	5.13	14.87	26.14	
3.09	0.00	0.00	-0.06	-0.08	0.00	0.20	1.26	1.030177	0.89	1.072865	1.291074	5.07	10.67	
3.67	0.00	0.00	-0.28	-0.61	-0.80	-1.18	-3.15	-1.37357	-0.67	-0.42915	-0.21518	0	-8.70	

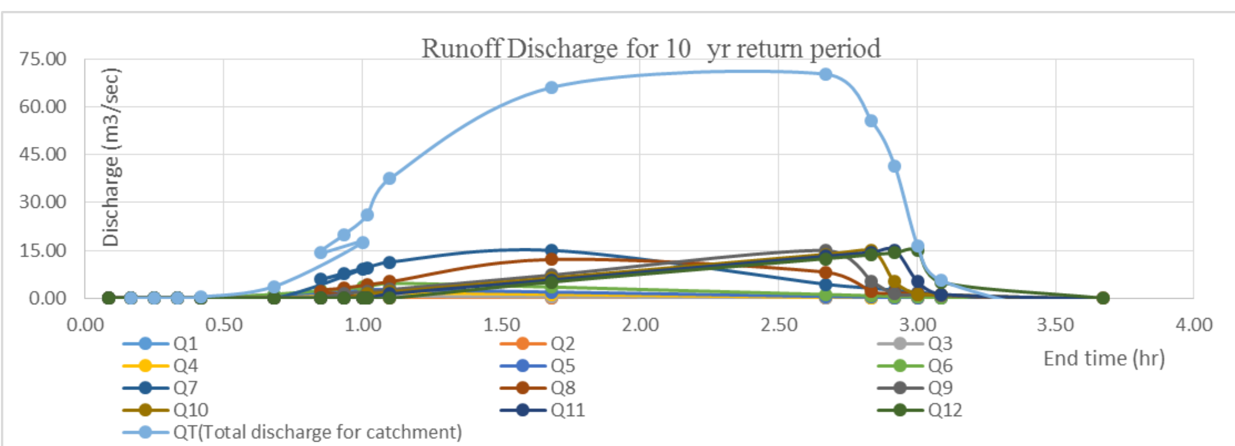

Table 8: DischargeComparisons by SCS,Rational method and SWMM for different year RT

Sub-catch ID	QR-10 year	QSW-10 year	QSC-10 year	QR-25 year	QSW-25 year	QSC-25 year	QR-50 year	QSW-50 year	QSC-50 year	QR-100 year	QSW-100 year	QSC-100 year
CC-SC-1	2.76	2.71	2.60	3.25	3.43	3.39	3.82	3.66	3.62	4.76	4.17	4.14
CC-SC-2	7.15	7.11	5.69	8.39	8.90	7.31	9.87	9.45	7.81	12.27	10.71	8.93
CC-SC-3	1.18	1.21	0.87	1.39	1.49	1.12	1.65	1.58	1.19	2.07	1.78	1.36
CC-SC-4	1.18	1.09	0.82	1.40	1.35	1.05	1.65	1.43	1.12	2.08	1.61	1.28
CC-SC-5	0.60	0.56	0.39	0.71	0.69	0.51	0.84	0.73	0.54	1.06	0.82	0.62
CC-SC-6	1.32	1.14	0.84	1.56	1.41	1.08	1.84	1.50	1.15	2.32	1.69	1.32
CC-SC-7	0.70	0.62	0.47	0.82	0.77	0.60	0.98	0.81	0.65	1.23	0.92	0.74
CC-SC-8	1.27	1.20	0.95	1.49	1.50	1.23	1.76	1.59	1.31	2.20	1.80	1.50
CC-SC-9	0.79	0.77	0.54	0.93	0.96	0.70	1.10	1.01	0.75	1.38	1.14	0.86
CC-SC-10	0.77	0.70	0.49	0.91	0.87	0.64	1.08	0.92	0.68	1.35	1.03	0.78
CC-SC-11	1.85	1.71	1.19	2.18	2.10	1.53	2.58	2.22	1.63	3.25	2.49	1.87
CC-SC-12	0.84	0.80	0.58	0.99	0.99	0.74	1.17	1.05	0.79	1.48	1.18	0.91
CC-SC-13	2.15	2.02	1.87	2.52	2.55	2.40	2.95	2.71	2.56	3.65	3.09	2.93
CC-SC-14	2.76	2.92	2.74	3.24	3.71	3.58	3.81	3.95	3.82	4.73	4.52	4.37
CC-SC-15	3.88	3.72	2.73	4.57	4.62	3.50	5.41	4.89	3.74	6.80	5.51	4.28
CC-SC-16	3.09	3.16	3.63	3.61	4.07	4.65	4.21	4.35	4.96	5.14	5.01	5.68
CC-SC-17	1.79	1.85	1.62	2.10	2.34	2.08	2.47	2.49	2.22	3.06	2.83	2.54
ASC-1	2.65	2.40	3.25	3.08	3.08	4.17	3.60	3.29	4.45	4.39	3.78	5.10
ASC-2	3.81	3.72	2.64	4.50	4.61	3.39	5.33	4.87	3.62	6.70	5.49	4.14
ASC-3	2.60	2.29	2.82	3.04	2.92	3.62	3.56	3.11	3.82	4.38	3.57	4.42
ASC-4	8.57	8.60	12.42	9.98	11.06	15.95	11.64	11.83	17.02	14.23	13.63	19.5
LSC-1	2.52	2.51	3.01	2.94	3.22	3.86	3.42	3.44	4.12	4.18	3.96	4.71
LSC-2	4.78	4.67	3.25	5.64	5.77	4.17	6.67	6.11	4.45	8.40	6.88	5.09
LSC-3	2.52	2.53	3.42	2.94	3.26	4.39	3.42	3.49	4.69	4.16	4.02	5.36
LSC-4	3.46	3.46	4.41	4.03	4.46	5.67	4.70	4.77	6.05	5.74	5.50	6.93
LSC-5	3.57	3.65	5.69	4.13	4.73	7.30	4.79	5.06	7.78	5.77	5.86	8.91

Note
QR- Discharge by Rational method
QSW- Discharge by SWMM
QSC –Discharge by SCS curve method

Areal to Point Ratio (%)

Area km²	Duration t(hrs)	0.50	1.00	2.00	3.00	4.00	5.00	6.00	9.00	12.00	15.00	18.00	21.00	24.00
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	61	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	98
275		42	54	65	70	74	76	78	81	84	85	86	87	88
300		41	53	54	70	73	75	77	81	83	65	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	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

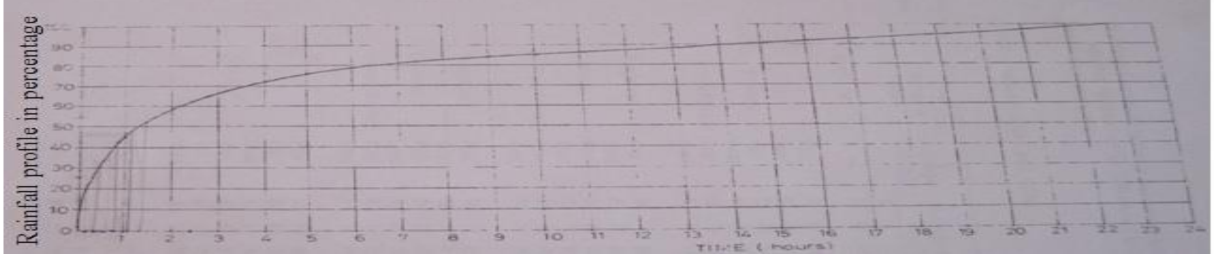

Figure 9: Incremental rainfall preparing chart in SCS runoff curve number method

K_T **values for Pearson Type III distribution (negative skew)**

Skew coefficient C_s or C_w	2	5	10	25	50	100	200
	Exceedence probability						
	0.50	0.20	0.10	0.04	0.02	0.01	0.005
−0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
−0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388
−0.3	0.050	0.853	1.245	1.643	1.890	2.104	2.294
−0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
−0.5	0.083	0.856	1.216	1.567	1.777	1.955	2.108
−0.6	0.099	0.857	1.200	1.528	1.720	1.880	2.016
−0.7	0.116	0.857	1.183	1.488	1.663	1.806	1.926
−0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
−0.9	0.148	0.854	1.147	1.407	1.549	1.660	1.749
−1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
−1.1	0.180	0.848	1.107	1.324	1.435	1.518	1.581
−1.2	0.195	0.844	1.086	1.282	1.379	1.449	1.501
−1.3	0.210	0.838	1.064	1.240	1.324	1.383	1.424
−1.4	0.225	0.832	1.041	1.198	1.270	1.318	1.351
−1.5	0.240	0.825	1.018	1.157	1.217	1.256	1.282
−1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
−1.7	0.268	0.808	0.970	1.075	1.116	1.140	1.155
−1.8	0.282	0.799	0.945	1.035	1.069	1.087	1.097
−1.9	0.294	0.788	0.920	0.996	1.023	1.037	1.044
−2.0	0.307	0.777	0.895	0.959	0.980	0.990	0.995
−2.1	0.319	0.765	0.869	0.923	0.939	0.946	0.949
−2.2	0.330	0.752	0.844	0.888	0.900	0.905	0.907
−2.3	0.341	0.739	0.819	0.855	0.864	0.867	0.869
−2.4	0.351	0.725	0.795	0.823	0.830	0.832	0.833
−2.5	0.360	0.711	0.771	0.793	0.798	0.799	0.800

K_T **values for Pearson Type III distribution (positive skew)**

Skew coefficient C_s or C_w	2	5	10	25	50	100	200
			Exceedence probability				
	0.50	0.20	0.10	0.04	0.02	0.01	0.005
3.0	−0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	−0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	−0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	−0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	−0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	−0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	−0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	−0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	−0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	−0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	−0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	−0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	−0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	−0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	−0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	−0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	−0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.3	−0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	−0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	−0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	−0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	−0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	−0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	−0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	−0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	−0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	−0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	−0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.2	−0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	−0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0	0.842	1.282	1.751	2.054	2.326	2.576

Figure 10: Field surveyed and observed flood associated problems and photos

FloodingAssociated problems of Hossana town during rainy season

Field data collecting mechanisms, and materials;   Sample of Flooding problem of the study site