PERFORMANCE ASSESSMENT OF STORM WATER DRAINAGE SYSTEM OF HOSANNA TOWN THE CASE OF GOFER-MEDA SUB-CITY USING STORM WATER MANAGEMENT MODEL 5.1

M.Sc. THESIS

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LIST OF ABBREVIATION AND ACRONYMS

AACRA	Addis Ababa City Road Authority
AMC	Antecedent moisture condition
BDG	Building
CN	Curve Number
CSA	Central Statistics Agency
DQ	Design Quality
EMA	Ethiopian Map Agency
ERA	Ethiopian Road Authority
FAO	Food and Agricultural Organization
FHWA	Federal Highway Administration
FUPI	Federal Urban planning Institute
HSG	Hydrologic Soil Group
GIS	Geographic Information System
GPS	Ground positioning System
GTZ	German Technical Cooperation
GEV	Generalized Extreme-Value
LID	Low-Impact Development
LULC	Land use/Land cover
NRCS	Natural Resource Conservation Service
RF	Rainfall
SCS	Soil conservation service
SDS	Sustainable Drainage System
SNNPR	Southern Nation Nationality People Region
EPA SWMM	Environmental protection Agency Storm Water Management Model
UN	United nation.
USACE	United States Army Corps of Engineers
USW	Urban storm water

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDIX TABLES

LIST OF APPENDIX FIGURES

DECLARATION

I declares that, this thesis which I submit to the school of Graduate studies of Hawassa University in partial fulfillment of degree of masters of Science in Hydraulics Engineering. I reasonably ensure that the work is original and to the best of my knowledge and has not been taken from other source except where such works have been cited and acknowledged with in the text.

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ABSTRACT

Urban storm water drainage systems are an increasingly important part of urban infrastructure that conveys storm water away from urban areas. Since such infrastructure play a key role in preventing urban floods, their performance should be monitored and quantified. In the study area, there is improper interconnection between road and urban storm water drainage system, lack of periodic maintenance, waste damping and absence of adequate size drain which result flooding during rainy season and damages properties and road users. The objective of this study was performance assessment of storm drainage system of Hosanna town for the case of Gofer-meda sub-city. The various Meteorological data and spatial data were collected to assess performance of drainage system. Rainfall data for period of (1988- 2017) was used to identify probability distribution and to develop the intensity duration frequency (IDF) curve for Hosanna town. Log-Pearson type III have better R^2 *value in range(0.6-0.88) and best fit of goodness of easy fit professional software and also used for this study to analysis of design RF for required return period 10 year for design and 25 year to check. The IDF Curve developed for various return periods to analyze RF and determine adequacy of existing drainage system and Compute peak runoff using rational method and SWMM 5.1. Observation and field visit method were used to explore and describe existing condition of drainage system. Digital elevation model (DEM) used as input data for ArcGIS software which used to delineate the runoff contributing areas and to prepare the SWMM input parameter. Storm water management model (SWMM5.1) was used to simulate 1-D view of the flood depth in the drainage system. Manning's equation used for compute storm water drainage system capacity. Based on primary and secondary data collected, Storm water drainage system were designed with inadequate hydrological data and hydraulic analysis, the existing drainage system problems associated with in study area categorized as construction, community awareness and design. Total estimated peak discharge by rational method was 43.89m3/s and calculated existing drainage system capacity by manning's formula was 33 m3/s. So, approximately 25% additional drainage system provision needed to handle coming flood. Therefore; this study recommends improvement of existing storm water drainage system, integration of solid waste management system in order to prevent the over flowing of floods as a result of blockage of drains, the drainage structures should be carefully cleansed, and suitable measures should be taken in order to make and drainage structures serve for intended purposes.*

Key words: *Storm drainage system, LULC, sub-catchment, IDF Curve, Google earth pro, EPA SWMM 5.1*

1. INTRODUCTION

1.1 Background

Urban drainage systems are an increasingly important part of urban infrastructure that conveys storm water away from urban areas. Drainage systems are required in developed urban areas for the safety road and proper environmental condition. Urbanization along with its impermeable structures and improper design are the major causes of flooding in most developing urban areas. Moreover, this urbanization affects the performance of the drainage lines (Dagnachehu, 2011).

Urbanization leads to impermeabilization of the soil by means of paving and soil compaction from the passage of vehicles and from buildings, among other things, which impedes rainwater infiltration (Silveira *et al*., 2001). Inadequate urban storm water drainage problems represent one of the most common source compliant from the citizens in many towns of Ethiopia (GTZ-IS, 2006).

In many towns of developing countries the main cause for the urban flooding is the result of improper drainage system and poor integration of road with drainage system (Anteneh, 2015). In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall. Therefore, to control runoff at source and its effects and to safeguard the disposal of runoff through proper drainage facilities becomes essential (Mitiku and Mekdes, 2017). Storm water drain networks in towns are usually designed to collect and convey excess surface runoff effectively in order to pervert urban flooding (Gouri and Srinivas, 2015).To overcome the problem of urban flooding, engineering prediction is essential to control the peak flow and the maximum depth allocation. Several mathematical models was used to generate or simulate the dynamics of rainfall runoff and flood modeling. For this study SWMM 5.1 was used to simulate 1-D view of the flood depth in the drainage system and to delineate the extent of the inundated area by the flood.

The service level of drainage system can be reduced due to degradation, improper maintenance, inappropriate design and sedimentation. Therefore, this thesis focus on assessing the performance of the storm drainage system in Hosanna town particularly, Gofer-meda sub city.

1.2 Statement of the problem

Ethiopian Cities and towns are at large, suffering with storm water leading in to floods especially during the rainy season, problem is more critical in cities of highland regions. The storm water drainage system problem of Hosanna town currently facing represents one of the most common sources of complaints from the public. Hosanna town is like other towns of Ethiopia has a lot of problems regarding drainage infrastructure like inadequacy and quality problem. In addition to this increased population density and impermeability of the urban landscape, the planning as well as implementation of storm water protecting structures and management is poor.

In previous times, a little in number of drainage systems have been constructed in Hosanna town to be used for storm drainage system. Most of such storm water drainage systems were designed under inadequate hydrological data and hydraulic analysis.

In the study area due to lack of proper interconnection between road and drainage system, poorly maintenance of existing storm drainage system and solid waste damping in drainage systems, street flooding is common during and after the rainfall. Miss-management of waste disposal and sediment in the drainage canal further aggravate to decrease the capacity of canals.

Currently some of the problems that have been observed in the Hosanna town especially Gofer-meda sub-city are: Edge crack, Shoulder erosion, Abutment damage and Silted drainage ditches which cause over toping of rainy water from drainage system. Another problem that observed in Gofer meda sub-city is that inadequate dimension and placement of the curb inlet to remove overland flow from road surface. Assessing performance of existing drainage system is very essential to perspective of public safety and safeguarding property. However, performance assessment of storm drainage system in hosanna town is not yet sightseen. Consequently, this research used the existing climatic and spatial data to assessing performance of storm drainage system in Hosanna town.

1.3 Objectives of study

1.3.1 General objective

The main objective the study was to assess a performance of storm drainage system of Hosanna town the case of Gofe-Meda town using Storm Water Management Model (SWMM 5.1).

1.3.2 Specific objectives

1. To assess the existing condition and major Causes of problem associated with drainage system performance of Gofer-Meda Sub-city.
2. To develop the intensity duration Frequency (IDF) Curve of the Hosanna town for various return period.
3. To evaluate hydraulic performance of existing drainage system and delineate areas which contribute flow toward drainage system.
4. To estimate peak discharge by SWMM 5.1 and identify flooded junctions (flood prone area).

1.4.Research Questions

The fundamental questions that was addressed and investigated are.

1. What it looks like the existing condition and Source of problem that are associated with the drainage system performance?

2. Do existing storm water canals could able to carry a flood within a design period?

3. What was peak discharge and where was flood prone area?

1.5.Scope of the Study

This research focused on assessing performance of existing storm drainage system by aspect of construction, design, management and operation to solve flooding problem. The study was geographically limited to Hosanna town, particularly in Gofer meda sub-city it the most flooded sub-city and densely urbanized than other parts of town. Based on available data; the design, construction and the operational management of the drainage system are evaluated and a possible mitigation measure is recommended. This study comprises thirty year daily maximum RF data to develop IDF Curve by using log Pearson type III frequency analysis method to estimate peak discharge for different return period by rational method and SWMM. Manning's equation used for calculate drainage system capacities. This study does not include structural design of all types of drainage structures except assessing performance of the existing drainage system.

1.6.Significance of the Study

The study will encourage to reduce the environmental, health safety problems, to create awareness for stakeholders about drainage systems function and purpose. To prepare IDF Curve for specific station Hosanna town by using 30 years daily maximum rainfall which is used for hydrological design of peak discharge. The concerned body and municipality can use this as a reference for proper design, implementation and maintenance of urban storm water drainage system. To differentiate flood prone areas and inform for concerned body to take appropriate measure to prevent flooding problem.

2. LITERATURE REVIEW

2.1.Definitions of Urban Drainage System

Drainage has meaning to remove or to divert water. In generally, defined as a technical measure to reduce the excess water whether from rain, seepage or excess water irrigation from an area or land as a function of undisturbed area. Surface drainage systems usually applied in relatively flat lands that have soils with a low or medium infiltration capacity, or in lands with high -intensity rainfalls that exceed the normal infiltration capacity, so that frequent water logging occurs on the soil surface (Subandiyah, 2016).

Urban drainage includes the removal of all unwanted water from urban areas. It includes wastewater including sewerage and grey water and storm water. Surface storm drainage involves collecting the water from the road surface, road shoulders, side slopes and adjacent areas and carrying it away via downhill slopes, roadside ditches and pipes. (McDonough and Braungart, 2002).

2.2.Components of Urban Drainage Structures

2.2.1.Major and Minor Drainage Systems

As studied by Alderson (2006), the collection and discharge of storm water from the Road infrastructure, complete storm drainage system design includes consideration of both major and minor drainage systems. The minor system consists of curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc. The minor system is normally designed to carry runoff from 10 year frequency storm events. The major system provides overland relief for storm water flows exceeding the capacity of the minor system. This usually occurs during more infrequent storm events, such as the 25, 50, and 100 year storms. The major system is composed of pathways that are provided knowingly or unknowingly for the runoff to flow to natural or manmade receiving channels such as streams, creeks, or rivers (Habtamu, 2017).

The design of road drainage systems varies with factors such as road importance and age, traffic load and rural/urban area. A surface drainage system (ditch) collects and diverts storm water from the road surface and surrounding areas to avoid flooding. It also prevents damage to subsurface drains, water supplies (wells) and other sensitive areas adjacent to roads. It decreases the possibility of water infiltration into the road and retains the road bearing capability (Faisca *et* al., 2009).

Subsurface drainage system drain water that has infiltrated through the pavement and the inner slope, but also groundwater .Subsurface drainage system usually comprise culverts and have direct linkage to surface drainage systems(Dawson,2009).

As stdied by Kalantari (2011), in the title of Adaptation of road drainage structures to climate c hange, Different types of structures are employed in the drainage systems are open channels whether artificial or natural convey the flows of water; surface and sub-surface drainage systems; culverts and bridges convey flows under road cross-section; energy dissipaters, used to control the velocities of flows, especially at culvert outlets.

According to Reddy and Nora (2018), Performance Assessment of Road Drainage Systems of Burayu town, Oromia Region, Ethiopia. Different types of structures are employed in the drainage systems are open channels whether artificial or natural convey the flows of water; surface and sub-surface drainage systems; culverts and bridges convey flows under road cross-section; energy dissipaters, used to control the velocities of flows, especially at culvert outlet. The study focus on the performance assessment of road drainage system. Analysis of the collected data was carried out by rational method dividing the study area in two equal parts in 30 ha/0.3km^2 and comparing with SCS methods in entirely 60ha/0.6km^2 finally they got the same result in both methods.

According to Linmei, (2003) and Rossman (2016), the major urban drainage system includes catchment, nodes, conduit, links and outlet.

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

Nodes: are junctions to link the storm drains. They provided storm water transition between surface and subsurface systems.

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

Outlet (outfall): is the most downstream component of drainage system or flow removal point

2.3.Urban Storm Water Drainage Practice in Ethiopia

In Ethiopian context, where watersheds of many urban centers receive significant amount of annual rainfall and where rainfall intensity is generally high, control of runoff at the source, flood protection, and safe disposal of the excess water/runoff through proper drainage facilities become essential(FUPCoB,2008). Storm water drainage problem is one of the major challenges facing many countries in Ethiopia. So, the study focuses on the assessment of storm water drainage system using statistical analysis, geographic information system (GIS) and other materials (Biniyam, 2016). A statistical analysis was applied to analyze the existing condition system and to analyze the peak discharge for the town and to fix the diameter of the pipe. GIS software was used to delineate watershed by combining DEM and aerial photograph. Based on primary and secondary data collected, the problems in the areas are categorized as, management and design problem. The method used to investigate management problem is direct field data collection and site visit but the construction problem is analyzed using field survey as well as comparison of design with what is implemented in the ground. Design of the study area is evaluated by redesigning of the system using the computation method used in (ERA, 2013) drainage design manual.

For hydrologic analysis some factors should be evaluated and included because they have a significant effect on the final results; they are Drainage basin characteristics, shape, slope land use, geology, soil type, surface infiltration. Stream channel characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradation, degradation. Meteorological characteristics including precipitation amount and type. (ERA, 2013).

2.4.Urban Drainage system and its Importance

As studied by Kokeb (2016), assessment of road and surface water drainages system in Ethiopia with special reference of Assosa town. Adequate drainage systems are needed in urban areas because of the interaction between human activity and the natural water cycle. This interaction has two main forms: the abstraction of water from the natural cycle to provide a water supply for human life, and the covering of land with impermeable surfaces that divert rainwater away from the local natural system of drainage.

The first type, wastewater, is water that has been supplied to support life, maintain a standard of living and satisfy the needs of industry. After use, if not drained properly, it could cause pollution and create health risks. The second type of water requiring drainage, storm water, is rainwater (or water resulting from any form of precipitation) that has fallen on a built-up area. To describe and investigate the existing condition, coverage and level of integration between road and storm drainage infrastructures, exploratory and descriptive types of study design were used. Field survey and an in depth interview with respondents were methods of primary data collecting.

2.5.Urban Storm Water Drainage Problems

According to Novatech, *et al*. (2001) the practice of urban drainage in developing countries encounters more serious problems than those of developed countries, because urban development occurs under more difficult socioeconomic, technological and climatic conditions.

2.5.1.Solid waste management operations

Lack of coordination and integration between urban storm water management activities and other urban service can affect the sustainable and cost-efficient operation of urban drainage systems (Parkinson, 2006).

Urban areas in developing countries have significant proportions of exposed soil liable to erosion and giving rise to large quantities of sediment. Building sites, whether in areas where the city is expanding or within the developed urban area, do not normally have controls for erosion prevention or for retaining sediment so that it does not reach the streets, storm drains and urban rivers. It is no exaggeration to say that 10 to 15% of urbanized area in developing countries contributes extensively to sediment production and transport. The amount of garbage entering the drainage network is reduced corresponding to a production of 0.4 to 0.8% of total garbage produced (Tucci, 2000).

2.5.2.Absence of Community Participation

Lack of community participation in the examination for permanent solutions for urban drainage problems is one of the main obstacles preventing the success of modern storm runoff control measures, whether by structural or non-structural measures. In most developing countries this has been a problem for sustainable storm water drainage management. Lack of community participation leads to the repetition of earlier errors in solving drainage problems, to the discredit of public action, and lack of concern with environmental questions (ibid).

It can also bring about low investment in urban facilities (Novatech, *et al*. 2001). The community involvement is often suggested as a way of ensuring maintenance is carried out in low-income areas (Alahji Isa, 2010). In most developing countries it is the urban authorities that are responsible for waste management.

Non-structural approaches to storm water management can work well, provided the municipal the ruling classes are respective to involvement of community group in project implementation. Collaboration between government agency and non-governmental organizations, in conjunction with communities, is essential but often challenging (Alahji Isa, 2010).

Public participation is an opportunity for urban authorities to assess the social feasibility of storm water management system and flood response strategies. Urban drainage system cannot be designed in isolation from the community that they serve (NVSWCD, 1994).

2.5.3.Lack of Appropriate Technology

For the environmental approach to be successful, a change of technical culture is required through training (capacity building at all levels, for district engineers and urban planners) and environmental education for the people (Jones, 196).

The drainage system design: the life span of different components in the system can different a lot, and storm water pipes especially in old parts the town can be very old. The design criteria and urbanization have probably changed a bit since the first pipe were placed in the ground, which decrease to convey unexpected events.

Cross connections in the pipe (storm, and drainage pipes) can be problem due to a large variety of trash materials and in the system (treatment will be difficult).

2.6.Development of Storm Water Drainage Systems in Ethiopia

In many towns of Ethiopia the major challenges face regarding with storm water drainage system. The significant challenges of storm water drainage problems related with designing of sustainable and effective functioning of the drainage system (Mitiku and Makdes, 2017).The most difficult issues a community can face is the question of street flooding and its causes in most emerging towns of Ethiopia (Kokab, 2015).

2.6.1. Studies Related to Drainage system problem

In order to found the fact that drainage problem exists in the town and to understand the works that are done, literatures are reviewed. The literatures showed no doubt on the existence of drainage problem in the city. The appearance of the problems in the literature is presented either in the form of malfunctioning of specific component of the urban transportation or larger problems on urban drainage systems themselves. For instance, a few numbers of studies are performed on the drainage problems by post graduate students of Addis Ababa University and Hawassa University. The first thesis is titled, "Study of the Urban Drainage System in Addis Ababa, Yeka Sub-city" (Dagnachew, 2011).

The second thesis is titled, "Investigation on Storm Drainage Problem of Addis Ababa Case Study at Gotera-Wollo Sewer, Saris-Gotera and Ring Road" (Desalng, 2011).

2.7. Effects of poorly designed and constructed Drainage System on urban Roads

According to Jitendra, et al. (2013), the effect of drainage quality on structural and functional performance of pavement, by identifying a simple framework for quantification of the effect of drainage quality on structural as well as functional performance of the pavement. They presented the structural and functional performance of the pavement in predicted terms of deflection and roughness respectively.

Siddhartha, et al. (2012), carried out a research on drainage and flexible pavement performance; they pointed out that providing adequate drainage to a pavement system has been considered as. Important design considerations ensure satisfactory performance of the pavement, particularly from the perspective of life cycle cost and Serviceability.

According to Getachew, et al. (2015), based on their research the assessment of the effect of urban road surface drainage: a case study at Ginjo Guduru Kebele of Jimma Town. They assessed the pavement damage due to improper drainage, identified areas most prone to flooding problems, assessed the existing condition of road and surface drainage infrastructure, examine the impacts of road surface drainage structures integration on road performance and related social as well as environment issues and make recommendations on urban road and drainage structures integration, their provision and management. Dipnoan,(2014) studied highway surface drainage system and problems of water logging and concluded that adverse roadway elements contributing to highway accidents were substandard roadway alignment, lack of shoulder defects, absent or inappropriate pedestrian facilities, narrow and defective land bridges/bridge approaches, roadside hazards, undefined pavement center and edge lines, poor sight distance and visibility, unmarked and inappropriate design of intersections, serious allocation deficiencies along the route, haphazard bus shelters/stops, and others are causes of water logging problem in highway. This research traced that Proper drainage is a very important consideration in design of a highway.

Magdi (2016) investigated surface drainage problem of roads in Khartoum state, Sudan. A comprehensive study was done using appropriate land and traffic survey equipment. Several roads were surveyed which lead to conclusions as; The drainage problem is highly compounded in Khartoum state because of inadequate drainage system thereby resulting in damages of pavements and leading to unhealthy environment, poor drainage conditions especially during rainy seasons, force the water to enter the pavement from the sides as well as from the top surface. The most common causes of road drainage problem were fund related to improper road geometry, insufficient capacity of drainage structures, poor construction, and lack of proper maintenance.

2.8. Influence of urbanization and modernization on drainage system

As studied by Dagnachew (2011), Road and urban storm water drainage network integration in Addis Ababa: Addis Ketema Sub-city, with urbanization impermeability increases because of the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.). This in turn changes the drainage pattern, increases overland flow resulting in flooding and related environmental problems.

According to Habtamu (2017), Investigation of flooding problems in urban drainage system the case of Zenbe work in Addis Ababa, Ethiopia, the increased impervious areas such as roads, roofs and paving due to increasing development densities means more run-offs. Some major hydrological effects on urbanization are: increased water demand, often exceeding the available natural resources; increased wastewater, burdening rivers and lakes and endangering the ecology; increase peak flow (Debu, 2016).

2.9.Estimating Missing RF data and Adjustment of records

A complete measured precipitation data are important to many problems in hydrologic analysis and design but there are missing values, the causes of missing rainfall data due to failure of observer, vandalism/damage of recording gages and translation/shifting of gage (Garg, 2005). Some methods estimating missing Rainfall values are: station average method, Normal ratio method, inverse distance method. Station average method is simple but may not accurate when annual RF of catch at any n regional gages differ from the annual catchment the point of interest by more than 10%. When the average annual catches differ by more the normal-ratio method is preferable (Sabarmaniya, 2008).

2.9.1.Outlier test

An outlier test is an observation that deviates significantly from the bulk of the data, helps to avoid those data lie out the rang in the between the lowest datum and the highest datum, which may be due to errors in the data collection, recording or due to natural causes. The presence of outliers in the data causes difficulties when fitting a distribution to the data. Low and high outliers are both possible and have different effects on the analysis (Chow, 1988).

2.9.2.Test for Consistence of Recorded Rainfall Data

The continuity of a record may be broken with missing data due to many reasons such as damage so,before starting any procedures, it is important to check whether the data were homogenous, consistence and complete with no missing data. If the missing data existing it should be estimated using the data filling methods. The checking for inconsistence of recorded done by the double- mass curve technique (Asawa, 2005).

2.9.3.Double Mass Curve

Double Mass curve are very basic analysis tools. A mass curve is a plot of cumulative values against time; a double mass curve is plot of cumulative values of one variable against the accumulation of another quantity during the same time period (Searcy and Hardison, 1960). Double mass curve analysis are often used to adjust precipitation records. Precipitation data can be very inconsistent due to non-representative factors, such as change in location or exposure of the rain gage (Chow, 1988). The theory behind double mass curve is that by plotting the accumulation of two quantities the data will plot as straight line, and slope of proportionality between the two quantities. A break in slope indicates a change in the constant of proportionality (Searcy and Hardison, 1960).

2.9.4. Design of Rainfall

When selecting a design frequency, potential up stream land use which could reasonably occur over the anticipated life of the drainage facility shall be considered. (ERA, 2013). The purpose of frequency analysis is One of the important problems in hydrology deals with interpreting past records of hydrologic events in terms of a future probabilities of occurrence. Any probability distribution can be used as a model but the reliability of the distribution is checked by the goodness of fit tests by professional software Easy fit 5.6.

Gumbel and Log Pearson Type III methods are used as suggested by Ethiopian Drainage Design Manual (ERA, 2013, Chow et al., 2012). The Log person type III was practically most preferable to determine maximum flood.

2.9.5. Selection of Frequency Distribution of RF

The choice of distribution is influenced by many factors like method of estimation parameters, the availability of dataset. The distribution functions most often used when the estimating hydrological events were: Normal, Log Normal, Pearson Type III, Generalized Extreme –Value, and Gumble Type I. Annual precipitation events tend to follow the normal distribution, and distribution varies over a continuous range and is systematic about the mean but allows negative values. However, hydrological variables tend to be skewed and all are non-negative values. The log normal distribution eliminate the problem of non-negative variable, permits the skins of the data, and requires the data to be symmetric about the logarithm of the mean. According, U.S Army crop of engineers (1977) recommended the log Pearson Type III distribution be used in an effort to promote consistency for flood flow analysis.

2.10. Development of IDF Curve

Design of hydraulic structures for conveyance of storm water requires to know the magnitude of the incoming peak flood in a certain return period. This is mainly conducted to determining the discharge capacity of storm water and sewerage system, drainage system like channels, culverts and in the design of bridge (Fasikaw and Tsegamlak, 2017).

2.11. Hydrological Modeling for Peak Runoff Determination

Many hydrologic methods are available for estimating peak discharge, the following method are estimating peak flood: Rational method; NRCS Runoff Curve number method and regional regression equations (ERA, 2013).

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

2.11.1. Rational Method

The Statistical rational method is still being widely used as the preferred method in different parts of the world for pipe design in urban drainage systems instead of the more advanced runoff routing procedures which produce hydrographs (Goyen et al., 1989).

Aitken (1975), in his study found that the rational Formula method as deterministic model was of value in the urban situation. As a statistical model, it was found have some merit. An Australian Rainfall and Runoff, (1987) recommends the Rational Method (SRM). Rational Method provides peak runoff rates for small urban and rural catchment areas; less than 50 hectares, but modified rational method could be used for size of drainage area greater than 50 hectors. It shall be used with mistake if the of concentration exceeds 30 minutes(ERA,2013).

An assumption inherent in the rational method is, Peak flow occurs when the entire water shed is contributing to the flow. Rainfall intensity is the same over the entire drainage area. Rainfall intensity is uniform over time duration equal to the time of concentration, tc. **Time of concentration** is the time required for water to travel from the hydraulically most remote point of the basin to the point of interest (Mays, 2004).

The time of concentration (T_c) is used in the rational method to determine the critical rainfall duration, which can then be combined with an appropriate rainfall intensity duration frequency (IDF) relation to establish the required design RF intensity(ERA,2013).The popularity equations can be found in the most hydrology texts books, such as Kirpich(1940) formula for well–defined and steep slopes(3%-10),California Culvert practices(1942) for small basin in California(U.S. Bureau of Reclamation).

According to ERA (2013), for the varied shape and topography of the surface three paths of flow were selected to determine T_c, these three paths of flow located in the top, middle and bottom of the watershed. There are three components of T_c: sheet flow, shallow concentrated flow and open channel flow. This three paths are sum to get total time of concentration with in catchment. (ERA, 2013).

2.11.2. Hydraulics of storm water drainage systems

The hydraulic capacity of storm drain is controlled by its size, shape, and slope and friction resistance. Several flow friction formulas have been advanced which defined 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 (ERA, 2013).

Manning Equation is used to determine the channel capacity of discharge and that is essential to examine the existing drainage channel to sustaining coming discharge without causing any flooding problem to the environment. It is mainly dependent on the roughness characteristic of channel system (Brown et al., 2013).

Manning's roughness Coefficient: in Appling manning equation, the greatest difficulty lies in the determination of roughness coefficient, 'n' there is no exact method of selecting the n value. Selecting n value actually means to estimate the resistance to flow in a given channel, which is rely a matter of intangibles (NRCS, 2010).

2.12. Urban Drainage Computer Models and Selection Criteria

As level of software availability, accuracy, cost and time of continent the modeling should be selected for flooding problem to investigate (Deepak, 2009). Many storm water management models are used in different parts of world, like (WP Software, 1991), HEC-RAS (Hydrologic Engineering Centre, 2000), Rational Formula method (Aitken, 1975) were used as urban drainage computer models for design and analysis.

Most widely used urban drainage simulation models in Australia and overseas in modeling event hydrographs are AUSQUAL (White and Cattell, 1992), STORM (U.S. Army Corps of E ngineers, 1977), some are commercial and some are open-source. Open source model require a nominal cost; however, they provide very little technical support for users. In contrast, commercial models support the beginners well, but their cost is often too high for widespread use (Zoppou, 2001).

SWMM is a widely accepted model and is currently applicable in Ethiopia for planning, analysis and design related to water system in urban areas (Getachew, *et.al*. 2015).

Therefore, storm water management model(SWMM5.1) was selected for this study due to simplicity and easily understandable, availability, use both hydrological and hydraulic characteristics of the catchment, and also it mainly used for flood plain mapping of channel system, design and sizing of drainage system components(Niyonkuru *et al.*, 2018).

2.13. EPA's Storm Water Management Model (SWMM)

2.13.1. EPA SWMM Model Description

The EPA SWMM model was first developed in 1971 (SWMM1) by association of American engineers for United State Environmental Protection Agency (US. EPA) and since it has undergone four major upgrades: SWMM2; SWMM3; SWMM4; and SWMM5.the current version (SWMM5.1) has many attractive features including an integrated graphical environment to do the modelling task in a more convenient way. It is used in the united State for the design of storm water drainage systems and has been incorporated in regional water quality management planning,design and rehabilitee purposes(Reza,2016).

It also offers several post-analysis options such as displaying simulation result using graphs and tables showing drainage area maps, profile plots, (Sharif et al., 2018).

The EPA (SWMM 5.1) is dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from primarily urban areas. SWMM model was set up based on the topographical and drainage network data and parameters derived based on the properties of the sub-catchments (Niyonkuru *et al.*, 2018). The runoff component SWMM operates on a collection of sub catchment areas that receive precipitation through a system of pipes, and channels (Rossman, 2017). This model is used for planning analysis and design related to storm water runoff, combined and sanitary sewers and other drainage systems in urban areas and as well as non-urban areas (Rossman,217). Running under Windows, SWMM 5.1 provides an integrated environment for editing study area input data, running hydrologic, hydraulic and water quality simulations and viewing the results in a variety of formats. These includes: - color-coded drainage area and conveyance system maps, time series graphs and tables, profile plots and statistical frequency analyses (Rossman, 2017). EPA SWMM model most widely used by researchers to model rainfall-runoff processes in urban areas (Niyonkuru et al., 2018).

2.13.2. SWMM system representations

SWMM model a conveyance network as a series of nodes connected by links. Land inks control the rate flow from one node to the next and typically conduits (e.g. open channels or pipes). The flow conveyed through the links and nodes of the model is ultimately discharged to a final node called the outfall. Outfalls can be subjected to alternative hydraulic boundary conditions (e.g free discharge, fixed water surface, time varying water surface (Rossaman, 2016).

The SWMM is chosen for this study to simulate all the hydrologic and hydraulics elements involved in the phenomenon of urban drainage.

2.13.3. Infiltration model of sub-catchment

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub catchment areas. SWMM offers three choices for modeling infiltration:

A. Horton's Equation

This method is based on empirical observations showing that infiltration decreases exponentially from an initial maximum rate to some minimum rate over the course of a long rainfall event. Input parameters required by this method include the maximum and minimum infiltration rates, a decay coefficient that describes how fast the rate decreases over time, and the time it takes a fully saturated soil to completely dry (Rossman, 2016).

B. Green-Ampt Method

It is a physical based model which can give a good description of infiltration model. This method for modeling infiltration assumes that a sharp wetting front exists in the soil column, separating soil with some initial moisture content below from saturated soil above. The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front. The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity (Rossman, 2016).

C. Curve Number Method

This approach is adopted from the NRCS (SCS) Curve Number method for estimating runoff. It assumes that the total infiltration capacity of a soil can be found from the soil's tabulated Curve

Number. During a rain event this capacity is depleted as a function of cumulative rainfall and remaining capacity. The input parameters for this method are the curve number and the time it takes a fully saturated soil to completely dry (used to compute the recovery of infiltration capacity during dry periods). SWMM also allows the infiltration recovery rate to be adjusted by a fixed amount on a monthly basis to account for seasonal variation in such factors as evaporation rates and groundwater levels. This optional monthly soil recovery pattern is specified as part of a project's Evaporation data (Rossman, 2016).

Imperviousness

The percent of imperviousness of a sub-catchment is another parameter that can, be measured accurately from aerial photos or land use maps. Table 1below.

Table 1: Percent impervious area for different land uses

Land use	Percent Imperviousness
Commercial	56
Industrial	76
High density residential	52
Low density residential	19
Institutional	34
Agricultural	2
Forest	1.9
Medium density residential	38

Source:-Rossman, 2016).

2.13.4. GIS Application for urban flooding

The use of GIS in the urban storm water model has been limited due to the need for large, detailed and expensive database that are usually difficulties to assemble. (Micheal, 2017). Runoff modeling needs integration of GIS and remote sensing (RS) and has two process (hydrologic parameter determination using GIS (2) hydrologic modeling within GIS. Remote sensing is used for obtaining land cover data each year and for obtaining information about the nature, rate and location of land use/land cover change (Misganaw, 2016).

Contributing Areas (Drainage Basin): contributing area identified through topographic mapping, survey data aerial photographs, and street, streams and drainage ways watercourse mapping and less commonly through field reconnaissance. Accurate flow estimation for basin depends significantly on recognizing areas with uniform distribution of important characteristics within that basin (McCuen et al., 2002).

3. MATERIALS AND METHOD

3.1 Description of the Study Area

3.1.1 Location

Hosanna town is the administrative and commercial center of the Hadiya Zone. The town got its new administration structure in 2004. It is located southwest of Addis Ababa at a distance of 232 km and North East of Hawassa at 168 km. The absolute geographic location of Hosanna is from 7 ° 30′ 00" to 7 ° 35′ 00 ″ North latitude and from 37° 49′ 00″ to 37 ° 53 ′ 00 ″ East longitudes orientation. The town consists three sub –cities namely Addis sub-city, Gofer meda sub city and sechi-duna sub-city. Gofer meda sub-city is one of mainly flood exposed area in a town (Hosanna town municipality, 2018).

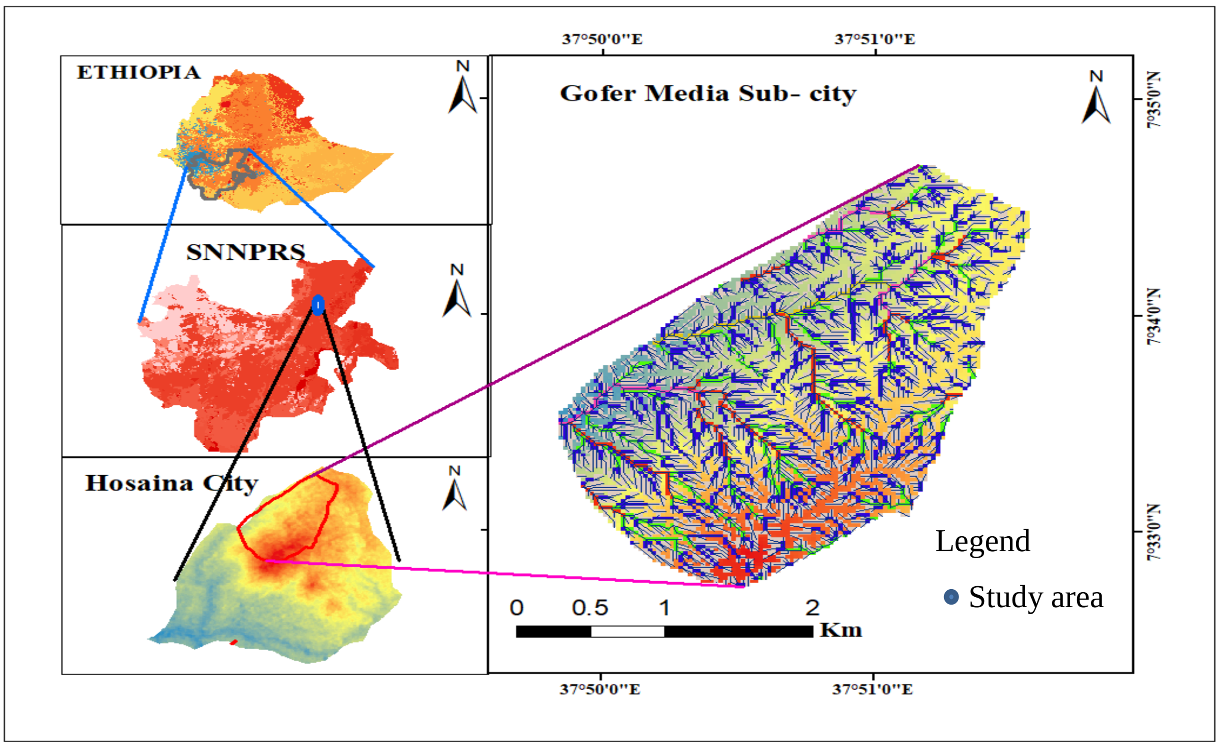


Figure 1: Location Map of Study area

3.1.2 Topography

The elevation of Hosanna town ranges from 2130-2417m above mean sea level. Most stream flow from north west south east due to steep slope of north west, most parts of the town is steep slopes. The average slope of the town is 4 %-8 %.

The northeast part of the town contains river and the drainage out lets was to the directions of river (Hosanna town municipality 2018 and field survey).

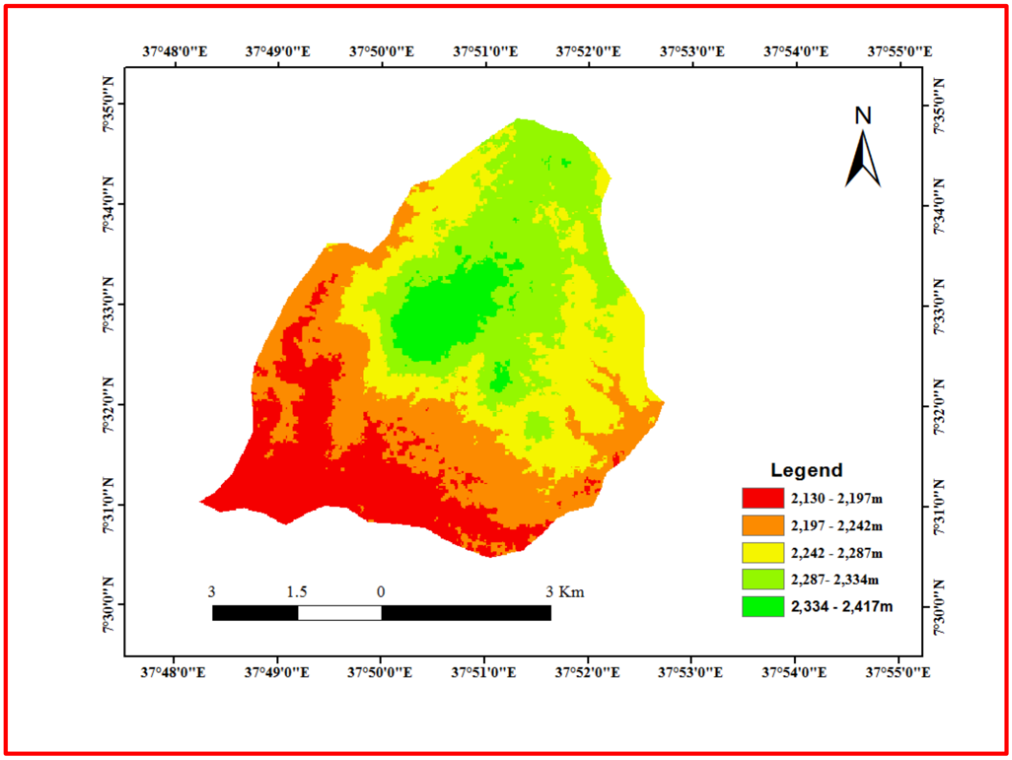


Figure 2: Elevation map of Hosanna town

3.1.3 Climate

Those climatic elements such as rainfall, temperature, relative humidity, wind direction.

3.1.3.1 Temperature

According to SNNPR meteorological Service agency (2018), the annual max temperature of Hosanna town ranges from 21.1°C to 23.7°C during. The annual minimum temperature ranges from 10.1°C to 11.9°C.

3.1.3.2 Rainfall

Based on RF data obtained from SNNP regional meteorological agency (2018),for 30 years rain fall data, the rainfall distribution of the town can be summarized in to the following three seasons.

A. Summer (June, July and August) like other parts of Ethiopia summer is the main rainy season. According to SNNPR meteorological agency (2018), during summer the town receives averagely 52.556% total annual rainy season of the town. It is the main rainy season of the town.

B. Belg (March, April and May) it is the second rainy season following karat. During this season the town gets average 36%of the total annual rainfall.

C. Winter is opposed to other places; Hosanna town gets moderate rainfall during winter. The average amount of rain in this season is 11.4% of the total annual rainfall.

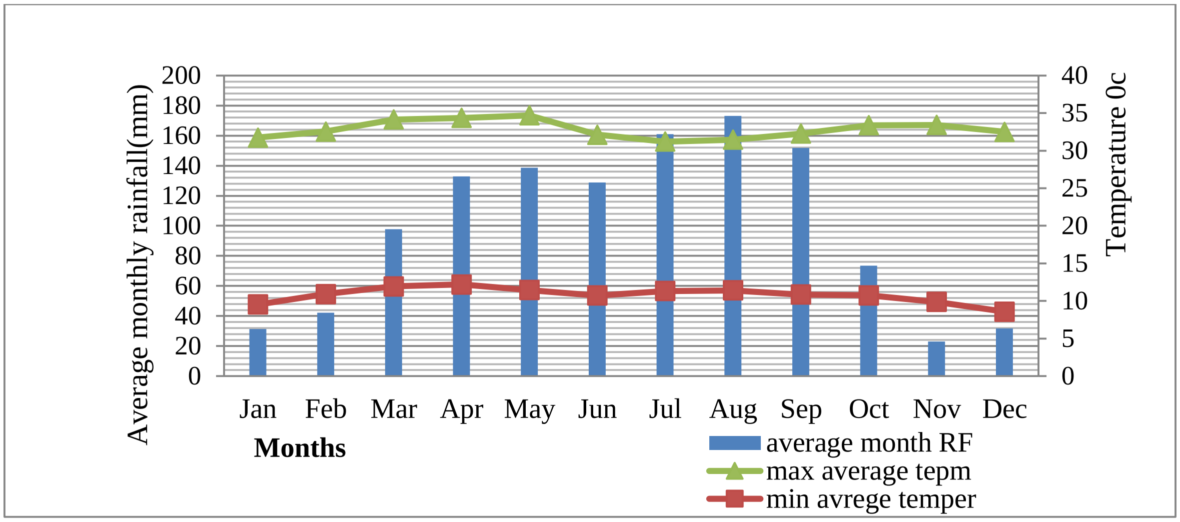


Figure 3: Monthly Average Rainfall and Average Temperature of Hosanna town

3.1.4 Land Use/Land Cover

Hosanna town has experienced rapid physical expansion in East, West, North and south direction. The total population of Hosanna town was 13467 and 31701 in 1984 and 1994 respectively (CSA, 1984 and 1994) with in 10 year the town population reached 69,995(more than double) (CSA, 2007).Based on CSA 2007 the population census result, the population of was projected to 133,800 at the end of 2015. The old and central part of the Hosanna town is dominated by mixed land use, poor housing condition, and irregular plot size and road network. However, the other part of the town is comparatively arranged by regular block structure on the place. Major urban function and their characteristics are discussed in percentage (Table 2) below. Different land use was done by field survey and with help of Google earth map, the land use land cover for catchment area grouped in to mixed residences, residential, commerce, Administration, road (cobble stone, asphalt), Recreation and open areas.

Table 2: Existing Land Use of Hosanna Town

No	Land use	Area(ha)	In percent (%)
1	Residential	422.79	19.45
2	Commerce	44.02	2.02
3	Administration	13.53	0.62
4	Service	213.12	9.8
5	Manufacturing	21.47	0.99
6	Road	180	8.28
7	Recreation	8.47	0.39
8	Buffer zone	40.5	1.86
9	industrial	30	1.37
10	Forest and agriculture	1200.1	55.45
Total		2174	100

(Source: - Hosanna Town Municipality 2018)

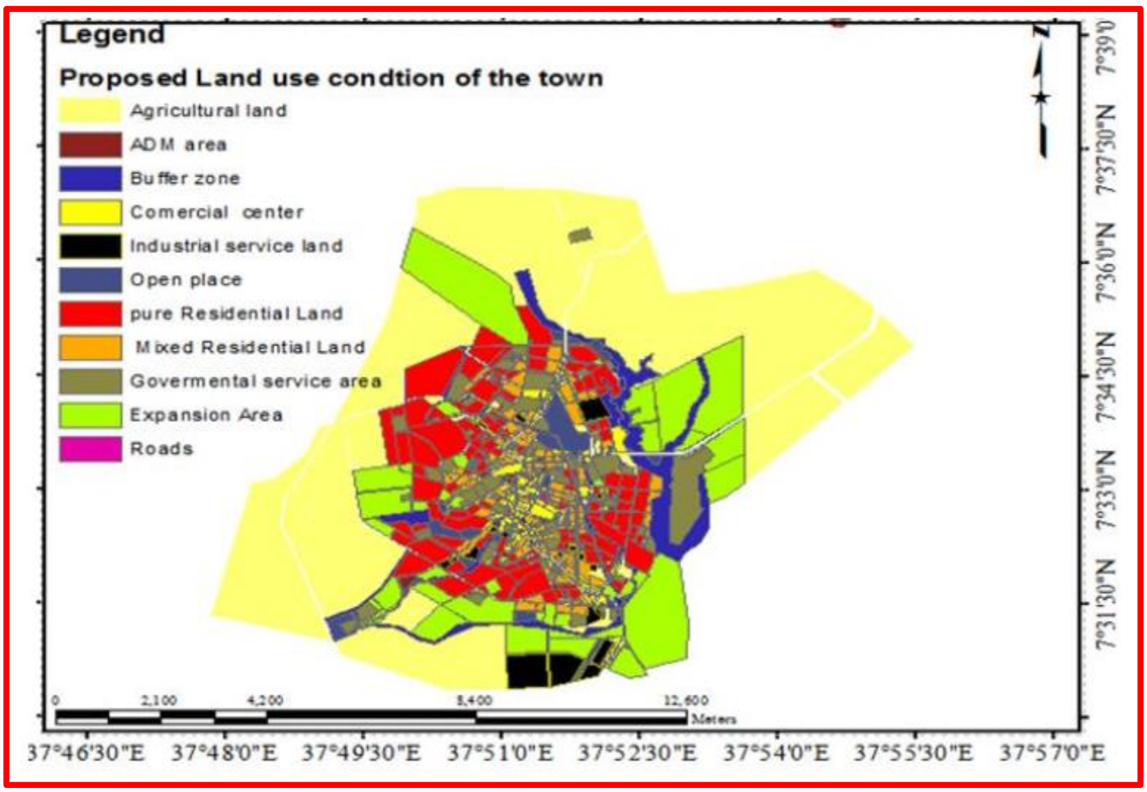


Figure 4: Existing land use of Hosanna Town
(Source: Hosanna Town municipality, 2017)

3.2 Data Collection and analysis

3.2.1 Study design/type

In this study, descriptive and exploratory types of study methods were employed. The descriptive type was used to describe the existing condition and coverage of urban storm drainage facilities. Whereas, the exploratory type was particularly used to explore the existing condition by making some required physical measurements, and compare with standards.

3.3 Sources of data

Data for this study collected from two sources: primary sources and secondary sources.

Primary Sources: - these include: - field observation, measuring of storm drainage system depth, width and length during field survey.

Field survey/study area observation

This method was the dominant method of data collection in this study using base map and checklist. In this study, photos of different scenario as it relates to different drainage issues were taken to show the real status of study area and identifying major sources of problems associated with drainage system. The sizes of urban storm drains were measured to check their length, width, height and/or radius. Topography field visiting of the study area was carried out to assess existing condition and major source of problem associated with storm drainage system failure drainage structures. Field survey was also carried out to confirm the patterns in order to accurately describe the sub-catchment physical characteristics.

Secondary sources:-. The secondary sources of information were including Meteorological data, Drainage network data, and urban plan document, ERA drainage design manual and socio economic data.

3.4.Types of Data

3.4.1.Rainfall data (meteorology data)

For hydrological analysis of peak discharge and to develop intensity duration frequency curves rainfall data is used. For specific time of concentration, duration of rainfall is assumed the same and intensity is estimated from intensity duration frequency curve. Due to in completeness of data additional three rain gauge station data are used to for this study. The Angacha, Durame, and Gimbichu station rainfall were used to fill missed rainfall data of Hosanna station

(Appendix Table1b). Consequently, this study used rainfall data from the SNNPR Meteorology agency for Hosanna town it has thirty year's data (from 1988 to 2017) of daily rain fall depth record. The time series data arranged and ordered by hour, day, month and yearly (Justine, 2016). But due to rain gauge station in the study area is one the outlier test is appropriate to check the quality of the data(chow,1988).For this study taking maximum daily rainfall for the design.

3.4.1.1.Estimating missing rainfall data

The continuity of recording precipitation data may have been broken with missing data due to several reasons such as damage in rain gauge during certain period (Garg, 2005). There are different approaches for estimating missing rainfall. These are: arithmetic mean method, normal ratio method and inverse distance weighing method. Arithmetic mean method can be used to fill in missing data when normal annual precipitation is within 10% of the gauge/station for which data are being reconstructed. The normal ratio method is used when the normal annual precipitation at any of the index station differs from that of the precipitation station by more than 10%. In this thesis normal ratio method is used to fill missed data (Chow,1988). (Appendix Table1B).

$$P_X = \frac{N_x}{3}\left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \frac{P_3}{N_1} ... + \frac{P_n}{N_M}\right] \quad (3.1)$$

Where: P_X =missing rainfall data (daily RF of Hosanna station) P_1, P_2 and P_3 rainfall data at nearest different station (daily RF of Gimbichu, Hangaca and Durame) Nx - mean annual rainfall at Hosanna) station. N_1, N_2, and N_3- mean annual rainfall at different nearest station (Gimbichu, Hangaca and Durame respectively).

Check quality of Data: An outlier test is an observation that differs significantly from the bulk of the data helps to avoid those data lie out the range in the between the lowest and highest datum, which may be due to errors in the data collection, recording or due to natural causes(Chow,1988).

Let Y= logx……………………………………………………… (3.2)

Lowest datum $RL = 10^{YL}$……………………………………… (3.3)

Highest datum $RL = 10^{YH}$……………………………………… (3.4)

Where: - $YL = Y_{avg} - K_n\delta_{n-1}$……………………………… (3.5)

$52 - 14 * 2.54 = 16$

$YH = Y_{avg} + K_n \delta_{n-1}$…………………………………………………………..…. (3.6)

$52 + 14 * 2.54 = 87.56$ Where: Kn = factor used for different sample size outlier (chow, 1988) for sample size 30 it is value 2.563 (See Appendix table 2B).

δ_{n-1}= standard deviation, YH=Highest daily rainfall (mm)

YL=Lowest daily rainfall value in chronological order (mm)

Y_{avg} =average of daily max RF (mm).

The RF data lies within the domain.

Homogeneity of data the similarity and uniformity of recorded data of hosanna station with surrounding stations, it is checked by double mass curve.

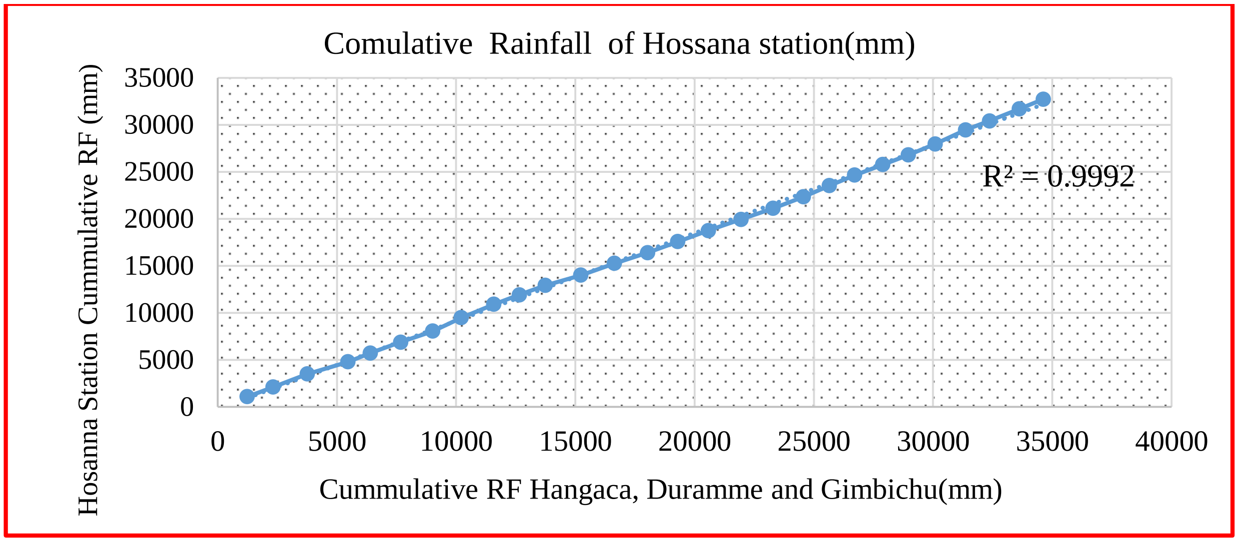


Figure 5: Homogeneity RF data for Hosanna and surround station.

3.4.1.2. Adjust for inconsistence of recorded rainfall data

Numerous factors could affect the consistence of recorded data in given station:

- Damage and replacement of a rain gauge
- Change in the gauge location or elevation
- Change in measurement procedure.

The checking for inconsistence of recorded data done by the double- mass curve technique (Garg, 2005). The procedure consists of comparing: the accumulated annual (seasonal, weekly, daily or hourly) RF at the station in question with; the accumulated annual (seasonal, monthly weekly, daily or hourly) RF for group of surrounding stations. If the change in graph occurs it should be corrected by using following equation.

$P_X = P_O \frac{M_C}{M_O}$ ………………………………………………………..…………………. (3.7)

Where: - P_X is adjusted precipitation value at station x, Po is original precipitation value of station x Mc is adjusted slop, Mo original slope C.

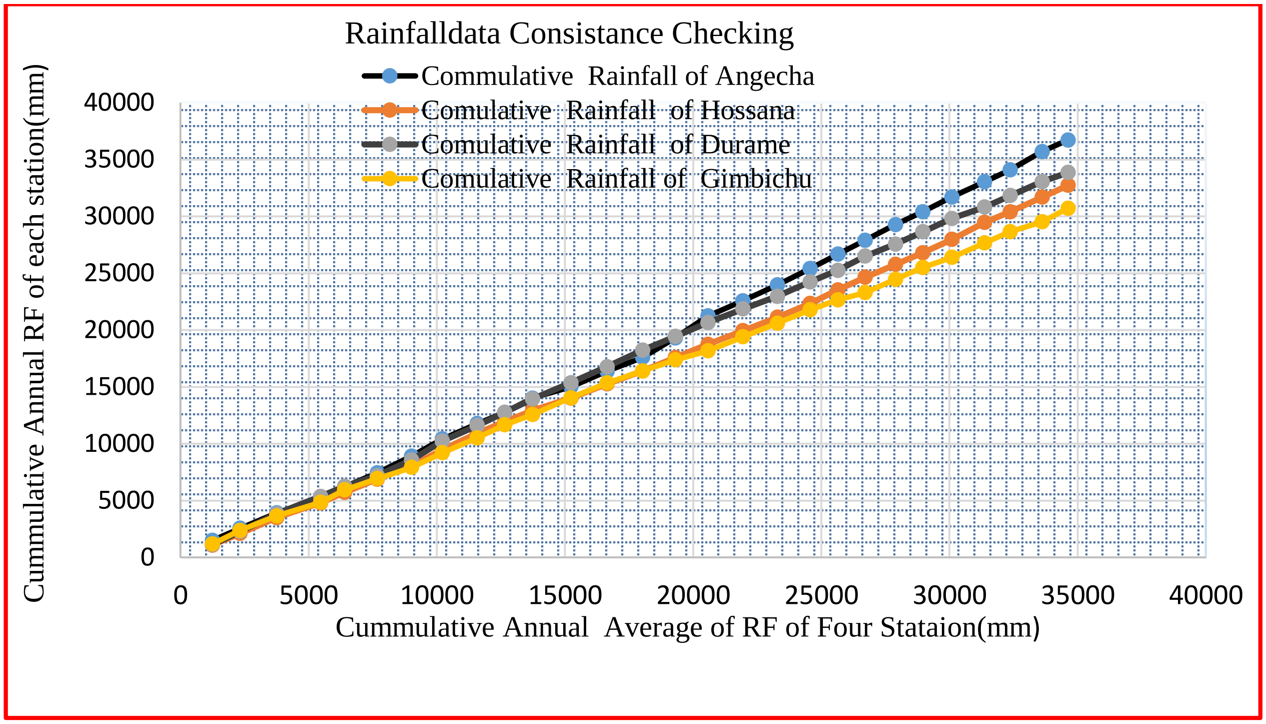


Figure 6: RF data consistence of stations

3.4.1.3.Rainfall frequency analysis

To develop the intensity duration frequency curve for specific station of Hosanna Town by using simple empirical formula for this thesis it employed arrange the given daily max rainfall in descending order of magnitude and assign an order number m.

Thus for the first entry m=1, for the second entry m=2 and so on till the last event for m=N=number of years record. The probability p of an event equal to or exceeded is given by the formula (Chow, 1988).

$P = \frac{m}{N+1}$……………………………………………………………… (3.8)

Where: p =probability of an event, m=rank of event=number of years record. Recurrence interval

$T = \frac{1}{P}$... (3.9)

Any probability distribution can be used as a model but the reliability of the distribution is checked by the goodness of best-fit R^2 is chosen to estimate design rainfall depth for different return periods that is used to develop IDF Curve, R^2 is unit less and describes the value ranges from 0(poor equation) to 1.0(perfect equation) (Fasikaw and Tsegamlak, 2017).

3.4.1.4.Selection criteria for frequency distribution of rain fall data

The purpose of frequency analysis is to interpret recorded hydrologic events in terms of future occurrence. The Log Pearson type III and Gumble methods are most common to analysis hydrological data. (ERA DDM, 2013; Ashraful et al., 2018). But for this study Log Pearson type-III is used.

To select the best frequency analysis of rainfall data based on;

- ❖ Using easyfit professional 5.6 software
- ❖ Using coefficient of determination or goodness of fit R^2
- ❖ Recommendations' of Ethiopian Road authority Drainage Design manual,(ERA, 2013).

Recent data analysis demonstrate Log-pearson type III distribution gives a best fit to the recorded data, should be used (ERA, 2013).

The log-pearson Type III distribution methods better to fit with observed RF data compared to Gumble methods from (Fig.8) below.

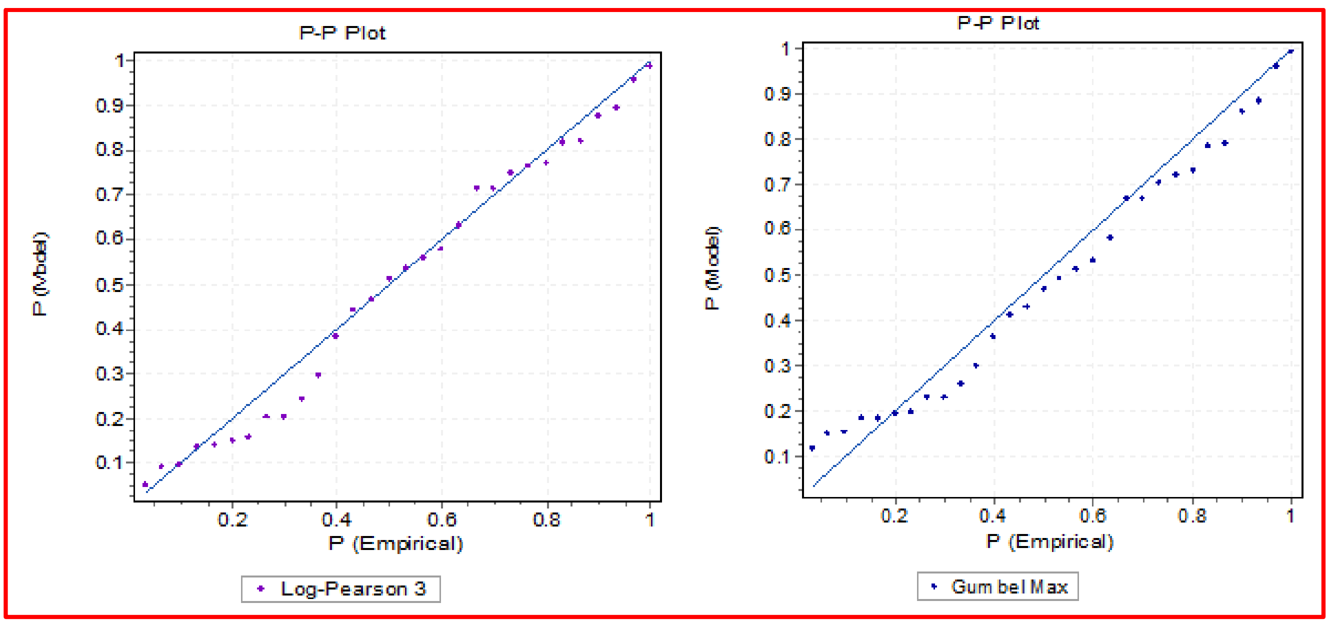


Figure 7: Log Pearson- III and Gumble distribution Goodness of fit observed for data.

Log Pearson Type III: is a three-parameter gamma distribution with a logarithmic transform of the variable. Mean, standard deviation and coefficient of skewness are the three necessary parameters that are necessary to describe the distribution.

The method employed in excel are as follows.

The frequency factor K_T depends on the return period T and the coefficient of skewness C_S. when $C_S = 0$, the frequency factor is equal to the standard normal variable Z. When Cs≠0, KT is approximated as follows.

- The log of annual maximum values are calculated, $Y = logX$
- The average (=average ()), standard deviation (=stdev ( )) and standardized skew (=skew ( )) of the data set are obtained using the respective excel functions.
- Coefficient k to be used in calculating K_T is obtained as $\frac{skew()}{6}$
- Coefficient Z to be used in calculating K_T is obtained using excels = Normsin($\frac{1}{T}$) function where T is the desired return period the value of Z corresponding to an probability of Exceedence P(P = 1/T)
- can be calculated by finding the value of an intermediate variable w:
- $w = [ln\frac{1}{p^2}]^{0.5}$ ……………………………………………………….………………....(3.10)

Then calculate Z using the approximation

- $Z = w - \frac{2.515517+0.802853w+0.103228w^2}{1.1.432788w+0.189269w^2+0.001308W^3}$……………………………………..…(3.11)

Coefficient K_T is calculated as

$K_T = Z + (Z^2 - 1)K + \frac{1}{3}(Z^3 - 6Z)K^2 - (Z^2 - 1)K^3 + ZK^4 + \frac{1}{3}k^5$………………….....(3.12)

Where: $K = {C_S}/{6}$ ……………………………………………………………………….........(3.13)

Y_T is calculated using Y_T=average ( ) +K_T*stdev ();

That is $Y_{T=} \bar{y} + K_T * S$…………………………………………………………..….............. (3.14)

Magnitude of precipitation with a return period T, X_T is obtained by transforming Y_T. finally

$X_T = 10^{Y_T}$ ……………………………………................………………………….........… (3.15)

3.4.1.5.Goodness of fit test

For design rainfall depth and IDF Curve development for different return period, commercially available statistical analysis software, easy fit 5.6 professional has been used. The reliability of distribution is checked by goodness of fit (GOF) tests. It test measures the compatibility of a random sample with theoretical probability distribution function. In other words, these tests show how well the distribution we selected. The Anderson-darling, the Kolmogorov-Smirnov, and the chi-square tests. These parameter unique to each methods is calculated for the required distribution types and ranked based on their fitness methods (ERA, 2013)

i. Anderson-Darling(A-D) test

The A-D test statistics, the quadratic class of the empirical distribution function (EDF) test statistics, is expressed as A as below;

$$A^2 = \sum \left[(2i - 1)\ln Fx(xi) + \frac{\ln[1 - fx(xn + 1 - i)]}{n} \right] - n] \qquad \text{.....................(3.16)}$$

Where: Fx (xi) is the continuous distribution factors (CDF) of the proposed distribution at xi, for i=1,2,3..n the observed data must be arranged in increasing order, as x1<x2<xn (Ashraful et al., 2018). In all three tests a parameter or statistics unique to each method is calculated for the well-defined distribution type and distributions' are ranked based on their parameter values. The best fit PDF was selected by using easy fit 5.6 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. After preparation IDF curve for study area it was be re-checked by ERA pre-prepared IDF curve for region "B2".

ii. Kolmogorov-Smirnov Test

This test is used to decide if a sample comes from a hypothesized continuous distribution.it is based on the empirical cumulative distribution function (ECDF). Assume that we have a random sample X1……..Xn from some continuous distribution with CDF F(x).the empirical CDF is denoted by

$$F_n(X) = \frac{1}{n}[\text{Number of observations} \leq X] \qquad \text{.. (3.17)}$$

The Kolmogorov-Smirnov statistic (D) is based on the largest vertical difference between F(x) and F(x) and Fn(x).it is defined as

$D_n = \sup_x\ [Fn(x) - F(x)]$ When comparing different distribution, lower statistics means better fit.

iii. Chi-squared Test

The chi-Squared test is used to determine if samples comes from a population with specific distribution. This test is applied to binned data, so the value of the test statistics depends on how the data is binned. Easy Fit employs empirical formula;

$k = 1 + \log 2 * N$……………………………………………………………..…….(3.18)

The data can be grouped in to intervals of equal probability or equal width.

The chi–squared statistics defined as $X^2 = \sum_i^K 1 \frac{(O_i - E_i)^2}{E_i}$ where Oi is the observed frequency for bin I, and Ei

E_i =F(x2)-F(x1) ……………………………………………………………….…… (3.19)

Where F is the cumulative distribution function of the probability distribution being tested, and X1, X2 are the limits for bin i. when comparing different distribution, lower statistics means better fit. Easy fit 5.6 professional software is used for testing goodness of the recommended log person type-III, General Ext ream Value (GEV), Log normal, Gumble and Weibull methods for this study.

In all three tests parameter or statistic unique to each method is calculated for the required distribution types and these distribution are ranked based on their parameter values. The best-fit PDF was selected by using Easyfit5.6 professional software.

Table 3: Goodness of Fit test in Different Distribution Methods

Distribution	Kolmogrov-Simrnov		Anderson-Darling		Chi-Squared	
	Statistics	Rank	Statistics	Rank	Statistics	Rank
Log P. III	0.09538	1	0.27186	2	0.9451	3
Gumbel	0.11528	4	0.51999	3	0.41789	1
G.E. Value	0.093595	3	0.29412	4	0.86789	2
Log normal	0.1	2	0.318	1	1.5637	5
Weibull	0.11694	5	1.4649	5	1.294	4

3.4.1.6. Development of intensity- duration- frequency (IDF) curve

The frequency analysis of hydrologic data is used to relate the magnitude of extreme events and frequency occurrence through the use of probability of distribution. The historical data use to design rainfall frequency analysis. An IDF Curve is a plot of average rainfall intensity (rainfall depth is averaged over the duration and the duration in minute (Garg, 2005).

$$\text{Average intensity} = \frac{\text{Rainfall Depth}}{\text{Duration}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.20)}$$

i. Design rainfall of shorter duration

Rainfalls for shorter duration are not available for most of Ethiopian stations. Consequently, a convenient method of converting 24-hr rainfalls depths into smaller rainfall depth of different durations was adopted to constitute 5-min, 10-min 15-min and other smaller duration's rainfall depths required in constructing IDF curves. The rainfall ratio method is briefly presented below. (ERA DDM, 2013) suggests that following equation for calculation of shorter duration rainfall from 24hour duration rainfall.

$$R_t = t\frac{(b+24)^n}{24(b+t)^n} * R_{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.21)}$$

By substituting Intensity (mm/hr) in the above equation

$$I_t = \frac{R_t}{t} = \frac{t\frac{(b+24)^n}{24(b+t)^n}*R_{24}}{t} = \frac{(b+24)^n}{24(b+t)^n} * R_{24} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(3.22)}$$

Where: I_t $(\frac{mm}{hr})$, t = (hrs.) Using b = 0.3, and n = 0.92 (0.78<n<1.09) (recommended for east Africa by ERA Drainage manual, 2013).

As suggested by (ERA,2013), manual for region B2 short duration RF IDF Curve developed for 3hr or 180 min, results are tabulated for rainfall durations 5, 10, 20, 30, 150,160 and 180 minutes. The methods employed to develop IDF curve for shorter duration events using the equations (3.22) as above discussed.

Using the trend line equation obtained from Log-Pearson Type III frequency analysis its goodness checked by best-fit R^2 value greater than Gumble's Extreme value. R^2 is unit less and describes the portion of total variance by equation, its range is from '0" (poor equation) to 1 is perfect (Fasikaw and Tsegamlak, 2017). For storm water drainage design short periods of rainfall are very important than monthly or annual rainfall (ERA, 2013). Therefore in this study log Pearson Type III R^2 is 0.7853. The mean maximum daily rainfall of hosanna town is 52 mm and with standard deviation 14.

In shortly steps used to prepare design RF depth and IDF curve are:

Checking quality and consistency of meteorological data.

Design rainfall analysis by using easy fit 5.6 professional software and then check Goodness of fit for each formula.

Develop RF depth for given return periods 2, 5, 10, 25, 50 and 100 years.

Finally compare the study site IDF Curve with ERA, 2013 regionalized curve (B2 region).

3.4.2. Spatial data

3.4.2.1.Soil type Data

This data is essential to differentiate study area hydrologic soil group. This used for estimating peak discharge by SWMM to determine infiltration characteristics and imperviousness of soil chrematistics. The 97% of town Covered with pelvic verticals and (3%) covered with orthic solonchaks type soil or hydrologic soil group of D and B respectively. See appendix, table14. According data obtained from Ministry of Water, Irrigation and electricity (MoWRE). Pellic Verticals, heavy clay soils with a high proportion of swelling clays. These soils form deep wide cracks from the surface downward when they dry out are medium.

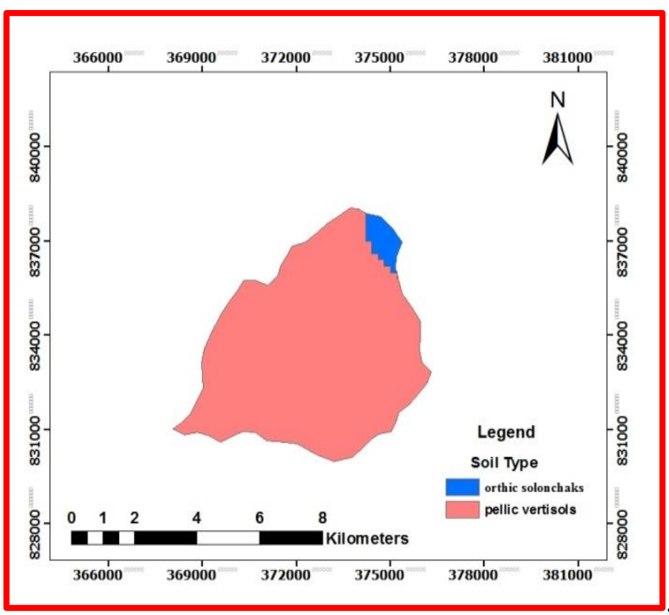


Figure 8: Soil map and hydrologic soil group

3.4.2.2. Digital elevation data (DEM)

DEM data used to describe topographic characteristics such as contour, slope and elevation difference. Since the analyses that are to be done are primarily elevation based, topography of the study area is important global elevation maps from satellite imagery are readily available in a raster and digital elevation format.

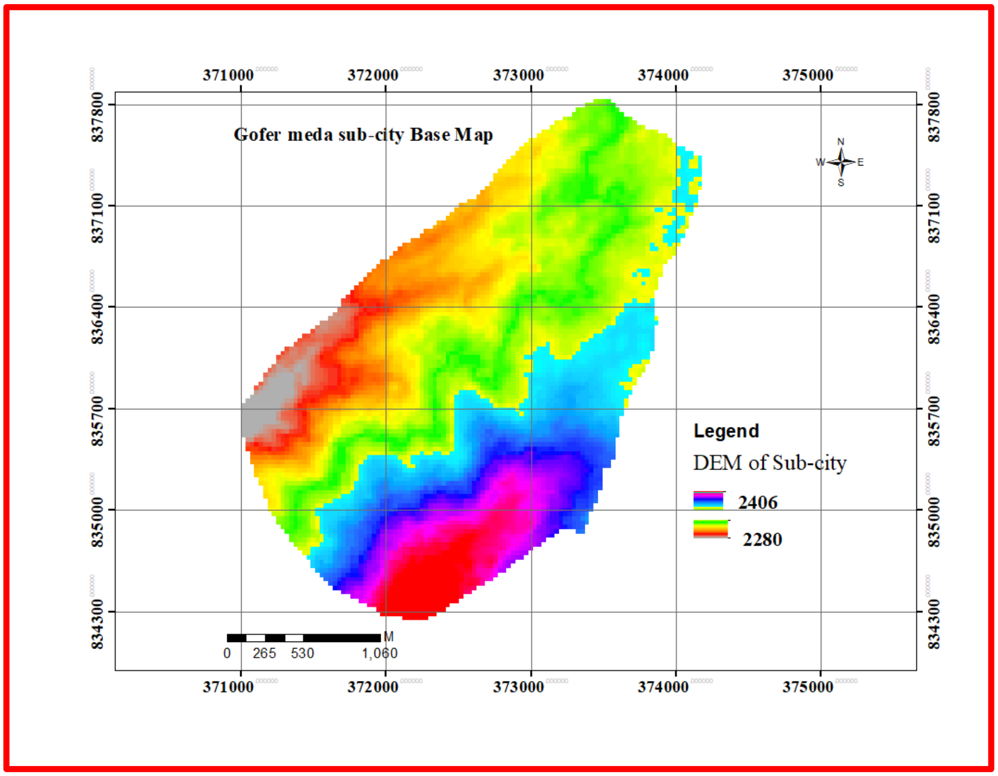


Figure 9: DEM of study area (Gofer meda)

The digital elevation model is the basic dataset used in this study for basin delineation and for extracting characteristics of the study area. But DEM for study site (Gofer-meda) was developed from contour map of the sub-city. DEM is an essential tool for presenting and analyzing the features of topography (Aitken, 1975).

3.5. Materials and Software's Used

The materials were used for this study are: - **Arc-GIS** were used to obtain hydrological, topographic, physical parameters and spatial information of the catchments of the study area. DEM 30m by 30m data was used as an input data for Arc-GIS software and estimation of catchment characteristic and **Google Earth pro** software was used to verify watershed, compare and identify the different features, divides of catchments of the study area.

Digital camera; used to capture photos of drainage system, road conditions and waste disposal sites. **GPS device and measuring tape**: - used to collect elevation, coordinates of junction, links and nodes drainage system ground, and to measure the size of drainage system respectively.

SWMM 5.1:-is used for rainfall-runoff filtration simulation and canal/pipe flow calculation. It provides an integrated environment for editing study area input data, running hydrologic, hydraulic and water quality simulations and viewing the results in a variety of formats.

Easy fit 5.6 professional software:-used to determine statistical analysis of rainfall data and used to select best-fit distribution method.

MS-Excel:-used to calculate geometry of drainage system and arrange bulk RF data with in a friction of time and to construct graphs easily.

3.6. Methods

The methodological frame work for this study as follow(Fig.11).

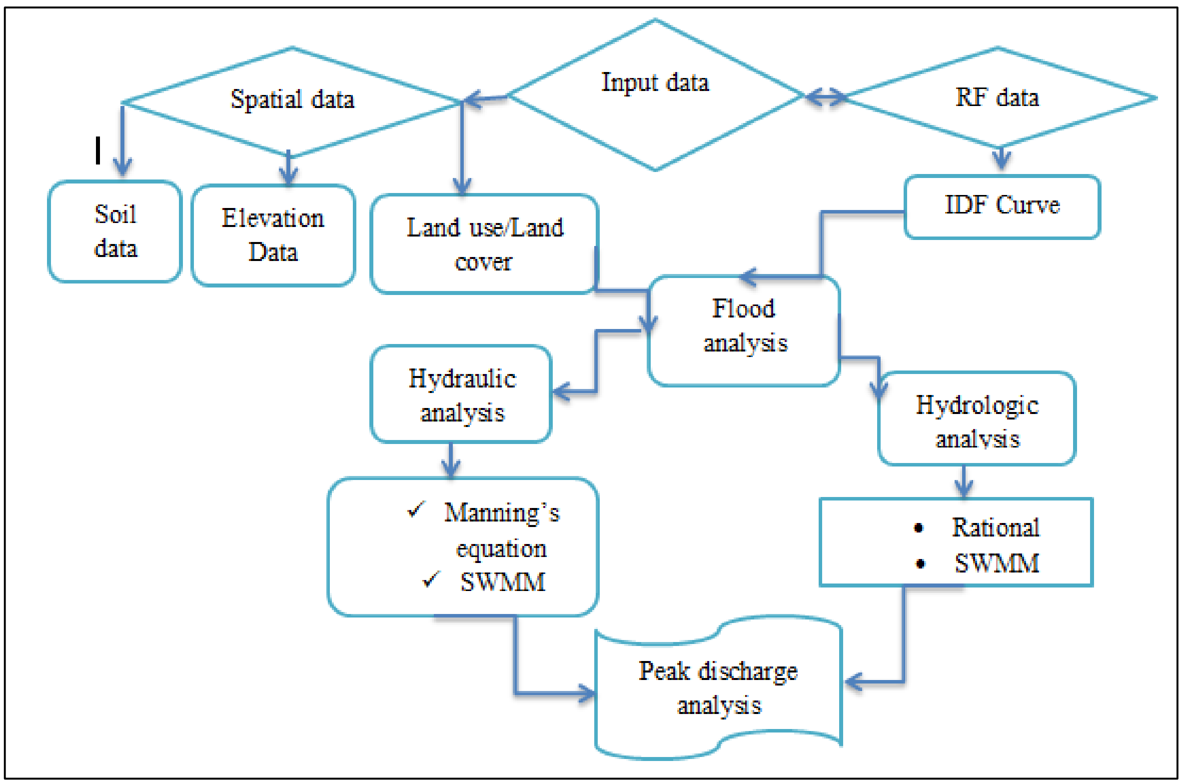


Figure 10: Methodological framework

3.6.1 Delineation system of study area

Hydrological modeling starts from delineating streams and watersheds, drainage pattern of urban area is influenced by, slope, flow length, flow direction, stream network. Each sub-catchment area is delineated digital to determine accumulated discharge at the end of which then be removed through outfall structure. In urban area catchment delineation depends on topographical and drainage network data and it further divided into small sub-catchments connected to the appropriate location in the drainage channel (Aitken, 1975).

The steps in terrain pre-processing are as follow

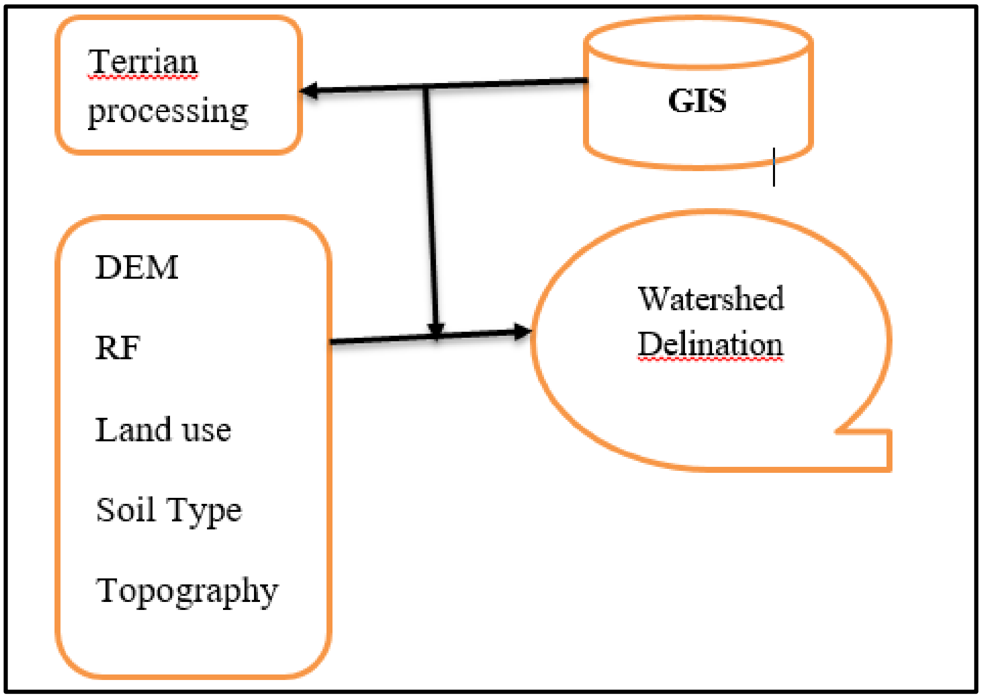


Figure 11: Watershed delineation
(Source: ArcGIS10.3)

The catchment area delineation was taken based on its classes of study area, flow direction elevation data used for the modeling that obtained from field survey and structural plan of Hosanna town. The areas for each category are calculated with the help of AUTOCAD (Havin g collecting coordinates by GPS), and Google earth pro software's to prepare the SWMM input parameter. The properties of nodes, outlets, and sub-catchment, catchments and boarders were surveyed from field and it was used to analysis of the catchment flow direction and drainage system.

Figure 12: Delineated Map of Gofer-meda sub city watershed based on class of study area. (Source: Google earth pro)

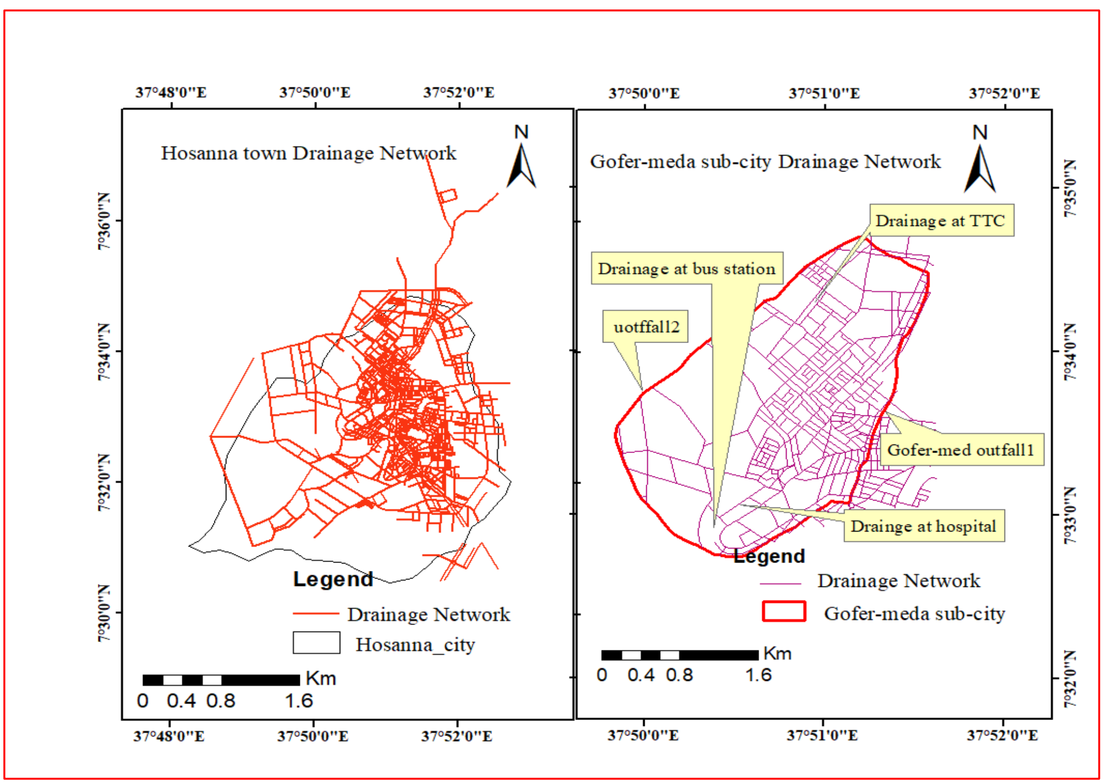


Figure 13: Drainage network of study area.

3.6.1.Run off amount determination using the rational methods

The idea behind the rational method is that for a spatially and temporally uniform rainfall intensity (I) which continues indefinitely, the runoff at the outlet of a catchment would increase until the time of concentration (tc), 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 Qp.

Steps of runoff computation in rational method

1. Determine Catchment area:- A Catchment area is the upslope area contributing flow to a given location, and can also be referred to as a basin, catchment, sub-watershed, or contributing area. Collect required data from each sub-catchment, drainage area, land uses soil types.
2. Determine longest path and elevation. Distance from most remote point to outlet and difference in elevation from the farthest point of the drainage to pint of discharge.
3. Determine catchment property:- The catchment property includes hydrologic soil group, slope, flow length and flow direction
4. Use appropriate runoff coefficient (C)
5. Determine peak flow(Q_{peak})

$Q = 0.00278CIA$………………..…………………………...………………………....(3.23)

Where:

Q=maximum rate of runoff, $\frac{m^3}{s}$, C=dimensionless runoff coefficient,

I= average rainfall intensity for a duration equal to the time of concentration

A=Catchment area tributary to the design location, (ha).

Runoff Coefficient(C)

The runoff coefficient, C, is a dimensionless ratio intended to indicate the amount of runoff generated by a watershed given average intensity of precipitation for a storm. The runoff coefficient values for different drainage areas are different .The runoff coefficient represents the fraction of rainfall converted to runoff. A weightage 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. (Chow, et al, 2012).

$$C_w = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i} = \left(\frac{A_1C_1+A_2C_2+A_3C_{3+}\ldots\ldots\ldots A_nC_n}{A_1+A_2+A_3+\cdots\ldots\ldots\ldots A_n}\right) \ldots\ldots\ldots\ldots\ldots\ldots (3.24)$$

Where, $C_{w=}$ weighted C; Ci=runoff coefficint for parts of drainage area

A_1, A_2……. $A_{n=}$parts of drainage area with different runoff coefficients

Time of concentration

The time of concentration (Tc), is the time required for water flow from the most distant point of the basin to the location being analyzed/outlet (ERA, 2013). The time of concentration (Tc) is used in the rational method to design determine the critical rainfall duration, which can then be combined with an appropriate rainfall intensity duration frequency (IDF) relation to establish the required design RF intensity. In varied shape and topography watershed has three paths of flow. Time of concentration based up on slope, characteristics of land cover, the rainfall pattern of an area and the flow length.

Travel Time

Water moves through a catchment area as sheet flow, shallow concentrated flow and open channel flows.

The type that function of the conveyance system and is best determined by field inspection.

Travel time is the ratio of flow length to flow velocity.

There are three components of TC

1. Initial or overland flow time: this is a flow in a plane surface it will vary with surface characteristics such as, slope, depression storage and infiltration capacity of soil (NRCS, 2010).

$$T_i = \frac{3.26*(1.1-C)\sqrt{Li}}{S^{0.33}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.25)$$

Where: Ti=overland flow time (minute),

Li= over land flow length (m), S=slope of catchment along flow path (m/m).

C=runoff coeifficent(from table)

Overland flow is flow over plane surfaces and shall not be use over distances exceeding100 m for urban area (Hydrologic Analysis and Design, 2010).

2. Shallow concentrated flow

After maximum of 100m sheet flow become shallow concentrated flow. And the average velocity for this flow function of water course slope and type of channel (ERA, 2002).

$$T_{c2} = \frac{L}{3600v} \quad \text{……………………………………………………………………(3.26)}$$

$$V_{paved} = 6.1967s^{1/2} \quad \text{……………………………………………………………3.27)}$$

Where: S=waterway slope, Tc_2 =travel time,(hr.)

L=flow length (m), V=average velocity (m/s), 3600= conversion factor from second to hours. After a maximum of 100 m, sheet flow usually becomes shallow concentrated flow.

3. Open channel flow

   Open channels are assumed to begin where surveyed cross section information has been obtained, where channels a

$$T_{c3} = \frac{L}{3600v} \quad \text{…………………………………………………………………..…..(3.28)}$$

Where: $v = \frac{1}{n} R^{2/3} s^{1/2}$ ………………………………………………………………...(3.29)

S= slope of channel, R= Hydraulic radius

The time of concentration is the sum of T_c values for the various consecutive flow segments:

Total time concentration (TC) $= T_{C1} + T_{C2} + T_{C3}$………………..…………………….………(3.30)

Different slopes along the hydraulic length of the catchment were calculated from field survey data. The weighted average of these slopes is taken to calculate (estimate) the time of concentration.

The minimum time of t_c value of 5 minutes for urbanized areas and 10 minute for areas not considered urban (UDFCD Drainage manual, 2016). However, according to ERA drainage design manual (2013), minimum time of concentration taken 15 minute when calculation result was lesser time of concentration (Appendix Table 6).

Storm Intensity (I)

Intensity of rainfall is the average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from RF IDF Curves (ERA, 2013).

However, this parameter is usually not available in most meteorological stations in Ethiopia; therefore, empirical relationships have been used to drive shorter duration of rainfall intervals from 24hr maximum rainfall (ERA, 2013) which discussed previous section. Using statistical analysis, rainfall intensity-duration curves have been developed for commonly used design frequency.

3.7. Drainage Network Analysis

3.7.1.Hydraulic capacity of drainage canal

The storm drainage sizing is done based on the computation excel macro sheet prepared by the researcher based on ERA (2013) drainage manual. For the existing drainage canals geometry, position and canal type data collected from Hosanna town Municipality office, field survey and its capacity checked by most widely used formula for gravity and pressure flow Manning's formula as shown equation 3.31. Hydraulics analyses were used to determine the ability of a drainage channel to accommodate the discharge of storm water.

Flow Velocity

Manning's formula is used to determine flow velocity of drainage.

$$V = \frac{1}{n} R^{2/3} . S^{1/2} \quad (3.31)$$

$$Q = \frac{A}{n} . R^{2/3} . S^{1/2} \quad (3.32)$$

Where: Q = discharge (m3 / sec),

A = sectional area of the channel (m2)   n = Roughness coefficient of wall (Manning)

R = hydraulic radius=A/P (m) P=wetted perimeter

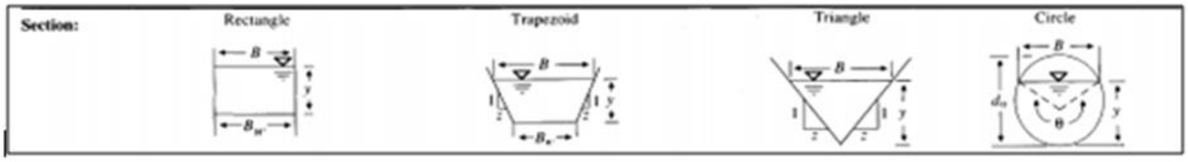
S = friction slop, it equals to the channel longitudinal slope because the flow in channels is

Roughness coefficient (n)

The hydraulic friction is determined by roughness of drain channels. Selected roughness coefficient depends on the smoothness of the material (Appendix 10).

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

Table 4: Geometric property of different channels.



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 θ D
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	$\left(\sin\frac{\theta}{2}\right)*D$
Hydraulic depth	h	$\frac{(b+mh)h}{b+2mh}$	½*h	$\frac{\theta - sin\theta}{sin\theta/2} * \frac{D}{8}$

3.7.2.Possible engineering approaches and Design consideration of hydraulic structures as per the standards

a) The visual observation of hydraulic structures

The visual observation of hydraulic structures at the different junction points, at an inlet and outlets of the canals as well sub-catchments.

b). Validation as per design document as well as design criteria's

The data obtained from visual observation could be validated by comparing with the design document as per the standards. If the design analysis is accurate as per the design theory, procedure and construction is as per the design, then cross check variables such as runoff coefficient, rain fall intensity whether they are changed through the process of urbanization. For this study, little design analysis and as-built drawing is available at the stakeholder or municipality but, many road side ditches are built by micro and small enterprises in Hosanna town so to compare between the constructed and the designed one is challenging. Based on availability of design data in some areas existing, design and deficit of storm drainage size was compared. Therefore, the procedure to collect actual data of size and slope of the side drainage ditch, length, width; size of the catchment; land use land cover data; estimate rainfall intensity and analyze the run off produced within the catchment.

3.8. Storm water Drainage Operation and Management Evaluation

The storm water drainage operation and management system in study area is evaluated by direct observation of inlets condition, disposal of solid and liquid wastes, accumulation of debris and silt, condition of ditch after construction. The data collected from the site assessment compared with the design data in the technical drawings of all related drainage structures. The Current conditions of storm drainage system evaluated using GTZ, 2006 Standards (Appendix 11).

3.9. Storm water management model (SWMM)

Since SWMM is a dynamic rainfall-runoff simulation model used for single event simulation of runoff quantity and quality from primarily urban areas (Rossman,2017).

The two principal components of SWMM conveyance system network are nodes and links. **Junction node**:-represent the end points of conveyance links that form connection between two or more links. Principal input parameters for junction node are;

- Invert(channel or manhole bottom) elevation
- Height between its invert and the ground surface

Conduit links: are conveyance element that transport flow between nodes (Rossman, 2017) The required input parameters for a conduit link are:

- Conduit length
- Manning’s roughness coefficient
- Cross-sectional shape and dimensions

Outfall or outlets Outfall nodes are terminal nods of the drainage system used to define final downstream boundaries (Rossman, 2017).

3.9.1.Input parameters of the SWMM and their properties

i. **Rain gage properties**

The user assigned rain data format are Intensity (mm/hr) and RF depth (mm). Cumulative RF in mm. Rain interval time interval gage reading in decimal hours, minute format. Data source of RF data is TIMESERIES for user-supplied time series data.

Precipitation supplied from rain gauge more than one catchment, but the rainfall data for this study is time series data and one rainguge record.

ii. Sub-catchment properties

Sub catchment is hydrologic unit of land whose topography and drainage system elements direct surface runoff to a single discharge point (Rossman, 2017).

Table 5: Input parameters for SWMM

Name	User assigned sub catchment name
Area	Area of the sub catchment in acres or hectors.
Width	Characteristics with of the overland flow path for sheet flow runoff (feet or meters).This parameters is important in the modeling of flood peaks.
% slope	Average percent of slope of sub catchment.
% Imperviousness	Percent of land area which is impervious.it is measured from Arial photo or land use map. The run off volume and flow rates area strongly sensitive to estimates of imperviousness (ApendixTab7).
N-Imperviousness	Manning's n for impervious portion of sub catchment (Appendix Table 10).
N-perviousness	Manning's n for pervious portion of sub catchment.
Dstore-Imperviovsness	Depth of the depression storage on the impervious of the sub catchment (inch or mm).
Dstore-perviousness	Depth of the depression storage on the pervious of the sub catchment (inch or mm).
%zero-Imper	%of impervious area with no depression storage.
Infiltration	Used to edit infiltration parameters for the sub catchment. The Green-Ampt equation is used for this study because the method reserved considerable attention to calculate infiltration urban catchments (Rossman, 2016). It is also physically based model which can give good description of infiltration process.

3.9.2.SWMM Input Sources

During the field observation and topographic map, the Sub catchment property like; Slope of catchments, conduit flow node directions, Conduit length, width, depth, side slope and Manning's n for overland flow both impervious and pervious areas were data collected.

Table 6: Conduit Dimensions and nodes in study area

Conduit Code	Link flow		Shape	Width (m)	Depth (m)	Side slope	Length (m)	Type
	From	TO						
C1	J11	J12	Rect	0.5	0.5		370	Masonry
C2	J12	J9	Rect	0.5	0.6		320	Masonry
C3	J9	Out1	Rect	0.5	0.7		290	Masonry
C4	J8	J9	Rec	0.4	0.5		505	Masonry
C5	J7	J8	Rect	0.45	.6		611	Masonry
C6	J10	J3	Trape	0.4	0.5		258	Masonry
C7	J2	J13	Trape	0.5	0.6		380	Masonry
C8	J3	J13	Trape	0.4	0.5		140	Masonry
C9	J14	J13	Trape	0.5	0.4		432	Masonry
C10	J5	J6	Rect	0.7	0.6		395	Masonry
C11	J6	J7	Rect	0.5	0.6		445	Masonry
C12	J4	J1	Rect	0.7	0.4		500	Masonry
C13	J1	J2	Rect	0.5	0.5		300	Masonry
C14	J15	J1	Recta	0.5	0.5		403	Masonry
C15	J13	Out2	Trape	0.7	0.5	0.05	400	Masonry
C16	J17	J16	Recr	0.5	0.4		300	Masonry
C17	J16	J2	Rect	0.6	0.4		400	Masonry
C18	J18	J10	Rer	0.4	0.2		374	Masonry
C19	J19	J5	Rect	0.5	0.4		408	Masonry
C20	J20	J12	Trape	0.4	0.4	0.05	389	Masonry
C21	J21	J3	Trap	0.5	0.6	0.04	230	Masonry
C22	J22	J7	Trape	0.5	0.4	0.05	260	Masonry
C23	J23	J21	Trape	0.5	0.3	0.05	189	Masonry

Note: Rect=rectangular, Trape=trapezoidal channel shape

3.9.3. Setting model simulation option

3.9.3.1 Flow routing

Flow routing with in 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 sophisticated used to solve these equations. The choices are steady flow routing, kinematic wave routing and Dynamic wave of routing method used to route flows through the conveyance system(Rossman, 2016).

Dynamic flow routing method

Dynamic Wave routing solves the complete one-dimensional Saint Venant flow equations and therefore produces the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. Flooding occurs when the water depth at a node exceeds the maximum available depth, and the excess flow is either lost from the system or can pond atop the node and re-enter the drainage system (Rossman, 2016). Dynamic wave routing can account for channel storage, backwater effect, entrance/exit losses flow reversal and pressurized flow. Because it combines together for both water levels at nods and flow in conduits it can applied to general network layout. Therefore, this method was selected for this thesis due to its accuracy result (Rossman, 2016).

Infiltration

Infiltration is the process of rainfall penetrating the ground surface into the unsaturated soil zone of pervious sub catchments areas. SWMM offers three choices for modeling infiltration: Horton's equation, Green-Ampt method and Curve number which discussed in literature part.

Green-Ampt Method

The input parameters required are the initial moisture deficit of the soil, the soil's hydraulic conductivity, and the suction head at the wetting front. The recovery rate of moisture deficit during dry periods is empirically related to the hydraulic conductivity. It is a physically based model which can give a good description of infiltration process. In this study Green-Ampt method was used.

3.10. Model parameterization

EPA SWMM requires three major parameters categories for runoff quantity modelling including the physical catchment characteristics', rainfall and infiltration data. The physical catchment data required for runoff modeling include catchment area, percentage of impervious area, catchment width, average slope, surface depression storage and surface roughness. Other input parameters for catchment properties were adopted from the range provided in SWMM User's manual below,

4. RESULT AND DISCUSSION

4.1.Existing Condition of Storm Drainage System

The existing storm drainage facilities in study area are; Junctions, open and closed drainage system Fitted RC precast inlets are in some areas especially along main road only Hosanna-Wolikite road. Closed drainage channel lines are found also only along main road side. However, drainage provision in the town is considered to be inadequate in terms of quality and coverage. As a whole drainage facility in the hosanna town requires construction of large number of drainage system for existing and new unconstructed areas. The existing condition of the slotted/fitted reinforced concrete precast inlets in Goffer-Meda Sub-city along main road of Hosanna- Wolikite is presented below. During field survey the existing conditions of storm drainage system is evaluated using GTZ-IS, 2006 Standards (Appendix 11).

Table 7: Existing Inlet Conditions of the Study Areas

S.no	Drainage segment	condition	Types of inlet	Numbers
1	Gombora	Good	Fitted RC precast cover with 3cm opening	14
2		Light	Slotted RC precast cover with 3cm opening	18
3	Wachamo	Light	Fitted RC precast cover with 3cm opening	10
4	TTC	Light	Fitted RC precast cover with 3cm opening	7
5		sever	Fitted RC precast cover with 3cm opening	12
3	Jerusalem	Sever/out of function	Slotted RC precast cover with 3cm opening	28

(Source: field survey)

Based on GTZ-IS, 2006 standard, the Storm water drainage system in study area is categorized into three conditions (Fig.16) below. The first category of storm water drainage systems working without problem.

Second category of storm water drainage inlets system work with problem because of lack of awareness, knowledge and continues inspection and follow up to clear blockage and the third category of storm water drainage system is completely out of services due to blockage by debris.

Figure 14: Great inlet conditions

4.1.1. Major problem associated with storm water drainage system that caused failure

The existing drainage system problem in study area have been seen in three dimensional aspects such as; design aspect, construction and management aspects.

4.1.1.1. Poor design of storm water drainage system

Proper design of storm water drainage system is essential component for road structure to give service without traffic interruption.

The existing conditions of drainage system at the study area were investigated in detail. The problems associated with drainage system; drainage systems having the inadequate capacity to carry storm water corresponding with catchment area size; hence resulting over flow and Ponds. If appropriate hydrological and hydraulic calculations were not practiced storm drainage system design, either under designed or over designed would occur which leads to excess cost in long term basis. A storm drainage systems are designed to convey coming flood in permissible return period to safe properties and life from accidental flood risk. Most designed storm water drainage system in the study area are overestimated and underestimated. For instance, the drain ditch along large “Adabay” to St. Mariam church, Jerusalem Hotel to Bobicho Branch Commercial bank and Gombora to Taxi “Mazoria” Hospital to “Godgade” Heath Science College to “Hame” Hotel are cited as underperforming channels and in contrary some areas like Wachamo highschol to TVET, at Diaspora, Arada and Bole overdesigned drainage system. This is due to mismatching of the design capacity of the drainage system with the flood generated area, land use land cover features/run off coefficient in the sites and RF intensity. This is strongly related with Lack of detail information about extreme flood and its recurrence interval and inadequate hydraulic design. This finding linked with Kolsky (1998) studies in that conventional drainage system designs are inappropriate because the miscarry to take the potential for flooding in to account, as well as the frequency of flooding, the duration, RF depth and extent of flooding, should be considered.

Figure 15: Areas Exposed to Flooding problem due to under performance.

(Source: - Own field survey 2018).

4.1.1.2.Poor Construction of Drainage Lines

During the field survey, it was observed that the drainage system has serious problems related with poor construction practices and lack of appropriate maintenance. Most drainage ditches construction in the study site was undertaken by small micro enterprises with less construction supervision and management. Most of the enterprise members lack professional background and their main target is business rather than worrying about strength, durability and serviceability of the structure. In addition to quality, other challenges have been faced during construction misinterpretation of design document and what would be applied on ground. There is no technical staff or committee to check is it a construction done as per design standard or not. Figure 18 below, shows there is a variation in size between design and practical application on ground in normal condition i.e. without silt or sediment in drainage system. The proper construction practice is important after proper design for storm water drainage system to function properly. This must be carried out by skilled person according to design. During field survey in some area existing drainage sizes on ground are vary from that of design document. They are oversized, undersized and in some area rarely equal with design document. The negative sign indicates oversized and uneconomical drainage system.

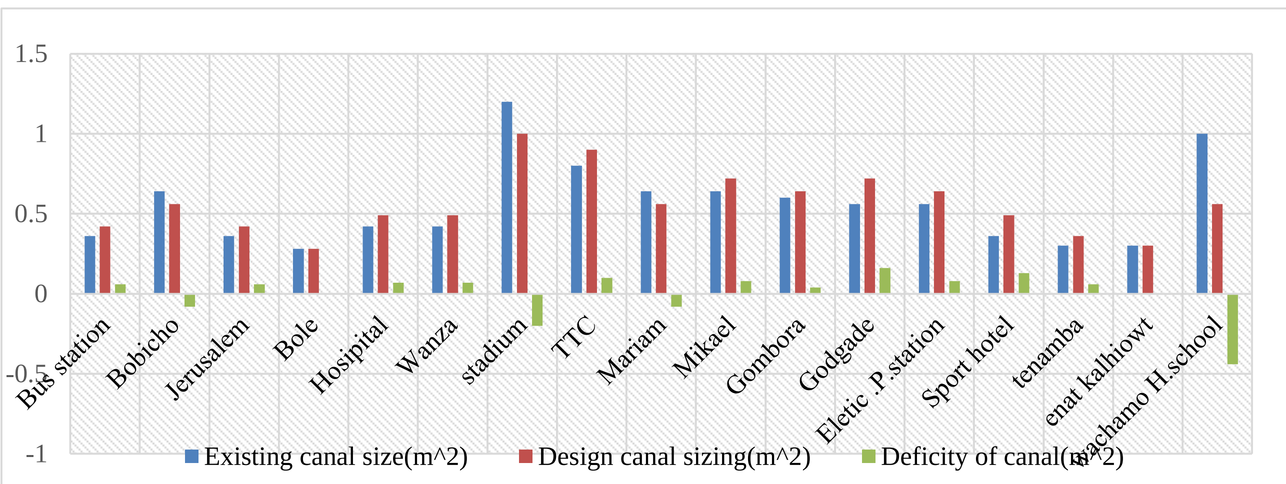


Figure 16: Existing canal size, Design canal size and Deficit of canal size.

Figure 17: Abutment erosion (left) and cracked slab (right).

Due to poor construction, many drainage systems were failed before their design period.

Hosanna town; base failure, edge cracks, gully formation and abutment erosion are common problems related to poor construction. As seen in figure 19 above, Poor construction of pre-cast slab across the drainage channels without considering of heavy vehicles movement or passage through it. In (Fig.20) below at study area previously constructed ditches are seriously damaged; know time existing drainage infrastructures are incapable to convey flood away from road surface and surrounding. Due to this splashing and ponding of storm water on road surface and causes risk on life and properties of residents of town and also challenges movement of vehicles. This Result matches with Getachew’s and Tamene’s (2015) study on assessment of urban road drainage using GinjoGuduru kebele of Jima town in Ethiopia. It was found that road drainage structures of study area was found to be inadequate due to insufficient road profile, insufficient drainage structures, provision, improper maintenance and lack of proper interconnection between the road and drainage infrastructure thereby resulting damaging road surfacing material and flooding problems in area.

Figure 18: Ponding of Storm Water on Asphalt Road and deteriorated road side ditches. (Field survey, 2011).

I. Lack of Community Awareness to Clear Drainage System

Community awareness is one of most proactive measures for the sustainable urban drainage management. Due to lack of regular clearance of drainage lines they become out of services .Sediment load and solid wastes blocks many drainage systems. This paves away for the diminishing of the service life of ditches. At the study area most of existing drainage ditches have been filled by sand and other rubbish materials as seen in (Fig.21) below. The growth of weeds, bush, and trees in a drainage channel is common that reduce its hydraulic efficiency and performance of drainage system to carry storm water in the study area. In addition to this, waste materials thrown from hotels, small business enterprises, shops and wastes like dugs, ashes, plastics from residential houses which block the storm drainage structure and accumulate the storm water within drainage system which exposes to environmental pollution. From the study result a community participation to clean and remove sediment and waste material is low.

Figure 19: Drainage Facility Filled by Sediment and Solid Waste

(Source: field survey)

Figure 20: Drainage Nodes are blocked by solid wastes in study area

(Own field survey,2011)

One of the major problems in relation to environmental problem is the shortage of both public toilet and shower. It is the most important elements that plays its own role in waste management. As shown figure 22 above, many residents of the town choose storm drainage channel as waste disposing site. This study finding interrelated with idea of Faisal et al.(1999) non-structural storm water managements for mitigation of flood impacts focuses up on preventive action and rely predominantly on behavioral changes in order to be effective and concluded.

That well-coordinated and balanced combination of both structural and non-structural measures is required as part of long term flood mitigation strategies in Bangladesh. Correspondingly the UNCHS (1986), study supports this study findings the successful construction of a drainage system in a neighborhood does not guarantee a successful drainage project. Users need to be aware of operation and maintenance requirements at the neighborhood level. The responsibility for solid waste management is virtually always the responsibility of another department in the municipality. Effective drainage area planning requires careful coordination between the relevant institutions responsible for urban drainage in different areas.

In general dominant drainage system problems in the study area are:

- Flooding or inundation,
- Sedimentation,
- Traffic disturbance, flood overtopping, deterioration of roads, pollution and unsanitary condition.
- Properties damage, like existing drainage structures, houses and buildings

The common sources of waste in Hosanna town are:-

A. Commercial institutions:- the commercial institutions comprised of bars, hotels restaurants. Almost all of the commercial institutions have poor liquid waste management techniques. As a result they generation waste water out septic tank, thus, they are becoming one of the pollution source to the town.
B. Individual house: - the shortage of toilet in the town, the some residents of the town generate their waste to the nearby open space, ditches and road sides.
C. The slaughter or butchers house: - the slaughter house in hosanna town is the main source of environment pollution to the town. It is found in closer proximity to residential areas. There is no attempt to adopt modern technology to reduce the

degree of pollution by the municipality .One of the most significant findings was the lack of clear lead in urban drainage management.

Generally, the performance of the current drainage system is poor because from the findings it is observed that the existing of the drainage system is full of problems are above listed.

4.2. Intensity Duration Curve

4.2.1. Hydrology and hydraulics

Hydrologic and hydraulic calculation carried out using rain fall data relate with empirical equation such as Log Pearson type III for frequency analysis and rational method for peak discharge computation. Which distribution fits to the theoretical probability distribution, goodness of fit test using Easy Fit 5.6 professional software to test different methods was identified in methodology part. The caliculated value at (Table 8) using equations at (Apendix Table 3 and 4) shawn the log perason type III distribution gives maximum design rainfall for Hosanna town than gumble's extrem value and it used to develope IDF curve .

Table 8: Comparison of LPT- III and Gumble Value distribution maximum RF.

Return (years)	Design maximum 24 hr point rainfall	
	Log Pearson- III	Gumble's Ext ream value
2	50.68	51.2
5	67.37	62
10	77.84	70.3
25	91.47	80.3
50	102.3	87.2
100	113.7	94.6

Note: LPT-III =log Pearson Type III

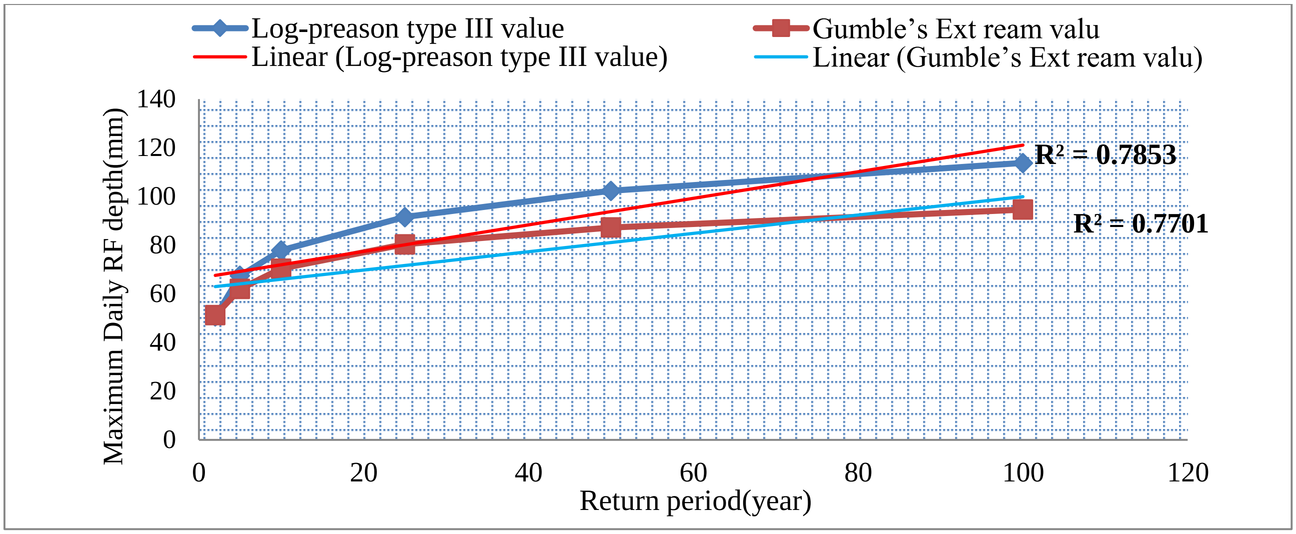


Figure 21: LogPT-III and Gumble Extreme value maximum daily RF depth.

The comparison graph shows above (Fig.23) the trend line value (R^2) was grater for log preason type III PDF than gumble's PDF. The value of R^2 approaches to 1 is perfect equation. Therefore, log preason type III was taken.

Table 9: Comparison of RF depth (ERA, 2013) and Hosanna station rainfall depth.

Return periods in years	2	5	10	25	50	100
ERA (RF depth for RB)	55.26	69	79	92	102	111
Hosanna station (Gumbl's methods)	51.2	62	70.3	80.3	87.2	94.6
Log-P type III value	50.68	67.37	77.84	91.47	102.3	113.7

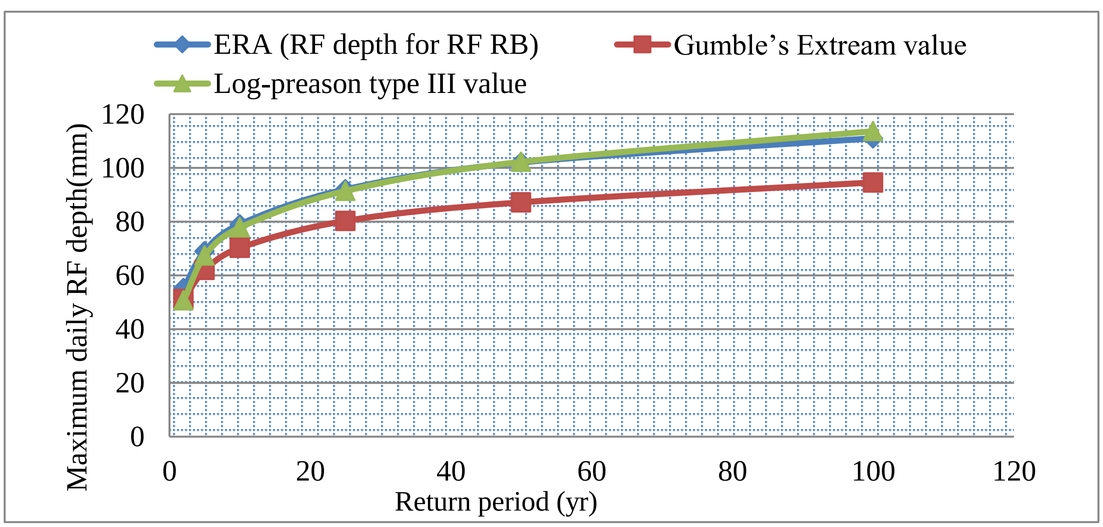


Figure 22: ERA (2013) Vs. Hosanna RF depth, by Gumble and log preason Type-III. (sorce:ERA,2013 and own computation)

The result obtained from easy fit 5.6 professional software and R^2 that, Log-pearson type III was selected as best-fit probability density function(PDF) to compute maximum design discharge. The (Fig .24) above, shows that to reduce the problems that associated with flooding of Storm water, maximum design rainfall log person Type III was selected to prepare IDF curve for specific place hosanna station and to determine maximum discharge for Hosanna town rather than (ERA, 2013), developed regionalized IDF Curve and Gumbl's extreme value distribution, because Hosanna station receives maximum daily rainfall. In (Fig. 23) and table 9 above, the log preason type III gives maximum daily rainfall value. This result related with other Ethiopian town as studied by Habtamu (2017) in Addis Ababa investigation of flood problem a case of Zeneba work, Binam(2016) in Assessment of storm water drainage problem in case of Kamise town and Mulualem & Nega(2018) performance assessment road drainage system in case of burayu Town based on better R^2).

4.2.2. Intensity- Duration- Frequency (IDF) Curve For Hosanna station

The IDF Curve developed for Hosanna station from 24 hour maximum daily rain fall data of (1988-2017) obtained from SNNPR rainfall gauge location in Hosanna, Ethiopia. By using appropriate reduction equation described in the methodology part. The purpose IDF Curve development for this specific study area (Hosanna town) is to compare the result of IDF Curve that developed by (ERA, 2013) and design of peak discharge for different return period. IDF Curve value for Hosanna station developed by (ERA, 2013) is applicable. Hosanna town is located at region B2, the IDF Curve developed for this study region is not much different of that of (ERA, 2013) IDF Curve developed. However, for accuracy and design purpose to determine peak discharge IDF curve developed for specific station. ERA develop IDF curve (2013) value is compiled/amassed I.e. considered as similar region and the same curve value. In ERA drainage design manual Ethiopia is divided in to several hydrological regions which show similar rainfall patterns. Therefore, the developed IDF curve for this specific station (Hosanna town) used to design peak discharge of this specific station.

Table 10: Short duration RF depth for different time intervals at Hosanna station

Maximum Rainfall depth						
Time minute	T2	T5	T10	T25	T50	T100
	RF(mm)	RF(mm)	RF(mm)	RF(mm)	RF(mm)	RF(mm)
5	7.96	10.1	11.67	13.7	15.345	17.055
10	16	17.6	20.5	24	26.83	29.83
20	22.6	26.6	30.6	36.33	40.6	45
30	26	32	37	44	49	54
40	30.3	36	41.3	49.33	54.66	61.3
50	33.5	39.2	45	52.5	59.16	65.8
60	35	42	47	56	63	70
70	36.17	43.16	49	58.33	65.33	73.5
80	37.3	44	50.6	60	68	74.6
90	37.5	45	52.5	61.5	69	78
100	38.33	46.6	53.3	63.33	71.66	80
110	40.33	47.67	55	64.17	71.5	80.6
120	40	48	56	66	74	82
130	41.167	49	58.5	67.17	75.83	84.5
140	42	49.8	58.6	67.67	77	85
150	42.5	50	58.7	70	77.5	87
160	44.9	51.8	59.8	70.5	78.76	87.3
170	46.2	52.2	60.27	71	79.2	88.3
180	47.8	52.9	60.9	69	80.1	89.1

Rainfall Intensity Duration frequency curve for Hosanna station by using log Pearson Type III frequency analysis for different return period used to estimate peak discharge. To develop IDF Curve taking 3hr RF dividing appropriate sub minutes 5, 10, 20, 30……, 180minute, and RF duration as recommended ERA, 2013 manual.

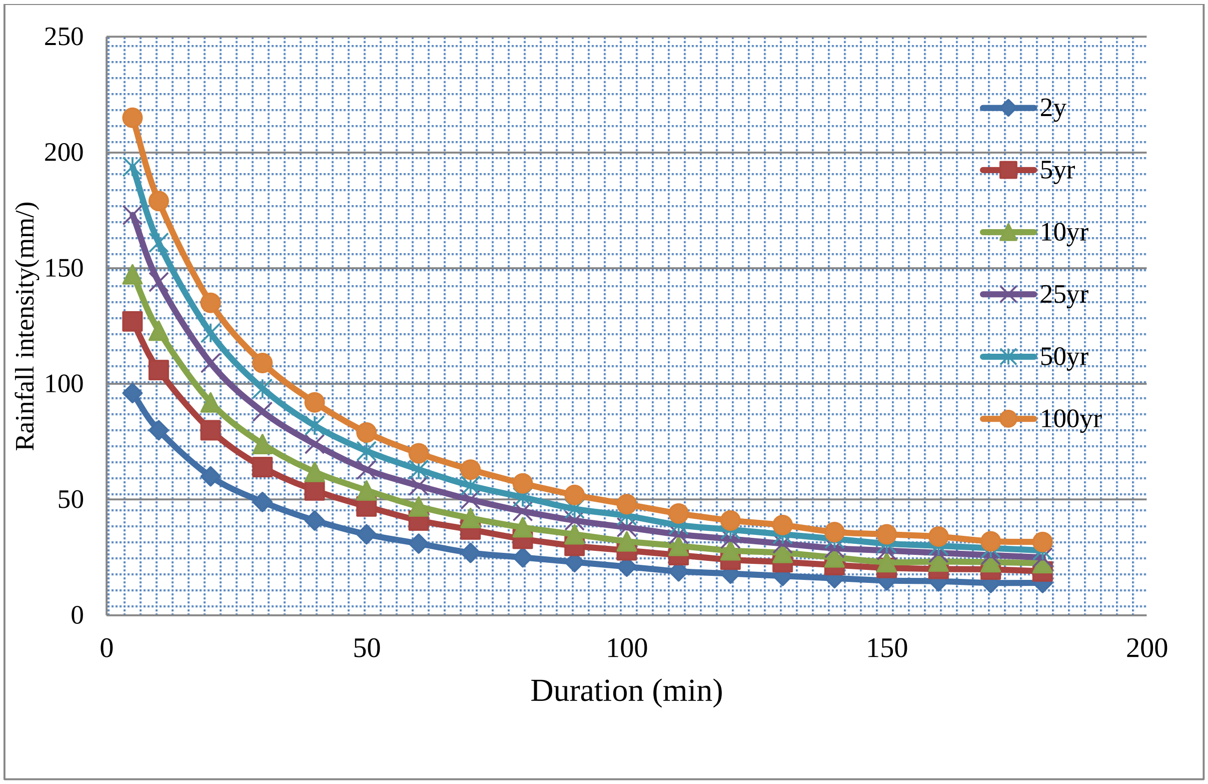


Figure 23: Developed IDF Curve for Hosanna station (1988-2017).

As recommended by ERA, 2013 for small drainage systems and road side ditches were designed for 10 year return period the detail was shown (Appendix Table 15), from this point of view by using 30 year daily maximum rainfall data to develop IDF curve to estimate peak discharge of study area shown(Fig.26)below. Designers should use this IDF Curve to estimate peak discharge of study are.

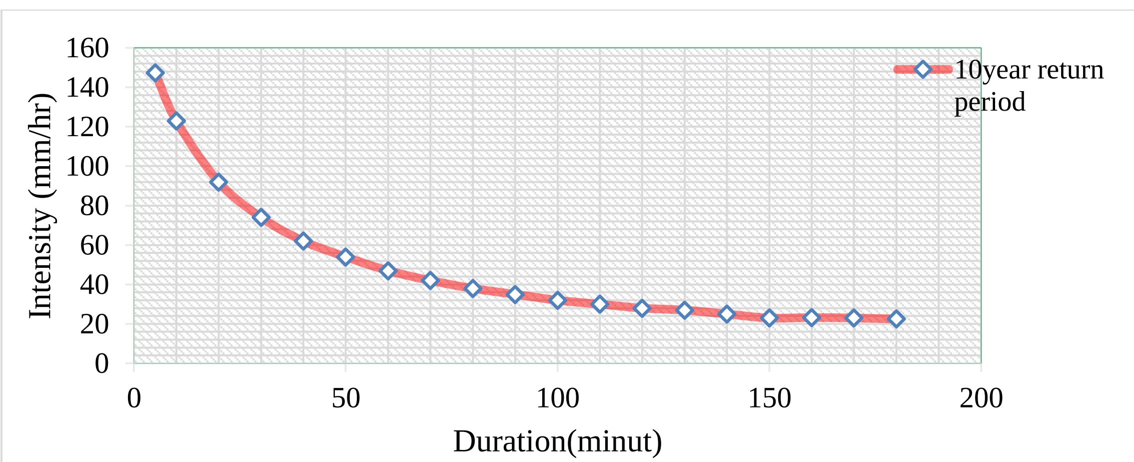


Figure 24: For 10 year return period IDF Curve for Hosanna station.

Table 11: Rainfall intensity for Hosanna station (1988_2017)

	Return periods					
Duration(min)	T2	T5	T10	T25	T50	T100
	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)
5	96	127	142.4	157	168.8	187
10	80	106	123	144	161	179
20	60	80	92	109	122	135
30	49	64	74	88	98	109
40	41	54	62	74	82	92
50	35	47	54	63	71	79
60	31	41	47	56	63	70
70	27	37	42	50	56	63
80	25	33	38	45	51	57
90	23	30	35	41	46	52
100	21	28	32	38	43	48
110	19	26	30	35	39	44
120	18	24	28	33	37	41
130	17	23	27	31	35	39
140	16	21.7	25	29	33	36
150	15	20.5	23	28	31	35
160	14.7	20	23.2	27	30	34
170	14	19.8	23	26	29	32
180	14	19	22.5	25	28	31

Table 12, below shows short duration time RF intensity developed by ERA, 2013 for region B2 is nearly the same as that of short duration time RF intensity developed by researcher for study region Hosanna town.

Table 12: (ERA, 2013) Developed RF Intensity for short duration in region B2

	T2	T5	T10	T25	T50	T100
Duration(minute)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)	I(mm/hr)
5	98.79	129.68	154.68	168.78	183.07	216
10	92.8	115.2	131.2	151.5	166.84	197.52
20	65.30	82.6	94.5	108.7	119.21	141.33
30	53	66.24	76.24	84	96	115
40	44.72	55.59	65.59	72.67	82.09	98.07
50	38.67	48.17	58.17	64.27	79.33	87.56
60	34.1	42.5	48.5	56.3	61.9	68.4
70	30.36	38.45	44.05	50.45	55.1	67.54
80	28	35	39.8	46.73	50.8	56.84
90	25.373	31.67	34.27	42.67	45.7	52.33
100	23.23	28.96	33.96	43.53	45.4	48.52
110	21.41	27.59	31.59	36.47	39.34	45.01
120	20.15	25.5	28.6	33.12	36.2	43.2
130	18.9	23.93	26.93	32.381	35	41.96
140	17.86	22.36	24.31	27.07	34.9	38.75
150	16.84	21.24	23.24	26.4	32	36.74
160	16.02	20.27	21.27	23.97	29.36	34.8
170	15.35	19.399	20.39	22.28	28.1	30.63
180	14.63	18.43	18.43	21.43	26.87	29.26

The catchment areas that contribute storm water to a specific drainage segment are the basic parameter in estimating the peak discharge for the hydraulic sizing of conveyance channels and junctions. The runoff that generated from watershed doesn't flow direct towards outlet because of terrain characteristics of the catchment. Depending up on the entrance node and drainage line availability the study area was divided in to eighteen sub-catchments are selected in Gofer-meda sub city to compute storm drainage capacity by rational method, SWMM and Manning's equation. The study area has two outfalls known by Ajo spring (outfall1) and Abera River (outfall2). Figure 28 below shown.

The ERA, 2013 IDF Curve developed for region B2

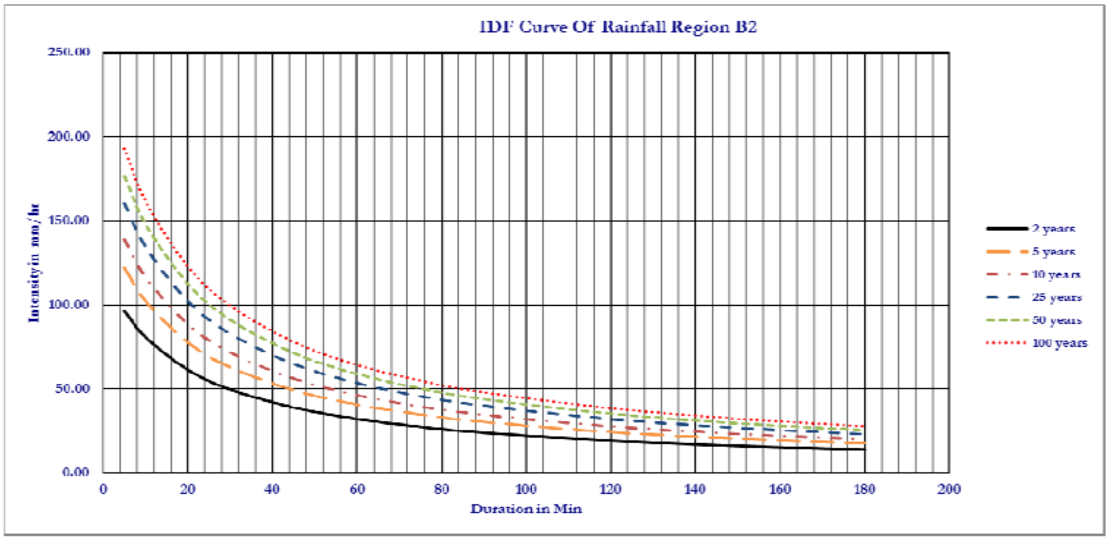


Figure 25: ERA, 2013 IDF Curve for region B2
(Source; ERA, 2013 DDM).

4.3. Hydraulic Performance of e Storm Drainage System in study area

4.3.1. Delineation of Drainage catchment

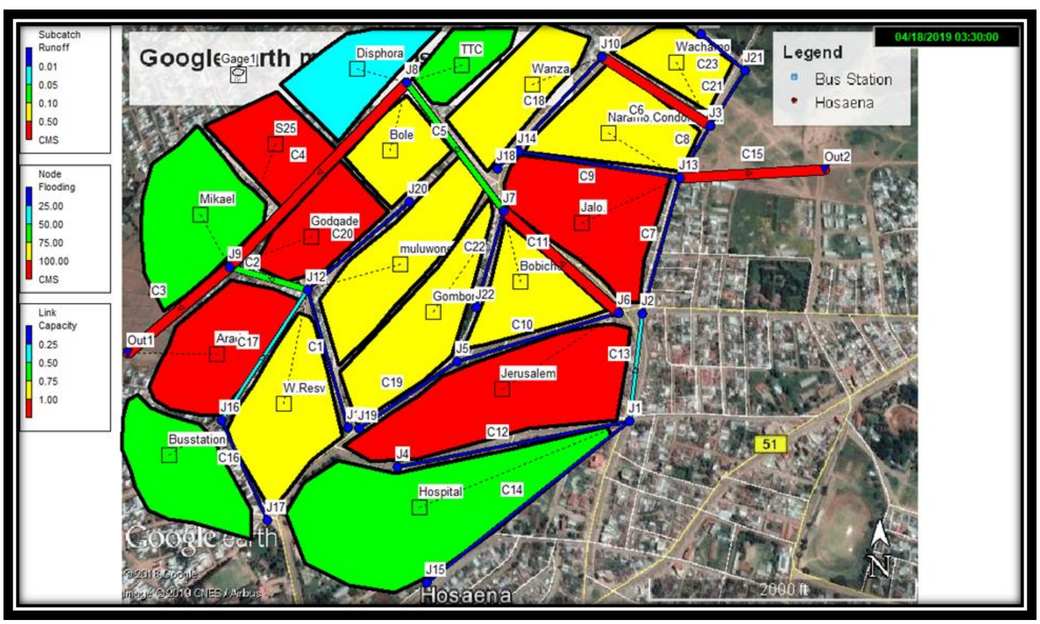


Figure 28: Selected sub-water shed drainage networks and nods.

4.3.2. Runoff computation by rational method at Gofer meda sub-city.

The weighted runoff coefficient (C_w), perecentage of imperviouseness, length of conduit/canals and slop of land used for peak discharge (Appendix table 6).

Table13: below, indicates the computed discharges for each sub-catchments using 3-hr RF intensity duration of time equal to time of concentration by using rational method.

Table 13: Peak discharge computation using Rational Method

No.	Sub catch	area (ha)	I-10yr	I-25yr	I-50yr	R.cof(c)	N	Q(m3/s(10year)	Q(m3/s)25year
1	Bus Station	7.87	105	115	128	0.74	0.011	1.7	1.87
2	Jalo	13.86	72	84	92	0.67	0.011	1.88	1.84
3	Jerusalem	13	73	88	97	0.79	0.011	2.11	2.25
4	Gombora	10.88	89	108	125	0.76	0.011	2.05	2.5
5	Bole	9.36	100.1	109	118	0.78	0.011	2.03	2.26
6	Arada	15	84.6	109.1	97.8	0.68	0.011	2.4	2.84
7	Disphora	17.62	68	80	102	0.63	0.011	2.1	2.03
8	S25	15.8	78	92	104	0.73	0.011	2.5	2.86
9	Hospital	10.4	115	126	140	0.77	0.011	2.6	2.34
10	Godrgade	14	78	86	100	0.88	0.011	2.72	2.91
11	W.Res	13.4	92	106	120	0.76	0.01	2.62	3.1
12	NC	12	101	120.2	129	0.8	0.01	2.7	3.25
13	TTC	11.7	103.2	122.3	130	0.74	0.011	2.64	3.14
14	Mikaiel	12.5	109.5	128.9	137	0.76	0.011	2.9	3.5
15	wanza	12	102.3	122	134	0.8	0.011	2.75	3.2
16	Muluwongel	13	100.4	118.9	127	0.77	0.011	2.8	3.1
17	Bobicho	12.8	103	122.1	129	0.68	0.011	2.63	2.8
18	Wachamo	12.4	108.9	127.8	137	0.753	0.011	2.81	3.3

Note: TTC= Teachers training college, W. Res =water reservoir, NC=Naramo Condominu

4.3.2.1.Adequacy and Availability of Drainage system

The result of drainage canal capacity computed by Manning's equation, most of existing drainage channels are incapable to carry coming flood during rainy season, design rainfall intensity with in return period at 10 year. (See Appendix Table8). (Fig. 28) below shows; typical examples of the most flood prone areas that needs special consideration for the problem of flooding during rainy season are: Bus station, wanza, TTC, Jerusalem, Hospital, Mikael and Gorgoade. In these areas storm water flooding problem is an eye burning issue during rainy seasons. The storm water overtops from drainage system. In another hand some storm water drainage systems were oversized.

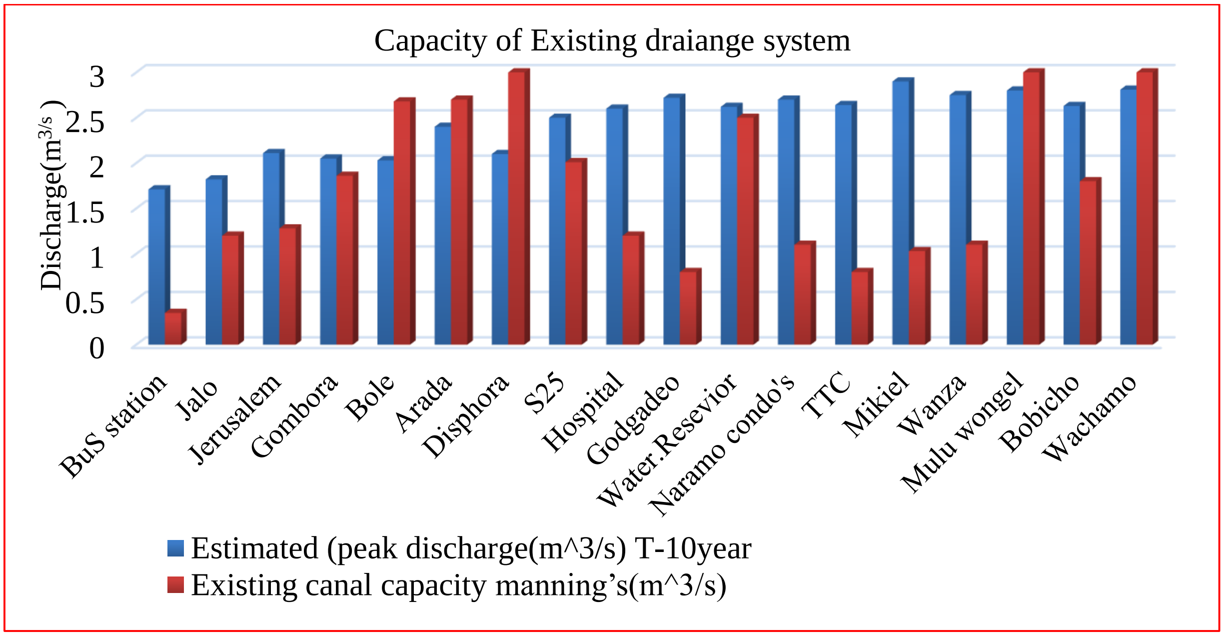


Figure 26: Existing drainage capacity vs. estimated peak discharge for 10 year return period.

4.4. Storm Water Management Model Simulation Result

4.4.1. Flood level in nodes and design depth in sample sub-catchments

According to ERA, 2013 drainage design manual drainage system were designed for only 10 year and checked for 25 year return period's RF intensity. SWMM Simulated result shows total volume of flooding and time for flooded nodes in sample catchments for 10 year return period was shows in (Appendix table 16).

The study area sample watersheds and their runoff characteristics are presented by using the SWMM. The simulation results of SWMM a 3-hr duration rainfall intensity in 5-minute time interval, considering present land use land cover and effects of urbanization on drainage system, the storm water runoff from Sub catchments of TTC, Diaspora and Bole joining at J8 is as shown (Fig.29). As can be read from the graph there is flooding in this joint. I.e. this junction is incapable to carry entering storm water from these catchments. The vaue in Fig 29.(number y-axis at right side is depth in meter).

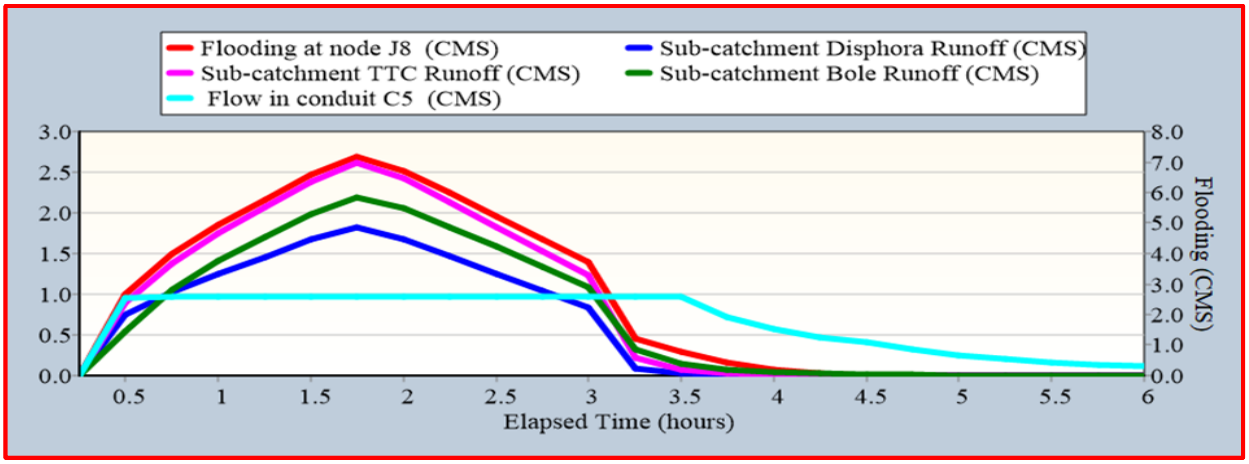


Figure 27: Runoff hydrograph for selected sub-catchments for 3-hr RF intensity.

Figure 30 below, the coming flood from surrounding sub-catchments of Arada, Godgade and and Mikael then joins at junction J9.

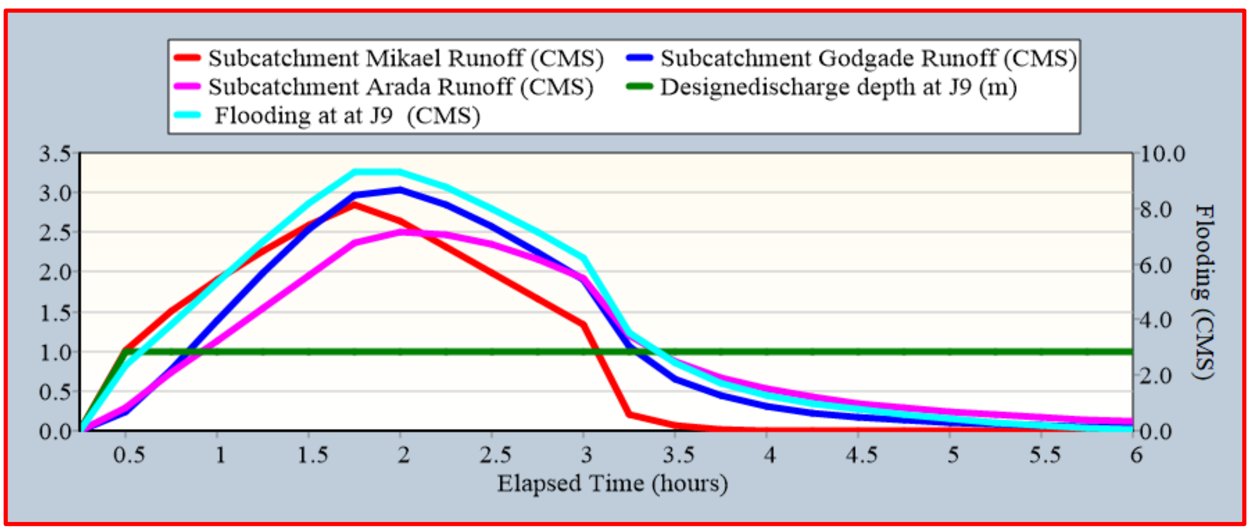


Figure 28:Sub-catchments flood Vs. flooding node

During field observation in study area at rainy season many Junctions were filled by solid wastes; for instance at sub catchment Jalo, storm water flow in conduit (C7 from Jalo and C9 from Naramo condominium) as seen (Fig.31) below, C7 and C9 designed discharge carrying capacity is enough, but at J13 over flooding happen during rainy time, due to silt and chocked by of solid wastes by ill managed waste system. The SWMM 5.1 simulates peak runoff at catchment Jalo and Naramo Condominium shown in (Fig.31) below, the runoff from these two sub-catchments enters and joins at point J13.

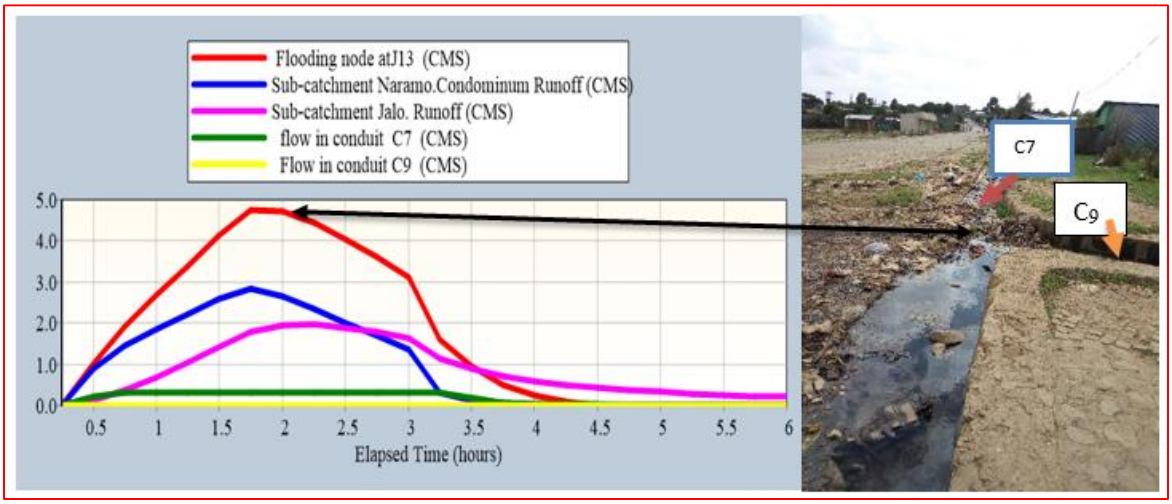


Figure 29: Flooding node at J13 vs. Sub-catchment runoff

(Source: field survey and SWMM5.1 analysis).

At sub-catchment "Wachamo" as shown (Fig.32) below, the maximum designed depth is greater than flood level at J10 and based on the simulated result of SWMM 5.1 for considering current land use situations and 3-hr intensity for 10 year return period, in this catchment there is no flooding of the storm water and there little water pass through conduit (C18).

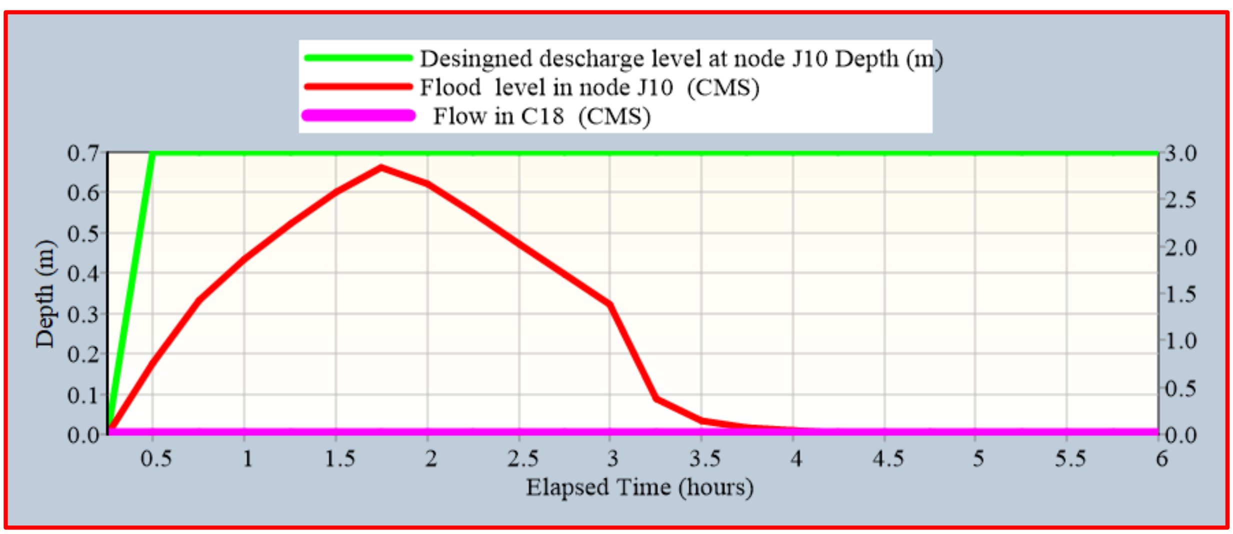


Figure 30: Node depth and flood in junction at "Wanza" sub-catchment

Figure 33 below, Junction (J3) in front of Shambalala Hotel is unable to carry coming flood at now time. It is filled by sediment and solid wastes there is also lack of proper maintenance. Due to this it become out of service. From the SWMM model simulation 3-hr RF intensity for 10 year return period flooding is happening in this node. Consequently, the storm water is being stored for long period of time.

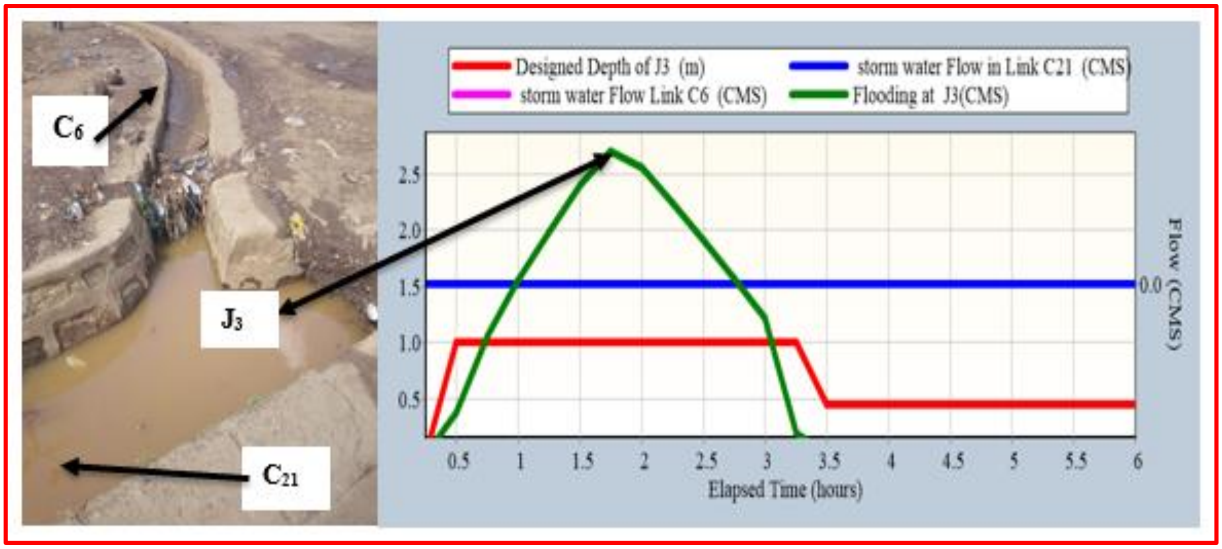


Figure 31: Flooding node at J3

4.4.2.1. Locations of flooded Junctions

Table 14: absolute locations of flood prone junctions

Flooding node for 10yr	Max Rate flood (CMS)	Time of max flooding Occurrence day, hr: min	Easting	Northing	elevation
J1	2.17	0 ;00:45	373401	834929	2318
J2	0.182	0;00:35	373432	834929	2317.5
J3	2.61	0;01:15	373604	835666	2317
J6	1.78	0;02:30	373375	835160	2316
J7	4.108	0;02:30	373070	835160	2314
J8	7.14	0;01:30	372650	835948	2310
J9	8.78	0;01:15	372180	835535	2309
J10	2.78	0;02:30	373367	835859	2317
J12	5.15	0;01:30	372656	835253	2316
J13	4.31	0;02:00	373545	835532	2313

4.4.3. Water elevations profiles and links capacity

Due to lack of proper bed slope of canal, less integration of drainage network with each other and road network causes flooding to road surface and create potholes on asphalt road at rainy season as shown (Fig.34) below.

Jerusalem sub-catchment in front of Mobile Hotel to St. Mariam church the existing canal is aged, abutment damaged and pre-cast slabs are broken down. There is no regular maintenance of road and drainage structures at this site as it was investigated during field study. This results over flow of water on road system and creates potholes on road. From SWMM 5.1 model red color line indicates storm water flow level and blue color one shows designed depth of canal, water level is greater than channel depth.

Therefore flooding occurs in this site. As shown (Fig.34) below link flow greater than designed depth.

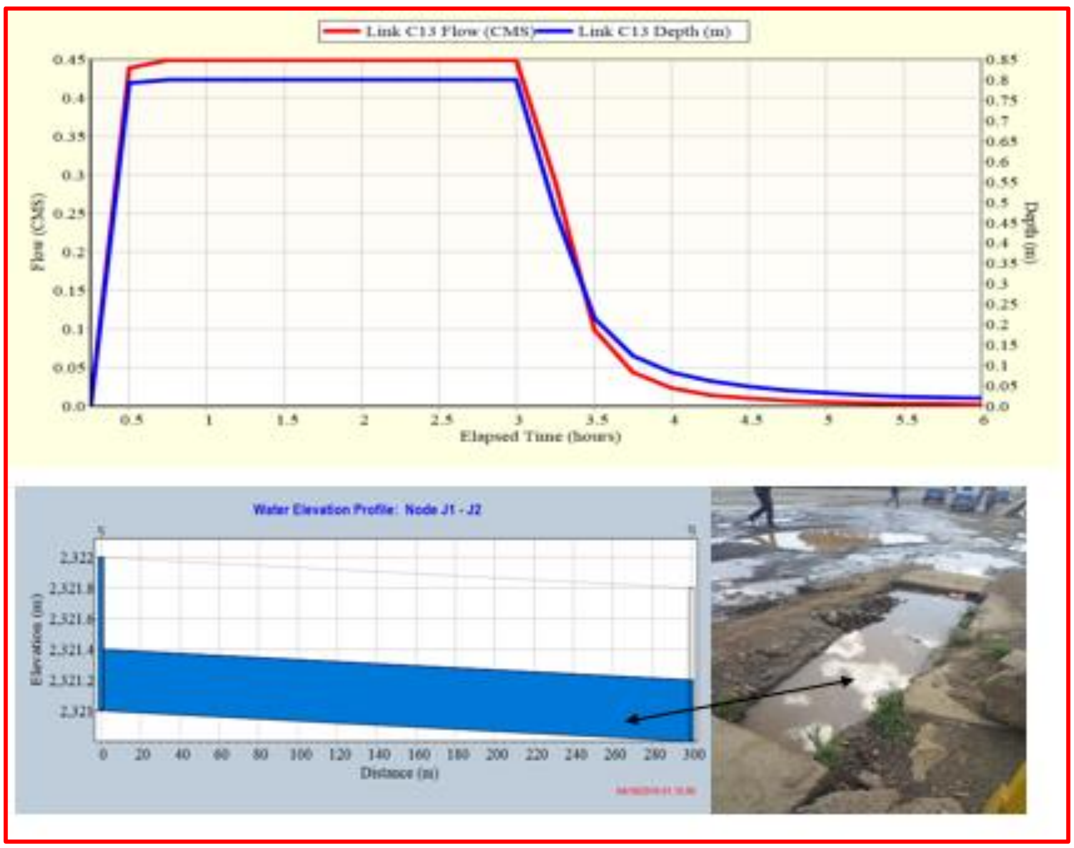


Figure 32: Flood and elevation profile in front of Mobile Hotel Jerusalem Catchment.

(SWWM and field survey 2018).

From field observation made, some ditches were constructed for nothing as there is no inlet or opening to collect storm water from the surrounding area or road.

(Fig.35) blow shows; that in front of Ashem Hotel to Omedad café at the main road of Hosanna to Addis Ababa, due to absence of proper curb inlets and well integration road network with drainage system to take away storm water form asphalt road surface, the storm water splash on asphalt for extended period, in addition to this, the canal itself has no enough bed slope.

The SWMM model shows that the blue color part in channel the level of storm water and the grey one shows free space that has not water reach. Due to inadequate integration between road and urban storm water drainage system, the major portion of the area exposed to flooding hazard. According to Faisca *et al., (*2009) study at Netherlands, their result is similar with this study; a surface drainage system (ditch) collects and diverts storm water from the road surface and surrounding areas to avoid flooding, otherwise water retains on road system which decreases bearing capability of road.

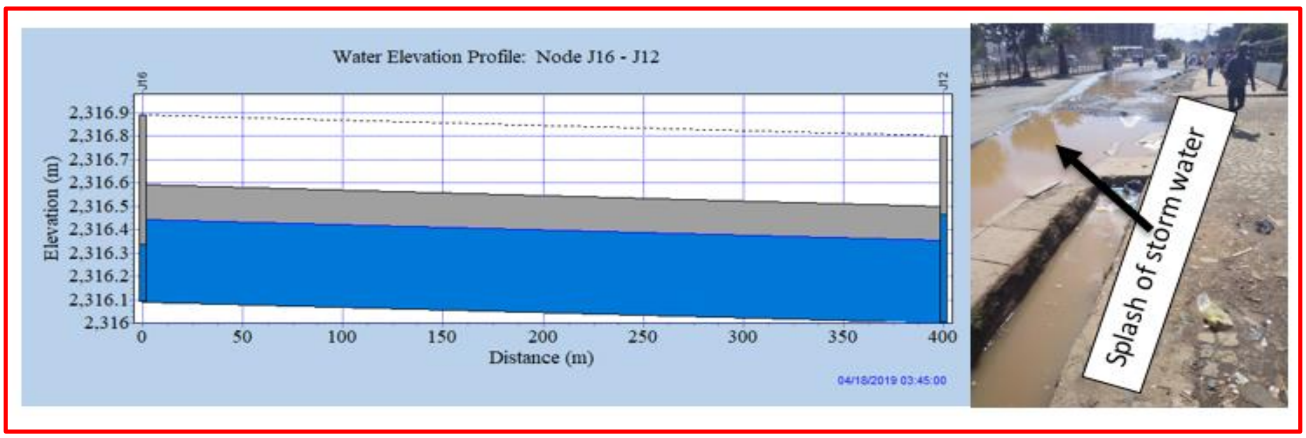


Figure 33: splashing of storm water on road surface

(Source: SWMM and field survey, 2018)

As it was observed in field survey and SWMM model simulation result there is no design as well as construction problems at Muluwongel sub-catchment, drainage system from the SWMM a drainage channel cross section and slope to convey storm water was adequate as shown (Fig. 36) below. But, currently this drainage system is out of function because of complete blockage by waste materials from the surrounding residents. The channel is being used as waste disposal site rather than conveying storm water. Also a Tucci(2000), studies result shows true in this study finding the amount of garbage entering to drainage network reduces efficiencies of channel performance .

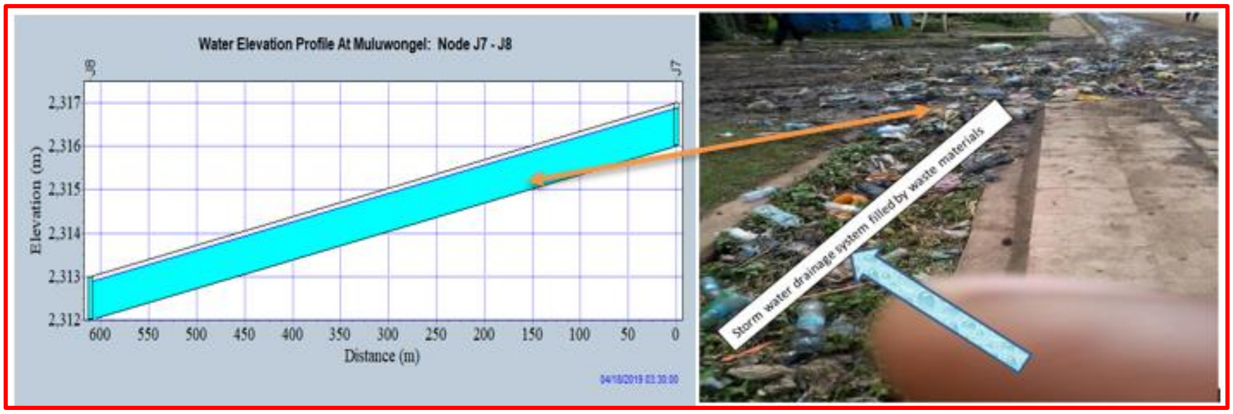


Figure 34: Storm Drainage System is filled by waste materials

(source: SWMM analysis and Field survey;2018)

At Jerusalem sub-catchment, Hosanna to wolikite main road side ditch J1 to J2 conduit 13 (Fig. 37)below in existing conditon, the canal is incapable to convey coming flood in appropriate way because it is completely filled by silt. As a result, the coming flood is over flowing above canal and splash to road system. Therefore, at rainy time vehicles face challenges to use this asphalt road. The SWMM model simulation confirm the grey color in drainage channel shows occupied by silt.

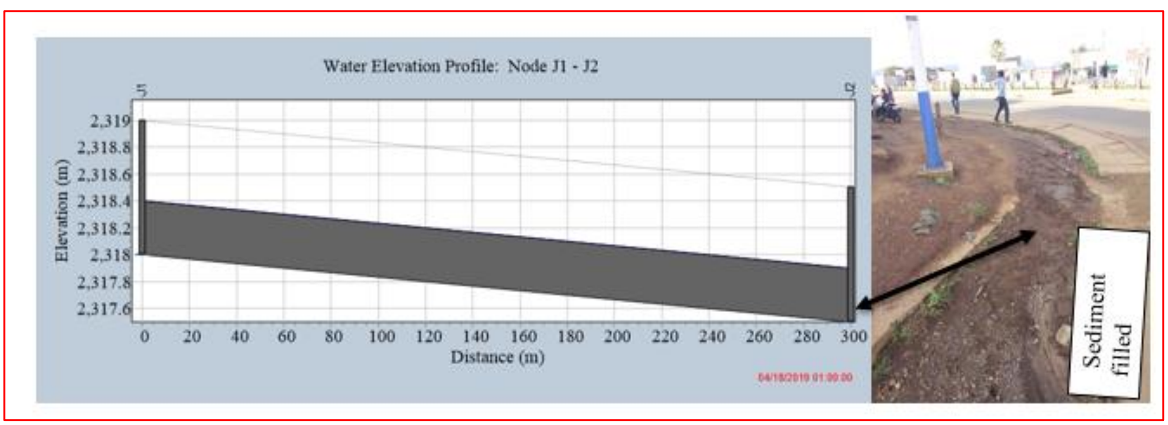


Figure 35: Silted Canal at Jerusalem sub catchment

(Source:SWMM and field survey)

In some incidences very steep slopes of channels causes deterioration and bed erosion. As shown (Fig.38) below, SWMM model simulation the steepest slope of the channel accelerates stormwater flow in channel system and the storm water rapidly reaches downstream causing da mage to outlet structure. Around "Batena Loji" due to steeply sloped channel constructed, flood over tops from cannels and stored at bridge site is challenging passage of vehicles and pedestrians during rainy season.

Similarly, Magdi (2016) in his study recommends a good drainage alignment with respect to slope of road is important to avoid silting, ponding and as well as deterioration and bed erosion.

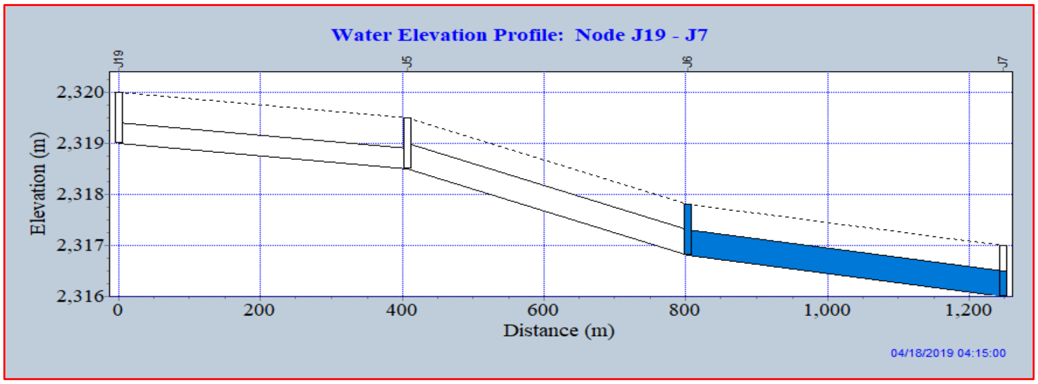
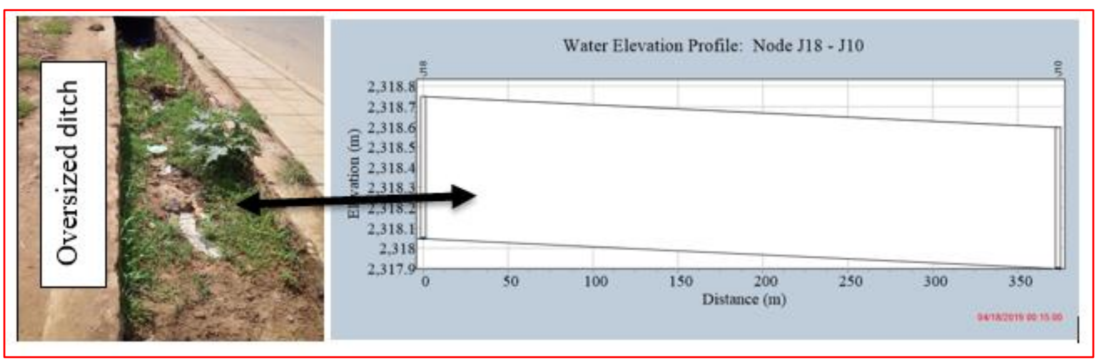


Figure 36: Steep sloped Water Elevation profile

In some occasion, at the study area during field survey there were un econoiacl designed canals have been observe, they are over sized, the the existing canal capacity is much greater than estimated peak discharge. As shown (Fig.39(a)) below at in front Wachamo High school and (Fig 39(b))“Masalamia to Tinsae Hotel” nowadays existing canal are not use for storm drainage system also SWMM model simulation shows in 3hr RF intensity no storm water conveyed through these channels. Beside, of over size these channels are also less linked with other drainag networks and road system. According to ERA,2002 Drainage design manual basic design techniques in roadway drainage system should be developed for economical design of surface draiange structures.



A. Oversized canal at in front of Wachamo high schoo
Figure 37: oversized canal

(Source:SWMM and field survey,2018)

4.4.4. Overview of Storm drainage System and Capacity

At the time of field survey the drainage system properties were collected like, inverted elevations of node, maximum water depth of nodes, the length and shapes of conduits to evaluate discharge carrying capacity of by mannigs and EPA SWMM. The cross setional shapes of conduits surveyed were mostly rectangular and trapezoidal. During rainy season many draiange lines are suffering from conveying storm water flooding (Fig. 40) Red colored conduit are over flooded totally blocked by solid waste and sediments the are out of function,yellow conduits are somewhat filled with silt and partially functinal and blue conduits are works properly.

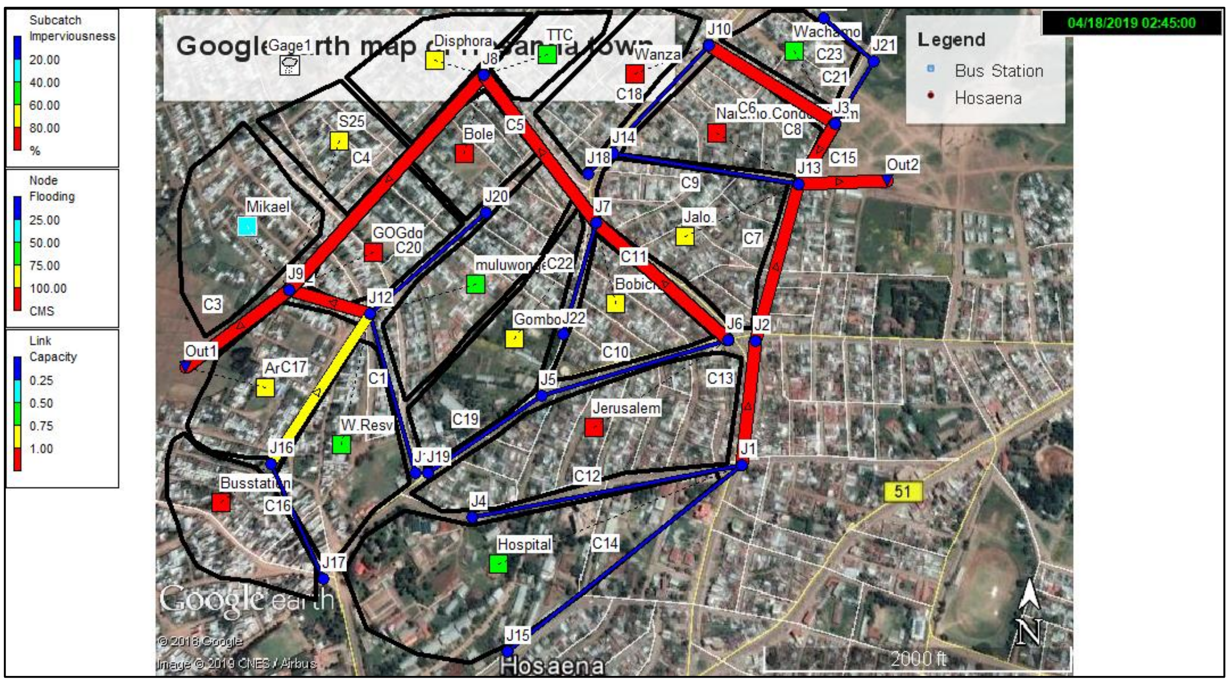
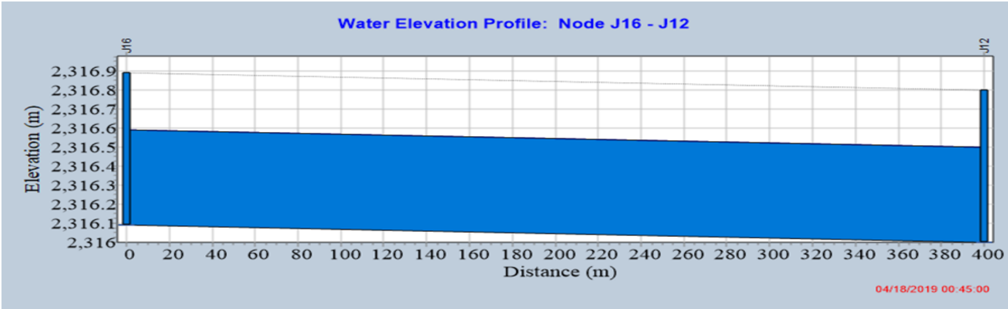


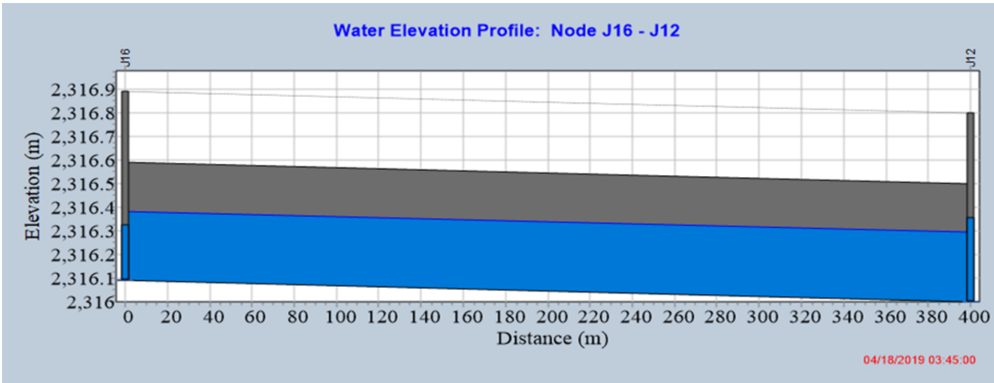
Figure 38: Determination of nodes and conduits carrying capacity using SWMM 5.1.

Rainfall intensity effects at different sub-chatchments links

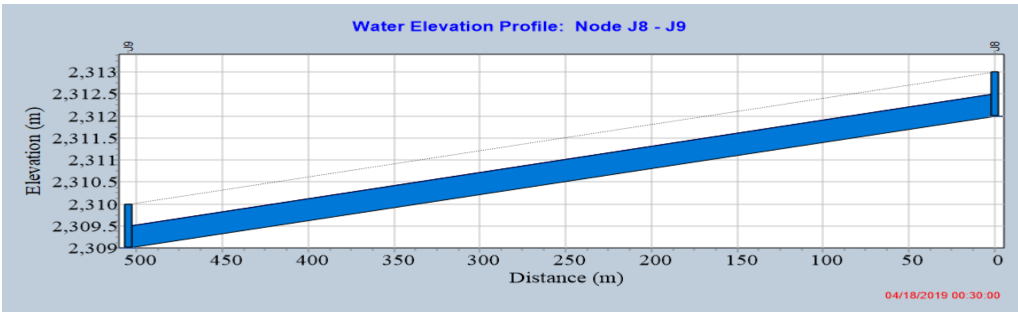
During heavy rainfall for short duration rainfall intensity(Fig 41 A-D) below shows 0.5 hr rate of flood occurance(time to peak) is very fast at many channels and it release (falls) with fast is rate. This is true for most of simulated junctions, causes flooding problem.



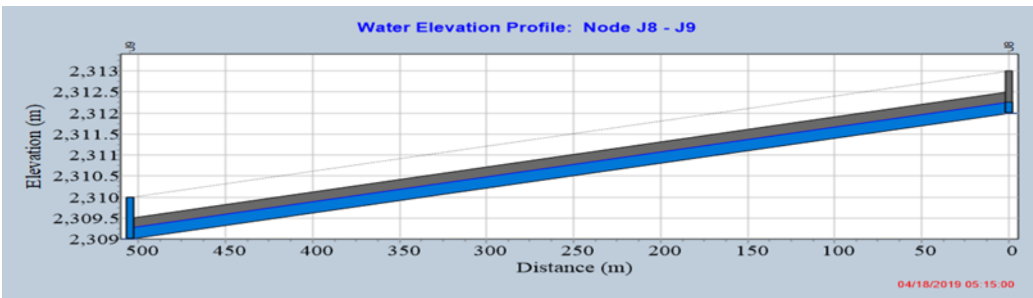
A. Rainfall intensity effect for 0.5 hr at Arada L-17



D. Rainfall intensity effects for 1hr at Bobicho L-17



C.Rainfall intensity effect for 0.5 hr at Godgade L-4



D. Rainfall intensity effect for 1hr at Godgade L-4

Figure 39: Rainfall Intensity characteristics

The simulated discharges values of all sub-catchment with SWMM compared with peak discharges by rational method for 10 year return period follows.

Table 14: Comparison of Peak discharge with Simulated SWMM and rational method

No.	Sub catch	Q(m3/s(10year)	Q(m3/s)25year	Q10 SWMM 5.1
1	Bus Station	1.71	1.87	1.72
2	Jalo	1.82	1.84	1.82
3	Jerusalem	2.11	2.25	2.16
4	Gomombora	2.05	2.5	2.1
5	Bole	2.03	2.26	2.02
6	Arada	2.4	2.84	2.4
7	Disphora	2.1	2.03	2.3
8	S25	2.5	2.86	2.51
9	Hospital	2.6	2.34	2.5
10	Godrgade	2.72	2.91	2.69
11	W.Reserviour	2.62	3.1	2.63
12	N.Condom	2.7	3.25	2.69
13	TTC	2.64	3.14	2.63
14	Mikaiel	2.9	3.5	2.92
15	wanza	2.75	3.2	2.77
16	Muluwongel	2.8	3.1	2.85
17	Bobicho	2.63	2.8	2.62
18	Wachamo	2.81	3.3	2.8
19	Total	43.89		44.13

The simulated peak discharge by SWMM was compared with rational method, to ensure SWMM 5.1 model was combatable with study area parameters for existing land use land cover and 3-hr rainfall intensity, the coefficient of determination (R^2) with in allowable limit.

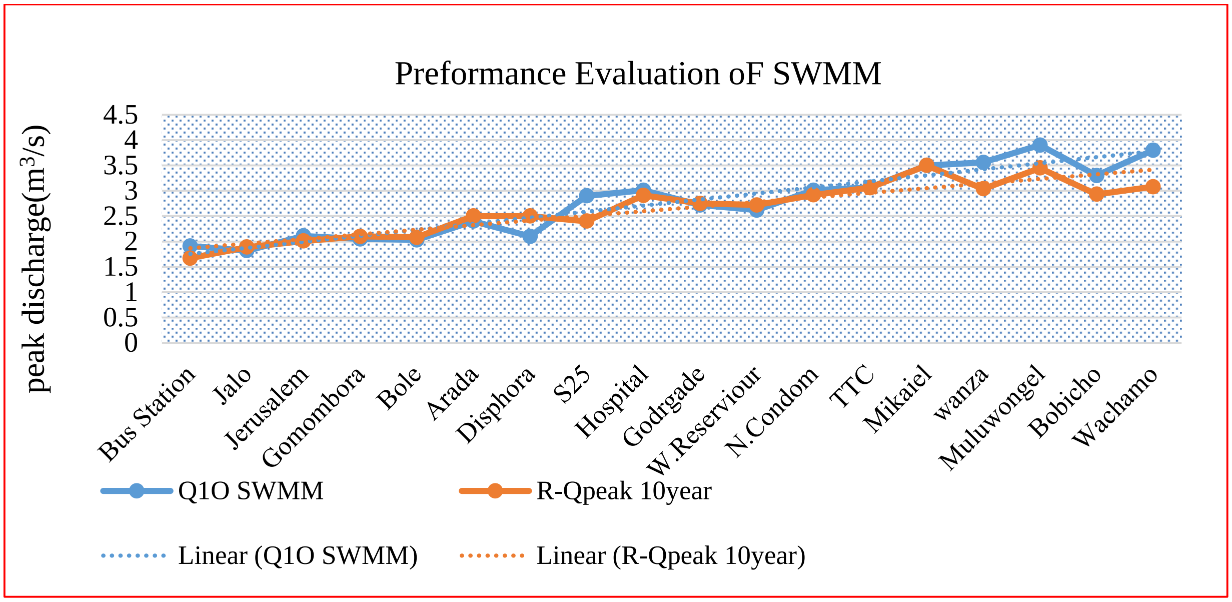


Figure 40: Peak Discharge with SWMM vs. Rational method

5. SUMMARY AND CONCLUSIONS

5.1. Summary

The provision of urban storm water drainage system is a challenging problem in most towns of Ethiopia and the situation in Hosanna town is not different from this. The EPA SWMM 5.1 computer based mathematical models were used to simulate catchment runoff in urban areas. It is widely used urban flood planning and management. In study area Dynamic wave routing and Modified Green Ampt infiltration approach were applied to analyze flow routing the catchment. According to this thesis result most of the existing storm water drainage systems are incapable and inadequate to convey the peak discharge within 10 year design period. Based on the result, problems are associated with lack of community awareness, design and construction practices are adopted by ignoring of hydrology and hydraulic analysis. Rapid population increase and urbanization can decrease capacity of storm water drainage system.

Rainfall data for 30 years (1988-2017) was analyzed by log-preason type III distribution method for 2, 5, 10, 25, 50 and 100 years return period and used to develop appropriate IDF Curve for Hosanna station to estimate peak flow. Depending up on entrance node and availability of drainage line, the study was conducted by dividing total catchment area in to 18 sub-catchments. The SWMM 5.1 simulate result Shows total flooding at nodes is 40.7m^3/s from this each node accounts J1(5.1%), J2(0.56%), J3(4.1%), J6(4.5%), J7(10.08%), J8 (18%), J9(21.6%), J10(6.7%) , J12(13%), J13(13.4) and J16 (4.3%) m^3/s over flooded and their absolute location was determined with help of GPS.

The intensity duration frequency curve (IDF) is developed in this thesis can be the input for Hosanna town's designers and planners during construction of asphalts, cobble stones, culverts and other hydraulic structures.

Estimated peak discharges by rational method and EPA SWMM 5.1 are compered by existing canal capacity computed by Manning's equation. Generally, it can be concluded that storm drainage systems in study area found to be inadequate due to insufficient elevation profile, lack of proper interconnection between the road and drainage infrastructures there by resulting damages road surface materials and flooding problem in the area.`

5.2.Recommendation

Based on the findings of study outcomes the following recommendations are drawn out:-

- Provision of proper connections or integrations between the road network and drainage network is required with regular maintenance.
- IDF Curve prepared for Hosanna town is give better and accurate reading result comparing with those ERA regionalized IDF curve, therefore the designers and planners had better to use this developed IDF curve to design RF intensity. During design stage of drainage structures, all parameters such as appropriate catchment size determination, variability of climate, future urbanization shall be considered.
- Training and technical support should be provided to those Micro small Enterprise (MSE) to enable them to conduct the construction of storm water drainage systems with good quality and applied as per design standard. Previously constructed under sized drainage systems should be scaled up to cope with the anticipated area characteristics.
- This was study limited in Hosanna town Gofer-meda sub-city. It will be better further researchers account 2D view to show problems related storm drainage system design and plan for whole Hosanna town. For further research, and accurate designing and planning of urban infrastructure, gauge site should be provided in town.
- As a whole storm drainage system construction in Hosanna town Gofer-meda sub-city should have better take notable consideration for future urban development.

Limitations

This research work comes through with certain limitation. Which come across:-

There is no recorded runoff data for historical RF at different junction and outfalls to calibrate the simulated result of the model at the nods and outfalls. Since, SWMM technology is new for our country case, so measuring runoff depth at nods and junction is not adopted in Hosanna town. The SWMM model needs high quality primary data to minimize errors related with topography. Collecting each watershed property at rainy time is so much tedious work and also to do this practically the limitation of material, finance and time to collect necessary data from field. Therefore, the model results are simulation without calibration. Beside of this, the model limitation is that the model is (1D) view of flow in canal it does not show the flood inundation area with it.

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7. APPENDICES

Table 1a: Daily max Rainfall of Hosanna station (1988-2017)

year	Dail max RF	Desinding(RF)	Rank(m)	T=(n+1/m)	log(RF)	{log(x)-ave(logx)}	{log(x)-ave(logx)}^3	p=1/T
1988	67.1	87.1	1	31	1.94	0.24	0.013	0.032
1989	37.8	77	2	15.5	1.808	0.182	0.0064	0.065
1990	62.3	73.9	3	10.33	1.83	0.166	0.0047	0.096
1991	61.2	67.1	4	7.75	1.829	0.126	0.00203	0.129
1992	39.4	66.8	5	6.2	1.825	0.1247	0.00194	0.16
1993	63.9	63.9	6	5.16	1.8054	0.105	0.0011	0.193
1994	37.6	62.8	7	4.43	1.79	0.0979	0.00094	0.22
1995	51.3	62.3	8	3.87	1.79	0.0945	0.00084	0.25
1996	42.3	61.2	9	3.44	1.78	0.0867	0.00065	0.29
1997	87.1	60	10	3.1	1.771	0.078	0.00047	0.32
1998	37.1	59	11	2.818	1.77	0.0708	0.00035	0.35
1999	35.4	54.6	12	2.583	1.73	0.037	5.14483E-05	0.38
2000	40.6	52.2	13	2.388	1.713	0.017	5.51756E-06	0.419
2001	50.4	51.3	14	2.228	1.711	0.01	1.03562E-06	0.45
2002	77	50.4	15	2.067	1.702	0.002	1.43584E-08	0.48
2003	49.5	49.5	16	1.9375	1.69	-0.005	-1.5701E-07	0.51
2004	39.3	47.8	17	1.871	1.674	-0.02	-8.7065E-06	0.544
2005	60	47	18	1.72	1.672	-0.027	-2.1726E-05	0.583
2006	73.9	45	19	1.642	1.65	-0.046	-0.000102	0.612
2007	47.8	42.3	20	1.55	1.624	-0.07	-0.00039	0.61
2008	52.2	40.6	21	1.47	1.608	-0.09	-0.00076	0.671
2009	47	39.4	22	1.491	1.596	-0.1045	-0.00114	0.709
2010	33.5	39.4	23	1.342	1.592	-0.104	-0.00114	0.74
2011	37.2	37.8	24	1.27	1.573	-0.122	-0.0018	0.774
2012	45	37.6	25	1.24	1.574	-0.124	-0.00196	0.806
2013	35.6	37.2	26	1.192	1.5701	-0.129	-0.0021	0.83
2014	66.8	37.1	27	1.1485	1.569	-0.13	-0.0022	0.87
2015	54.6	35.6	28	1.107	1.55	-0.148	-0.0032	0.903
2016	62.8	35.4	29	1.06	1.54	-0.15	-0.0034	0.93
2017	59	33.5	30	1.033	1.525	-0.17	-0.0053	0.96

Table1b: Annual average rainfall of Hosanna and near to stations.

No	Rainfall stations	Annual average rainfall(mm)	Total	Distance from Hosanna station(km)
Hosanna(Nx)		1173	35195	
1	Gimbichu(N1)	1096	33678	31
2	Angacha(N2)	1425	42763	29
3	Durame(N3)	1148	34453	48

Table 2a: Annual cumulative Rainfall of Study area and Surrounding Stations.

Cumalative Rainfall Average	Commulative Rainfall of Angecha(mm)	Comulative Rainfall of Hossana(mm)	Comulative Rainfall of Durame(mm)	Comulative Rainfall of Gimbichu(mm)
1238.9	1495.1	1089.6	1132	1189.344
2320.533333	2560.5	2115.1	2286	2374.344
3754.1	3912.4	3503	3846.9	3648.344
5461.666667	5226.7	4800.4	5357.9	4846.344
6400.1	6230.1	5720.4	6249.8	5944.344
7671.733333	7435.2	6884.1	7195.9	6924.344
9015.766667	8918.7	8057.3	8571.3	7914.344
10213.511	10422	9486.8	10236.2	9209.044
11578.561	11767.1	10920	11603.9	10523.244
12646.861	12741.7	11923.6	12757.9	11664.244
13731.236	14004.7	12915.5	13942.5	12562.244
15227.986	15006.7	14009.4	15359.5	14036.344
16638.011	16375.2	15260.3	16765.6	15350.944
18034.636	17597.7	16406.6	18232.2	16402.044
19297.761	19311.9	17569.1	19428	17382.044
20575.436	21240.9	18744.1	20653.5	18163.244
21941.761	22571.2	19941.8	21853.6	19400.444
23286.311	23964	21120.7	22964.3	20596.244
24561.136	25407.6	22331.3	24233.5	21772.144
25646.961	26667.5	23525.7	25242.5	22652.144
26710.6585	27901.8	24647.29	26495.8	23297.744
27891.3585	29275.5	25770.19	27560.2	24459.544
28959.2335	30403.3	26791.49	28643	25499.144
30090.326	31694.1	27963.86	29812.8	26390.544
31368.601	33042.9	29462.96	30813.5	27655.044
32369.701	34098.9	30407.76	31822	28650.144
33615.2735	35699.4	31683.86	33056.59	29521.244
34624.6985	36716.6	32722.56	33848.39	30711.244

Table 2B: Outlier test Kn Value

Sample size n	K_n	Sample size n	K_n	Sample size n	K_n	Sample size n	K_n
10	2.036	24	2.467	38	2.661	60	2.837
11	2.088	25	2.486	39	2.671	65	2.866
12	2.134	26	2.502	40	2.682	70	2.893
13	2.175	27	2.519	41	2.692	75	2.917
14	2.213	28	2.534	42	2.700	80	2.940
15	2.247	29	2.549	43	2.710	85	2.961
16	2.279	30	2.563	44	2.719	90	2.981
17	2.309	31	2.577	45	2.727	95	3.000
18	2.335	32	2.591	46	2.736	100	3.017
19	2.361	33	2.604	47	2.744	110	3.049
20	2.385	34	2.616	48	2.753	120	3.078
21	2.408	35	2.628	49	2.760	130	3.104
22	2.429	36	2.639	50	2.768	140	3.129
23	2.448	37	2.650	55	2.804		

Source: U.S. Water Resource Council, 1981.This table contains one-sided 10-percent significance level Kn values for the normal distribution

Table 3: Log-Pearson type III distribution parameters.

Log Pearson type III method		Return period	1/T	w	z	KT	Y_T	X_T
$\bar{Y}$	1.17	2	0.5	1.17	-1.01E-07	-0.055	1.712	50.68
S_Y	0.13	5	0.2	1.794	8.41E-01	0.82	1.79	67.37
C_S	0.78	10	0.1	2.14	1.28E+00	1.31	1.84	77.84
K	0.13	25	0.04	2.53	1.75E+00	1.86	1.89	91.47
		50	0.02	2.797	2.05E+00	2.22	1.93	102.3
		100	0.01	3.035	2.33E+00	2.567	1.967	113.7

Table 4: Gumball's Distributions parameter

Gumball method Calculation				
		Return period	YT	XT (mm)
$\bar{x}$	52	2	0.37	51.2
δ_x	14	5	1.5	62
		10	2.25	70.3
		25	3.2	80.3
		50	3.9	87.2
		100	4.61	94.6

Table 5: Recommended percentage of impervious value.

Land Use Conditions	Percent Impervious Area
Institutional	34
Industrial	76
Low density residential	19
Medium density residential	38
Open Urban Land	11
Paved surface with gravel joint	70
Roofs	90
Commercial	56
Concrete and asphalt surface	80
Gravel path with undeveloped part of soil	20
Gravel road sharply slope mountain Road	40
High density residential	51

Source: NRSC, 2010

Table 6: Time of concentration, length of flow and slope of different watershed.

No.	sub catch	Cw	Aver. Slope	longest channel length(m)	Timeof concen.(t1+t2+t3)minut	Alloweable (tc)min	velocity of channel
1	BuS station	0.74	0.0934	540	14	15	3.4
2	Jalo	0.68	0.06578	590	26	26	2.23
3	Jerusalem	0.71	0.06453	768	24	24	4
4	Gombora	0.77	0.07653	602	21	21	3.4
5	Disphora	0.78	0.0778	340	15	15	2.65
6	Arada	0.62	0.056	280	22	22	3.87
7	S25	0.69	0.0932	388	30	30	2.6
8	Bole	0.69	0.0781	483	27	27	2.478
9	Hospital	0.89	0.087	687	14	15	2.2
10	GODgod	0.61	0.0634	408	27	27	3.67
11	WR	0.72	0.0898	467	20	20	3.31
12	Naramo	0.71	0.0769	642	16	16	2.1
13	TTC	0.42	0.067	420	13	15	1.1
14	Mikael	0.42	0.056	478	16	16	1.9
15	Wanza	0.6	0.078	360	14.3	15	1.3
16	Mulwongel	0.6	0.05	330	12	15	1.47
17	Bobicho	0.65	0.076	450	14	15	1.84
18	Wachamo	0.39	0.045	489	13	15	2.1

Note: NC:-Naramo Condominium, TTC=Teachers training center, WR=water reservoir

WC=weighted coefficient

Table 7: Computations of area, runoff Coeff and avg. imperviousness

catch ID	LU type	Area (m^2)	R.coff	impe%	Ave.cw	Ave.impe	Area(ha)
Bus station					0.75	60	7.8
polygon	Residential	74582	0.65	50			
polygon	Residential	1243.1	0.65	50			
polygon	Commercial	2174	0.7	68			
Jalo.					0.735	62.2	13.8
polygon	Residential	13061.7	0.6	50			
polygon	Residential	12387.2	0.63	50			
polygon	coble road	13939.3	0.8	74			
polygon	coble road	17751.12	0.55	62			
polygon	Coble road	9090.84	0.8	52			
polygon	Coble road	10098.7	0.7	70			
polygon	Residential	26671.6	0.7	70			
polygon	Residential	34999	0.67	70			
Gombora					0.57	54.2	11
polygon	commercial	7834.21	0.55	60			
polygon	Residential	40904	0.5	51			
polygon	Residential	34261	0.6	51			
polygon	Residential	27000	0.6	51			
Jerusalem					0.55	43	12.4
polygon	KG school	34245.2	0.65	55			
polygon	Residential	29356.23	0.5	35			
polygon	coble road	12134.6	0.7	60			
polygon	Gavel road	23456	0.45	35			
polygon	play ground	24807.97	0.35	30			
polygon	Residential	6000	0.5	45			
Disphora					0.56	44	17.6
polygon	Residential	5456.56	0.6	35			
polygon	mixed res	4375.45	0.65	40			
polygon	coble road	56476	0.7	60			
polygon	play ground	37695	0.35	30			
polygon	school	71997	0.5	55			

Note: R.Cof= Runoff coefficient, Ave.cw =Average weighted runoff coefficient, %Imp= percentage of imperiousness.

Table 8: Existing canal capacity Vs Estimated sub-catchment discharge

No.	Sub-watershed code	Estimated (peak discharge(m^3/s) T-10year	Existing canal capacity manning's(m^3/s)
1	BuS station	1.71	0.35
2	Jalo	1.82	1.2
3	Jerusalem	2.11	1.28
4	Gombora	2.05	1.86
5	Bole	2.03	2.68
6	Arada	2.4	2.7
7	Disphora	2.1	3
8	S25	2.5	2.01
9	Hospital	2.6	1.2
10	Godgadeo	2.72	0.8
11	Water.Resevior	2.62	2.5
12	NC	2.7	1.1
13	TTC	2.64	0.8
14	Mikiel	2.9	1.03
15	Wanza	2.75	1.1
16	Mulu wongel	2.8	3
17	Bobicho	2.63	1.8
18	Wachamo	2.81	3
Total		43.89	31.41

Note: NC=naramo condominium,TTC=techers training center

Table 9: General Runoff coefficients for the rational method

Type of Drainage Area	Runoff coefficient (C)
Business	
Commercial	
Neighborhood areas	
Residential	
Single family areas	0.3-0.5
Multi-units, detached	0.25-0.4
Apartment dwelling areas	0.5-0.7
Industrial	
Light areas	0.5-0.8
Heavy areas	0.6-0.9
Parks, Cemeteries	0.1-0.25
Play ground	0.2-0.4
Streets:	
Asphaltic	0.7-0.95
Concrete	0.8-0.95
Brick	0.7-0.85
Drives and walks	0.75-0.85
Roofs	0.75-0.95

Source: Hydrology, Federal Highway Administration, HEC No. 19, 1984.

Table 10: Roughness Coefficients (Manning's n) for sheet flow**.**

Surface Description	n'
Smooth surface (concert ,asphalt, gravel, or bare soil	0.011
Fallow(no residue)	0.05
Cultivated soils:	
Residue cover≤20%	0.06
Residue cover ≥20	0.17
Grasses:	
Short grass	0.15
Dense grasses	0.24
Light underbrush	0.4
Woods	
Dense underbrush	0.8
Dense grasses	0.24

Source: engineering hydrology, Sabarmaniya, K (2008)

Table 11: GTZ Standards and Check List

Indicators Classification	Surface condition
Very good	Shapes of USWD lines as still in original design condition
Good	No significant depression, undulations and deformation
Light	Shape of the USWD lines deteriorate, but still sheds water.
Severe	Total collapse of the USWD lines structure and barely passable.

Source: GTZ, 2006

Table 12: Meteorological regions of Ethiopia

Meteorological Region	Station	Years of Record	Meteorological Region	Station	Years of Record
A1	Axum	18	B	Bedele	19
	Mekele	35		Gore	45
	Maychew	24		Nekempte	27
A2	Gondar	40		Jima	45
	Debre Tabor	22		Arba Minch	11
	Bahir Dar	35		Sodo	28
	fiche	25		Awasa	26
	Addis Ababa	3310	C	kombolicha	46
A3	Nazareth	40		Woldiya	23
	Kulumasa	31		Sirika	17
	Robe/Bale	19	D1	Gode	29
A4	Metehara	28		Kibre Dihar	38
	DireDawa	46	D2	Kibre Mengist	24
	Mieso	35		Negele	45
				Moyale	29
*max 24 hour rainfall not given				Yabelo	34

Source; (ERA, 2013).

Table 13: Rainfall depth of different region of Ethiopia.

Return Period in years						
	2	5	10	25	50	100
RR-A1	50.3	66.02	76.28	89.13	98.63	108.06
RR-A2	51.92	65.52	74.45	85.7	94.07	102.45
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37
RR-B2	55.26	69.95	79.68	92.03	101.29	120.07
RR-C	56.52	71.04	80.54	92.52	101.48	110.5
RR-D	56.23	76.84	90.37	107.46	120.23	133.05

(source: ERA,2013)

Table 14: Hydrological Soil group

Group	Descriptions	Rate of transmission (mm/hr)
A	Sand, loamy sand or sandy loam. Soil has low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels.	>7.62
B	Silt loam, or loam. Soil has a moderately low runoff potential due to moderate infiltration rates. low bulk density, or contain greater than 35 percent rock fragments	3.81-7.62
C	Sandy clay loam. Soils have a moderately high runoff potential due to slow infiltration rates. Soils typically have between 20 percent and 40 % clay and less than 50 sand and have loam, silt loam, sandy clay loam, clay loam, and silt clay loam textures.	1.27-3.81
D	Clay loam, silt clay loam, sandy clay, silt clay or clay. Soils have a high runoff potential due to very slow infiltration rates. These soils have a very low rate of water transmission.	(0-1.27).

(Source: Natural soil Conservation, 2007).

Table 15: Road drainage system design period

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

(Source ERA, 2013)

Table 16: Flooded nodes SWMM Hydraulic analysis

Flooding node	Hours flooded	Max Rate flood (CMS)	Time of max flooding Occurrence day, hr: min	Total Flood volume 10^ 6ltr
J1	3.2	2.17	0 ;00:45	15.47
J3	3.34	2.61	0;01:15	19.488
J6	4.2	1.78	0;02:30	16.569
J7	3.2	4.108	0;02:30	29.876
J8	5	7.14	0;01:30	58.963
J9	5.5	8.78	0;01:15	87
J10	5.51	2.78	0;02:30	21.8
J12	3.48	5.15	0;01:30	38.42
J13	3.98	4.31	0;02:00	36.9

(Source: SWMM 5.1)

Table 17 : Junctions of Gofer-meda sub-city watershed

Geographic locations of Junctions			
ID	easting	Northing	elevation(m)
J15	379052	834821	2319.2
J1	373401	834929	2318
J2	373432	834929	2317.5
J13	373545	835532	2317.4
J11	372756	824893	2318.5
J16	372352	834820	2316.09
J17	372567	834701	2320
J19	372769	834886	2319
J5	373004	835058	2318.5
J6	373375	835160	2316.09
J7	373070	835160	2316
J8	372650	835948	2312
J9	372180	835535	2309
Out2	372060	835321	2316
J10	373367	835859	2317.9
J12	372656	835253	2316
J18	373126	835621	2318.05
J3	373604	835666	2317.7
J20	372824	835804	2316.7
J23	372824	835983	2318.1
J21	373716	835839	2317.9
OUT1	374494	835659	2308.6

Appendix Figures

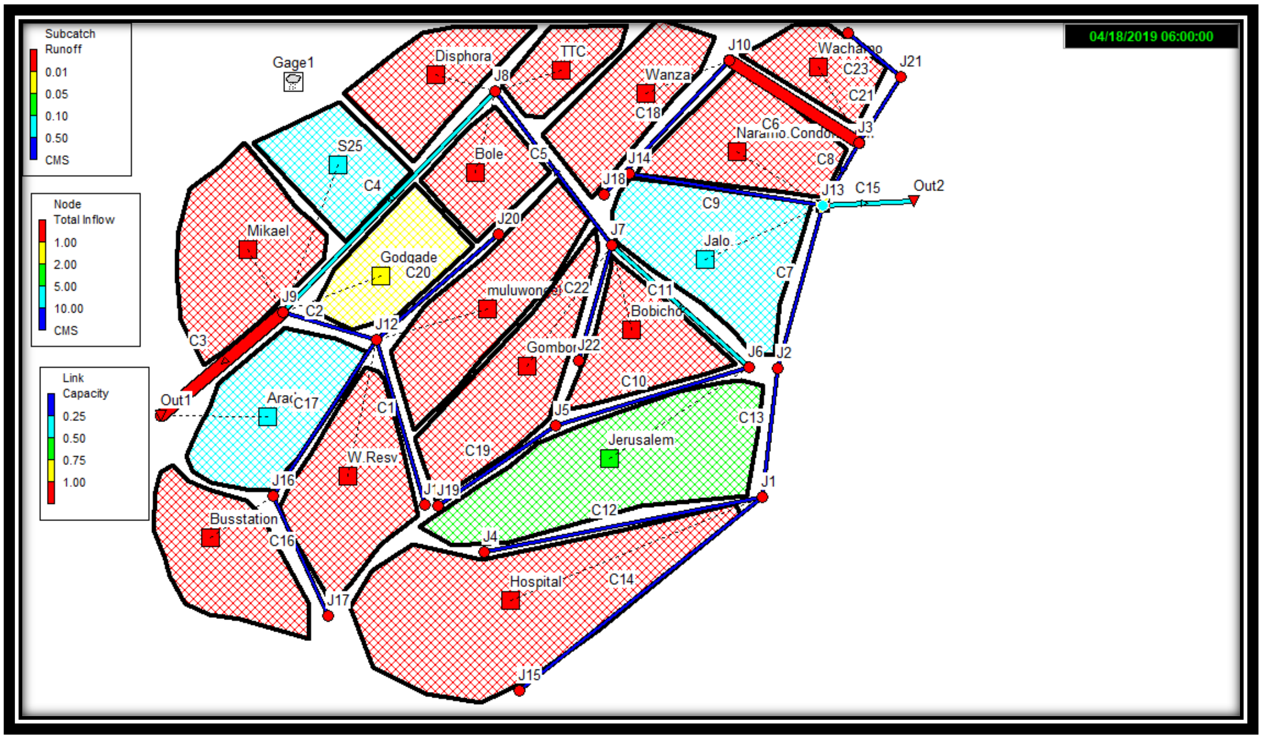

Figure1a: Runoff contributing areas

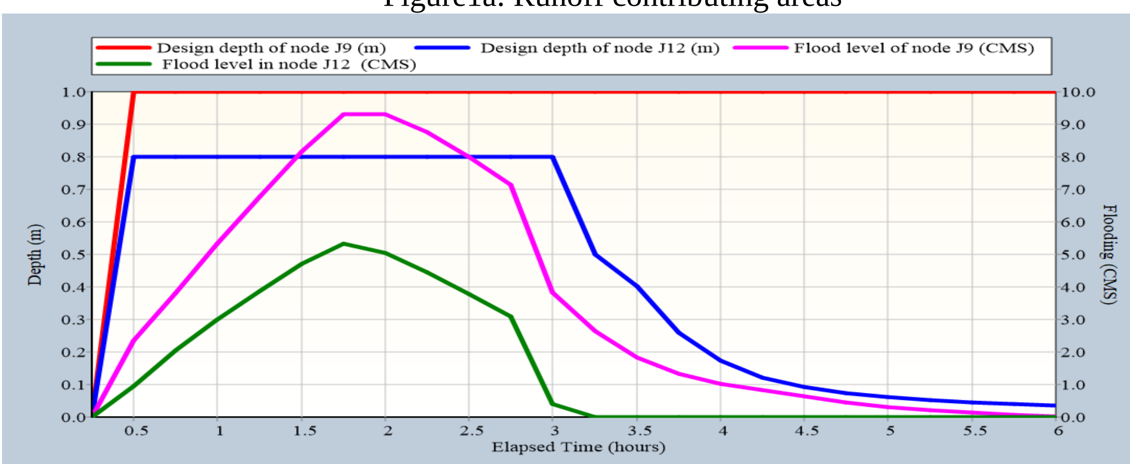

Figure 2: Design depth and food level in node J9 and J12

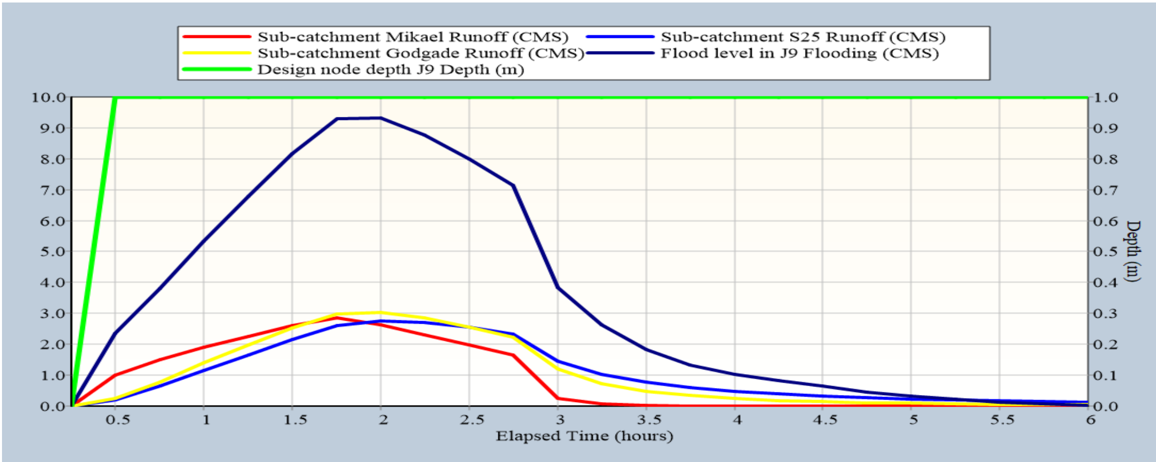

Figure 3: Sub-catchment runoff and Flood level nod J9

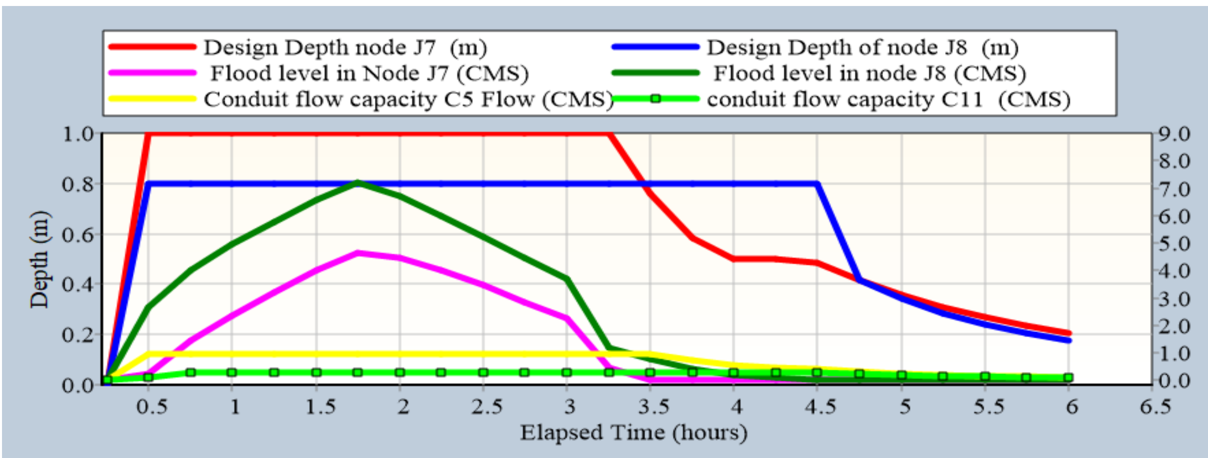

Figure 4: Water level and flood level in junction (J8 and J7)

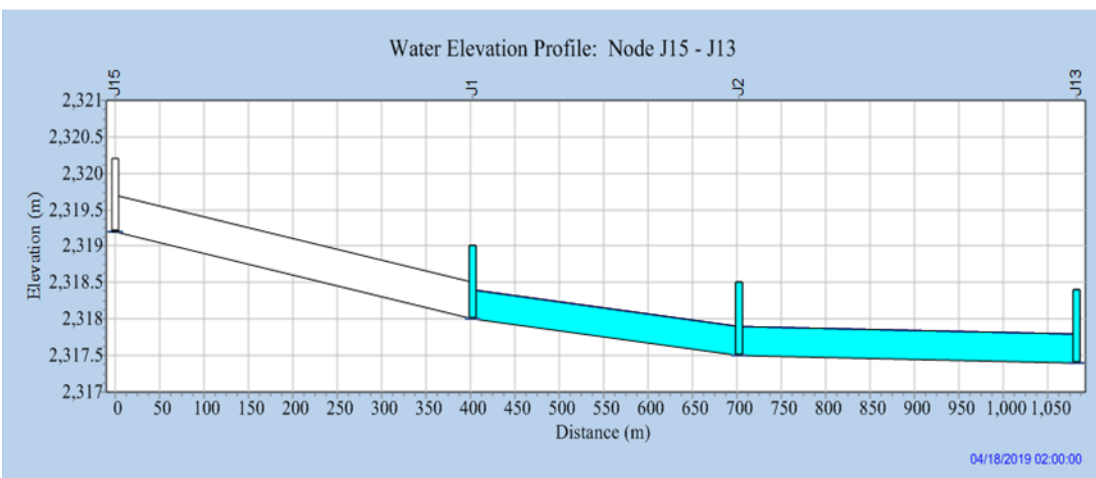

Figure 5: Storm water elevation profile.

a.

Figure 6: Field survey and problem associated drainage System