ADDIS ABABA UNIVERSITY

ADDIS ABABA INSTITUTE OF TECHNOLOGY

AFRICAN RAILWAY CENTER OF EXCELLENCE

Flood Analysis and Hydraulic Competence of Drainage Structures along Addis Ababa Light Rail Transit

(A Case Study on Mesualekia-Nefas Silk2 Stretch)

A Thesis in Railway Engineering (Civil Infrastructure)

By Kiwanuka Moses

July, 2019

Addis Ababa

A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science in Railway Engineering (Civil Infrastructure)

The undersigned have examined the thesis entitled '**Flood Analysis and Hydraulic Competence of Drainage Structures along Addis Ababa Light Rail Transit**' presented by **Kiwanuka Moses**, a candidate for the degree of **Master of Science in Railway Engineering (Civil Infrastructure)** and hereby certify that it is worthy of acceptance.

Dr. Yilma Seleshi	__________________	____________
Advisor	Signature	Date
Dr. Admasu Gebeyaw	__________________	____________
Internal Examiner	Signature	Date
Dr. Belete Birhanu	__________________	____________
External Examiner	Signature	Date
Mr. Zewdie Moges	__________________	____________
Chair person	Signature	Date

UNDERTAKING

I certify that research work titled "**Flood Analysis and Hydraulic Competence of Drainage Structures along Addis Ababa Light Rail Transit**" is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

……………………………………………

Kiwanuka Moses

ABSTRACT

This study is on flood analysis and hydraulic competence of the existing drainage structures on some roads of Addis Ababa City after an integration of Roads and Addis Ababa Light Rail Transit Drainage Systems. Flooding in any circumstance causes major stresses on the economic, social and environmental regimes of the affected area. Addis Ababa has a pronounced rainfall peak which is common during the summer time months of July to August. Flooding is a major problem for Addis Ababa city roads. Therefore, this study was carried out in-order to assess the flood analysis and hydraulic competence of drainage structures along Mesualekia - Nefas silk2 stretch for the different rainfall intensities corresponding to different return periods. The existing side drains and cross drainage structure located within the study area were inspected and assessed in order to ascertain different aspects relating to their performance. The hydrological analysis was assessed in-order to determine the different watershed contributing to the flows. This was done by use of Geo spatial tools like ARCGIS, ARCSWAT, and HEC GEOHMS. The flows were obtained by using the rational method for areas less than 0.5km^2 and HEC HMS for areas greater than 0.5km^2. Rational method was used to obtain different design flows for watershed 1 to 4, including flow from the carriageway runoff and the following were obtained as 2.66, 0.08, 3.23, 6.20, 2.12, 0.08, 0.08, 0.08 m^3/s corresponding to a 10-year ARI. HEC HMS was used to obtain a design flow of 29.52 m^3/s corresponding to watershed 5. A design flow of 46.3m^3/s was used for assessing the cross culvert which was the summation of all the flows from side drains and carriageway runoff. The hydraulic analysis was carried out using HY-8 for the culvert and Hydraulic tool box for existing side drains. From field survey and measurements, it was discovered that the existing side drain of 900mm circular pipe couldn't convey a flow of 6.2 m^3/s. The existing box culvert comprised of one barrel of 4000mm x 3300mm, which was sufficient in conveying the designed flow because its headwater elevation was lower than that of the roadway elevation. However, observations like presence of accumulated silt, debris within most side drains and an undersized side drain reduced its hydraulic competence. Based on the findings, recommendations were made including regular desilting, screens at inlets of side drains and at some distance before the entrance of the culverts, periodic maintenance of the existing drainage structures and widening of the pipe conduits for the side drains to about 1200mm for easy maintenance. There is also need to monitor performance of existing side drains and culverts, that, basing on field data assessed, might have resulted in under- or overdesigns. Regarding the on-track drainage network, this research provided for aggregate drains of size 400mm and the existing longitudinal drains were efficient to convey the flow.

Keywords: Lancha**,** Circular side drains, Box culverts, headwater and roadway elevations, Hydraulic tool box software, HY-8 software, Design flow, Aggregate drains

ACKNOWLEDGMENTS

Firstly, I thank the Almighty GOD for his unspeakable gift of life, help and His protection.

Secondly, I express my sincere gratitude and appreciation to Dr. Yilma Seleshi for his guidance and support throughout the entire preparation of this thesis.

I acknowledge the World Bank through the African Railway Center of Excellence for my scholarship, without which this study would not have been possible.

Finally, I am deeply grateful to my classmates, my family and those who supported me throughout this master's program.

Thank you.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ACRONYMS

AACRA - Addis Ababa City Road Authority

AALRT - Addis A baba Light Rail Transit

ARI- Average Recurrence Interval

CN - Curve Number

DEM - Digital Elevation Model service

GIS - Geographic Information System

HEC HMS - Hydrologic Engineering Center Hydrological Modeling Software

IDF - Intensity-Duration-Frequencys

masl - meters above the sea level

SCS - Soil Conservation Service

USGS- United States Geographical Society

PDF - Probability Density Function

CDF- Cumulative Distribution Function

MoU- Memorandum of Understanding

MoT- Ministry of Transport

NMA – National Meteorological Agency

CHAPTER 1 INTRODUCTION

This chapter will describe the historical events of the study which are the background, study area, objectives, scope of the study and many others.

1.1 Background

Urban drainage systems play a vital city infrastructure role which is to collect and convey storm water and wastewater away from urban areas. Many factors contribute to flooding in urban catchment areas like Hydrological factors which cause rapid flood runoff and flood discharge grows with an increase in impervious areas, and concentrated population and assets are also important social aspects of flooding. Climate change is considered an important factor that increases flooding that's to say with an increase in the frequency and their intensity of torrential storms (IPCC, 2007, Patrick W and Jonas, 2007). Floods vary considerably in size and duration. In order to reduce on the problem of flooding, national and local governments have been implementing structural measures, constructing flood control reservoirs and infiltration and storage facilities with the funds they have available for flood prevention.

In general, there is a need to study the flood analysis and hydraulic competence of drainage structures along the AALRT in order to determine whether they are able to convey the storm water for the integrated road drainage system with that of the AALRT without flooding.

1.2 Statement of the problem

Like any other country, the Federal Republic of Ethiopia is prone to natural disasters which include floods, drought and many others. This could be due to various factors like torrential rains especially in a rainy season with a high runoff volume within Addis Ababa causing flooding in several streams originating from the nearby mountainous ranges.

The reduced capacity of the drainage structures is due to the continuous siltation, heavy storms, inadequate storm drainage systems, and the concentration of population and assets thus degrading the urban environment when flooding occur. Climate change is also a real threat, bringing heavier and more frequent storms. Due to increased urbanization, there is an increased run off and less infiltration of rain water because of the impervious surfaces like paved surfaces (asphalt, concrete), roads, parking lots and buildings which leads to an increased intake for the drainage structures. The increased run-off currently cannot be easily conveyed by the incapacitated drainage systems thus leading to continuous flooding which leads to destruction of property, infrastructure, lives and acts as a barrier to economic development. These floods normally destroy properties ranging from houses to personal belongings and in extreme cases, the loss of lives. Increase in population and hence urban development has led to changes in land use and land cover. This is because people are changing floodplains to industrial and residential use. Removing vegetation and soil, grading the land surface, and constructing of infrastructure and drainage networks increase runoff to streams from rainfall. As a result of the above, the peak discharge, volume, and frequency of floods increase in nearby streams. Changes to stream channels during urban development limit the capacity of these channels to convey floodwaters (Frimpong, 2009).

In addition to the above, the implementation of the storm water drainage structures isn't sufficient. The newly constructed Addis Ababa Light Rail transit lacks its own drainage system, the line uses a system for the existing highway which is integrated to work as the drainage system for the Light rail.

Therefore, due to that, the existing drainage system may not be able to accommodate the entire runoff which calls for a study in-order to ascertain the flood analysis and hydraulic competence of the existing drainage system.

1.3 Objectives

1.3.1 General objective

The main aim of the study is to carry out a flood analysis and hydraulic competence of existing drainage structures along the AALRT due to the integration of the highway drainage system with that of the newly constructed AALRT. It further aims at determining drainage structures that are prone to flooding.

1.3.2 Specific Objectives

- To carry out hydro statistical analysis in order to obtain different flood volumes for different return periods.

- Determine a hydrological model for the catchment and identify the most prone points facing floods along the line.
- To assess the hydraulic capacity of the existing drainage structures in order to determine their response to the flood storms and suggest different mitigation measures for curbing down flooding.

1.4 Research Questions

Which rainfall intensities received within the catchments correspond to the different return periods of 5, 10, 15, 25, 50, and 100 years?

Which particular points along the railway line does the natural drainage of the catchment crosses the line?

Which points are most prone to flooding?

Which appropriate methods are used for assessing the hydraulic capacity of a given drainage structure?

What are the mitigation measures for flood control?

1.5 Significance of Study

Management of urban drainage structures plays a significant role for viable environment by keeping the service life of the urban infrastructures such as buildings, roads, telephone lines and many others.

The successful implementation of the project will;

- Enable the determination of hot spot areas of flooding along the Mesualekia - Nefas silk2 stretch of AALRT in order to ease maintenance works on those particular sections.
- Enable the implementation of the appropriate mitigation methods to curb down flooding in case they occur.

1.6 Scope of the Thesis

This thesis was limited to the analysis of the actual response of the different drainage structures to the different flood storms within the study area. It also suggested appropriate mitigation measures to curb down flooding. The study area stretches from Mesualekia-Nefas silk2 stations. This research didn't consider the details of the effect of flooding on the entire track structure.

1.7 Justification

This thesis is to address the different continuous problems affecting the railway drainage structures which are integrated with the highway drainage. This will offer a better ground for decision making among the different stakeholders of this country like the AACRA, ERC, ERA and AALRT on sustainable and easily achievable management solutions regarding storm water. The problem was addressed through two techniques, the descriptive and explanatory. Through the descriptive technique, the research addressed the extent of flood occurrence within this particular community while with the explanatory approach, the project addressed the causes of flooding, relationship of flooding with the existing drainage systems and identified the need and propose mitigation measures to curb down floods.

CHAPTER 2 LITERATURE REVIEW

This focuses on reviewing existing literature on the use of drainage analysis soft wares like EPA- SWMM, HY8, MOUSE and many others in the determination of performance evaluation and assessing the flood analysis of drainage structures and particularly that of the selected AALRT stretch.

2.1 An overview on flooding

Floods are one of the most recurring disasters which affect human lives and cause severe economic damage throughout the entire world at large. They can be dated as far back in history from the time of Noah when they covered the entire earth was covered and only the family of Noah with a few animals survived.

The floods are commonly caused by heavy rains and overflowing of rivers and lakes during rainy seasons.

Table 2-1: Factors contributing to floods (WHO, 2008)

Meteorological factors	Hydrological factors	Human factors aggravating natural flood hazards
Rainfall	Soil moisture level	Land use changes e.g surface sealing due to urbanization, deforestation
Cyclonic storms	Groundwater level prior to storm	Occupation of the flood plains obstructing the flows
Small scale storms	Natural surface infiltration rate	Inefficiency or non-maintenance of infrastructure
Temperature	Presence of impervious cover	Too efficient drainages upstream areas increase the flood peaks

Meteorological factors	Hydrological factors	Human factors aggravating natural flood hazards
Snowfall and snowmelt	Channel cross-sectional shape and roughness	Climate change affects magnitude and frequency of precipitations and floods
	Presence or absence of over blank flow, channel network	Urban microclimate may enforce precipitation
	Synchronization of runoffs from various parts of watershed	
	High tide impending drainage	

From the above table, the most important factors contributing to floods are the changes in the urban planning and climate change. This is because they are today's most influential factors determining the urban flooding (Semadeni-Davies, A., Hernebring, C., Svensson, G. & Gustafsson, L.G. 2008). Flooding is generally a temporary condition of partial or complete inundation of normally dry areas from overflow of inland or tidal waters or from unusual and rapid accumulation or runoff (Jeb and Aggarwal, 2008).

About 15% to 20% of the total rainfall ends up as surface runoff in rivers, the remaining part of the rainfall water infiltrates into the ground or returns back to the atmosphere by means of evaporation and transpiration from plants (Plummer and McGeary, 1993). The amount of rainfall runoff runs between 2% - 25% with variations in climate, elevation/slope, soil and rock type, infrastructure and vegetation.

Having some fairly detailed knowledge and understanding of flood analysis relevant to different communities is good for an appropriate implementation of flood prevention or reduction measures like emergency systems, development planning and timely or early warning measures.

2.1.1 Flooding in Global Perspective

According to the Belgium-based Centre for Research on Epidemiology of Disasters (CRED), the world's most disastrous floods to have occurred in terms of the number of people who lost their lives happened in the year 2004 in Haiti (CRED, 2011). It also indicated that for a fortnight, there were continuous and heavy rains which caused rivers to swell and subsequently overflowing of river banks mostly in the southeastern parts along the areas that share borders with the Dominican Republic. The continuous precipitation for that fortnight generated floods that killed over 2,400 people, the Guardian Newspaper reported (CRED, 2011). The monsoon rains generated the floods that occurred in India in 2005 (CRED, 2011). These caused approximately 1,000 deaths, placing this disaster in second place after the floods that hit Pakistan. The monsoon rains in the region normally carries on into September and the help/aid workers feared the number of lives lost to flood occurrence could increase annually. Notably, one-third of Pakistan (an area close to the size of England) is under water (CRED, 2011).

Reports indicate that 7 out of the 11 worst floods on CRED's list for the decade between 2000 and 2009 occurred in India (CRED, 2011). Guha-Sapir asserts that countries like India, Pakistan and Bangladesh appeared top of the list of numbers of lives affected by floods due to the high concentration of relatively poor rural people living along and within some distance from the river banks. Between 1996 and 2005, floods have had serious devastating effects on the continent of Africa. In this seven-year period, approximately 290 flood-disasters have been recorded across the continent of Africa. A greater number of people were reported dead approximately 8,183; 23 million people affected and there were reported economic losses worth approximately $1.9 billion (Satterthwaite et al., 2007).

Several media and aid organizations have widely reported a lot of flooding incidences in Sub-Saharan Africa. These floods are mostly flash floods which resulted from several and continuous days of rainfall. Mozambique is one of the most affected countries. It’s consistently affected by flooding almost annually and in the year 2000, the recorded losses totaled to millions of dollars. Approximately 800 lives were lost and consequently there was the need for setting up a community-based early warning system. These community-based early warning systems have dramatically reduced the number of casualties as well as fatalities in Mozambique‘s yearly flood season (Wisner, 1979). In 2013, the Ghana News Agency (GNA) reported the flooding incidence in Central Nigeria‘s Plateau State which resulted in the loss of at least 39 human lives. Floods are an inconvenience and at times a danger to the residents. This is more dangerous if it's a combination of both floods occurring at the same time leads to the largest inundations that are destructive (van Overeem & Steenbergen, 2015).

2.1.2 Flooding in Ethiopia

Addis Ababa's undulating topography and haphazard urban expansion has created significant flooding in the city. The most flood-prone areas in Addis Ababa include the middle reaches of the Little Akaki and Bantiyketu rivers and the lower reaches of Kechene and Kurtume rivers. Floods in Addis Ababa are exacerbated by inadequate urban drainage systems. The sewer lines collect runoff water and discharge it into nearby streams and rivers. The urban stormwater drainage network, however, has limited capacity due to the obstruction of pipes and street inlets with debris and sands (Wondimu and Alfakih, 1998), and most of the roads do not have drainage systems. The major causes of flooding in Addis Ababa were found to be the blockage of urban stormwater drainage lines and poor integration between road and urban drainage infrastructure, which continues to lag behind the rapid urban population growth (Belete, 2011).

On 7 August 2014, the different sub cities within the city of Addis Ababa flooded. Within Bole sub city woreda 13, different property was damaged like Sunrise building property whereby about 700 thousand birrs was damaged because of the flood (Ethiopian Opinion, 2014). The case study area is found within Nefas silk-Lafto areas whereby the main causes of flooding is as a result of blocked drainage structures because of accumulated debris, silt and rubbish.

2.2 Effects of urbanization in flooding events

The evolution of the land use is basically much more related with urban development and the increment of floods derived from it. Within the undeveloped areas, the water from precipitation infiltrate into the soil filling up the holes between particles up to saturation. After reaching the saturation point, runoff occurs on the surface. However, within urbanized areas the paved and other impervious surfaces hinder the capacity of the soil to absorb water. Consequently, as a result of urbanization the velocity of the runoff is increased leading to peak discharges and greater amount of water in the surface (EPA, 2003). This can be seen in the figure where the influence of urbanization causes runoff generation. As can be seen, the water cycle balance is modified since the groundwater table level decreases and the runoff is raised instead.

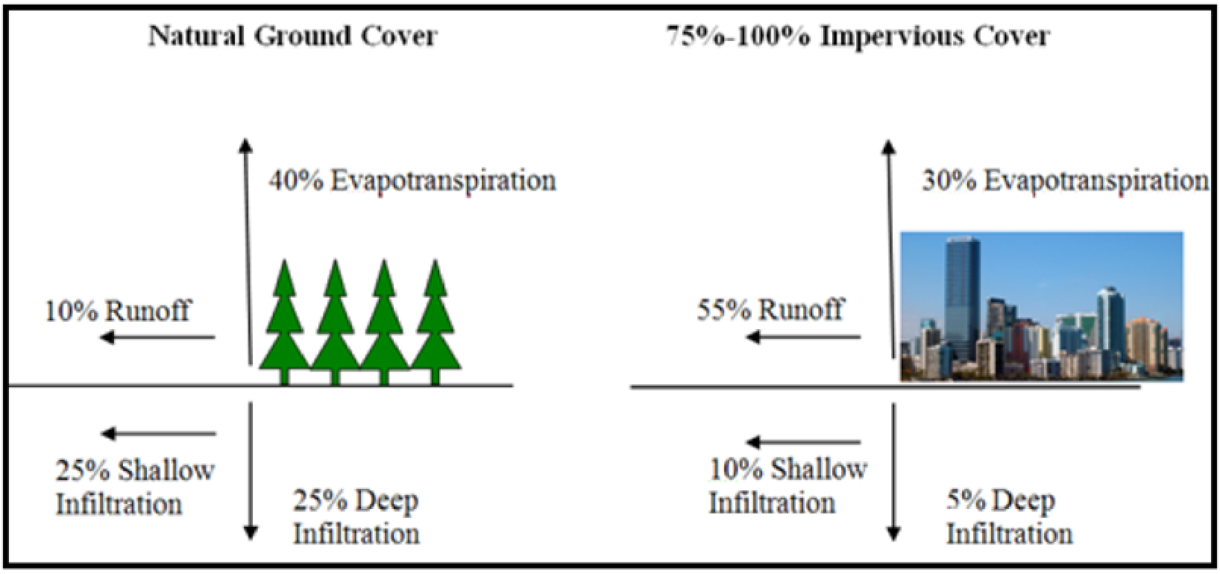


Figure 2.1: Influence of urbanization in runoff generation (modified from EPA, 2003)

2.3 Urban drainage components

Urban drainage refers to the process of transporting waste and storm water outside the urban area and is directly related to urban flooding. Urban drainage systems are normally used in developed urban areas because of the interaction between human activity and the natural water circulation. 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.

According to AMK Associates (2004), the urban storm water drainage system consists of two distinct components. These are surface and subsurface storm sewer network. The surface is also known as the "major" system which is composed of street ditches and numerous channels which are designed to handle events of 25-100-year return period. The subsurface sewer network is also known as the “minor” component, which is designed to carry the runoff from a storm of 2-10 years return period.

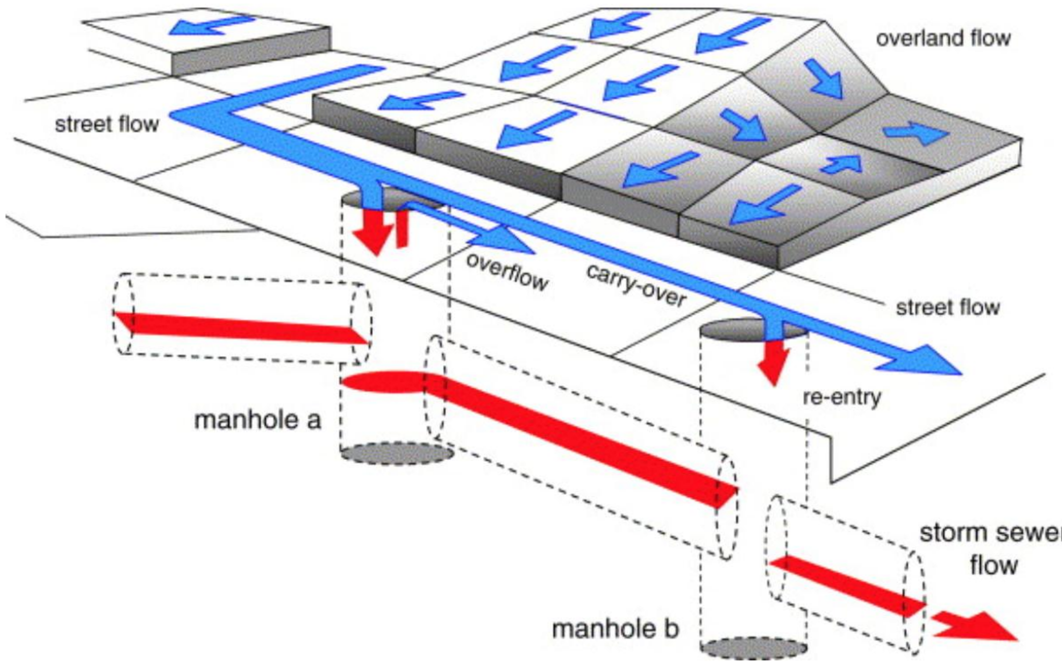


Figure 2.2: Idealized surface (blue) and storm sewer (red) flow components in dual drainage systems (M. Smith, 1993)

These systems can either be artificial, or combination of manmade sewer facilities and natural watercourses. The drainage system is represented as a network which consists of catchments and sub catchments, nodes, links and outlets. The different components are briefly explained below (Linmei Nie, 2003).

Catchment; this refers to the area collecting water from nearby higher terrain surface, which is delineated by topographic contour lines. It’s normally described by its parameters like catchment area, percentage composition of the impervious surface, average slope, the longest flow length and approximate shape.

Nodes; these are junctions which link the sewers. They also provide storm water transition between surface and subsurface systems. Manholes are typical examples of nodes which are provided at every intersection of storm water drainage structure. They are normally at points where a storm water drains changes direction/gradient, on long straight lengths, and at different sizes of storm water drains.

Links; these normally transport flow in the system. They are made up of open channels or closed sewers with regular or irregular cross sections.

Outlet; this refers to the most downstream component of a given urban drainage system. Its main task is to discharge the sewage from the system to receiving waters.

2.3.1 Drainage Inlets

The hydraulic capacity of any given storm drain inlet depends on its geometric properties and the characteristics of the gutter flow. Storm drain inlets are used to collect runoff and discharge it to an underground storm drainage system. These are briefly described as shown below;

Kerb-opening inlets; Normally, these inlets are vertical openings in the curb covered by a top slab and are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. They are less affected by blockage.

Grate inlets; these consist of an opening in the gutter or ditch which is covered by a grate. They are effective in intercepting gutter flows, and they also provide an access opening for maintenance. However, in some scenarios they are prone to blockage.

Combination inlets; the most efficient drainage inlet is the combined grate and kerb. It's commonly used on urban roads wherever possible.

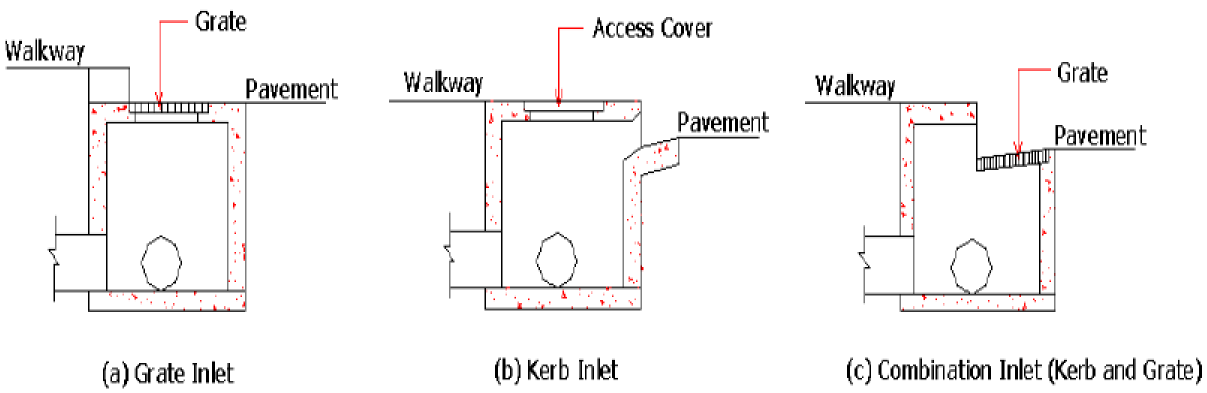


Figure 2.3: Drainage inlets

(Source: Urban Stormwater Management Manual)

2.4 Types of Track Drainage

An appropriate drainage system must be installed in a newly built or upgraded track structure. In-case of inappropriate track drainage system, it will lead to weakening of the track structure hence subsequent failure. Drainage planning is required in-order to come up with a safe, sustainable and resilient drainage systems to enable water to flow through the railway infrastructure from point of entry to point of exist in such a way as to allow earthworks, track and structures / works to perform in an optimal manner. When ditches along the track line are not well maintained, debris settles in them whereby the ditches end up holding water at a higher level hence obstructing drainage from the track section. This retained water weakens the sub-ballast and subgrade. It's sad that often times the track inspectors do not easily notice the drainage problem until the mud appears in the ballast section which means that the water has saturated the subgrade. Then, under normal loading, the subgrade is overloaded and begins to move from its original position. When this occurs, ballast pockets begin to form which has proved to be a world-wide problem on most railway lines.

Track drainage system is made up of two types.

- Surface drainage
- Sub-surface drainage

2.4.1 Surface Drainage

This drainage system conveys surface runoff before it ends up into the track structure, as well as collecting water percolating out of the track structure. It begins with the basic grading of the ground on either side of the track, and allows water flowing out of the track structure to be removed. A shoulder grading can be used in very flat areas where it is difficult to get sufficient fall for either surface or subsurface drains. Cross drainage structures like culverts and bridges can be used for disposing off surface water

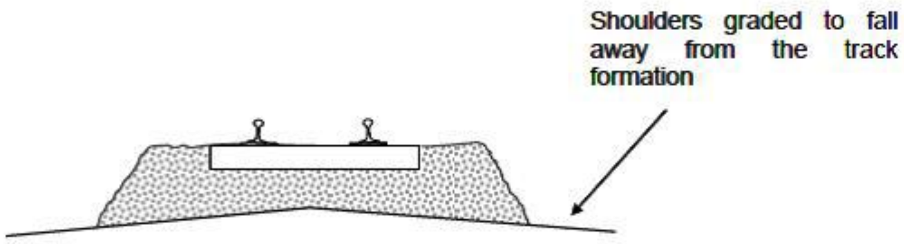


Figure 2.4: Typical Track Formation

2.4.2 Subsurface Drainage

Subsurface drainage is mainly for maintaining the integrity of the track formation and ensuring that earth slopes are more stable. They are mainly used where adequate surface drainage cannot be provided due to some restriction, or lack of available fall due to outlet restrictions. Locations where these circumstances may occur are, track junctions, turnouts, platforms, multiple tracks and cuttings (ARTC, 2006). Subsurface drainage is used for;

- ❖ Drainage of the track structure

- Controlling of ground water
- The draining of slopes

Subsurface drainage systems are provided in area where the water table is on or near earthworks level.

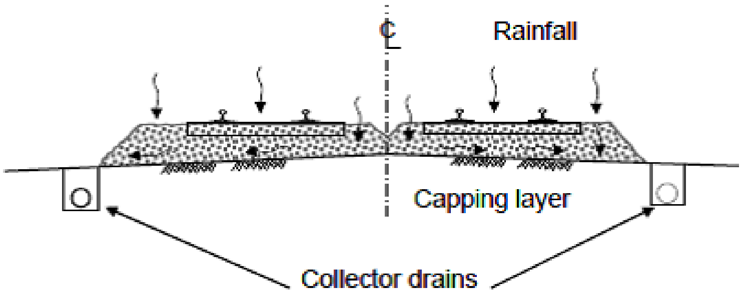


Figure 2.5: Subsurface Drainage

2.5 Flooding on railways

The main purpose of track drainage system is to convey water away from the track support System. The main sources of water that can affect the track support system include; Precipitation on the track, run off from areas adjacent to the track including catchments external to rail boundaries, groundwater from underlying permeable layers, and perched water tables and infiltration through the ballast and ditches.

Figure 2.6: Flooded railway track

Source: (Network rail website)

Proper surface water control and discharge from railway property is critical to maintaining the stability of railway embankments. Without this, track is prone to flooding and becoming structurally unsafe. Any blockage of ditches and drains can allow flooding of track and adjacent land.

2.5.1 Forms of Floods

These flood events were defined based on the hydro-meteorological conditions and the resulting catchment states in them (Yeshewatesfa, 2017).

Short-rain floods; its defined as a flood event caused by rainfall of duration less than or equal to a day. This normally result from intense rainfall within a duration of a few hours.

Long-rain floods; these are caused by rainfall with duration of several days. The intensity could be much lower but with time it gradually saturates the watershed and may ultimately result in flooding.

Snowmelt floods; this normally occurs when there is accumulated snow within the watershed and the temperature rises above a freezing point.

Rain-on-snow floods; the snowmelt process may be enhanced during rainfall due to the additional latent heat the rain provides to the snowpack. Together with the incoming rainfall, the snowmelt can result in considerable runoff.
However, this particular case study area is commonly affected by short rain floods and in some cases long rain floods occur.

2.6 Rainfall-based Flood estimation methods

Flood prediction and modeling generally involve approximate descriptions of the rainfall-runoff transformation processes. Some of the methods used are described below;

2.6.1 Rational method

The rational method dates back to the middle of the 19th Century when an Irish engineer called Mulvaney in 1850 first wrote about its principles. The method is based on the assumption that a constant intensity of rainfall is spread over an area, and the effective rainfall is falling on the most remote part of the basin takes a certain period of time which is known as the time of concentration (T_c), to arrive at the basin outlet. If the input rate of excess rainfall on the basin continues for the period of time of concentration, then the part of excess rain that fell on the most remote part of the basin will just begin its outflow at the basin outlet and with it, the runoff will reach its ultimate and the maximum rate. The

Rational Method is used for flood estimation and can also be used for estimating storm water runoff peak flows for the design of gutter flows, drainage inlets, storm drain pipe, culverts and small ditches. It's most applicable to small (normally less than 0.5km^2 area as recommended by Ethiopian Road Authority drainage modeling manual) and highly impervious areas. The main considerations of the Rational Method are as follows; (GMDID, 2010)

- The peak runoff rate, Q_p - this refers to the function of the average rainfall rate which flows after the period, T_c (time of concentration).
- Rainfall intensity, i - assumes that the rate of rainfall is constant during the time of concentration T_c, and that all the measured rainfall over the area contributes to the flow.
- The runoff coefficient, C - is the proportion of rainfall that contributes to runoff from the surface. The coefficient accounts for the initial runoff losses (depression, storage) continuing losses (surface infiltration) and implicitly accounts for the hydrodynamic effects encountered as the water flows over the catchment surface. Runoff coefficients are theoretically restricted to the range of 0 to 1.0 though it's hard to have a coefficient of exactly 0 and 1. Each part of the watershed can be considered as either pervious or impervious. The pervious part is the area where water can readily infiltrate into the ground. The impervious part is the area that does not readily allow water to infiltrate into the ground, such as areas that are paved or covered with buildings and sidewalks or compacted unvegetated soils (Rugumayo, 2010).
- Catchment Area, A - the boundaries of the complete catchment to be drained can be got by field survey or use of contour maps. The entire catchment is divided into sub catchment areas draining towards each pipe or group of pipes in the system. The sub areas can be measured using a planimeter if using paper maps or automatically if using GIS based package.
- Time of Concentration (T_c) – this refers to the time required for the surface runoff to flow from the remotest part of the catchment area to the point under consideration. This means every point in the catchment has its own time of concentration. The time of concentration has two components and these are the overland flow time known as the time of entry, t_e, and the channel or sewer flow time, the time of flow t_f (Albert, 2010).

Therefore; $T_c = t_e + t_f$

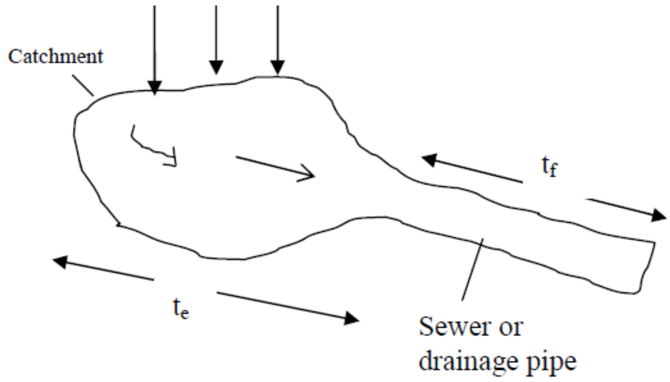


Figure 2.7: Time of concentration in a catchment

The time of concentration can be got using the most widely used formula given by Kirpich (1940). However, for small drainage basins, the lag time for the peak flow can be taken to be equal to the time of concentration.

$$\mathbf{Tc = 0.01947 * L^{0.77} * S^{-0.385}} \ldots\ldots \qquad \textbf{2-1}$$

Where;

T_c = time of concentration (min)

L = maximum length of travel of water (m)

S = slope of the drainage basin = H/L

H = difference in elevation between the most remote place in the basin and the outlet (m)

The probabilistic Rational Method has been developed with the runoff coefficient for different return periods. A return period (recurrence interval) is an estimate of the likelihood of an event such as flood to occur. It is a statistical measurement typically based on historic data denoting the average recurrence interval over an extended period of time, and is usually used for flood analysis. The rational formula is given by the formula below;

$$\mathbf{Q_p(m^3s^{-1}) = 0.278C\,I\,A} \ldots\ldots\ldots\ldots \qquad \textbf{2-2}$$

Another method used for calculating the time of concentration is TR55 is as described below;

Sheet Flow Time; this refers to flow over plan surfaces. It usually occurs in the headwater of the streams (usually for the first 100-130m run). With sheet flow, the friction value (Manning's roughness coefficient, n) which takes into account the effect of raindrop impact, drag over the plan and other ground cover barriers has a significant impact on the overall sheet flow travel time determination. Manning's kinematic solution (Overton and Meadows 1976) is used to compute sheet flow travel time.

$$\mathbf{T_C = \frac{5.476}{P_2^{0.5}}\left[\frac{nL}{\sqrt{S}}\right]^{0.8}} \text{..............................} \mathbf{2\text{-}3}$$

Where;

T_t = travel time (hr), n = Manning's roughness coefficient, L = flow length (m), P_2 = 2-year, 24-hour rainfall (mm) and S = slope of hydraulic grade line (land slope), m/m.

Over Land Flow Time -This refers to the time required for runoff to flow over the surface from the end of sheet flow to the nearest channel inlet. This is primarily a function of the length of overland flow, slope of the drainage basin, and surface cover. The overland flow velocity for the overland flow distance is estimated by plugging equation below developed for pervious surface flow velocity estimation purpose.

$$\mathbf{V = 4.91778(S)^{0.5}} \text{.........................} \quad \mathbf{2\text{-}4}$$

Then, the velocity is divided to the flow distance using equation below to determine the total overland flow time.

$$\mathbf{T_{travel} = \frac{D}{60xV}} \text{..........................} \qquad \mathbf{2\text{-}5}$$

Where; T_{travel} - time of concentration (minutes), D - Overland flow Distance (m), V - approximate flow velocity over the surface (m/s) based on catchment Characteristics (land use of the area).

Channel Flow Time; this refers to the time required for the runoff to flow from the channel inlet to the outlet. The time of concentration is calculated by the Kirpich formula as stated below;

$$\mathbf{T_{travel} = 0.02L^{0.77} * S^{-0.385}} \text{...............} \quad \mathbf{2\text{-}6}$$

Where; T_{travel} - time of concentration (min), L - maximum length of travel (m) and S = slope. In order to minimize error in calculating time of concentration particularly when the average basin slope varies significantly from the mean channel slope, time of concentration is calculated for two sections. The first one is from the outlet point to 0.7 of the channel flow length and the second one is from 0.7L to the end of the channel. Therefore, the channel flow time is summed to the overland flow time to obtain the total time of concentration. Hence, the time of concentration for the overall catchment flow system was determined by taking the summation of all travel times under different flow system.

2.7 Assumptions of Rational method

The basic assumptions for the application of the Rational Method include, (ERA, 2013);

- The computed maximum rate of runoff to the design point is a function of the average rainfall rate during the given time of concentration for that particular point.
- The hydrologic losses in the catchment are assumed to be homogeneous and uniform. The runoff coefficients vary with respect to type of soils, imperviousness percentage, and rainfall frequencies.
- The depth of rainfall used is one that occurs from the start of the storm to the time of concentration. The design rainfall depth during that period is converted to the average rainfall intensity for that period which is assumed to be uniform.
- The maximum runoff rate occurs when the entire area is contributing to the flow. However, this assumption is not valid where a more intensely developed portion of the catchment with a shorter time of concentration produces a higher rate of runoff than the entire catchment with a longer time of concentration.

2.8 Limitations of Rational method

These include the following;

- The method is limited to catchments smaller than $0.5km^2$ especially under the assumption of uniform hydrologic losses hence limited to small areas only.
- Under the condition of composite soils and land uses, use an area-weighted method to derive the catchment's hydrologic parameters.
- It normally provides only one point (the peak flow rate) on the runoff hydrograph. When the areas become complex and where sub catchments come together, the Rational Method will tend to overestimate the actual flow, which results in oversizing of drainage facilities.
- The Rational Method provides no means or methodology to generate and route hydrographs through drainage facilities.
- Another disadvantage of the Rational Method is that with typical design procedures, one normally assumes that all of the design flow is collected at the design point and that there is no water running overland to the next design point. This is not an issue of the Rational Method but of the design procedure. Use additional analysis to account for this scenario.

2.8.1 SCS Method

The Soil Conservation Service (1972) developed a method for computing abstractions from storm rainfall. The volume of storm runoff can depend on a number of factors. Certainly, the volume of rainfall will be an important factor. For very large watersheds, the volume of runoff from one storm event may depend on rainfall that occurred during previous storm events. However, for smaller watersheds, design hydrologists usually assume that runoff for the current storm event is independent of the rainfall of previous storm events. There are other factors which affect the volume of runoff in addition to the rainfall. A very common assumption in hydrologic modeling is that the available rainfall for runoff is separated into three different parts namely; direct runoff, initial abstraction, and losses. The different factors which affect the split between losses and runoff include the volume of rainfall, land cover and use, soil type, and antecedent moisture conditions. Land cover and land use will determine the amount of depression and interception storage. The process of hydrologic modeling involves the acceptance of a number of simplifying assumptions because of the large number of factors to be considered (McCuen, 1998).

For the storm as a whole, the depth of excess precipitation or direct runoff P, is always less than or equal to the depth of precipitation P_e. After run off begins, the additional depth of water retained in the watershed, F_a is less than or equal to some potential maximum retention S. there are abstractions, I_a which are made which are losses from the precipitation depth.

$$\mathbf{P_e = \frac{(P-I_a)^2}{P-I_a+S}} \quad \text{2-7}$$

Where;

P_e is the excess precipitation, $P$is the cumulative precipitation at time t; S is potential maximum retention/soil moisture, which is defined as a measure of the ability of a watershed to abstract and retain storm precipitation. $\mathrm{I_a}$ is the initial abstraction (L) or the amount of water before runoff, such as infiltration, or rainfall interception by vegetation; historically, it has generally been assumed that $\mathrm{I_a} = 0.2S$ although more recent research has found that $\mathrm{I_a} = 0.05S$ may be a more appropriate relationship in urbanized watersheds where the CN is updated to reflect developed conditions (Hawkins et al., 2003).

The maximum retention S is determined following USACE, (2000) equation shown below;

$$\mathbf{S} = \frac{\mathbf{25400-254CN}}{\mathbf{CN}} \text{..................................} \quad \mathbf{2\text{-}8}$$

Where; CN is the runoff curve number, and has a range from 30 to 100; lower numbers indicate low runoff potential while larger numbers are for increasing runoff potential. The initial abstraction and CN are the required parameters (Zhang et al., 2013).

Soil conservation Service (SCS) model has been applied widely during the estimation of flood discharge mainly from small watershed. This method requires numeric catchment characteristics which act as basis of catchment runoff determination. The main objective of this method is to obtain the right curve number for the catchment of interest which defines the runoff potential. The basic catchment characteristics for curve number calculations are hydrologic soil group number, land use type, vegetation cover, soil conservation measures and antecedent soil moisture conditions. It's important to determine and calculate the above listed characteristics accurately during the calculation of flood discharge.

2.8.2 Unit Hydrograph

The introduction of the unit hydrograph concept was brought up by an American engineer called Sherman in 1932. He defined the unit hydrograph as the storm hydrograph resulting from an isolated storm of unit duration occurring uniformly over the entire catchment area which produces a unit, 1cm depth of direct runoff. The unit duration is usually expressed in hours. The unit duration of the storm normally depends on the catchment area. A unit hydrograph is used to relate the direct runoff hydrograph and effective rainfall.

The unit hydrograph method is based on the following assumptions:

- There is a direct proportional relationship between the effective rainfall and surface runoff
- Superposition: the response to successive blocks of effective rainfall each starting at particular times may be obtained by summing the individual runoff hydrographs starting at the corresponding times.
- The effective rainfall-direct runoff relationship does not change with time.

2.8.2.1 Derivation of a Unit Hydrograph

The following data is required when deriving a unit hydrograph; Recorded hydrograph of the storm runoff, recorded storm rainfall data, average depth of rainfall and mass rainfall curves of one or more recording rain gauge stations located in the watershed.

Procedure

The unit hydrograph derivation from a simple storm proceeds in the following stages, (Rugumayo, 2010).

- The stream flow records are studied, and the storm hydrograph due to the selected storm is obtained.
- The base flow is separated from the storm hydrograph to obtain the direct runoff hydrograph.
- The ordinates of the direct runoff hydrograph are obtained by subtracting the ordinates of the base flow from the ordinates of the storm hydrograph.
- The volume of the direct runoff can be estimated from the ordinates O_1, O_2……O_n of the direct runoff hydrograph by applying the trapezoidal rule.

$$\mathbf{V = tx60x60\left[\left(\frac{Q_0+Q_n}{2}\right) + Q_1 + Q_2 + \cdots + Q_{n-1}\right]} \ldots\ldots\ldots \quad \textbf{2-9}$$

Where;

V (m^3) – volume of the direct runoff

t - Time interval (hours) between the successive ordinates of the direct runoff hydrograph.

This because in a direct runoff hydrograph, the end ordinates are zero, the volume of direct runoff is computed as;

Volume of direct runoff, V=3600*t*Q

Where; Q = discharge

- The depth of direct runoff is then computed by dividing the volume of direct runoff by the area of the catchment.
- The ordinates of the direct hydrograph are divided by the depth, d, of direct runoff to compute the ordinates of the unit hydrograph.

The application of the Unit Hydrograph Method requires; (GHKSAR, 2000)

- Loss Model: methods of determining the net rainfall hyetograph from the rainfall hyetograph
- Unit Hydrograph: The unit hydrograph for a catchment can be derived from rainfall-runoff monitoring.

2.9 Design of storm sewers

From the definition of hydrology which is the study of rainfall events and runoff as related to the engineering design of conveyance features. These features include drains, ditches and culverts. These features are typically designed according to a particular storm event or storm frequency.

In general, storm sewers are mainly designed to provide safe passage of vehicles, and to collect, convey and discharge for frequently occurring, low-return-period storms. A given design frequency is selected in order to match the facility's cost, amount of traffic, potential flood hazard to property, expected level of service, political considerations, and budgetary constraints, considering the magnitude associated with damages from larger flood events. Here are a couple of examples showing the different conveyance features with their respective design frequency as extracted from AREMA Manual, 2003.

Table 2-2: Designed return periods for different structures

Conveyance feature	Design frequency (years)
Culverts	50
Ditches	50
Storm water	10

Most damages of the conveyance features are assessed basing on the 100-year storm event by different agencies. This means that the designer should always check or test the system against the 100-year storm in-order to evaluate how the performance of the system under those conditions. If, for example, the tracks are well above the 100-year storm elevation, then the designer has completed his design. However, if the storm overtops the tracks, the person in-charge of the design work may reconsider designing the size of the culvert.

2.10 Modeling urban flooding

A model is a simplified representation of a complex system. It consists of variables, parameters, boundary conditions, and initial conditions. There are different models used when assessing urban drainage floods. Simulation models for flood analysis are required to accurately describe the hydraulic phenomena of surcharged and flooded drainage systems, some of them are listed below according to Schmitt T.G. et al., 2004.

- Identify the current capacity of the various drainage systems.
- The transition from free surface flow to pressure flow in the sewer pipes.

- The rise of water level above ground level with water escaping from the drainage systems.
- The occurrence of surface flow during surface flooding

2.10.1 HEC-HMS model description

HEC-HMS is a conceptual hydrological model which is used to simulate precipitation-runoff processes of single outlet watershed systems (HEC, 2006) using either lumped or semi-distributed model (Madsen, 2000; Scharffenberg et al., 2010). The HEC-HMS can be used to simulate single watershed or a system of multiple hydrologically connected watersheds. The first step in the application of HEC-HMS is defining the basin area and sub-basins, a stream network, and diversions and junctions. It also offers automated and manual calibration and is a numerical model that includes a large set of methods to simulate watershed, channel, and water-control structure behavior, thus predicting flow, stage, and timing. It is a continuous, distributed parameter and watershed scale model that simulates large river basin water supply and flood hydrology and small urban or agricultural watershed runoff (Abushandi and Merkel, 2013). HEC-GeoHMS, is a geospatial hydrology toolkit which allows users to visualize spatial information, document watershed characteristics, perform spatial analysis, delineate sub-basins and streams, and construct inputs for hydrologic models.

Steps in Using HEC HMS

The following steps are generally used for selected study area;

- Create a new project. This was used to give a name and locate the storage point for the project.
- Enter the shared project data. Shared data includes paired data, time series data and grid data. Precipitation, discharge, wind speed, stage, humidity and many others are examples of time series data; storage- discharge, elevation-area, elevation-storage, unit hydrographs curves and many others are examples of paired data; elevation, SCS curve numbers, impervious area, storage capacity and many others are examples of grid data.
- Describe the physical watershed. This was used in the basin model where the hydrologic elements were added and connected to one another to model the real-world flow of water in a natural watershed.

- Describe the Meteorology. The meteorological model was used to calculate the required precipitation input.
- Enter the simulation time windows. It was used for setting the time span and time interval of a simulation run. A simulation time window was created by adding control specifications to the project.
- Simulate and view results. This shows the output of the entire simulation process. For the simulation run to compute, the precipitation runoff response, meteorological model and control specification are required.
- Create or modify data. This involved changing properties of an existing basin model in-order to suit the project under consideration.
- Make additional simulations ad compare results. This was done by using new or modified model components. The results can be compared to one another in the same graph or time series table.
- Exit the program. This is done after saving the project.

Under this particular modeling, different methods are embedded within the software and these include;

Transformation method

The Soil Conservation Service (SCS) dimensionless unit hydrograph procedure was used in HECHMS as a transformation method. The dimensionless unit hydrograph used by the SCS was developed by Victor Mockus and was derived based on a large number of unit hydrographs from basins that varied in characteristics such as size and geographic location. Different unit hydrographs were obtained and averaged; the final product was made dimensionless by considering the ratios of q/q_p (flow/peak flow) on the ordinate axis and t/t_p (time/time to peak) on the abscissa, where the units of q and q_p are flow/inch of runoff/unit area. The derived final dimensionless unit hydrograph, which is the result of averaging a large number of individual dimensionless unit hydrographs, has a time-to-peak located at approximately 20% of its time base and an inflection point at 1.7 times the time-to-peak. (Donald E. Woodward, 1997).

Loss method

In any sub-basin element, it conceptually represents infiltration, surface runoff, and subsurface processes interacting together. However, the actual infiltration calculations are performed by a loss method contained within the sub-basin. A total of twelve different loss methods are provided within the HEC-HMS program. Some of the methods are designed primarily for simulating events while others are intended for continuous simulation and they conserve mass. Therefore, the sum of infiltration and precipitation left on the surface will always be equal to total incoming precipitation. In this particular case study, the soil conservation service curve number method was used. Soil conservation Service (SCS) model has been applied widely during the estimation of flood discharge mainly from small watershed. The right curve number for the catchment of interest was obtained which defines the runoff potential. The rainfall-runoff standard chart has been used mostly for estimating the runoff from watersheds for which composite CN's are obtained from listings like those in table of runoff curve numbers for hydrologic soil-cover complexes (USDA, 1973). Different antecedent moisture condition will used be considered in the determination of the appropriate curve numbers (CN). However, the simplest way of obtaining the peak discharge is by using appropriate numerical equations as stated below;

$$\mathbf{Q = \frac{(P-Ia)^2}{(P-Ia)+S}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}10}$$

Q = accumulated direct runoff.

P = accumulated rainfall (potential maximum runoff)

I_a = initial abstraction including surface storage, interception, and infiltration prior to runoff.

S = potential maximum retention

Where;

$$\mathbf{S = \frac{25400-254CN}{CN}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}11}$$

$$\mathbf{I_a = 0.2S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}12}$$

The CN is read from runoff curve numbers for hydrologic soil-cover complexes table. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

The run off curve numbers depends on various parameters such as land uses and hydrological soil groups. In order to select any given CN value, an average antecedent moisture conditions must be considered like dry, wet and normal conditions. They can only be selected only after a field inspection of the catchment area and a review of cover type and soil maps (ERA manual, 2013). The SCS has four hydrologic soil groups based on infiltration rates (Groups A, B, C, and D). The soils within in this catchment area are clay and loam which lie within hydrologic groups D and B respectively. Land use is the catchment area cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, and many others are all part of the land use.

Much care must be taken during the selection process of the curve numbers. This is because selection of an overly conservative curve numbers results in the estimation of excessively high runoff and consequently excessively costly drainage structures. It is better to use average values and design for a longer storm frequency. Often times the runoff computed using conservative CN's for a ten-year storm will greatly exceed the computed runoff for average CN's for a 25 or even 50-year storm. The different antecedent moisture conditions for Ethiopia are as below; dry for Region D1, wet for Region B1, and average or normal AMC for all other regions. The portion of Region A2 in the proximity of Bahir Dar should also be treated as wet (ERA, 2013).

Initial abstraction mainly depends on the determined potential maximum retention storage potential. It's obtained from the relationship below;

$$\mathbf{I_a = 0.2S} \qquad \mathbf{2\text{-}13}$$

The potential maximum retention, S is obtained after getting the run off curve numbers from the relationship below;

$$\mathbf{S = \frac{25400-254CN}{CN}} \qquad \mathbf{2\text{-}14}$$

The determination of the sub-basin percentage contributing to the land use was specified using the existing land Google earth program and field visits.

Routing Method; incase the analyzed watershed is divided into different sub-watersheds, the flow at the outlet of any upstream watershed has to be routed through the river channel in the downstream watershed. The models used in-order to simulate the routing process are called routing methods. The model is widely used as a practical (and research) rainfall runoff model in various countries, including USA, Europe and Asia. It has also been widely applied for humid, tropical, subtropical, and arid watersheds to simulate and forecast stream flow. Different versions of HEC-HMS have been developing and the currently available version is HEC-HMS 4.2.1(Feldman, 2000; HEC, 2006).

2.10.2 HY-8

This is used to automate culvert hydraulic computations. It's also known as HYDRAIN. It helps users to analyze the performance of culverts, multiple culvert barrels at a single crossing as well as multiple crossings, roadway overtopping at the crossing and develop report documentation in the form of performance tables, graphs, and key information regarding the input variables (FHWA, 2014). It has also the ability to define multiple crossings within a single project. A crossing is defined by 1 to 6 culverts, where each culvert may consist of multiple barrels. It automates culvert hydraulic computations and also enables the analysis of the following; Performance of the culvert, multiple culvert barrels at a single crossing as well as multiple crossing, roadway overtopping at the crossing and develop report documentation in the form of performance tables, graphs, and key information regarding the input variables (Nwaogazie and Agiho, 2019).

Steps used in HY8

- Starting a new project; this established the directory a user wanted to work in, it had different components like determining the unit system to be used.
- Entering new project data; this helped the user to set the required project data. The data included discharge data, tail water data, road way data, crossing data (culvert data, site data)
- Performing the analysis; once all the necessary data has been entered, the user was able to perform the hydraulic calculations by clicking on the analyze button.
- Viewing and printing results; this helped the user to see the result after the computations were done. Different plots can be made basing on the user's intentions.

2.10.2.1 Limitations of HY8

The main limitation is as below;

- **Entrance limitations**

Since HY-8 is not primarily a water surface profile computation program but is a culvert analysis tool, it assumes a pooled condition at the entrance to the culvert.

- HY-8 will compare the velocity and depth from the culvert that were computed using direct step against the velocity and depth in the cross-sections that were computed using manning's equation.

- Due to the way that the HY-8 engine is setup, it can be difficult for HY-8 to accurately compute low flows.

2.10.2.2 Assumption of HY8

Assumptions include the following;

❖ **Vena Contracta assumption**

In some cases, a vena contracta drawdown of the water surface profile could occur in a culvert barrel since the culvert has the potential to act as a sluice gate at the entrance. This drawdown at the entrance is sometimes called a vena contracta. A coefficient that is generalized for circular and box culverts is used to compute the location and depth of the vena contracta for all culvert shapes.

❖ **Brink depth**

For culverts with tail water elevations below the outlet invert of the culvert, water flowing out of the culvert would theoretically pass through a brink depth instead of through critical depth. In this case, HY-8 uses critical depth to determine the final culvert depth and velocity rather than the brink depth.

❖ **Culvert cross section**

HY-8 assumes the culvert cross section shape, size, and material does not change in the barrel except in the case of broken back runout sections, where the user can change the material and Manning's roughness in the runout (lower) culvert section.

2.10.3 Hydraulic tool box

This is an intuitive computer program containing a suite of many different calculators that can be used to perform many of the routine hydrologic and hydraulic computations needed by highway hydraulic engineers and roadway designers.

However, field measurements of drains, culverts and bridges in the project area was carried out. This was done to ascertain the hydraulic dimensions of the existing drainage structures before using them in hydraulic modeling. For underground drainage structures that may not be accessed during the field visits, the design technical drawings were used in ascertaining their structural sizes on an ‘as designed’ principle of analysis.

The procedure to be followed when using hydraulic toolbox is briefly described below (Hydraulic toolbox users’ manual);

2.10.3.1 Steps used in Hydraulic toolbox

Each calculator performs a specific type of analysis and only one calculator can be active at any given time. As the user opens the calculator windows, data inputs are displayed that are unique to the opened calculator. However, in this case study, the channel analysis calculator was considered.

- Starting a channel analysis calculator; this established the directory a user wanted to work in, it had different components like determining the unit system, full range of hydraulic parameters to be used.
- Entering channel lining; this helped the user to set to select the appropriate channel lining like fiber glass lining, woven paper net and many others.
- Entering flow data or known depth and other boundary conditions; this helped the user to enter the flow data which was used to ascertain whether the suggested diameter of the structure will be able to convey the flow. The other boundary condition includes manning's coefficient, slope and many others
- Performing the hydraulic calculation; once all the necessary data has been entered, the user was able to perform the hydraulic calculations.
- Viewing and printing results; this helped the user to see the result after the computations are done. Different plots can be obtained depending on the user's choice.

2.10.3.2 Limitation of Hydraulic tool box

- FHWA does not provide user assistance or support for this software.
- The application of this software is the responsibility of the user. It is imperative that the responsible engineer understands the potential accuracy limitations of the program results, independently cross checks those results with other methods, and examines the reasonableness of the results with engineering knowledge and experience.
- There are no expressed or implied warranties.

2.11 Model Selection

HEC HMS, Hydraulic tool box and HY8 are selected for this particular study because of the following reasons;

- They are public domain software and freely copied.
- They have a strong capacity for conducting hydrologic simulations.
- They are able to do complex works and leaves the user to focus on how best he can represent the watershed environment.
- They are simple and easily understood.

2.12 Data preparation

This involves dividing the data in shorter durations.

2.12.1 Estimation of Missing Rainfall Data

Before any analysis or any hydrologic modeling is done on the rainfall data of a given place or region, it is necessary to check for its continuity. This is because the data can be discontinuous when a particular rain gauge is out of operation for some period of the record. In such a case, it becomes necessary to supplement the missing data by an intelligent estimation. The missing data is usually estimated from the available data of the neighboring rain gauge stations called index stations. In order to estimate any missing data, the normal annual rainfalls of all the available rain gauge stations, including that particular station with missing data, are required. The normal annual rainfall of a station is the average value of the annual rainfall over a specified period of 30 years or soothe normal rainfall is updated every ten years. Below are the two main methods which will be used for estimating the missing data (Subramanya, 2001);

2.12.1.1 The Comparison method

In case the rainfall record of a given rain gauge station (say, X) is missing data for a relatively long period like a month or a year, the missing data can be estimated by comparing the mean annual rainfall of the station X with that of an adjoining station (say A). Thus;

$$\frac{P_X}{P_A} = \frac{N_X}{N_A} \qquad \textbf{2-15}$$

Where; P_x and P_A are the precipitations of the stations X and A for the missing period and N_x, and N_A are the mean annual rainfalls of the stations X and A.

2.12.1.2 The Normal Ratio method

In case of short break in the precipitation data of a rain gauge station, it can be estimated from the observed data of three adjoining index stations A, B and C which are evenly distributed around the stations X. However, these two cases must be dealt with separately;

When the mean annual rainfall at each of the index stations A, B, and C, is within 10% of the mean rainfall of station X, a simple average of the values of the index station is taken. Thus:

$$\mathbf{P_X = \frac{1}{3}[P_A + P_B + P_C]} \text{..............................} \quad \mathbf{2\text{-}16}$$

When the mean annual rainfall at each of the index stations differs from the station X by more than 10% the normal ratio method is used. Thus;

$$\mathbf{P_X = \frac{N_X}{3}\left[\frac{P_A}{N_A} + \frac{P_B}{N_B} + \frac{P_C}{N_C}\right]} \text{..............................} \quad \mathbf{2\text{-}17}$$

Where; the symbol N is used for the mean annual rainfall (also called average annual precipitation) and the symbol P is used for the precipitation.

When there are M Index stations

$$\mathbf{P_X = \frac{N_X}{M}\left[\frac{P_A}{N_A} + \frac{P_B}{N_B} + \cdots + \frac{P_M}{N_M}\right]} \text{.........................} \quad \mathbf{2\text{-}18}$$

2.12.2 Test for Consistency

From several years' records it may seem that annual rainfall is, say, declining. It is important to know that this trend is independent of the gauging, and is due to meteorological conditions only. For this to be ascertained, a double mass curve can be plotted as a check. This technique is based on the principle that when each recorded data comes from the same parent population, they are consistent. About a group of 5 to 10 base stations in the neighborhood of the problem station X is selected (Lubega, 2008).

A double mass curve method is often used for checking the consistency of the rainfall data. The procedure to be followed is as below;

- The annual rainfall data of station X and also the average rainfall of the group of base stations covering a long period are arranged in a reverse chronological order.

- The accumulated precipitation of the station X (say$\sum P_x$) and that of the average of the group of base stations (say$\sum P_{av}$) are obtained and calculated starting from the latest record.
- Values of $\sum P_x$ against $\sum P_{av}$ are plotted for various consecutive time periods.
- A decided break in the slope of the resulting plot which indicates a change in the precipitation regime of station X is to be calculated.
- The precipitation values at station X beyond the period of change of regime can be corrected by using the relation below;

$$\mathbf{P_{cx} = P_X \frac{M_c}{M_a}} \qquad \textbf{2-19}$$

P_{cx} - corrected precipitation at any time period t_1 at station X

P_x - original recorded precipitation at time period t_1 at station X

M_c - corrected slope of the double-mass curve

M_a - original slope of the mass curve

After obtaining the value of M_c, older records are brought up to the new regime of the station. It is apparent that the more homogeneous the base station records are, the more accurate will be the corrected values at station X. A change in slope is normally taken as significant only where it persists for more than five years. The double-mass curve is also helpful in checking arithmetical errors in transferring rainfall data from one record to another.

2.12.3 Disaggregation of Rainfall Data

In most hydrological applications, like flood forecasting, require rainfall data of shorter duration, whereas the network of recording rain gauges (providing short duration data) is small in comparison to that of daily rain gauges. Hence, it is often necessary to disaggregate or divide the daily rainfall data into shorter time intervals of usually one hour. The observed daily rainfall is distributed in hourly values to follow the same pattern as the observed hourly rainfall. The information of short interval rainfall is normally used together with the information of daily rainfall from nearby non-recording (daily) gauges. A mass curve is most common method used for data disaggregation. A mass curve is a graphical plot of accumulated rainfall at a station versus time. Mass curves of accumulated rainfall at (non-recording) daily stations and recording stations can be prepared by plotting the accumulated rainfall values against time for the storm duration under analysis.

2.13 Rainfall Frequency Analysis

Frequency analysis of hydrologic data is mainly done in-order to obtain the relationship in the magnitude of extreme events to their frequency of occurrence through the use of probability distributions (Chow, et al., 1988). Basing the on the available rainfall data of previous years, the design rainfall can be determined by frequency analysis. Furthermore, the Intensity Duration Frequency curves are developed using the frequency analysis.

Different probability distributions can be used in frequency analysis but their reliability is checked normally by carrying out goodness of fit tests. According to the Ethiopian Drainage Design Manual (ERA, 2013), the Extreme value type 1 or Gumbel and Log Pearson Type III methods are suggested.
Limitations of the statistical methods normally depend on the assumptions made when using them. The first assumption is rainfall isn't affected by climatic trends. However, this may be true in the previous centuries not the current one (21^{st}) simply because there is a great relationship between climatic changes and rainfall intensity. Another assumption, annual maximum series are normally considered as sample of random and independent events (The Hydrology Subcommittee, 1981). Historical data were used to obtain the required sample which is used to estimate the population for getting the magnitude and frequency of rainfall (Chow, et al., 1988). Furthermore, existing drainage structures are mostly designed based on historical precipitation statistics and experiences, whereas today the design needs to include future scenarios (Kassahun., 2015).

2.13.1 Gumbel / Extreme Value Distribution Type 1

This is a two-parameter distribution, whose form arises from consideration of the statistical properties of the sample extreme values and was first introduced into hydrology by Gumbel. The theory of extreme values considers the distribution of the largest or smallest observations occurring in each group of repeated samples. For example, the study of peak flows uses just the largest flow recorded each year at a gauging station out of the many thousands of values recorded.

Since these observations are located in the extreme tail of the probability distribution of all observations from which they are drawn (the parent population), it is not surprising that their probability distribution is different from that of the parent population. The equations used for obtaining the probability density function and cumulative distribution function are stated as below;

$$\mathbf{f(x) = \frac{1}{\sigma} exp(-\frac{x-\mu}{\sigma} - exp - \frac{x-\mu}{\sigma})} \text{..................} \quad \mathbf{2\text{-}20}$$

$$\mathbf{F(x) = exp(-exp\left[-\frac{x-\mu}{\sigma}\right])} \text{.........................} \quad \mathbf{2\text{-}21}$$

Where; 'σ' and 'μ' are the scale and location parameters, respectively.

This distribution can be used to calculate the maximum value of expected rainfall (X_T) corresponding to any return period (T) using the equation below;

$$\mathbf{K_T = \frac{\sqrt{6}}{\pi}\left[0.5772 + In\left\{In\left(\frac{T}{T-1}\right)\right\}\right]} \text{...............} \quad \mathbf{2\text{-}22}$$

$$\mathbf{X_T = \bar{X}(1 + C_{vy}K_T)} \text{.................................} \quad \mathbf{2\text{-}23}$$

Where $\bar{X}$ is the mean, 'C_v' is the coefficient of variation and 'K_T' is the frequency factor, which depends on the return period (T) and probability distribution.

2.13.2 Log Pearson Type III

This is a three-parameter gamma distribution with a logarithmic transform of the variable. The necessary parameters which are used to describe the distribution are mean, standard deviation, and coefficient of skew. It is used for approximation of frequency characteristics of measured annual flood peak data. This distribution has been widely adopted as one of the standard methods for flood frequency analysis. In this distribution the transform y = log x is used to reduce skewness. Although all three moments are required to fit the distribution, it is extremely flexible in that a zero skew will reduce the Log-Pearson III distribution to a Log-Normal and the Pearson Type III to a Normal. The probability density function and cumulative distribution function of the log-Pearson type-III distribution are calculated using the following equations (Rugumayo, 2010);

$$\mathbf{f(x) = \frac{1}{xI\beta I\Gamma(\alpha)}\left(\frac{In(x)-y}{\beta}\right)^{\alpha-1} exp\left(-\frac{In(X)-y}{\beta}\right)} \text{.......} \quad \mathbf{2\text{-}24}$$

$$\mathbf{F(x) = \frac{\frac{\Gamma(In(x)-y)^{\alpha}}{\beta}}{\Gamma(\alpha)}} \text{.......................................} \quad \mathbf{2\text{-}25}$$

Where; 'α', 'β' and 'γ' are shape, scale and location parameters, respectively. In the log-Pearson type-III distribution, the maximum value of expected rainfall (X_T) corresponding to any required return period (T) can be calculated using the following equations as stated below;

$$\mathbf{X_T = Antilog\ (X)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}26}$$

$$\mathbf{Log\ X = \bar{X} + K_T S_d} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}27}$$

$$\mathbf{K_T = \frac{2}{C_s}\left[\left\{\left(z - \frac{C_s}{6}\right)\frac{C_s}{6} + 1\right\}^3 - 1\right]} \ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}28}$$

Where;

$\bar{X}$ - Mean, S_d - Standard deviation and C_s - the coefficient of skewness of the rainfall data. K_T is the frequency factor.

2.13.3 Pearson Type III

The Pearson Type III distribution is a three-parameter one. The three parameters represent measures of location, scale and shape respectively. The probability density function is given by the equation below;

$$\mathbf{f(Q) = \frac{e^{-\frac{-(Q-Q_0}{\beta}}(Q-Q_0)^{\gamma-1}}{\beta^{\gamma}\,\Gamma(\gamma)}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}29}$$

Where;

Q_o= location parameter, in this case a lower bound, β = scale parameter, γ = shape parameter Γ(γ) = Complete Gamma Function

2.13.4 Log Normal Distribution

If the random variable Y=log X is normally distributed, then X is said to be log normally distributed. The lognormal distribution has the advantage over the normal distribution that it is bounded (X>0) and that the log transformation tends to reduce the positive skewness commonly found in hydrologic data, because taking logarithms reduces large numbers proportionately more than it does small numbers. Some limitations of the lognormal distribution are that it has only two parameters and that it requires the logarithms of the data to be symmetric about their mean. When the coefficient of skewness g = 0, then Log-Pearson Type III frequency distribution is reduced to log-normal distribution. A log-normal distribution plots as a straight line on logarithmic probability paper. The pdf and CDF equations are given as below;

$$\mathbf{f(x) = \frac{exp\left[-\frac{1}{2}\left(\frac{In(x-y)-\mu}{\sigma}\right)^2\right]}{(x-y)\sigma\sqrt{2\pi}}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \mathbf{2\text{-}30}$$

$$\mathbf{F(x)} = \emptyset\left(\frac{\mathbf{In(x-y)-\mu}}{\boldsymbol{\sigma}}\right) = \frac{\mathbf{1}}{\mathbf{2}}\left[\mathbf{erfc}\left\{-\frac{\mathbf{In(x-y)-\mu}}{\boldsymbol{\sigma}\sqrt{\mathbf{2}}}\right\}\right]\ldots \quad \textbf{2-31}$$

Where; 'μ' is the shape parameter, 'σ' is the scale parameter, 'γ' is the location parameter and 'Φ' is the Laplace Integral. The log-normal distribution assumes that Y=InX; therefore, the maximum value of expected rainfall (X_T) corresponding to any return period (T) can be calculated using the equation below;

$$\mathbf{X_T = exp(Y_T)}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{2-32}$$

$$\mathbf{Y_T = \bar{Y}(1 + C_{vy}K_T)}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{2-33}$$

$$\mathbf{K_T} = \frac{\mathbf{Y_T - \mu_y}}{\boldsymbol{\sigma}_\mathbf{y}}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad \textbf{2-34}$$

Where $\bar{Y}$ and 'C_{vy}' are the mean and coefficient of variation of 'Y', respectively. K_T is the frequency factor, which is the same as the standard normal variate and can be computed using the equation;

2.14 Goodness of fit test

This is designed to compare the sample obtained with the type of the sample one would expect from the hypothesized distribution and to check whether the hypothesized distribution function fits the data in the sample not for selecting the best distribution. These help in calculating the test-statistics, which are normally used to analyze how well the data fits any given distribution and can also describe the differences between the observed data values, and the expected values from the distribution being tested. This test is performed in order to test the following hypotheses; H_0; the precipitation data follow the specified distribution, H_1: the precipitation data do not follow the specified distribution.

Different goodness of fit tests are carried out which includes; Kolmogorov-Smirnov test, Anderson-Darling test and the chi-square test at significance level of 0.05 for choosing the best fit Probability distribution (Sharma and Singh, 2010). These tests are briefly described as below;

2.14.1 Kolmogorov-Smirnov Test

Kolmogorov-Smirnov test statistic (D) refers to a function of the greatest vertical distance between distribution functions, either hypothesized or empirical distribution functions. K-

S test calculates the maximum difference between the hypothesized distributions, $Z_i = F(x_i\, \hat{\theta})$ and empirical cumulative distribution function F_n (x_i) with x_i representing the ordered data. However, in case of small sample, it's more preferable over the chi-square test for determining the goodness of fit. This test is used to decide if a sample comes from a hypothesized continuous distribution (Chakravarti *et al.*, 1967).

2.14.2 Anderson – Darling test

It calculates the weighted squared difference of the hypothesized distribution $Z_i = F(x_i\,, \hat{\theta})$ and empirical cumulative distribution function F_n (x_i), the weighted function $\{F(x_i, \hat{\theta})[1 - F(x_i\,, \hat{\theta})]\}^{-1}$. Where x_i represents the ordered data. The Anderson Darling statistic (A) is defined as;

$$\mathbf{A^2 = -n - \frac{1}{n} \sum_{i=1}^{n} (2i-1)\{InF(X_i) + In[1 - F(X_{n-i+1)}]\}......} \quad \textbf{2-35}$$

This test normally compares the fitness of an observed cumulative distribution function to an expected cumulative distribution function. It gives more weight to the tails than the Kolmogorov-Smirnov test (Stephens, 1974, 1976, 1977).

2.14.3 Chi-square Test

This test assumes that the number of observations is large enough so that the chi-square distribution provides a good approximation as the distribution of the test statistic. The Chi-Squared statistic is defined as;

$$\mathbf{X^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i} ....} \qquad \textbf{2-36}$$

Where; O_i = observed frequency, E_i = expected frequency and 'i'= number of observations (1, 2…k), F = the CDF of the probability distribution being tested

$$\mathbf{E_i = F(x_2) - F(x_1)..............................} \qquad \textbf{2-37}$$

The observed number of observation (k) in interval 'i' is also computed from the equation below;

$$\mathbf{k = 1 + log_2 n...................................} \qquad \textbf{2-38}$$

Where; n is the sample size. This particular test is for continuous sample data only and only used to determine if the given sample comes from a population with a specific distribution (Sharma and Singh, 2010).

CHAPTER 3 STUDY AREA

This section describes the different aspects of the study area.

3.1 Brief description of Addis Ababa city

Addis Ababa city was established way back in 1886 by Emperor Menilik II. Being one of the oldest and largest cities in Africa (UN-HABITAT, 2008), it has played a historic role in housing different headquarters of big organizations like Economic Commission for Africa under United Nations and African Union which contributed to the decolonization of African countries, and later bringing Africa together.

Currently, Addis Ababa is one of the rapidly urbanizing cities of the world and it's geographically located in the center of Ethiopia. As a result of its location, Addis Ababa is a melting pot to hundreds of thousands of people, coming from all corners of the country in search of better employment opportunities and services (Anteneh, 2015). This has led to rapid population growth within the city hence leading to its fastest growth among the cities in Africa. However, it's also facing critical challenges which include inadequate transport, housing shortage and high unemployment rates. The newly constructed AALRT project is part of the cities urbanization process that is intended to solve the transportation problem of the capital (Fisha, 2015). The fact that the project is located within Addis Ababa is the reason of choosing the city as my study area. This thesis will provide some knowledge regarding the flood analysis and hydraulic competence of the integrated drainage system for light rail transit.

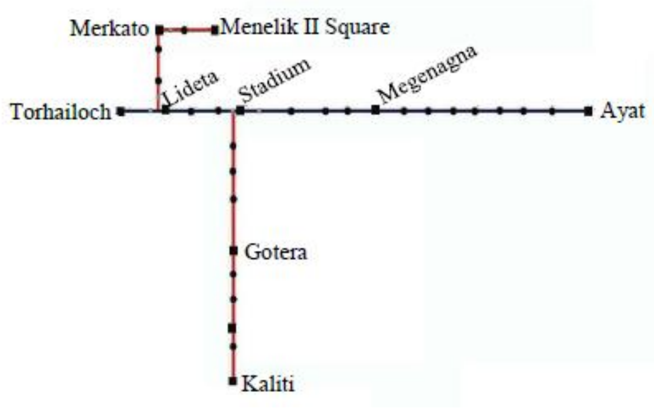


Figure 3.1: Complete AALRT System

3.1.1 Location of the project area

Addis Ababa is found within the center of the Federal Democratic Republic of Ethiopia at latitude of 8º 50’11”- 9º 05’29” North and longitude of 38º 39’40”- 38º 54’57” East on Universal Transverse Mercator projection. The city lies at the foot of Mount Entoto which is 3400masl and extends south wards to its lowest point near to 2000masl around Akaki (Anteneh, 2015). The study area of Mesualekia – Nefas Silk2 is found in the areas of Nefas silk – Lafto at latitude of 9º 0’ 17.9136” and longitude of 38º 45’ 31.612” at Mesualekia and latitude 8º 58’ 22.3536” and longitude of 38º 45’ 38.7” at Nefas- Silk2.

3.1.2 Climate

The climate of Addis Ababa doesn’t vary much because of its location around the Equator. As a result, its temperature remains nearly constant from month to month, with not more than 10ºC change. It also has a temperate climate due to its high-altitude location within the subtropics.

In the mid-November to January is a season for occasional rain. The highland climate regions are normally characterized by dry winters, and this is the dry season in Addis Ababa. During this season the daily maximum temperatures are usually not more than 23°C (73°F), and the night-time minimum temperatures can drop to freezing. The short rainy season is from February to May. During this period, the difference between the daytime maximum temperatures and the night-time minimum temperatures is not as great as during other times of the year, with minimum temperatures in the range of 10 - 15°C (50 - 59°F). At this time of the year, the city experiences warm temperatures and a pleasant rainfall. The long-wet season is from June to mid-September; it is the major winter season of the country. During this period, the temperatures are much lower than at other times of year because of the frequent rain and hail and the abundance of cloud cover and fewer hours of sunshine. This time of the year is characterized by dark, chilly and wet days and nights. The autumn which follows is a transitional period between the wet and dry seasons. The highest temperature on record was 32°C (90°F) on the 27th August 1996, while the lowest temperature on record was 0°C (32°F) on 23rd November 1999. The average monthly rainfall of each month together with the annual maximum precipitation time series are shown in the graphs below which is generated from the acquired raw data from National Meteorological Agency of Ethiopia particularly for Addis Ababa Observatory

Station. The data used for developing the graphs below is shown in the Appendix 1.

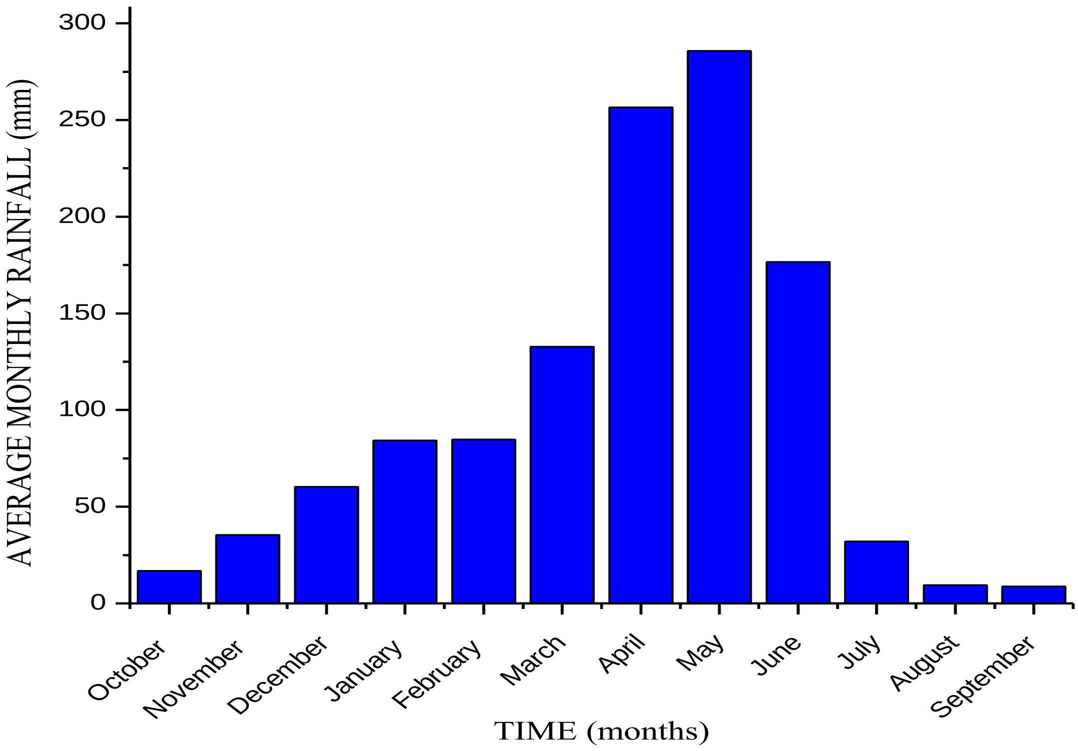


Figure 3.2: Temporal Rainfall

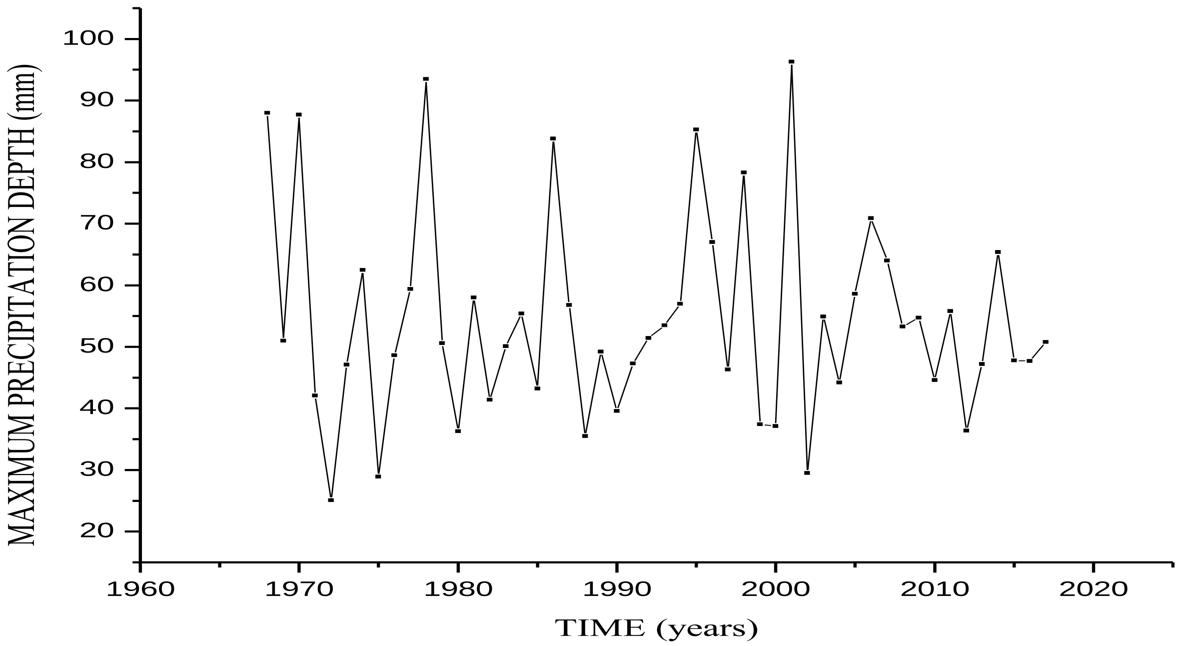


Figure 3.3: Annual precipitation series

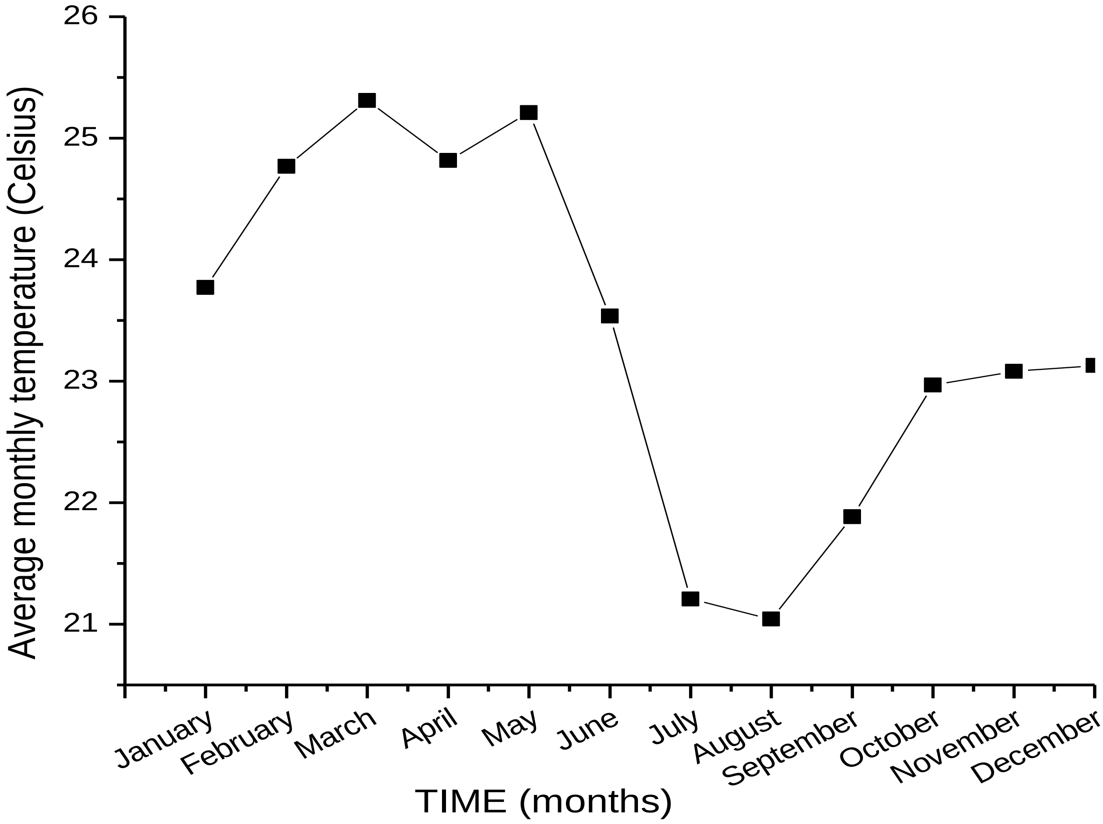


Figure 3.4: Average monthly maximum temperature

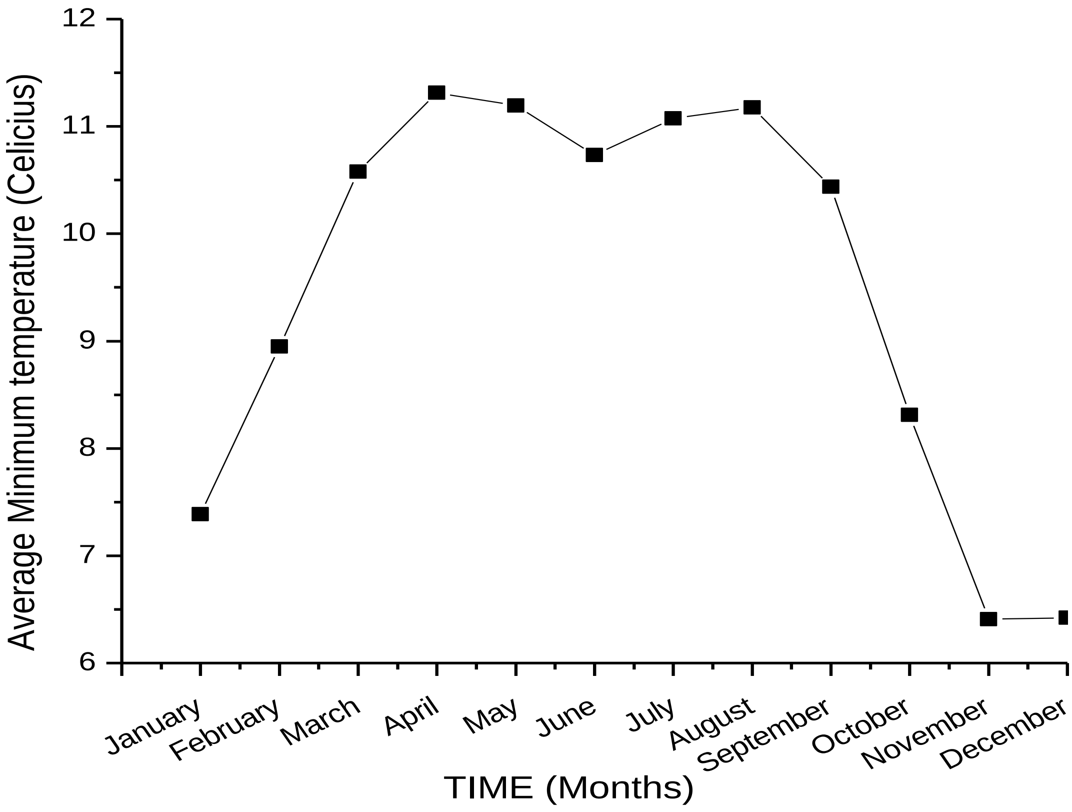


Figure 3.5: Average minimum temperature

The average minimum and maximum temperature of each month is shown in the graphs above was generated from the acquired raw data from National Meteorological Agency of Ethiopia for a period of 31 years from 1980 to 2010.

3.1.3 Demography

The population projection of Ethiopia for all regions at woreda level from 2014 – 2017 were carried out by Central Statistical Agency (CSA, 2013). The different population distribution of Addis Ababa for all ten sub cities in 2015 are shown. The route from Mesualekia -Nefas silk2 along the AALRT is located in Nefas silk lafto area, it's moderately populated implying that chances exist for more urbanization which would have an implication on future hydrological set up within the area. The city has an urban population of approximately 3.3 million.

Table 3-1: Population and population density of Addis Ababa by sub city for 2015

Sub City	**Population**	**Area**	**Density**	**Density rank**
Addis Ketema	305,058	7.408	41,180	1
Akaki Kality	216,538	118.080	1,834	10
Lideta	240,989	9.175	26,266	2
Arada	252,705	9.914	25,490	3
Kirkos	264,337	30.184	8,758	4
Gulele	319,712	120.610	2,651	9
Bole	369,189	120.610	3,061	8
Nefas Silk Lafto	377,892	68.301	5,533	6
Yeka	414,212	87.444	4,737	7
Kolfe Keraniyo	512,369	61.251	8,365	5
Total population	**3,273,001**			

3.1.4 Land Use and Land Cover

Land use in Addis Ababa ranges from agricultural to high density commercial areas. The land use and land cover of the city have evolved as forest and agricultural lands are built up. Figure 3.7 below shows the proposed land use map for Addis Ababa 2006, while Figure 3.8 indicates the current land use of Addis Ababa areas whereby the darker areas are highly urbanized whereas lighter areas are crop land, forest, green areas or areas along a river. Figure 3.9 shows the trend of urbanization land cover of Addis Ababa between 1986 and 2010 with the same description with figure 3.8.

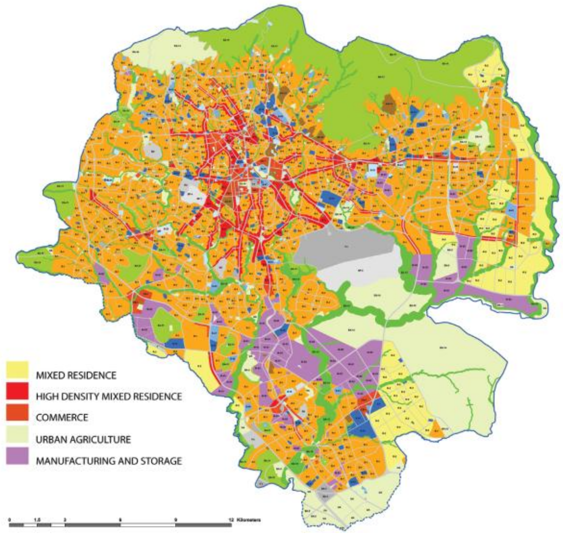


Figure 3.6: Proposed Land Use Map for Addis Ababa 2006
(Source: Addis Ababa City Master Plan Revision Project office)

Figure 3.7: Urbanization Map of Addis Ababa City in 2015

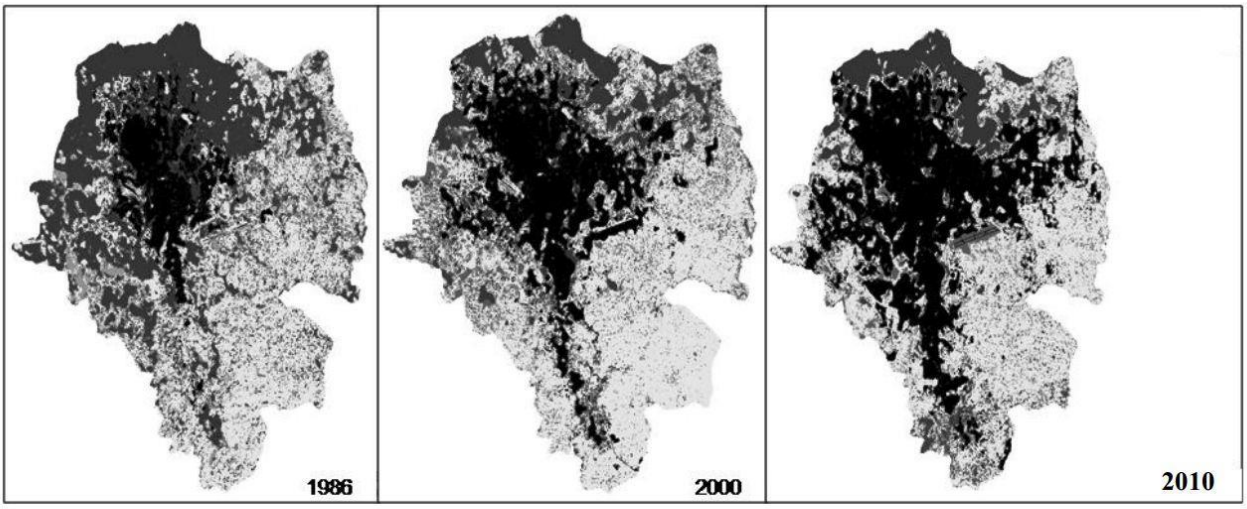


Figure 3.8: Urbanization Maps of Addis Ababa City in 1986, 2000 and 2010
(Source: Kasa, et al., 2013)

3.2 Addis Ababa Light Rail Transit

The Light Rail Transit (LRT) project is located in Addis Ababa, the capital city of Ethiopia. Addis Ababa is one of the fastest growing cities in Africa. The introduction of the Light Rail Transit project was to reduce on the problem of urban transport, especially in the downtown areas. The project has two routes, the East-West route and the South-North route. The line has two types of tracks i.e. a slab section and a ballast section. The slab section is found at the elevated sections while the ballast is found on the non-elevated ground. The first step in the launching of the Light Rail Transit System (LRT) for Addis Ababa was taken when an Ethiopian Railway Corporation (ERC) took charge of LRT in March 2008. The AALRT is a double track and it has a length of 34.25 km. The North-South line has a length of 16.9 km and the East- West line has a length of 17.35 km. the North-South line covers the areas through Menelik II Square, Mercato, Lideta, Legehar, Meskel Square, Gotera and Kality while the East-West line covers areas like Ayat Village to TorHailoch passing through Megenagna, Legehar and Mexico Square.

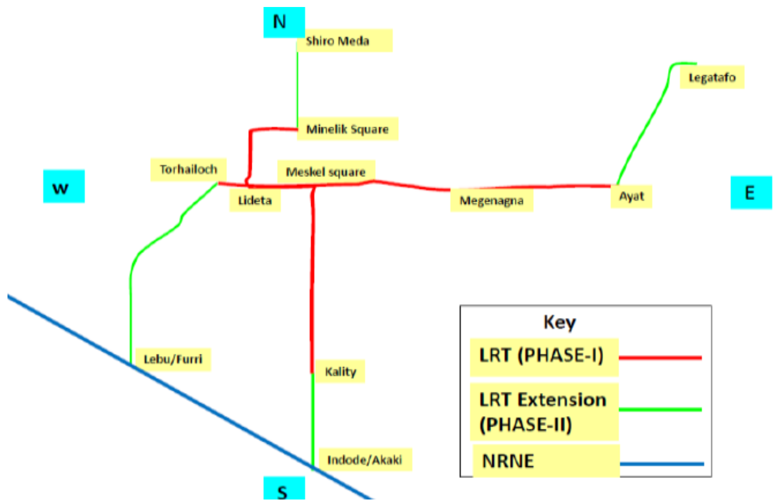


Figure 3.9: LRT alignment and its proposed extension

The north to south line starts from Menelik II square and ends at Kality. It passes through Mercato market, Meskel square and Saris. The total length of the line is 16.9km of which 11.6km is at subgrade and the rest is elevated section. There are also 22 stations on this line. The north to south line and the east to west lines share a common track of 2.7km, (Fisha, 2015).

3.3 Mesualekia-Nefas Silk2 Route Section

This study area is within the south of Addis Ababa, on the road from Meskel square to the terminal at Kaility. The project route is located in Nefas silk – Lafto area which is situated in the South Western part of Addis Ababa, bordered by Oromia Special Zone in the South, Kolfe Keranio in the North West, Bole and Akaki Kality in the East and Lideta and Kirkos in the North and it covers an area of 5876.25 hectares (Alamirew, 2016). It's a highly populated area after Yeka sub city (CSA, 2013). Due to the rapid growth of the city, that area is occupied mainly by both commercial buildings and residential buildings. This leads to an increase in the total runoff volume due the increased impervious surface which makes the existing drainage structures not able to carry or convey the runoff volume during heavy precipitation flows. The existing drainage for the highway was integrated to serve as both for the road and AALRT at large. The area is at an elevation of 2298-2355masl and the type of soil within that area is mainly clay. The major socio-economic activities of the population in the sub city are nonagricultural activities such as trade, manufacturing and services (Nigussie, 2018). The sub-city is divided has a total of 12 Weredas, 128 Subweredas, 397 Sefers, and 1059 blocks (Alamirew, 2016).

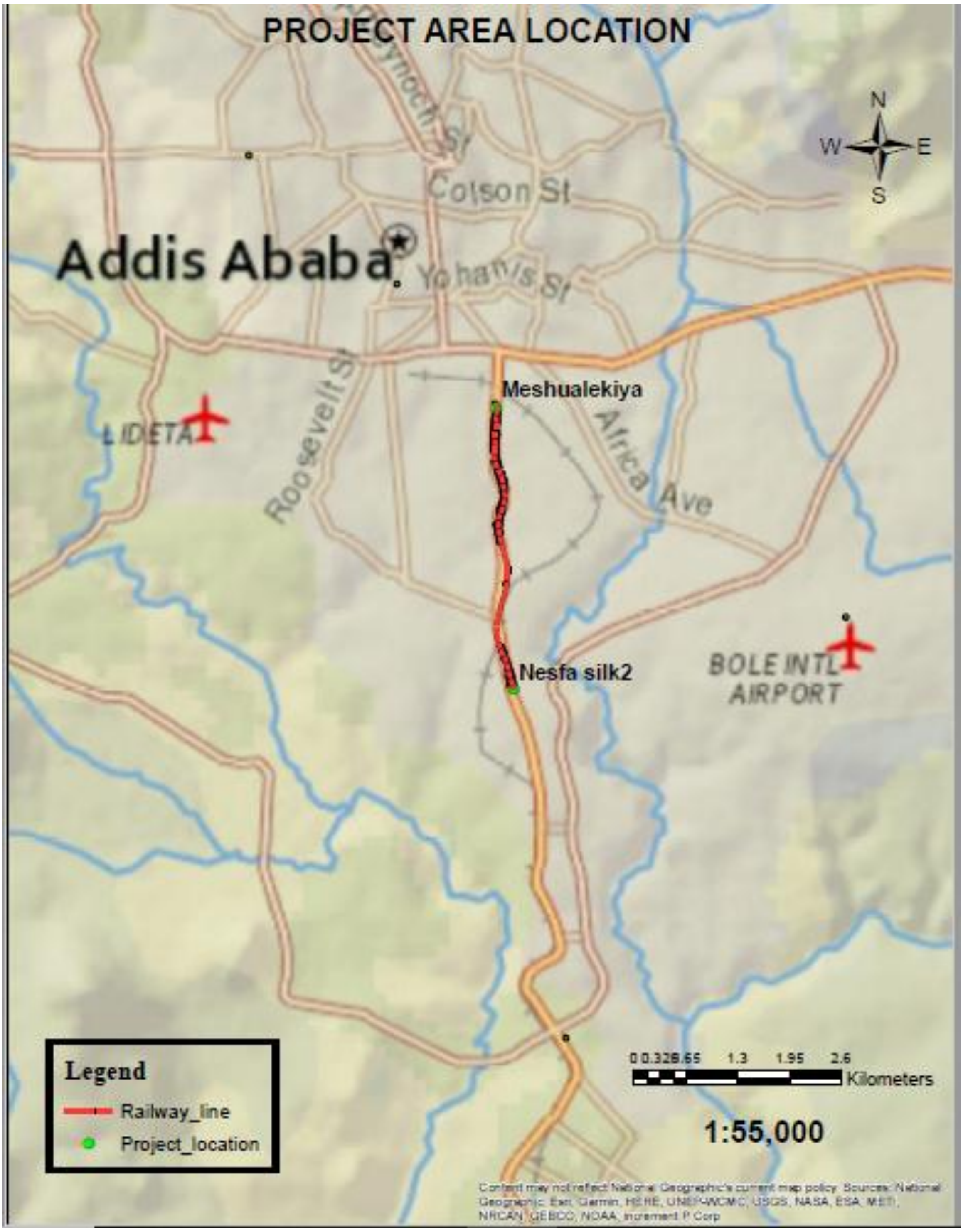


Figure 3.10: Location of project route

3.3.1 Location of Prevalent Flooding

As mentioned in the above section the route selected for this study is the Mesualekia - Nefas silk2 route**.** Due to consultation from the ERC drainage engineer, the most prone flood point along the study area is Lancha. This area suffers from flooding especially during the rainy season. This is due to the blocked manholes which are meant to convey the storm water off the surface. Most of the manholes are blocked with rubbish and siltation.

Figure 3.11: Covered manhole entry points

Figure 3.12: Rubbish within manholes

CHAPTER 4 METHODOLOGY AND MATERIAL

Under the methodology, HY8, Hydraulic toolbox, HEC HMS and Rational method of discharge calculation were used for this particular thesis.

4.1 Data Collection

This included the data required and ways of gathering it for this particular project. The data is described as shown below;

4.1.1 Meteorological data

The daily rainfall data for the case study was obtained from the National Meteorology Agency (NMA) of Ethiopia. The maximum daily precipitation depth within the data was selected and used as an input for simulation. The 24hr duration rainfall depth used during the analysis of was for 50 years. The chosen was Addis Ababa Observatory because of the following consideration; this is because basing on the developed catchments contributing to the different weather stations using the Theisen's polygon, the case study area was found to be within the Addis Ababa observatory weather station catchment.

4.1.1.1 Meteorological region of Ethiopia

There are different meteorological regions in Ethiopia. These are; A1, A2, A3, A4, B1, B2, C and D. This particular case study area is found within Addis Ababa which is region A2.

Figure 4.1: Location of Ethiopian meteorological regions
(Source: ERA, 2013)

4.1.2 Digital Elevation Model (DEM) of the study area

A Digital Elevation Model (DEM) is a specialized database that represents the relief of a surface between points of known elevation. The elevation is in form of grid of square cell. DEM is important to know the existing ground surface area flow of water, direction of flow and to identifying and selecting catchment locations. As per this case study, a USGS 12.5m x 12.5m is available on Alaska website. The particular study area was clipped out using ArcGIS 10.4 data management tools.

4.1.3 Data preparation

This involves estimating missing data, checking for its consistency as well as data disaggregation. The appropriate methods for the above three cases are briefly described as below;

4.1.3.1 Estimating missing data

This is done mainly in order to fill up missing data required for hydrologic modelling. This is because the datasets received from the national meteorological agency had missing records that could affect the results obtained hence influencing the design of the different storm water management structures. Therefore, the normal ratio method was adopted for filling in the missing daily rainfall data.

4.1.3.2 Testing for consistency

In order to carry out any statistical analysis for hydrological modeling, a data consistency check is required. A Double Mass Curve method was adopted for this check. In this particular study, the rainfall data obtained from the NMA was for only 5 stations within my study area and it was assessed to find out the consistency check. However, basing on the proximity of the different stations and my study area, Addis Ababa Observatory station was chosen. This means its data was adopted for further analysis.

4.1.3.3 Data disaggregation

This involves dividing of the rainfall data into shorter time interval and the common method used is the mass curve method. However, the rainfall data obtained from NMA was already recorded as daily rainfall data hence there was no need for further data disaggregation.

4.2 Rainfall Frequency Analysis Methods

This particular section involved the different ways of carrying out frequency analysis, and goodness of fit tests for the data.

4.2.1 Intensity-Duration-Frequency (IDF) Curves

This refers to the graphical representation of rainfall intensity, duration, and frequency. As per this study, the Log Pearson type III distribution was used for frequency analysis for the different ARI of 2, 5, 10, 25, 50 and 100-year. Under this, an already existing IDF for the case-study area was used. The number of years used for developing that IDF were 57. However, an IDF was also developed for the case-study area by using rainfall data obtained from Addis Ababa observatory station as shown in the Appendix 2.

4.2.2 Alternating Block Method

This was used to develop a hyetograph for the different return periods of 10, 25, 50 and 100-year rainfall intensity from an IDF curve which acted as an input in the modeling software for further analysis. The steps used to obtain the design storm hyetograph were as follows (Chow, 1988).

- Different design return periods were selected.
- Rainfall intensity was read from an already developed IDF curve corresponding to the calculated time of concentration for the delineated catchment.
- The corresponding precipitation depth was obtained, P as the product of the intensity read above and duration from the following equation;

$$P=i*T_d \qquad \textbf{4-1}$$

Where;

P=rainfall depth (mm)

i=intensity from the IDF curve (mm/hr)

T_d=duration time (hr)

- The differences between successive precipitation depths which gave the amount of precipitation to be added for each additional unit of time were obtained.
- The precipitation increments were recorded into a time sequence with the maximum intensity occurring at the center of the required duration.

- The remaining precipitation depth are arranged in a descending order alternately to the right and left of the central in-order to obtain the desired design precipitation hyetograph.

4.3 Catchment Area Delineation

In a hilly landscape, topographic ridgelines serve as natural drainage boundaries. A geographical area separated by surrounding ridges is a hydrological unit defined as a watershed. Normally, the entire surface area contributes its runoff to a common point (Manoj, 2015) and that common point is normally located at the lowest point of the catchment. The catchment characteristics vary because of the physical attributes within the environment like the topographic and climatic conditions (Manoj, 2015). As per this thesis, ARCGIS together with the embedded software like ARCSWAT and HEC-GEOHMS were used for catchment delineation. A DEM of 12.5mby 12.5m for the case study area was selected and used for the delineation. The hydrologic modeling tools in the ArcGIS Spatial Analyst extension toolbox were used for describing the physical components of the DEM and they were also used in identifying sinks, determining flow direction, calculating flow accumulation, delineating watersheds, and catchment area selection. The delineated catchments are shown in the result section of this report.

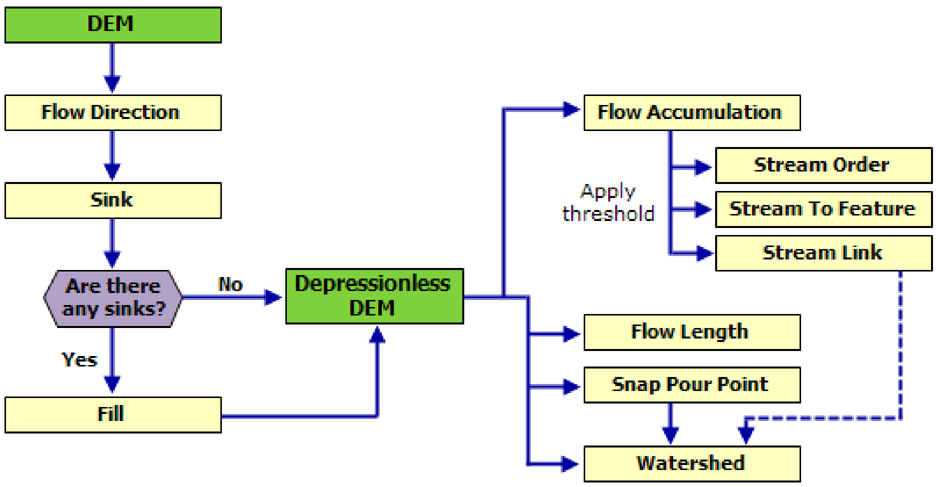


Figure 4.2: Hydrological modeling flowchart

4.3.1 Catchment characteristics

This describes the features that make up a delineated watershed and its parameters.

Drainage Area

This refers to the calculated area that makes up a catchment or watershed. It is a measure of the catchment size and a clear indication of the potential runoff obtained from the received precipitation.

Catchment length

This is the distance measured along the main channel from the watershed outlet to the basin divide. It's measured along the principal flow path. The watershed length helps in the calculation of travel time, lag time and time of concentration.

Catchment Slope

This normally shows the rate of change of elevation with respect to distance along the principal flow path. The magnitudes of the floods are normally as a result of the magnitude of the runoff generated throughout the entire watershed and slope is a main factor contributing to the momentum.

$$\mathbf{S = \frac{\Delta E}{L}} \text{...} \qquad \mathbf{4\text{-}2}$$

Where;

ΔE - difference in elevation

L - Hydrologic length of the flow path

River length

This refers to the hydrological distance measured along the main channel from the watershed outlet to the end of the channel.

River slope

The river slope is mainly responsible for the flow velocity as well as the local losses in drainage channels. If the channel slope is not uniform, a weighted slope may provide an index that reflects effect of slope on the hydrologic response of the watershed. This slope is computed by the equation below;

$$\mathbf{S_C = \frac{\Delta E_C}{L_C}} \text{...} \qquad \mathbf{4\text{-}3}$$

Where;

ΔE is the difference in elevation between the points defining the upstream and downstream of the channel, Lc is the horizontal length of the channel between the same two points.

The obtained catchment parameters are shown in the result section.

4.4 Goodness of Fit test

A goodness of fit test was done using three different methods to determine whether the sample data fits the chosen distribution. The commonly used methods included; Anderson-Darling, the Kolmogorov-Smirnov, and the Chi-Squared tests (Anteneh, 2015). In the tests, a parameter or statistic unique to each method was calculated for the required distribution types and these distributions were ranked basing on their parameter values.

Easy Fit 5.6 Professional software was used for this analysis and the common probability distributions used in hydrological modeling indicated a good fit for the dataset. Basic idea behind goodness of fit tests is that the "distance" between data and the distribution under test is measured; and is compared to a certain threshold value. If this distance (called "test statistics") is lower than threshold (critical value), distribution is considered as "good". The results are shown in the results section.

4.4.1 Reasons for selecting Log Pearson type 3

- The method is based directly on recorded precipitation data in the region.
- The method is easy and quick to apply.
- It is among the best proposed frequency analysis methods by most design standards including the Ethiopian Drainage Manual.

4.5 Data entry

This included the required data for this thesis.

4.5.1 Rational formula

The following is the application of the rational method to estimate peak discharges for catchments less than $0.5km^2$. The data required is as described below;

a) Site data

Regarding each inlet of manhole, the catchment area, slope, longest path is obtained from the delineation done in ArcGIS for this particular study area.

b) Land use type

Based on a field visit and with the help of Google Earth Pro software, the catchment area was divided into three different groups for easy computation of the runoff coefficient, C.

These included pavement (asphaltic), paved, unpaved (commercial areas) and the green area. The divided sections are shown in Appendix 3.

c) Runoff coefficient(C)

These values were obtained from the table for streets pavement (asphaltic) which is 0.95, for drives and walks paved, C = 0.85 and the remaining part of the catchment areas are out of the main road (commercial and residential area). The above values were chosen in-order to design for the worst scenarios because high runoff coefficients lead high discharges. The table for turnoff coefficient is shown in Appendix 4. The following methods were used;

Field visit; during the visit, the greatest part of the catchment areas was impervious with a few sections being pervious. The impervious areas are covered by roofs, asphaltic road, cobble stones and concrete.

Google Earth Pro; using this software, the delineated catchments were overlaid in Google Earth Pro software in-order to obtain the different land uses. This helped in the calculation of the average weighted runoff coefficient, C_w for the different land uses including pervious area, C = 0.3 (for green areas) because most of the runoff ends up infiltrating into the ground hence choosing a lower boundary value, and roads and residential/commercial roofs, C = 0.95 (ERA manual, 2013). This is because the runoff coefficient represents the runoff to rainfall ratio and it also represents the interaction of many factors which include the storage of water in surface depressions, infiltration, antecedent moisture conditions, ground cover, ground slopes and soil types.

Table 4-1: Coefficients for composite runoff analysis

Surface	Runoff Coefficients
Street: Asphalt	0.70-0.95
Concrete	0.80-0.95
Drives and walks	0.75-0.85
Roofs	0.75-0.95

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

d) Time of concentration (T_c)

Different values were calculated using the TR55 equations for each catchment delineated in ARCGIS and the lag time was obtained using $0.6*T_C$. This is because using simplified equations like the Kirpich equation can lead to common errors such as obtaining short time of concentration, particularly when the average basin slope varies significantly from the mean channel slope as in steep mountainous areas. Neglecting the overland flow time can also dramatically shorten the time of concentration thus increasing the design peak runoff (ERA, 2013). Also, this method (Kirpich) was developed from data for six agricultural watersheds in Tennessee, USA (ranging in size from 0.4 ha to 45 ha), with well-defined channels and slopes ranging from 3% to 10% (Viessman and Lewis 1996). The calculated time of concentration are shown as below;

Table 4-2: Catchment time of concentration

Catchment Point	Time of concentration, T_c (min)	Lag time (min)
WS1	11	7
WS2	15.00	9
WS3	37.80	23
WS4	31.00	19
WS5	38.68	23

e) **Determine Rainfall Intensity**

The rainfall intensity was used to obtain an average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period corresponding to the delineated catchments. This was calculated from an already developed IDF curves for rainfall region A2 in Ethiopia basing on the already calculated time of concentration.

f) **Determine peak Discharge (Q)**

This was calculated using a combined equation for different return periods.

$$\mathbf{Q_{peak} = \frac{C_f CIA}{360}} \qquad \textbf{4-4}$$

The results for discharges using this method are shown in the results section.

4.5.2 Hydrological analysis

This was done in-order to ascertain the flows from the delineated watersheds using the following methods;

4.6 The Rational Method

This is a popular approach for determining the peak runoff rates from small catchment areas less than 80 hectares. It was used for estimating discharges. The runoff coefficients for the study area was obtained from AACRA.

$$\mathbf{Q = \frac{CIA}{360}} \text{..} \quad \mathbf{4\text{-}5}$$

Where;

Q = Peak flow in m^3/s, C = Dimensionless runoff coefficient assumed to be a function of the watershed and often the frequency of the flood being estimated, I = rainfall intensity for the design storm (mm/hr), and A = catchment or drainage area (hectares).

In order to use the Rational Method, an average recurrence interval (ARI) was determined. This helped in estimating how often a particular event can reoccur on average. For example, an ARI of 1 in 50 years means that a particular storm event is likely to occur on average once in fifty years. The average intensity (mm/h) will determined by first identifying the necessary ARI and also calculate the time of concentration. Once the ARI and time of concentration has been determined it is possible to determine the intensity. Higher intensity storms normally require modification of the coefficient. This is mainly because infiltration and other losses which have proportionally smaller effect on runoff. The adjustment of the Rational Method for use with major storms was made by multiplying the right side of the rational formula by a frequency factor C_f. The rational formula now becomes (Kassahun, 2013).

$$\mathbf{Q = \frac{C_f CIA}{360}} \text{..} \quad \mathbf{4\text{-}6}$$

The different C_f values are listed below and the product of C_f and C shall not exceed 1.0.

Table 4-3: Frequency factors for rational formula (ERA MANUAL, 2013)

Recurrence interval (years)	C_f
5	1.0
10	1.0
25	1.1
50	1.2
100	1.25

(Source: ERA, 2013)

However, in cases where the catchment areas are greater than 0.5km^2, a SCS method in HEC HMS was used to determine the discharge.

4.7 Modeling using HEC HMS

This was used when determining the flows for catchment areas greater 0.5km^2. The first step in the application of HEC-HMS was defining the basin area and sub-basins, a stream network, and diversions and junctions. Under this particular modeling, different methods are embedded within the software and these include;

Transformation method

The Soil Conservation Service (SCS) dimensionless unit hydrograph procedure was used in HECHMS as a transformation method. The dimensionless unit hydrograph used by the SCS was developed by Victor Mockus and was derived based on a large number of unit hydrographs from basins that varied in characteristics such as size and geographic location.

Loss method

In this particular case study, the soil conservation service curve number method was used. Soil conservation Service (SCS) model has been applied widely during the estimation of flood discharge mainly from small watershed. The right curve number for the catchment of interest was obtained which defined the runoff potential. Much care was taken during the selection process of the curve numbers. The determination of the sub-basin percentage contributing to the land use was specified using the existing land Google earth program and field visits.

Routing method

In HECHMS, different routing methods are embedded in it. However, in this particular catchment, routing wasn’t necessary simply because there were no sub basins.

4.7.1 Calibrated Parameter and Hydrograph

There is a good fit of observed flow at Akaki station and good match between the simulated and observed flow was obtained. The hydrograph extracted from HEC-HMS is shown in Figure 4.3. It can be seen from the hydrograph that the model reasonably captured the peaks, as the difference between the simulated and observed peaks is negligible though with slight under estimations or overestimation in some peaks especially between 11 to 15 January. At the start, the model was warming (the process of simulation before it stabilizes). The rising and recession limbs were well captured reasonably well by the model, which indicated that the selected parameters and methods were appropriate for the catchment.

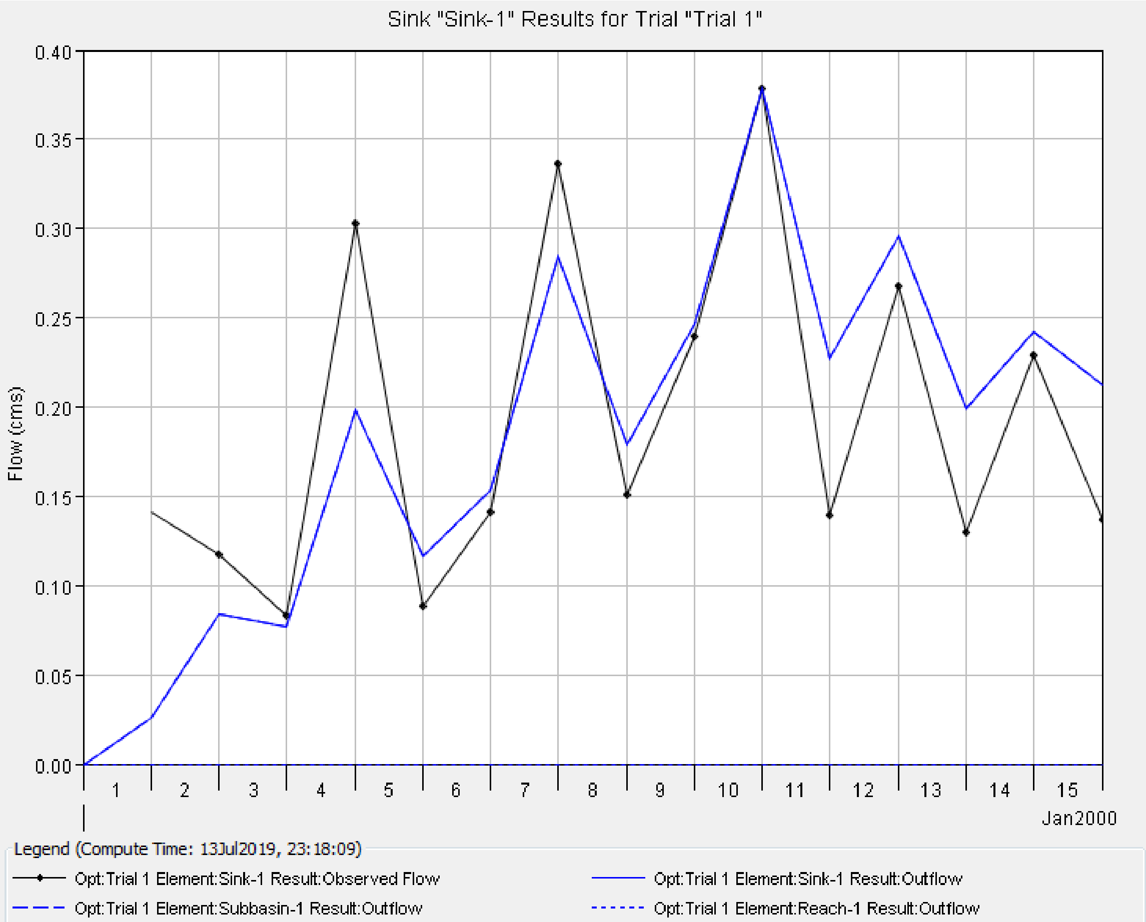


Figure 4.3: Model out showing simulated and observed flow at the outlet (HEC-HMS)

4.7.1.1 HEC-HMS Model Efficiency

The model efficiency was assessed using Nash-Sutcliffe (NS) efficiency and NS found to be 0.61. According to Zhong and Dutta (2015), the value of NS greater than 0.6 indicates appropriate agreement between simulated and observed data. However, the model slightly overestimated the cumulative volume.

4.8 Hydraulic modeling

The entire drainage system network of the case study area consists of curb-opening inlets made of concrete and gullies which act as collection for the runoff from the carriageway. These convey the flow into longitudinal pipe conduits which act as carriers. The flow through these pipe conduits ends up in the cross-drainage structure (culvert) at Lancha. However, basing on the numerous engagements with in charge at Ethiopian Railways Corporation regarding the drainage system network for the Railway line, there was no information regarding the interaction of the highway drainage with AALRT system especially for the on-track drainage. This may be because the information isn't available or they just didn't want to avail it to the researcher.

4.8.1 Modeling using Hydraulic tool box

Field measurements of longitudinal side drains and culverts in the project area was carried out. This was done to ascertain the hydraulic dimensions of the existing drainage structures before using them in hydraulic modeling. For underground drainage structures that may not be accessed during the field visits, the design technical drawings were used in ascertaining their structural sizes on an 'as designed' principle of analysis.

The procedure to be followed when using hydraulic toolbox is briefly described below (Hydraulic toolbox users' manual);

4.8.2 Modeling using HY8 software

This was used in the analysis of the cross-culvert performance. From the field measurements, the dimensions of the existing culvert were measured and inserted in the software for further analysis. The hydraulic modeling competence of all the existing structures within the case study were done as below;

4.8.2.1 Side drainage network assessment

These were found on either direction of the roadway. For easy calculation and assessment, they were divided into eight different sections. This is because there was no design information for the drainage from AACRA. The normal spacing between the different inlet points is 25m. However, these existing side drains were assessed basing on the data obtained from the field visits. This included their capacity to convey the accumulated runoff from the carriage way of 7.7m width on either direction or the corresponding watershed discharge along the carriage way to the existing drainage structure. The side drains of much interest were those at the upstream side of the carriage way. During the field visits, pipe sizes of 900mm in diameter were seen and recorded. For the existing longitudinal side drains, FHWA hydraulic tool box was used for the analysis.

4.8.2.2 Cross drainage structure

There was only one cross drainage structure which was a culvert located at Lancha. The hydraulic assessment was based on the measured dimensions from the field visit which might not have been very accurate. Using the measuring tape, the dimensions of the openings of each of this structure were measured and recorded. HY8 software was used for the cross-drainage structure in order to assess its hydraulic competence.

4.8.2.3 On track / Railway drainage system network

It comprises of both transverse and longitudinal drains that serve a similar purpose of draining flow from the tack in the shortest time possible in order to prevent destruction of other layers. In a typical drainage system, flow enters the collector pipes and is conveyed by carrier pipes. Collector pipes are laid within the ballast and are connected to a cross drainage carrier pipes to convey the flow to the existing longitudinal side drains. The main factor to focus is having a sufficient slope because it enables a smooth flow of the runoff. The transverse drains are mainly used to convey flow from the ballast bed into the longitudinal drains. The conveyed flow from the longitudinal drains ends up in the municipal storm water pipe network. From the field visit of the study area, the longitudinal drains were open drains with concrete covers. The existing transverse drains were not visible and it was hard to ascertain their design information because the responsible bodies claimed the contract is an EPC which is ongoing since the project is in its first phase, the researcher couldn’t be given the required information. Due to lack of design information for the on-track drains especially transverse drains, the researcher suggested the size of such drains basing on the calculated on-track flows. The flow to the longitudinal drains is what comes from the transverse drains.

Transverse aggregate drains

Basing on the calculated on-track flows using the rational method as shown in Appendix 5, it was seen that obtained flows were very minimal and aggregate drains were recommended.

Aggregate drains were sized by computing surface runoff that drains through the ballast and using the accumulated runoff to ascertain an optimal size of the aggregate drains. The capacity of the aggregate drains was computed using the Darcy's equation as shown below;

$$\mathbf{Q_1 = k * i * A}\text{...} \quad \mathbf{4\text{-}7}$$

Where:

Q_1 = flow (m^3/s), k = permeability of the aggregate, i = hydraulic gradient or slope, and A = Cross-section area (m^2).

The cross-sectional area used was based on the measured surface of the rail track bed. The permeability of the ballast aggregates from rocks mixed with scoria and Pumice was 0.25m/s. This was for aggregate sizes ranging between 28mm to 37.5mm after conducting numerous sieve tests. The above stated equation can be adjusted in-order to size of the aggregate drains as shown below:

$$\mathbf{Q_1 = K * A}\text{...} \quad \mathbf{4\text{-}8}$$

Where; K is a velocity constant that varies with the slope of the aggregate drain as shown in the table below. The two commonly used aggregate sizes are of 20mm and 53mm.

Table 4-4: Values of K=k*i (m/s) for various slopes

Slope	K = k i (m/s)	
	20 mm	53 mm
1 in 100	0.00150	0.0040
1 in 200	0.00075	0.0020
1 in 300	0.00050	0.0013
1 in 400	0.00038	0.0010
1 in 500	0.00030	0.0008

(Source**:** ARTC, June 2013)

Basing on the above, the computed Q_1 as shown below was used for sizing the aggregate drains by use of the above equation. The aggregate size used was 28mm and its permeability was 0.25m/s. Basing on the obtained slopes from a google earth elevation profile, a slope of 0.005 was chosen by the researcher for such drains. Basing on Table 4.5 below, the existing track parameters belongs to class 2 track type and this led to the design flow for 25-year return period.

Table 4-5: Typical recurrence intervals for various track classes

Track Class	Recurrence Interval
Class 1 and higher	50 years
Class 2	25 years
Class 3	10 years
Class 4	5 years
Class 5	5 years

(Source: ARTC, June 2013)

Longitudinal drains

Basing on the field work, the existing longitudinal drains were found to run throughout the entire stretch of railway line. The existing sizes of the longitudinal drains were 500mm by 800mm which are covered by concrete covers. The concrete covers help in keeping away the foreign material especially solid waste from being disposed into the drains which could have a permanent impact in the long run. Their hydraulic competence was assessed by use of Hydraulic toolbox as shown in the Appendix 6. Below are the general parameters which were used during the modelling software;

4.8.2.4 Slope, S

An appropriate slope was provided for both side drains and cross drainage structures like culverts. The proposed slopes were based on AACRA manual, railway line elevation profile obtained from google earth and ERA drainage manual. The minimum slope for the culvert should be 0.5% (AACRA MANUAL). This is because flatter slopes are prone to siltation which easily hinders the flow within the structure and difficult to construct. The Pavement longitudinal slope of the existing road ranges from 0 to 5.5 % as obtained from the google earth elevation profile. However, the slope adopted for the channel was 4%, and 5% for the culvert during the analysis in HY8. The slope used for side drains for the roadway and longitudinal side drains for the on-track was based on the google earth elevation profile and the contours generated as shown in the Appendix 7.

4.8.2.5 Manning's roughness coefficient, n

This is defined as a parameter representing the channel roughness and flow resistance. Manning's n-values are often selected from tables, but can be back calculated from field measurements. The factors affecting this coefficient are cross sectional geometry and boundary roughness surface roughness, vegetation cover, channel irregularity, channel alignment, size and shape of the channel (V.T CHOW, 1959). The coefficient roughness adopted was for concrete because it's the material which was used in this study for the drainage structures. This coefficient has an effect on the flow rate whereby the flow rate decreases as the manning's roughness coefficient increases and vice versa. The adopted manning's value was 0.08, (ERA, 2013) because the channel had light brush and trees and 0.012 for concrete for use in HY8.

4.8.2.6 Design flow

Different design flows corresponding to different return periods were used for sizing the drainage structures. This was done in the hydraulic modeling software. This was done by inserting in the dimensions of existing drainage structures to check their competence in conveying the obtained design flows. The design return period was based on the design storm frequency by geometric design criteria, (ERA, 2013) shown as Appendix 8. Below are the photos of the existing cross drainage structure and side drains;

For ordinary conditions, storm drains are always sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. The Manning's formula is recommended for capacity calculations (ERA, 2013).

CHAPTER 5 RESULTS, ANALYSIS AND DISCUSSIONS

This chapter shows the results obtained from the methodologies detailed in the previous chapter which are presented in order of their appearance. Furthermore, short discussion as to what the results meant is done here. This section discusses the specific objectives in their order.

5.1 Data Collection

Daily precipitation depth data for five different stations was obtained from the National Meteorological Agency of Ethiopia. The stations included; Addis Ababa Observatory station, Kality, Akaki, Bole international airport and Yekatit 23. The data obtained was arranged in an appropriate order for easy and quick analysis. Different tests were done on this sample to avoid statistical errors like the consistency test and goodness of fit test. As per this particular case study, the annual maximum precipitation data for a period of 50 years with a mean of 54.33mm and standard deviation of 16.74mm was used. The data used was from the Addis Ababa Observatory station. This is because the Theisen polygons developed in ARCGIS showed that case study area lies within the contributing area for that particular station.

5.2 Goodness of fit test

A goodness of fit test was carried out in order to check on the different distributions whether they fit the sample rainfall data from Addis Ababa Observatory station. This was done by the use of Kolmogorov-Smirnov, Anderson-Darling and Chi-Squared. The analysis was done using the Easy-fit professional software (ERA, 2013).

Table 5-1: Goodness of fit test results

Distribution	Kolmogorov-Smirnov	Anderson-Darling	Chi-Squared
Log Pearson 3	0.08612	0.43305	4.3243
Gumbel	0.07661	0.39068	2.8444

From the easy fit professional software, all the distributions fit the dataset analyzed and their graphs are shown in the Appendix 9; these graphs show how the distributions fit our data sample.

5.3 Frequency analysis

During the frequency analysis, different approaches were used in order to determine the best and appropriate methods for it. The distribution used included; Extreme value 1, Log Pearson type 3, Log Normal, and Normal distributions. Below is the summary of the design rainfall depth (mm) obtained from the above distributions; the values below are maximum between the calculated precipitation depth and those shown in the ERA, 2013 as shown in Appendix 10.

Table 5-2: Summary of design rainfall depth (mm)

Return period (years)	Extreme value type 1	Log Pearson type III	Log Normal
2	52	52	52
5	66	67	67
10	76	76	76
25	89	88	88
50	98	97	96
100	107	106	105

Based on the values in the table, the rainfall depth for Extreme value type 1 and Log Pearson III have a slight difference which makes them almost similar. According to the Ethiopian Drainage manual (ERA, 2013), Extreme value type 1 and Log Pearson type 3 are the suggested appropriate methods used in frequency analysis. Therefore, as per this project, Log Pearson III was considered for further frequency analysis because it has been widely and frequently used in hydrology and for hydrologic frequency analyses since the recommendation of this distribution by U.S. federal agencies (Amin, 2016).

Table 5-3: Using the Extreme value type 1 distribution

S/N	Frequency factor, K_T	T(Years)	Design Rainfall Depth, (mm)
1	0.164	2	52
2	0.719	5	66
3	1.304	10	76
4	2.043	25	89
5	2.591	50	98
6	3.135	100	107

Table 5-4: Using Log Normal distribution

S/N	T(Years)	P=1/T	P^2	In(1/P^2)	w	z	Rainfall Depth, mm X_T	Y=Anti Log X_T
1	2	0.500	0.25000	1.38629	1.1774	0.0000	1.72	52
2	5	0.200	0.04000	3.21888	1.7941	0.8415	1.83	67
3	10	0.100	0.01000	4.60517	2.1459	1.2817	1.88	76
4	25	0.040	0.00160	6.43775	2.5373	1.7511	1.94	88
5	50	0.020	0.00040	7.82405	2.7972	2.0542	1.98	96
6	100	0.010	0.00010	9.21034	3.0349	2.3268	2.02	105

Table 5-5: Using Log Pearson type III distribution

S/N	T(Years)	Skew Coefficient, Cs	K_T	Rainfall Depth, mm X_T	Y_T= Anti LogX_T
1	2	0.0440	-0.0075	1.71478	52
2	5	0.0440	0.8394	1.82529	67
3	10	0.0440	1.2864	1.88362	76
4	25	0.0440	1.7660	1.94620	88
5	50	0.0440	2.0773	1.98683	97
6	100	0.0440	2.3585	2.02352	106

5.4 Catchment delineation

A catchment is an area of land that drains water into a specific pond, stream or river for which it is named. Catchments have boundaries, called divides, located at relatively high elevations or ridges. Different catchments were obtained by using HEC GeoHMS which is a geospatial hydrology toolkit within ARCGIS software. In my case study, five catchments were obtained. This was done after a site visit where the different locations of the inlet curbs were recorded using a GPS together with their current condition from the time of their construction. In this particular project area, five different catchments were delineated as shown below;

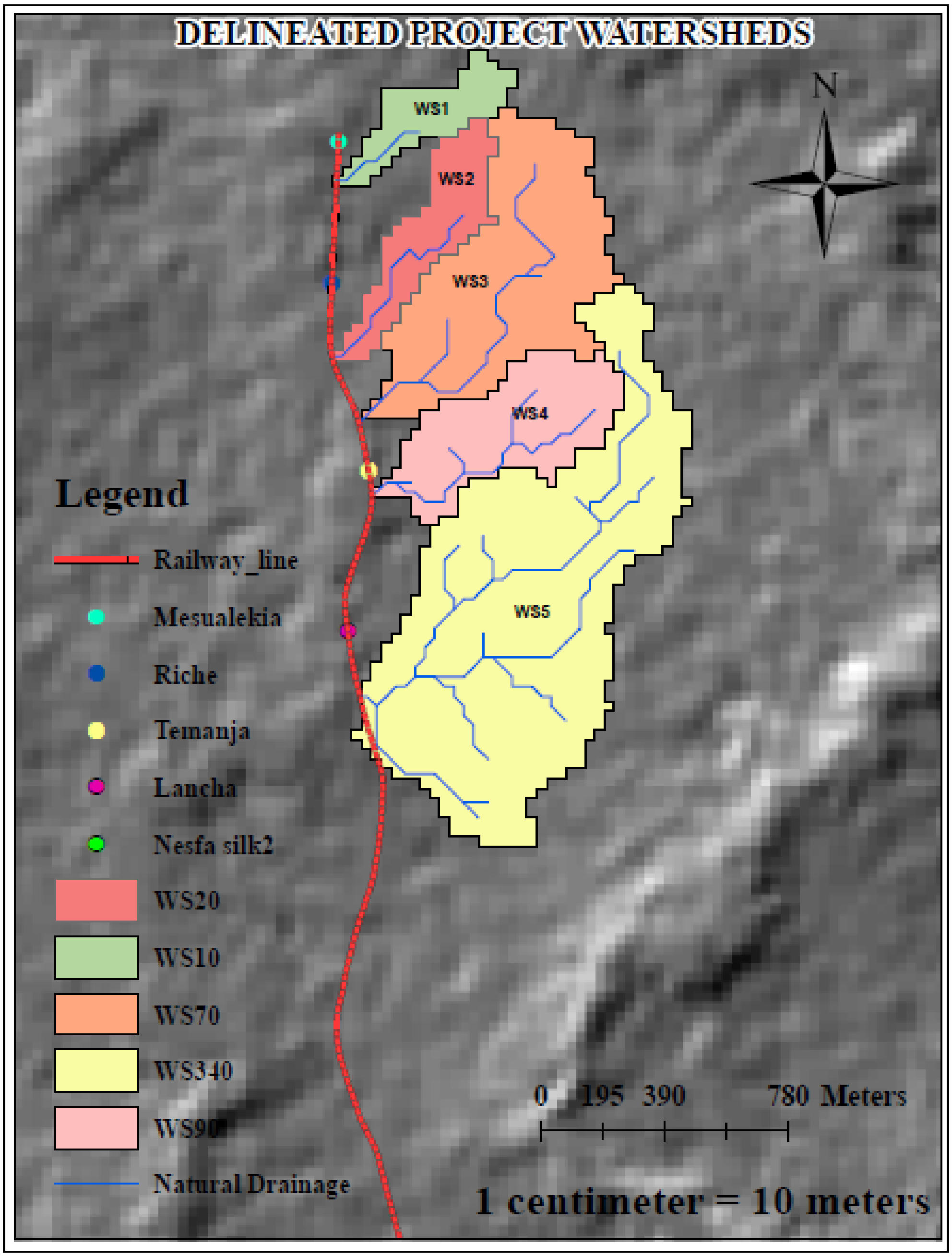


Figure 5.1: Delineated project watersheds

The flood prone areas are shown below;

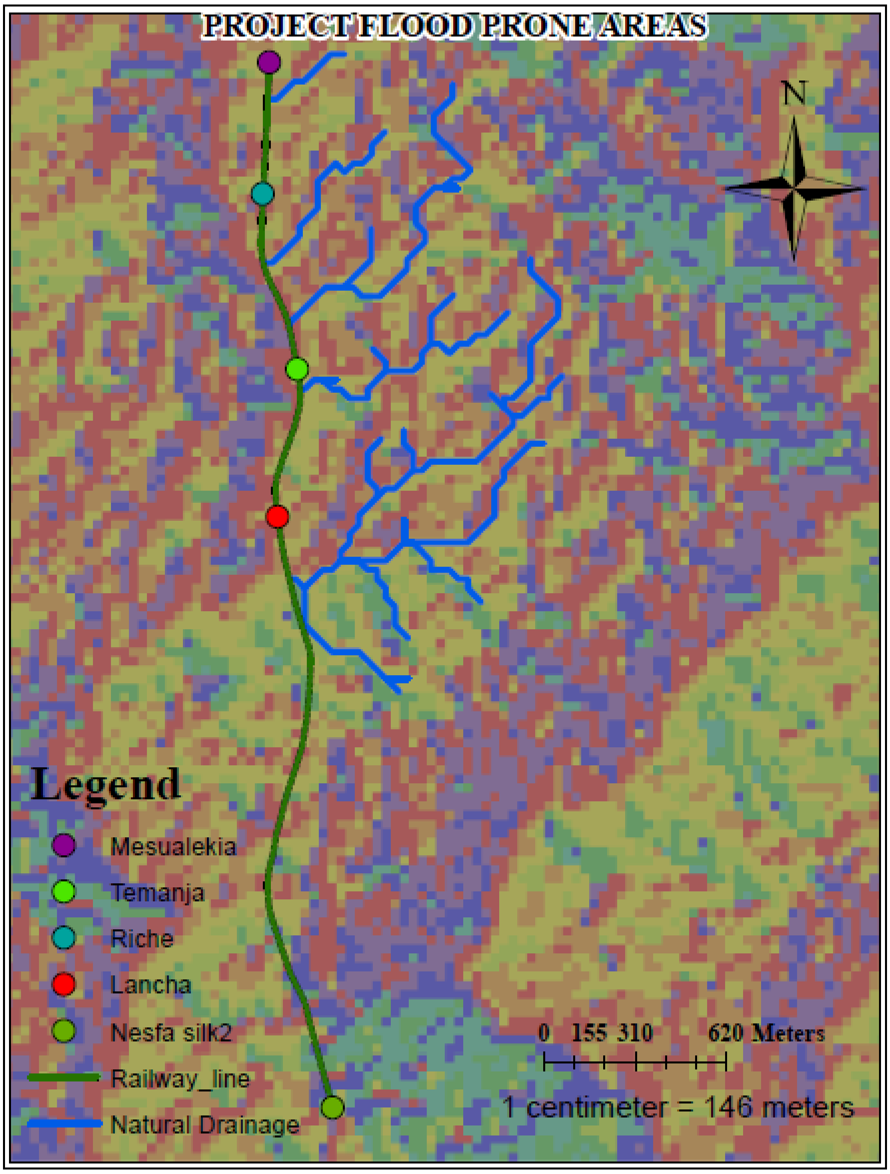


Figure 5.2: Flood prone areas

5.4.1 Catchment characteristics

This section shows the obtained results which makeup a watershed and its parameters. The obtained features include; drainage area, catchment length, catchment slope, river slope, river length. The stated parameters are were described in the methodology section.

Table 5-6: Catchment parameters

Catchment	Area (km^2)	Length(m)	Basin slope (%)	River length (m)	River slope (%)
WS1	0.0892	817.55	3.54	323.95	4.01
WS2	0.1315	1048	3.98	712.92	3.37
WS3	0.4076	1517.78	2.45	900.5	2.61
WS4	0.2216	1084.39	4.57	406.92	4.42
WS5	0.8631	1979.64	2.58	1377.69	2.86

5.5 Developing a Hyetograph

Different hyetographs were developed using an alternating block method. This was done by using the procedure described in the methodology section. From the delineation of the watershed in ARCGIS, five different catchments were developed. Four of the catchments had areas less than $0.5km^2$ whereas the other one had an area greater than $0.5km^2$. Under this section, the hyetographs developed was for watershed 5 which had an area greater than $0.5km^2$ for the different return periods of 2, 5, 10, 25, 50 and 100 years. This is because in order to use HEC HMS software to determine the amount of flow, a design hyetograph is required while for catchments with areas less than $0.5km^2$, a rational method was used for obtaining the discharge. Different values for the time of concentration were calculated using two different methods; for the catchments greater $0.5km^2$, a TR55 method was used while for those areas less than $0.5km^2$, a Kirprich method was used. Below are the hyetographs obtained using the alternating block method for watershed 5.

For a 2-year return period, Rainfall depth = 52mm

Duration (minutes)	Time(hr)	Time/24	Cumulative depth (mm)	Incremental depth (mm)	Precipitation (mm)
10	0.16667	0.00694	26.76	26.76	2.47
20	0.33333	0.01389	38.37	11.61	3.78
30	0.50000	0.02083	44.61	6.25	26.76
40	0.66667	0.02778	48.40	3.78	11.61
50	0.83333	0.03472	50.86	2.47	6.25
60	1.00000	0.04167	52.56	1.69	1.69

For a 5-year return period, Rainfall depth = 67mm

Duration (minutes)	Time(hr)	Time/24	Cumulative depth (mm)	Incremental depth (mm)	Precipitation (mm)
10	0.16667	0.00694	34.52	34.52	3.19
20	0.33333	0.01389	49.49	14.97	4.88
30	0.50000	0.02083	57.55	8.06	34.52
40	0.66667	0.02778	62.43	4.88	14.97
50	0.83333	0.03472	65.62	3.19	8.06
60	1.00000	0.04167	67.81	2.19	2.19

For a 10-year return period, Rainfall depth = 76mm

Duration (minutes)	Time(hr)	Time/24	Cumulative depth (mm)	Incremental depth (mm)	Precipitation (mm)
10	0.16667	0.00694	39.48	39.48	3.64
20	0.33333	0.01389	56.60	17.12	5.58
30	0.50000	0.02083	65.81	9.21	39.48
40	0.66667	0.02778	71.39	5.58	17.12
50	0.83333	0.03472	75.04	3.64	9.21
60	1.00000	0.04167	77.54	2.50	2.50

For a 25-year return period, Rainfall depth = 88mm

Duration (minutes)	Time(hr)	Time/24	Cumulative depth (mm)	Incremental depth (mm)	Precipitation (mm)
10	0.16667	0.00694	45.60	45.60	4.21
20	0.33333	0.01389	65.37	19.78	6.45
30	0.50000	0.02083	76.02	10.64	45.60
40	0.66667	0.02778	82.46	6.45	19.78
50	0.83333	0.03472	86.67	4.21	10.64
60	1.00000	0.04167	89.56	2.89	2.89

For a 50-year return period, Rainfall depth = 97mm

Duration (minutes)	Time(hr)	Time/24	Cumulative depth(mm)	Incremental depth (mm)	Precipitation (mm)
10	0.16667	0.00694	50.07	50.07	4.62
20	0.33333	0.01389	71.78	21.72	7.08
30	0.50000	0.02083	83.47	11.69	50.07
40	0.66667	0.02778	90.55	7.08	21.72
50	0.83333	0.03472	95.17	4.62	11.69
60	1.00000	0.04167	98.34	3.17	3.17

For a 100-year return period, Rainfall depth = 106mm

Duration (minutes)	Time(hr)	Time/24	Cumulative depth(mm)	Incremental depth (mm)	Precipitation (mm)
10	0.16667	0.00694	54.48	54.48	5.03
20	0.33333	0.01389	78.12	23.63	7.70
30	0.50000	0.02083	90.83	12.72	54.48
40	0.66667	0.02778	98.54	7.70	23.63
50	0.83333	0.03472	103.56	5.03	12.72
60	1.00000	0.04167	107.01	3.45	3.45

The combined hyetographs for different return periods for the catchment that was analyzed in HEC HMS is shown in Appendix 11.

5.6 Hydrological modelling

Different watersheds were modelled in HEC GEOHMS in order to obtain catchment parameters. Watersheds with areas >0.5km^2 were exported to HEC HMS in order to obtain the discharges well as those with areas < 0.5km^2, rational method was used to obtain their discharges. Under the HEC HMS, the SCS method was used as a loss method and the SCS unit hydrograph as a transform method.

5.6.1 Result of Discharge Obtained from Rational Method

The discharge calculated using this method was for areas less than 0.5km^2. The parameters involved in this calculation included Run-off Coefficient, Rainfall Intensity from an already existing IDF curve for this particular region. The different return period was fixed basing on ERA drainage manual. One of the most sensitive parameters is the run off coefficient which can change the value of run-off discharges at inlet of drainage structures. In this study, the run-off coefficient was calculated based on return period, slope of catchment vegetation and hydrological soil group.

The table below shows the calculated discharge.

Table 5-7: Rational method flows

Catchment	2 Year	5 Year	10 Year	25 Year	50 Year
Watershed1	1.57	1.98	2.15	2.72	3.24
Watershed2	1.85	2.36	2.63	3.32	3.97
Watershed3	3.46	4.41	5.11	6.38	8
Watershed4	1.17	1.48	1.71	2.16	2.54

5.6.2 Result of discharge Obtained from SCS Curve Number Method

As described in the methodology, for areas between 0.5km^2 to 65 km^2 SCS curve number method was used in this thesis. An SCS curve number and other parameters were used in the estimation of the discharge. Most runoff estimation techniques use the size of the watershed as the main factor. However, runoff rates and volumes normally increase with increasing drainage area. Different parameters were used in the calculation of the SCS CN which were described in the methodology earlier. The higher the CN, the higher is the runoff potential. The calculated curve number for watershed 5 was 87.74%. For each catchment, hydrological soil group, land use and land cover are indicated in Appendix 12.

The table below shows the calculated discharge.

Table 5-8: SCS method flows obtained in HEC HMS model

Catchment	2 Year	5 Year	10 Year	25 Year
Watershed5	13	17.5	20.5	24.6

The discharges obtained for the above catchments were up to a return period of 25 years. This is because in this particular case-study we only have drains as the existing drainage structures. The developed hydrographs in HEC HMS are shown in the Appendix 13.

5.7 Hydraulic modelling

Different side drains and cross drainage structures (culverts) were designed in Hydraulic tool box and HY8 respectively. Side drains were designed to convey the runoff into a cross drainage structure like culvert. For easy calculation, sections of about 250m were considered as location points for the manholes. However, the actual spacing between each manhole is about 25m-30m. Based on the side drains modelled within hydraulic tools, only one manhole was undersized which makes it more prone to flooding in case of heavy storms. This obtained manhole size from the simulated results was 1.2m instead of the existing 0.9m. The existing culvert was modelled in HY8 and the simulated results indicated that it was sufficient to convey the flow of a 50-year return period. However, flooding still occurs within this particular area. These could be the reasons causing the flooding as obtained after a field visit;

- Presence of debris, rubbish and different materials within the manholes.

- Presence of debris and rubbish at the upstream of the cross-drainage structure. This leads to massive collection of rubbish in the downstream and if it's not well controlled it can be disastrous in the long run.
- Existence of an undersized side drain located at 100m from Temanja railway station.
- Presence of silt within the manhole which hinders the drainage structure from operating at its designed capacity.
- Smaller entry width of the manhole which limits the runoff volume to be tapped especially the carriage way runoff during heavy storms.
- Some of the existing manholes have their entry points fully blocked by the rubbish or covered by concrete.
- Some of the manhole covers are widely broken, this leads to a massive entry of debris and rubbish.

With the above reasons as noted from the field visit, the infrastructure is prone to flooding especially at the low-lying area which is found at Lancha if the necessary precautions aren't put into consideration as described in the recommendations. The evidence of the above reasons is shown in the photos below;

Figure 5.3: Blocked and damaged manhole

(Source: Temanja station)

Figure 5.4: Measurement of entry width and dismantled cover of the manhole

(Source: Richie station)

Figure 5.5: Collection of rubbish in the manhole

(Source: Nefsi silk2 station)

Figure 5.6: Collection of rubbish in the upstream of existing culvert

Figure 5.7: Collection of rubbish in the downstream of existing culvert

(Source: Lancha)

Below is a summary of the results for the cross-culvert simulation in HY8; The used Crossing Discharge flow Data is defined as below;

Table 5-9: Summary of inputs in HY8

S/N	DESCRIPTION	METRIC	REMARK
1	Design flow	46.33 m^3/s	Calculated
2	Maximum Flow	59.27 m^3/s	Calculated
3	Channel type	Rectangular channel	Observation
4	Channel slope	0.004	Calculated
5	Channel invert elevation	2296.67m	Calculated basing on road profile
6	Manning's constant (channel)	0.08	ERA, drainage manual
7	Inlet invert elevation	2296.9m	Calculated basing on road profile
8	Outlet invert elevation	2294.6m	Calculated basing on road profile
9	Culvert slope	0.05	ERA, drainage manual
10	Manning's constant (culvert)	0.012	ERA, drainage manual
11	Roadway elevation	2302m	Calculated
12	Roadway surface	Paved	Observation
13	Culvert shape	Box	Observation
14	Material	Concrete	Observation
15	Culvert length	28m	Same as road width
16	Top width	28m	AACRA
17	Span	4000mm	Measured
18	Rise	3300mm	Measured
19	Bottom width of channel	7m	Measured
20	Crest Length	410m	Measured
21	Number of Barrels	1	Observation
22	Inlet configuration	Square Edge (90º) Headwall	Observation
23	Culvert type	Straight	Observation

The design flow for the cross-drainage structure (culvert) was calculated as a summation of the total flows resulting from the longitudinal side drains, carriageway runoff and flows form the delineated watersheds for a 25 year ARI whereas the maximum flow is a summation of the same flows but for a 50 year ARI.

The other different summary tables and other data used of the culvert flows are indicated in the Appendix 14.

5.7.1 Discussion of results from HY8

This particular software was used for modelling the cross-drainage structure (culvert). The headwater elevation obtained for the actual design discharge of 46.33 m^3/s is 2300.94m which is lower than the roadway elevation. This means that the culvert will be able to convey the design discharge without overtopping the roadway. Normally the roadway overtopping entails culvert headwater elevation been greater than roadway crest (Norman, 1985). The conveyed flow through the culvert barrel is either controlled by inlet or outlet conditions. This can be ascertained by the use of the Performance curve. It helps in determining the control condition that is prevalent at a particular headwater elevation with respect to discharge because of difficulty in predicting the actual condition that is governing. Basing on the obtained graphs from the software, the prevalent control condition at the design flow is the outlet control. This is because culverts with inlet control have high-velocity, low flow that is supercritical and the control section is at the upstream while those with outlet control have lower velocity, deeper flow that is subcritical and the control section is at the downstream. Since the existing condition is outlet control, the Froude number of 0.66 obtained confirms that the flow is subcritical with low velocity and deep flow. This is because Fr<1. One of the most important figures to note is the water profile of the culvert. This shows a plot of elevation against station. The station depicts the location of the culvert and its length along the entire roadway. The profile also reveals the roadway elevation. The headwater elevation of the box culvert at the design flow is 2300.94m and the tail water elevation is 2298.85m. Therefore, basing on the obtained results from HY8, the existing cross drainage structure is sufficient to convey the flow from the upstream to the downstream without overtopping the surface.

Table 5-10: Summary of Culvert Flows at Crossing Culvert

Headwater Elevation (m)	Total Discharge (cms)	Culvert Discharge (cms)	Roadway Discharge (cms)	Iterations
2296.90	0.00	0.00	0.00	1
2297.76	5.02	5.02	0.00	1
2298.26	10.03	10.03	0.00	1

Headwater Elevation (m)	Total Discharge (cms)	Culvert Discharge (cms)	Roadway Discharge (cms)	Iterations
2298.69	15.05	15.05	0.00	1
2299.09	20.06	20.06	0.00	1
2299.45	25.08	25.08	0.00	1
2299.80	30.10	30.10	0.00	1
2300.14	35.11	35.11	0.00	1
2300.49	40.13	40.13	0.00	1
2300.85	45.14	45.14	0.00	1
2300.94	46.33	46.33	0.00	1
2302.00	59.27	59.27	0.00	Overtopping

Rating Curve

The rating curve represents flow rate versus tail water elevation for the downstream channel. This helps in the computation of tail water elevation by using channel invert elevation (generally the same as the downstream invert of the culvert). A total rating curve is plotted as below;

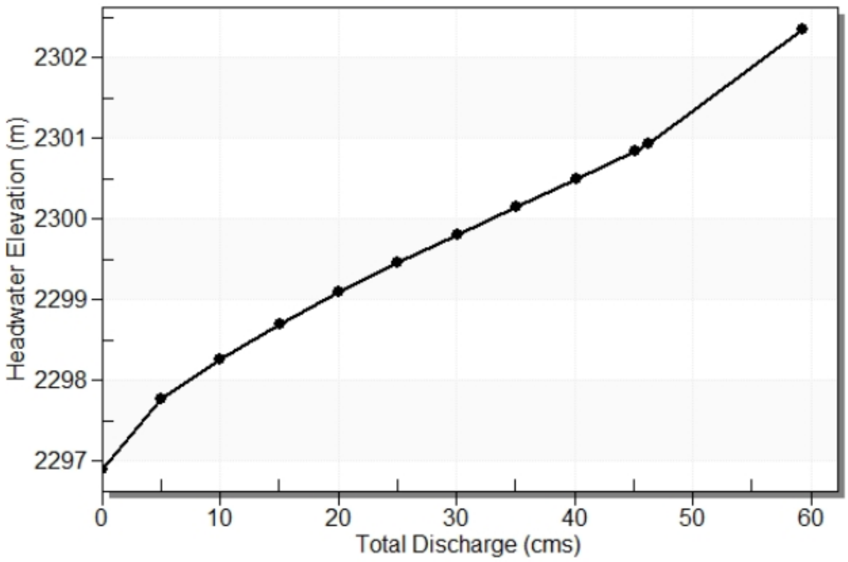


Figure 5.8: Total rating curve for the culvert

Culvert Performance Curve

A performance curve is a plot of headwater depth or elevation versus flow rate. The resulting graphical depiction of culvert operation is useful in evaluating the hydraulic capacity of a culvert for various headwaters. The culvert performance curve provides an easy way to determine the headwater elevation for any flow rate, or to visualize the performance of the culvert installation over a range of flow rates. For high discharges, the outlet control curve may have a very steep slope which means that the headwater will increase rapidly with increasing discharge. Since there is a probability that the design discharge will be exceeded over the life of the culvert, the consequences of that event should be considered. This will help to evaluate the potential for damage to the roadway and to adjacent properties. They are also useful in optimizing the performance of the culvert. Among its other uses, the performance curve displays the consequences of higher flow rates at the site and the benefits of inlet improvements.

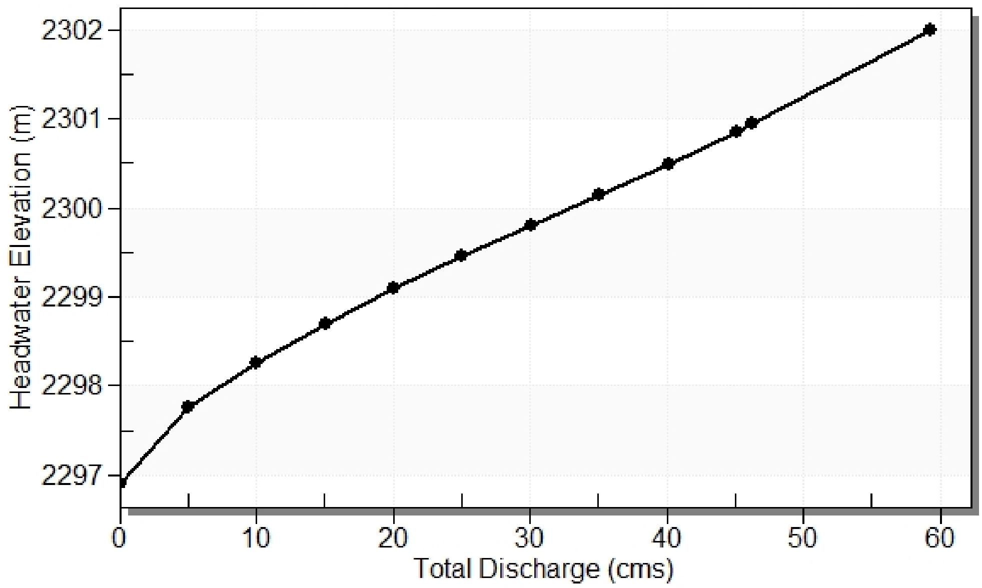


Figure 5.9: Performance curve for the culvert

Water Surface Profile Plot for Culvert

The surface profile indicates how the flow is conveyed from the upstream to the downstream within the culvert.

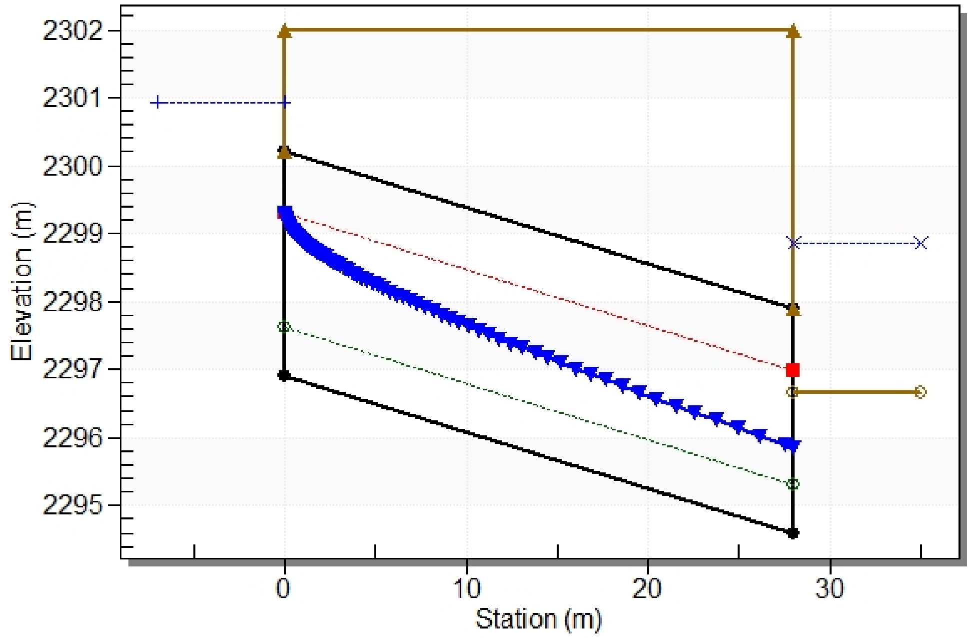


Figure 5.10: Water surface profile

Table 5-11: Downstream Channel Rating Curve

Flow (cms)	Water Surface Elevation (m)	Depth (m)	Velocity (m/s)	Shear (Pa)	Froude Number
0.00	2296.67	0.00	0.00	0.00	0.00
5.02	2297.17	0.50	1.44	195.39	0.65
10.03	2297.45	0.78	1.85	304.20	0.67
15.05	2297.68	1.01	2.13	396.39	0.68
20.06	2297.89	1.22	2.34	479.82	0.68
25.08	2298.09	1.42	2.52	557.67	0.67
30.10	2298.28	1.61	2.67	631.57	0.67
35.11	2298.46	1.79	2.80	702.46	0.67
40.13	2298.64	1.97	2.92	771.04	0.66
45.14	2298.81	2.14	3.02	837.68	0.66
46.33	2298.85	2.18	3.04	853.21	0.66

Below the rating curve developed from the above data;

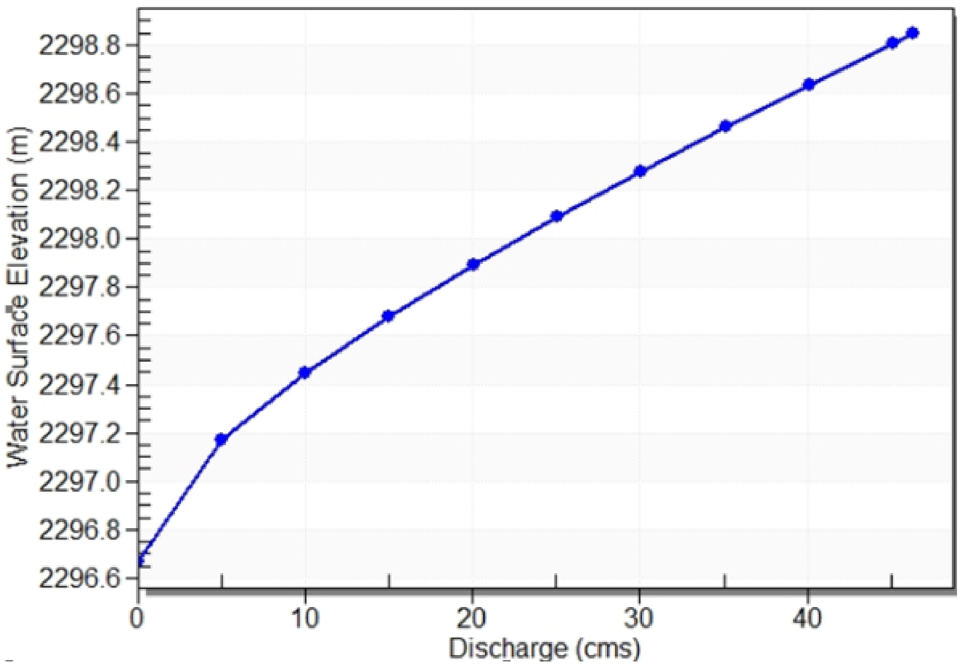


Figure 5.11: Downstream channel rating curve

5.7.2 Discussion of results from Hydraulic toolbox software

All storm drains have an outlet where flow from the storm drainage system is discharged. The discharge point can be a natural river or stream, an existing storm drainage system, or a channel which is either existing or proposed for the purpose of conveying the storm water away from the highway. Discharge should always be in the direction of flow of the river or stream. The risk of roads being washed away could be significantly reduced at a reasonable cost by identifying and rectifying weak points in the infrastructure system. Flooding is one of the main disasters that can wash away the infrastructure but also can lead to loss of human lives especially if it’s severe and also if it floods for a longer duration. This particular software was used to assess the hydraulic competence of the existing side drains along the study area. Different flow values obtained in the previous calculation techniques like rational and HECHMS were inserted into the software as shown in Table 5.13 below;

Table 5-12: Flow calculation table for side drains

									Intensity for carriage way (mm/hr)									Design Discharge for side drains				
Section	Chainage	Length (m)	Slope (%)	Slope (m/m)	Description	Road width (m)	Tc (min)	Adopted Tc (min)	5 Year	10 Year	25 Year	50 Year	Area (km^2)	Q_5	Q_{10}	Q_{25}	Q_{50}	Total flow	Design flow	Maximum Flow	Existing drainage size	Obtained size	Comment
1	0+250	250	3	0.03	Set up drains to convey storm water from the carriage way runoff and catchment 1	7.7	0.36	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	2.21	2.66	2.78	900	900	Appropriate
2	0+500	250	3.5	0.035	Set up drains to convey storm water from the carriage way runoff.	7.7	0.34	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	0.06	0.08	0.09	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe
3	0+750	250	5	0.05	Set up drains to convey storm water from the carriage way runoff and catchment 2	7.7	0.30	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	2.69	3.23	4.07	900	900	Appropriate
4	0+1000	250	4.2	0.042	Set up drains to convey storm water from the carriage way runoff and catchment3	7.7	0.32	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	5.17	6.20	7.74	900	1200	Under designed
5	0+1250	250	3.4	0.034	Set up drains to convey storm water from the carriage way runoff and catchment 4	7.7	0.34	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	1.77	2.12	2.68	900	750	Overdesigned however, for the sake of easy maintenance, provide 900mm diameter pipe
6	0+1500	250	3	0.03	Set up drains to convey storm water from the carriageway	7.7	0.36	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	0.06	0.08	0.08	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe

									Intensity for carriage way (mm/hr)									Design Discharge for side drains					
Section	**Chainage**	**Length (m)**	**Slope (%)**	**Slope (m/m)**	**Description**	**Road width (m)**	**Tc (min)**	**Adopted Tc (min)**	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**Area (km^2)**	**Q_5**	**Q_{10}**	**Q_{25}**	**Q_{50}**	**Total flow**	**Design flow**	**Maximum Flow**	**Existing drainage size**	**Obtained size**	**Comment**
7	0+1750	250	2.8	0.028	Set up drains to convey storm water from the carriage way runoff.	7.7	0.37	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	0.06	0.08	0.09	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe
8	0+2030	280	3.4	0.034	Set up drains to convey storm water carriage way	7.7	0.34	7.00	115	130	150	165	0.002	0.064	0.072	0.083	0.091	0.06	0.08	0.10	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe

Add 20% to the maximum discharge and reduce the minimum by 20% for climate change

Use a minimum tc value of 7 minutes for asphaltic and developed urban areas and a minimum t_c value of 15 minutes for areas that are not developed and intercepting catchments. (ERA, 2013)

Design flow = Total flow + 20% of total flow

Maximum flow = ((Q_{50} +Total flow) +(20%*(Q_{50} +Total flow))

The slope was calculated from a google earth elevation profile in comparison with the contours developed

Table 5-13: Flow calculation table for a 25-year Return period check for side drains

for 25year check									Intensity for carriage way (mm/hr)									Design Discharge for side drains					
Section	**Chainage**	**Length (m)**	**Slope (%)**	**Slope (m/m)**	**Description**	**Road width (m)**	**Tc (min)**	**Adopted Tc (min)**	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**Area (km^2)**	**Q_5**	**Q_{10}**	**Q_{25}**	**Q_{50}**	**Total flow**	**Design flow**	**Maximum Flow**	**Existing drainage size**	**Obtained size**	**Comment**
1	0+250	250	3	0.03	Set up drains to convey storm water from the carriage way runoff and catchment 1	7.7	0.36	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	2.21	2.66	2.78	900	900	Appropriate to convey even the flow at 25-year ARI
2	0+500	250	3.5	0.035	Set up drains to convey storm water from the carriage way runoff.	7.7	0.34	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	0.06	0.08	0.09	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe
3	0+750	250	5	0.05	Set up drains to convey storm water from the carriage way runoff and catchment 2	7.7	0.30	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	2.69	3.23	4.07	900	900	Appropriate to convey even the flow at 25-year ARI
4	0+1000	250	4.2	0.042	Set up drains to convey storm water from the carriage way runoff and catchment3	7.7	0.32	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	5.17	6.20	7.74	900	1200	Under designed, the suggested diameter is able to convey a flow of 25-year ARI
5	0+1250	250	3.4	0.034	Set up drains to convey storm water from the carriage way runoff and catchment 4	7.7	0.34	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	1.77	2.12	2.68	900	750	Overdesigned however, for the sake of easy maintenance and considering the flow of 25-year ARI, provide 900mm diameter pipe

for 25year check									Intensity for carriage way (mm/hr)									Design Discharge for side drains					
Section	**Chainage**	**Length (m)**	**Slope (%)**	**Slope (m/m)**	**Description**	**Road width (m)**	**Tc (min)**	**Adopted Tc (min)**	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**Area (km^2)**	**Q_5**	**Q_{10}**	**Q_{25}**	**Q_{50}**	**Total flow**	**Design flow**	**Maximum Flow**	**Existing drainage size**	**Obtained size**	**Comment**
6	0+1500	250	3	0.03	Set up drains to convey storm water from the carriageway	7.7	0.36	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	0.06	0.08	0.08	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe
7	0+1750	250	2.8	0.028	Set up drains to convey storm water from the carriage way runoff.	7.7	0.37	7.00	115	130	150	165	0.002	0.057	0.064	0.074	0.081	0.06	0.08	0.09	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe
8	0+2030	280	3.4	0.034	Set up drains to convey storm water carriage way	7.7	0.34	7.00	115	130	150	165	0.002	0.064	0.072	0.083	0.091	0.06	0.08	0.10	900	375	Overdesigned however, for the sake of easy maintenance, provide 750mm diameter pipe

Table 5-14: Flow calculation table for the cross culvert

					Intensity for carriage way (mm/hr)									Design Discharge for Cross drainage culvert		
Section	**Chainage**	**Length (m)**	**Road width (m)**	**Adopted Tc (min)**	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**Area**	**Q_5**	**Q_{10}**	**Q_{25}**	**Q_{50}**	**Total flow**	**Design flow**	**Maximum Flow**
1	0+250	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	2.22	2.67	4.56
2	0+500	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	0.07	0.09	0.10
3	0+750	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	2.79	3.35	4.86
4	0+1000	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	6.45	7.74	9.70
5	0+1250	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	2.23	2.68	3.15
6	0+1500	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	0.07	0.09	0.10
7	0+1750	250	7.7	7.00	115	130	150	165	0.0019	0.0568	0.0642	0.0741	0.0815	0.07	0.09	0.10
															29.52	36.22
8	0+2030	280	7.7	7.00	115	130	150	165	0.0022	0.0636	0.0719	0.0830	0.0912	0.08295	0.09954	
															46.33	**59.27**

Minimum flow is the discharge for 25year ARI and maximum discharge is 50 year

Add 20% to the maximum discharge and reduce the minimum by 20% for climate change

The cross-drainage structure is located 200m from Lancha station towards Nefsi silk2, its dimensions were assessed to see if it can accommodate the flow.

Use a minimum tc value of 7 minutes for asphaltic and developed urban areas and a minimum t_c value of 15 minutes for areas that are not developed and intercepting catchments. (ERA, 2013)

Total flow = Q_{25}

Design flow = Total flow + 20% of total flow

Maximum flow = ((Q_{50} +Total flow) +(20%*(Q_{50} +Total flow))

The slope was calculated from a google earth elevation profile in comparison with the contours developed

Based on the obtained results from the software, three important things were noticed in consideration of the existing side drains of 900mm diameter, these included; some of the side drains were oversized, undersized and others were appropriate. This meant that the appropriate designs could convey the flow without causing any flooding on the infrastructure, the overdesigned drains meant they can convey the flow but the economic aspect was high and the under designed drains meant that they can't convey the flow hence leading to flooding.

The over designed drains don't meet the economical aspect though they can convey the designed flow. It becomes clear that a lot of money was incurred during their manufacturing process. Appropriate sizes are suggested for such drains by considering aspects like maintenance of the drains.

The under-designed side drain can lead to flooding in-case of heavy storms. This is because the flow ends up toppling on the infrastructure hence leading to the different negative impacts caused by flooding. The negative effects include;

- Damage to surrounding or adjacent property resulting from water overflowing the roadway curbs and entering such property. This retards the development of a given country whereby much money will be needed to solve such effects.
- Risk and delay to traffic caused by excessive ponding in sags or excessive spread along the roadway and increased potential for accidents. In such situations people's lives are lost and much time is taken to navigate flooded areas.
- Weakening of base and subgrade due to saturation from frequent ponding over long durations. Under this, the structural strength of the pavements deteriorates more rapidly rather than gradually, as was originally predicted in their design which leads to the entire destabilization of the infrastructure.

The above negative effects may end up destroying people's property and their lives. In such case, the side drains below the undersized one should be enlarged to be able to convey flow up to the cross-drainage structure. From the field visit, it was clearly seen that the roadway infrastructure profile is at the same level with the station at Lancha which means incase the runoff from the carriage way isn't conveyed by the longitudinal side drains it ends up flooding the infrastructure (railway track). The existing small concrete wall of about 250mm that separates the station platform from the roadway can easily be over powered by the flow from the carriage way especially during heavy storms.

Figure 5.12: Presence of silt on the roadway

Figure 5.13: Road way profile at the same level with station at Lancha

Under this section, the results of the undersized side drain obtained from Hydraulic toolbox are shown below; there are different graphs that are shown below indicating how the design flow varies with depth, critical depth, critical velocity, average velocity, and critical slope.

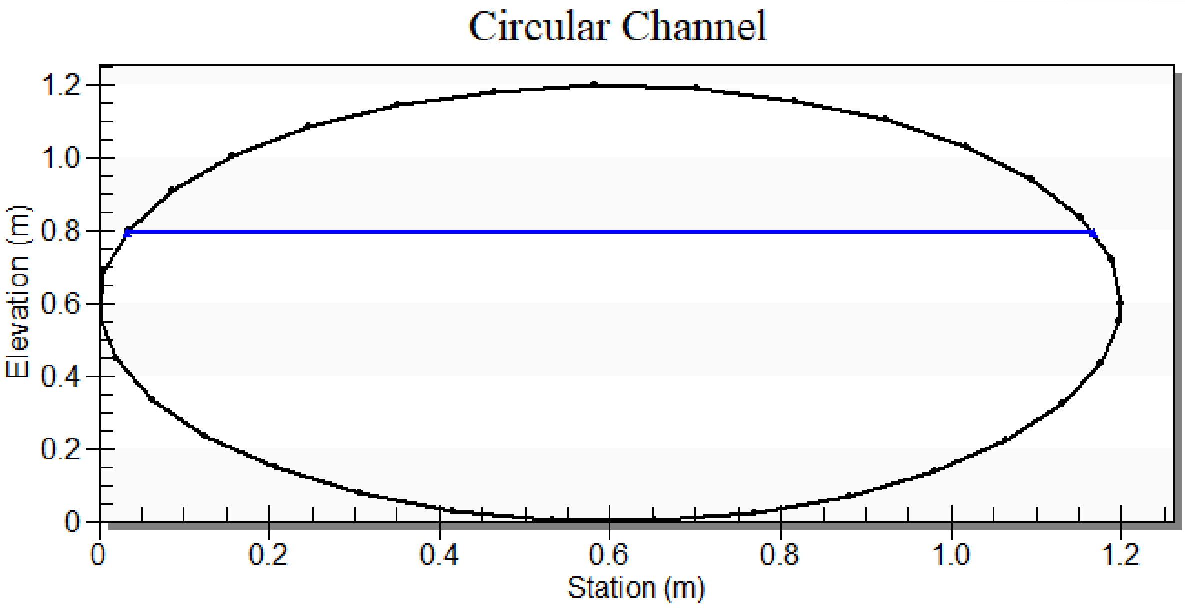


Figure 5.14: A graph showing a circular channel

Figure 5.14 above shows the geometry of the circular channel and the blue line indicates the maximum depth of the water flowing through it. This particular drain is of diameter 1.2m and it was found appropriate to convey the flow.

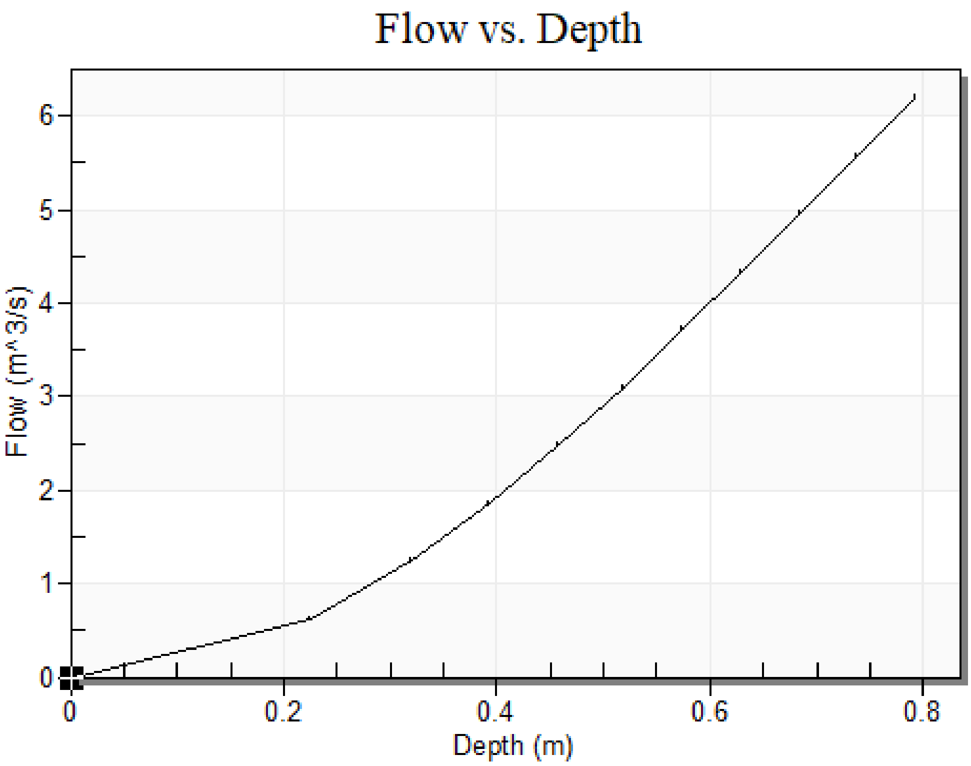


Figure 5.15: A graphical representation of flow and depth

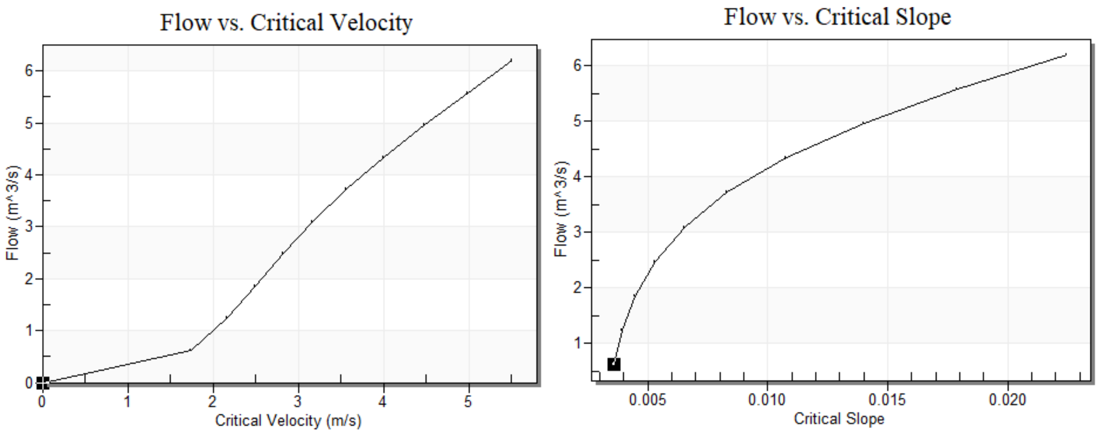


Figure 5.16: A graph of Flow varying with critical velocity and critical slope respectively

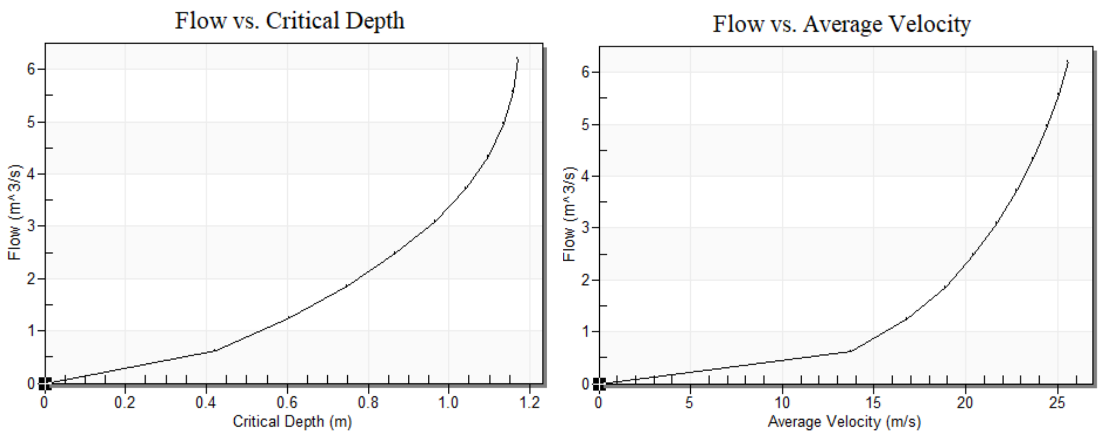


Figure 5.17: A graph of Flow varying with critical depth and average velocity respectively

Critical depth

The critical depth is the depth where the energy of the flow has been minimized (that's to say the critical depth maximized efficiency in the system). In cases where the depth of the flow is less than the critical depth a hydraulic jump may occur.

Critical slope

The critical slope (S_c) is the channel bottom slope that will result in critical flow conditions for a given flow rate in a channel of given Manning roughness, size, and shape.

Critical velocity

The critical velocity (V_c) is the liquid velocity for critical flow conditions in a particular channel with specified flow rate.

Froude number

This is a dimensionless parameter measuring the ratio of "the inertia force on an element of fluid to the weight of the fluid element". This means the inertial force divided by gravitational force.

Mean velocity

This refers to the average velocity in a fluid motion which can be understood from statistical definition of velocity in a fluid motion. For example, in a finite time range in fluid flow, ranges from, the mean velocity is obtained by averaging the velocity during those time range.

5.7.3 On-track drainage

The flows obtained on the track were minimal, this means an appropriate drainage network system can be provided. The type of system chosen for every location was dependent on the site restraints, water source, track structure and maintenance requirements. The subsurface drains are used where adequate surface drainage cannot be provided due to some restriction or lack of available fall due to outlet restrictions. The pipes under the track shall be of 375 mm minimum diameter, (ARTC, 2013). Basing on the obtained flows, aggregate drains were found suitable for use because of minimal flows or seepage. A typical example of an aggregate drain is a blanket drain, another type of aggregate drain is a French drain. A check should be made to ensure that the pipe selected satisfies all the above requirements (ARTC, 2013).

Since access to the actual on-track drainage design report or those as built wasn't availed to the researcher, different values of the flow were calculated using the rational method in-order to estimate the quantity of flow in-order to determine required type of drainage. The rational method helped in knowing whether the expected flows were much or less. After that the new flows were determined by Darcy's method. The track type belongs to class 2 hence the used flow is for the recurrence interval of 25 year. Basing on the calculated flows, the pipe size was taken to be 400mm for a ballast section and the flow on the ballast-less section is tapped by the installed circular PVC pipes of 200mm at an interval 0f 300mm which ends up in the collection pipes located along the pliers. This is because such pipe diameter eases maintenance works and can convey the expected flow. A check should be made to ensure that the pipe selected satisfies all the above requirements (ARTC, 2013). Whenever selecting a pipe size, the capacity of the pipe should be equal to or greater than the required capacity. The existing longitudinal side drains were assessed in hydraulic tool box and found sufficient to convey the flow from the track into the tapping point (cross drainage structure) which conveys it to the road way side drains. All this was seen basing from the field visit because there was no clear information from the relevant authorities. Below are the railway sketches used when determining the required dimensions for calculating the on-track flows.

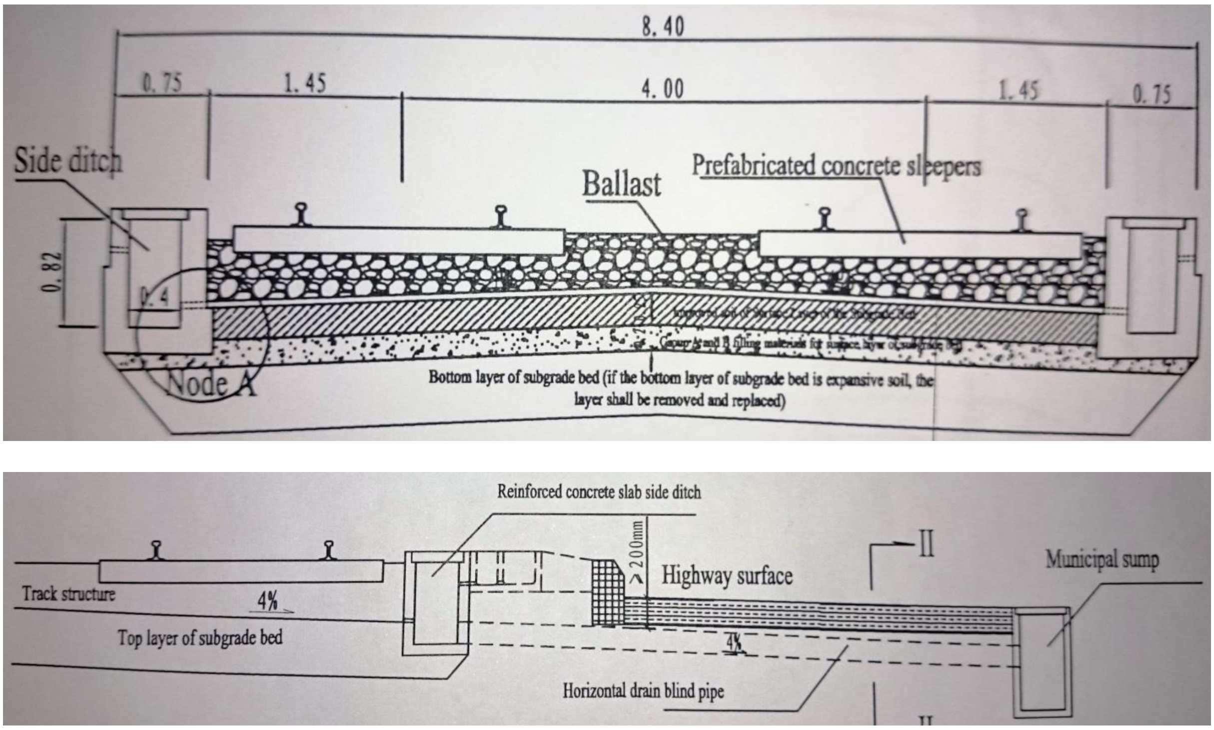


Figure 5.18: Railway track drawing

However, some other observations were seen during the field visits which included;

Around the station at Lancha, there is no inlet point for the carriageway runoff for a span of about 300m which means in case of heavy storms it can end up flooding the station. This is because the roadway surface is at the same level with the track profile. It makes more prone to flooding. The same observation was seen at Temanja station.

At Total petrol station, there is a drainage point which is widely open and incase of heavy storms the entire flow ends up on the roadway surface which means it can end up on the track structure.

Figure 5.19: Drainage point at total petrol station

(Source: Temanja station)

CHAPTER 6 CONCLUSIONS AND RECCOMENDATIONS

6.1 CONCLUSION

Based on the result of this study, the following conclusions can be drawn;

1. Log Pearson type 3 was used for the analysis and the obtained volumes were checked with the existing flood volume within the ERA manual. The greater volume of the two (calculated and read from the manual) was adopted. The flood volumes adopted for the different return periods of 2, 5, 10, 25, 50 and 100 as per Log Pearson type III included 52, 67, 76, 88, 97 and 106 respectively.
2. With the aid of geospatial tools like ARCGIS, ARCSWAT, and HEC GEOHMS, different catchments were developed in-order to identify the areas, streams contributing to the flow. The different flood prone areas were obtained by using ArcGIS software. Using the Rational and HECHMS, the flow values corresponding to different ARI were obtained depending on the sizes of the catchments/watersheds.
3. With the aid of Hydraulic tool box and HY-8 software, existing side drains and culvert were adequately analyzed respectively in-order to ascertain if they have the capacity to convey surface runoff from the delineated watersheds upstream. The existing box culvert comprised of one barrel of 4000mm by 3300mm which has the capacity of conveying the design flow of 46.33 m^3/s with a head water elevation of 2300.94m which is below the roadway elevation of 2302m. Otherwise, overtopping of the flow on to the roadway would be evident.
4. With the help of the Hydraulic tool box software, the existing circular side drains of 900mm each were analyzed. From the analysis only one side drain had no capacity to convey the flow. The remaining side drains had the capacity to convey the flows to the cross-drainage structure inlet point. The undersized side drain was redesigned in order to be able to convey the designed flow of 5.36 m^3/s by increasing its diameter to 1200mm. Some of the side drains were oversized however, due to maintenance works, it was appropriate to use oversized drains.

However, the challenge of flooding still exists especially at Lancha even when the cross culvert and side drains were able to convey the design flows. This could be due to; presence of silt within the manholes, debris at the upstream of the culvert and generally poor maintenance techniques.

6.2 RECOMMENDATIONS

Basing on the above conclusion, the following measures can be used to curb down the problem of flooding especially at the low-lying areas of Lancha. These include;

Recommend installation of screens on the cross culvert at an angle of 45° or 60° without blocking access for maintenance works; these normally serve two purposes such as retention of floating debris and restriction of access into the culvert by unauthorized personnel. This is done in-order to curb down the problem of accumulated rubbish from the upstream which ends up at the downstream.

Recommend regular desilting. Most of the side drains which convey the flow especially those around Lancha were blocked with silt which reduces on their capacity.

Recommend installation of grates at the inlet of the man holes along the carriage way. This is because the carriage way runoff carries fallen debris and any other material which ends up in the drains hence blocking the flow within them.

Recommend widening of the entry width from 6cm to 12cm of the manholes. This will help in tapping a greater amount of flow volume from the carriage way runoff hence preventing it from accumulating in areas with flat slopes like Lancha.

Recommend regular maintenance of the manholes and periodic maintenance at the culvert entrances. This is because most of the existing manholes along my study area are partly closed leaving a smaller percentage of the opening/ entry point.

Recommend bigger side drains of about 1200mm especially around Lancha area which floods often times.

Culverts must be kept free of obstructions. Sand or sediment deposits should be removed as soon as possible. During major storms, critical areas should be patrolled and the inlets kept free of debris. Inlet and outlet channels should be kept in alignment and vegetation should be controlled in order to prevent any significant restriction of flow.

There is need to monitor performance of existing side drains and the cross culvert as some were designed with limited field data resulting into either under or overdesigning.

REFERENCES

Abushandi, E., Merkel, B., 2013. Modelling rainfall runoff relations using HEC-HMS and IHACRES for a single rain event in an arid region of Jordan. Water resources management 27, 2391-2409.

Addis Ababa City Roads Authority (AACRA - 2003). Drainage Design Manual, Ethiopia

AMK Associates, L., International. (2004). Dual drainage storm water management model.

Anteneh Zewdu Deriba. 2015. Integrated Urban Drainage System; The case of Ayat to Megenagna light rail transit system route. Msc. Thesis, Addis Ababa Institute of Technology.

AREMA (2003). Practical Guide to Railway Engineering

Australian Rail Track Corporation LTD (2006). Engineering Practices manual.

Australian Rail Track Corporation LTD, June 2013. Engineering Practices manual.

Belete, D.A. (2011). Road and urban storm water drainage network interaction in Addis Ababa:

Chakravarti, Laha, and Roy, 1967. Handbook of Methods of Applied Statistics.

Chow, V. T., Maidment, D. R. & Mays, L. W., 1988. *Applied Hydrology.* 1st edition. New York:

CSA, 2013. *Population Projection of Ethiopia for All Regions at Wereda Level from 2014 – 2017*, Addis Ababa: Central Statistical Agency.

Daniel, A. (2007). Assessment of flood risk in Dire Dawa Town, Eastern Ethiopia, Using GIS Msc. Thesis, Addis Ababa university technology faculty.

Design Manual for Roads and Bridges, November 2004; Different standards for Roads and Bridges

Donald E. Woodward. *National Engineering Handbook, Section 4, Hydrology*, 1997

Environmental Protection Agency, Washington, 2003 Protecting water quality from urban runoff. United States Environmental Agency (EPA).

Ethiopian Opinion (2014). Around 765 thousand birrs damaged due to heavy flood in Addis Ababa, available at https://ethiopiantimes.wordpress.com/tag/addis-abeba/, last assessed July 18, 2015

Ethiopian Roads Authority, 2013. Drainage Design Manual.

Fantahun Alamirew, 2016. Impacts of informal settlement on environment (the case of Nefas silk Lafto sub city). MA. Thesis, Ethiopian Civil Service University. Addis Ababa university technology faculty.

Federal Highway Administration (2014). HY-8 User Manual (v7.3), Pennsylvania State University.

Feldman, A.D., 2000. Hydrologic modeling system HEC-HMS: technical reference manual. US Army Corps of Engineers, Hydrologic Engineering Center.

Federal Highway Administration (2014). Hydraulic Toolbox manual.

Fisha Hailemichael, 2015. Flood Assessment on Addis Ababa Light Rail Transit System (Case of Meshualekiya – Gotera). Msc. Thesis, Addis Ababa Institute of Technology

Frimpong, D. E. (2009). Eighty Houses to go Down – As Kumasi Metropolitan Assembly (KMA) begins demolition in Kumasi. Daily Graphic.

Government of Malaysia department of irrigation and drainage (2010). Rational method of flood estimation for rural catchments in Peninsular Malaysia.

Government of the Hong Kong Special Administrative Region (2000). Storm water drainage manual (Planning, Design and Management). Hong Kong.

Halcrow, (2009). ISIS Free & ISIS Professional Quick Start Guide. United Kingdom

Hawkins, R.H., Woodward, D.E., Jiang, R., Hjelmfelt, J., Allen T, Van Mullem, J.A., Quan, Q.D., 2003. Runoff curve number method: examination of the initial abstraction ratio.Proceedings of the Second Federal Interagency Hydrologic Modeling Conference, Las Vegas, Nevada. U.S. Geological Survey. doi:10.1111/j.1752-1688.2006.tb04481.x. Retrieved 24 November 2013., World Water & Environmental Resources Congress 2003, pp. 1-10.

HEC, 2006. Hydrologic modeling system HEC-HMS: user's manual. US Army Corps of Engineers, Hydrologic Engineering Center.

HEC, July 2016. River Analysis System user's manual. US Army Corps of Engineers, Hydrologic Engineering Center.

HEC, July 2019 Statistical Software Package user’s manual. US Army Corps of Engineers, Hydrologic Engineering Center.

Intergovernmental Panel on Climate Change (IPCC). Climate Change 2007: Fourth Assessment Report Climate Change 2007 Synthesis Report, Topic3. Available online: http://www.ipcc.ch/pdf/assessment-report/ar4/syr/ar4_syr.pdf (accessed on 14 September 2009).

Intergovernmental Panel on Climate Change, 2001: Impacts, Adaptation and Vulnerability. *J. Am. Stat. Assoc.*, 69, pp. 730-737

Jeb, D. N., & Aggarwal, S. P. (2008). Flood inundation hazard modelling of the River Kaduna using remote sensing and geographic information systems. Journal of Applied Sciences Research, 4(12), 1822–1833.

Kasa, L. et al., 2013. Impact of Urbanization of Addis Ababa City on Peri-Urban Environment and Livelihoods. *Proceeding of the Tenth International Conference on Ethiopian Economy,* II (1).

Kassahun Urgessa, 2015. Hydrologic and Hydraulic Analyses of Drainage Structures; In Case of Shishinda-Tepi Road Msc. Thesis, Addis Ababa Institute of Technology.

Kirpich, Z. P. (1940). Time of concentration of small agricultural watersheds.

Linmei Nie (2004). Flooding Analysis of Urban Drainage Systems, Dissertation paper. Trondheim, Norway

Lubega, F., Detection of Inhomogeneities in the National Climate Dataset (1902-2003) of Uganda, Proceedings, Groundwater and Climate in Africa, 2008, Kampala, Uganda, University College London/Ministry of Water and Environment, Kampala, 2008, Uganda.

M. T. Amin, M. Rizwan, A. A. Alazba, 2016. A best-fit probability distribution for the estimation of rainfall in northern regions of Pakistan.

Madsen, H., 2000. Automatic calibration of a conceptual rainfall–runoff model using multiple objectives. Journal of hydrology 235, 276-288.

Manoj Kumar, Rohitashw Kumar, P. K. Singh, Manjeet Singh, K. K. Yadav and H. K. Mittal. Catchment delineation and morphometric analysis using geographical information system. Udaipur: IWA Publishing, 2015. 72.72015.

McCuen, Richard H. 1998 Hydrologic analysis and design Prentice Hall

McGraw Hill. CRED, E. (2011). The OFDA/CRED International Disaster Database. Centre for Research on Epidemiology of Disasters–CRED, Université Catholique de Louvain, Brussels, Belgium.

Michael Nigussie. 2018. Roles and Constraining factors of Women and Children affairs for the advancement of gender mainstreaming and women empowerment: A case study of Nifas Silk - Lafto Sub city, Addis Ababa. MA. Thesis, Addis Ababa university technology, College of Development studies.

Normann, J.M., Houghtalen, R. J., and Johnston, W.J. Hydraulic Design of Highway Culverts,

Nwaogazie I. L, Agiho G. C. 2019. Performance analysis of box and circular culverts using HY 8 software for Aluu Clan, Port Harcourt

OPW (2009) The Planning System and Flood Risk Management, Guidelines for Planning Authorities Ireland.

Overton, D. E., and M.E. Meadows, 1976. *Storm Water Modeling*, *Academic Press*, New York.

Patrick, W.; Jonas, O.; Karsten, A.-N.; Simon, B.; Assela, P.; Ida Bulow, G.; Henrik, M.; Van-Thanh-Van, N. Impacts of Climate Change on Rainfall Extremes and Urban Drainage Systems; IWA Publishing: London, UK, 2007.

Plate, E.J. Flood risk and flood management. J. Hydrol. 2002, 267, 2–11.

Plummer, C. C., & McGeary, D. (1993). Physical Geology (6th Edition). England: Brown Publishers

Rugumayo, I. A. (2012). An Introduction to Hydrology and Water Resources Engineering in Uganda (1^{st} edition), Makerere University Printery,

Satterthwaite, D., Huq, S., Reid, H., Pelling, M., & Lankao, P. R. (2007). Adapting to Climate Change in Urban Areas: The Possibilities and Constraints in Low-and Middle-Income Nations, Retrieved from http://pubs.iied.org/pdfs/10549IIED.pdf

Scharffenberg, W., Ely, P., Daly, S., Fleming, M., Pak, J., 2010. Hydrologic modeling system (HEC-HMS): Physically-based simulation components. 2nd Joint Federal Interagency Conference, Las Vegas, NV.

Schmitt T.G. et al. (2004). Analysis and modeling of flooding in urban drainage systems, Journal of Hydrology. Kaiserslautern, Germany

Sharma, M.A., and J. B. Singh, 2010. “Use of Probability Distribution in Rainfall

Stephens, M. A., 1974. “EDF Statistics for Goodness of Fit and Some Comparisons”,

Subramanya, K., Engineering Hydrology, 2^{nd} Edition, Tata, McGraw Hill, 1994, New Delhi.

The Hydrology Subcomittee, 1981. *Guidelines for determining flood flow frequency Bulletin 17B,* Reston, Virginia: U.S. Interagency Advisory Committee on Water Data.

U.S. Department of Agriculture, April 1973 A Method for Estimating Volume and

UN-HABITAT, 2008. *Addis Ababa Urban Profile,* Nairobi, Kenya: United Nations Human Settlements Programme. University Press, Cambridge Upper Saddle River, New Jersey 07458

Urban Storm Drainage Criteria Manual Volume 1, august 2018.

V.T CHOW, 1959. Open channel flow, London: McGRAW – HILL, 11.95.99.136-140

Van Overeem, J., & Steenbergen, J. (2015, 3). Drr-team mission report (Unpublished document). Volume I, John Wiley and Sons, pp. 392-394

Wondimu, Abeje and Alfakih E. 1998. Urban drainage in Addis Ababa, Ethiopia: Existing situation and improvement ideas. Paper published in the fourth international conference on development in urban drainage modeling.

Xiaohui Zhong and Utpal Dutta (2015) 'Engaging Nash-Sutcliffe Efficiency and Model Efficiency. Factor Indicators in Selecting and Validating Effective Light Rail System Operation and Maintenance Cost Models', *Journal of Traffic and Transportation Engineering*, 3(5), pp. 255–265.

Yeshewatesfa Hundecha, Juraj Parajka, and Alberto Viglione. 2017. Flood type classification and assessment of their past changes across Europe. Hydrology and Earth System Sciences 21st July 2017

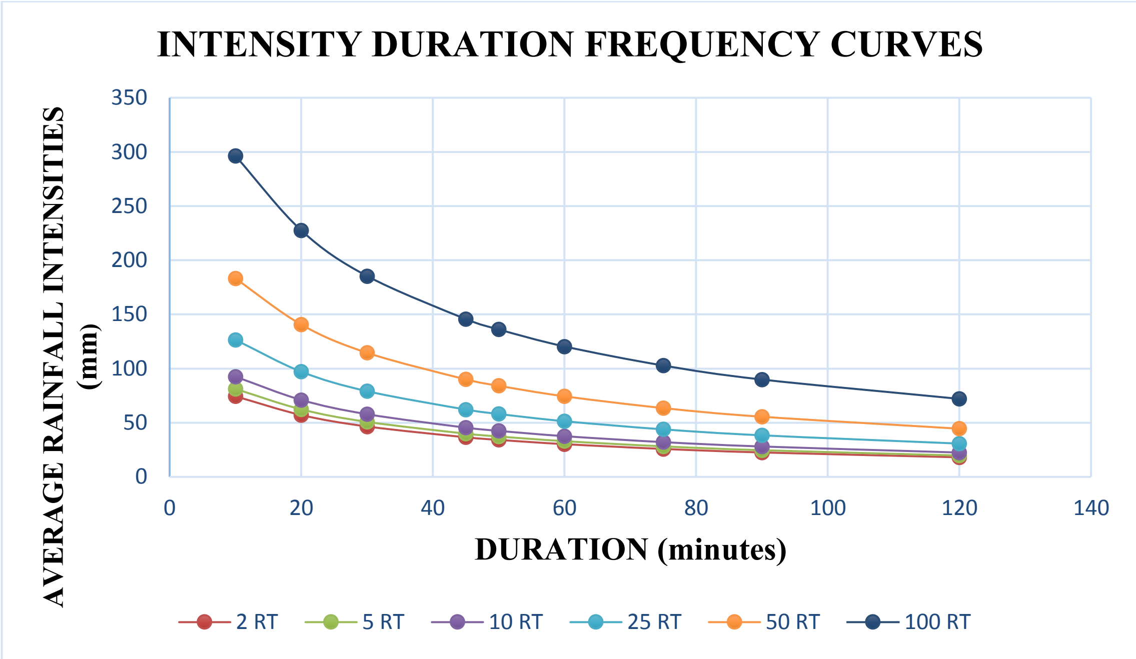
Zhang, H., Wang, Y., Wang, Y., Li, D., Wang, X., 2013. The effect of watershed scale on HEC-HMS calibrated parameters: a case study in the Clear Creek watershed in Iowa, US. Hydrology and Earth System Sciences 17, 2735.

APPENDIX

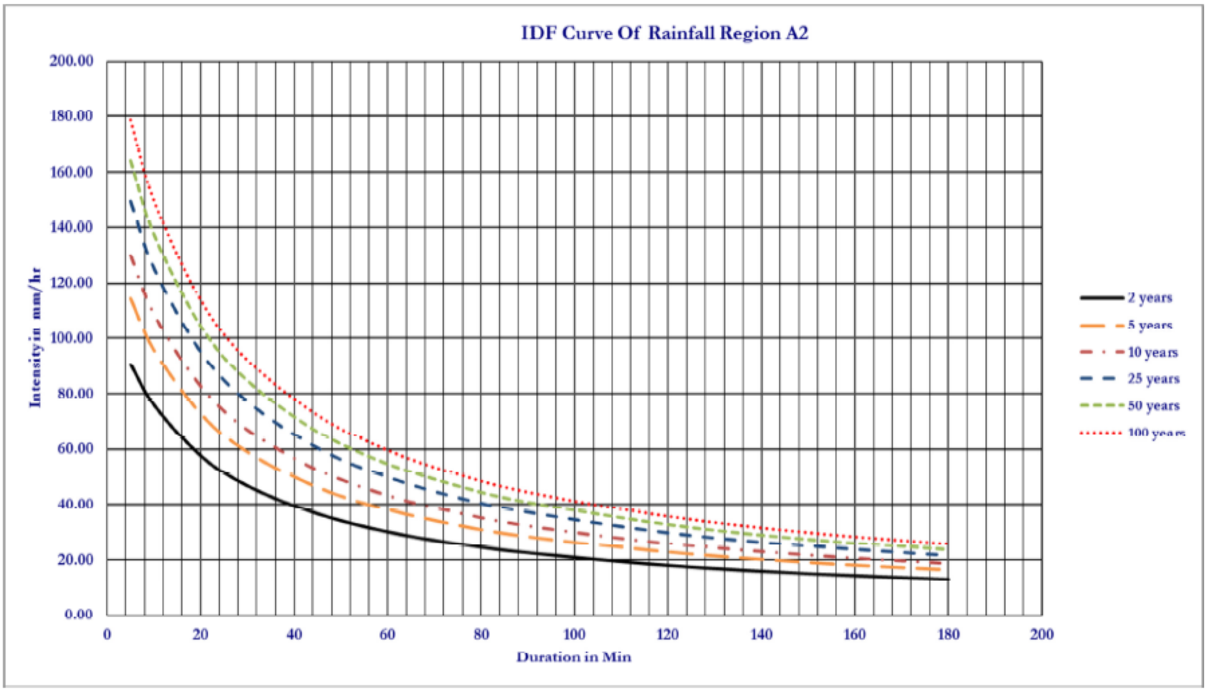
Appendix 1: Data for determining the rainfall for Addis Ababa Observatory station

YEAR	Maximum Daily Annual precipitation, X (mm)	YEAR	Maximum Daily Annual precipitation, X (mm)
1968	88.0	1993	53.5
1969	51.0	1994	57.0
1970	87.7	1995	85.3
1971	42.1	1996	67.0
1972	25.1	1997	46.3
1973	47.1	1998	78.3
1974	62.5	1999	37.4
1975	28.9	2000	37.1
1976	48.6	2001	96.3
1977	59.4	2002	29.5
1978	93.5	2003	54.9
1979	50.6	2004	44.2
1980	36.3	2005	58.6
1981	58.0	2006	70.9
1982	41.4	2007	64.0
1983	50.1	2008	53.3
1984	55.4	2009	54.7
1985	43.2	2010	44.6
1986	83.8	2011	55.8
1987	56.8	2012	36.4
1988	35.5	2013	47.2
1989	49.2	2014	65.4
1990	39.6	2015	47.8
1991	47.3	2016	47.7
1992	51.4	2017	50.8

Appendix 2: Intensity Duration Frequency

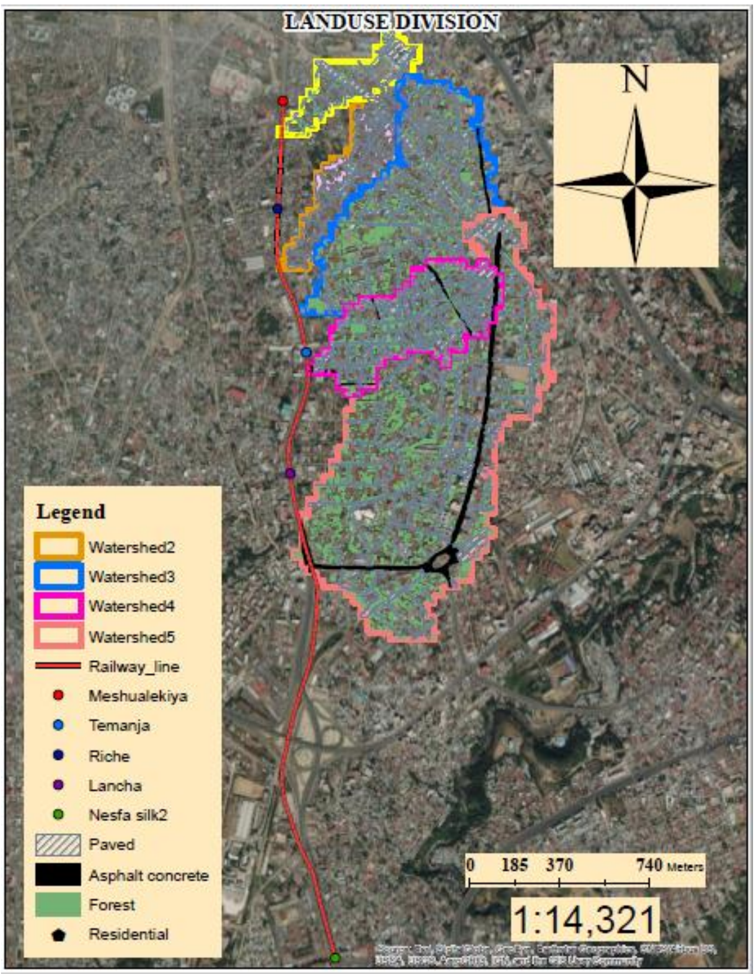


IDF curves of rainfall Region A2



(Source: ERA 2013)

Appendix 3: Developed landuse divisions in google earth.



Appendix 4: Recommended Runoff Coefficient C for Various Land Uses for Ethiopia

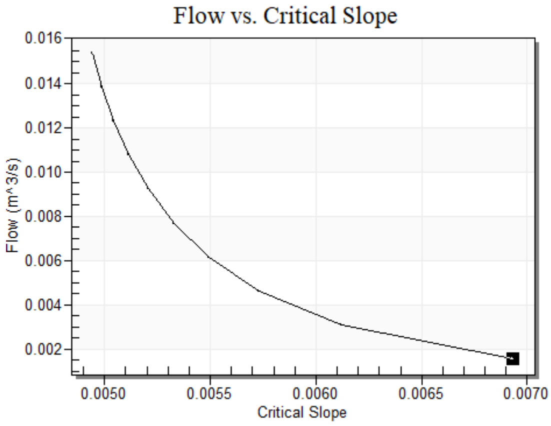
Description of Area	Runoff Coefficients
Business: Downtown areas	0.70-0.95
Neighborhood areas	0.50-0.70
Residential: Single-family areas	0.30-0.50
Residential: Multi units, detached	0.40-0.60
Residential: Multi units, attached	0.60-0.75
Suburban	0.25-0.40
Residential (0.5 hectare lots or more)	0.30-0.45
Apartment dwelling areas	0.50-0.70
Industrial: Light areas	0.50-0.80
Industrial: Heavy areas	0.60-0.90
Parks, cemeteries	0.10-0.25
Playgrounds	0.20-0.40
Railroad yard areas	0.20-0.40
Unimproved areas	0.10-0.30

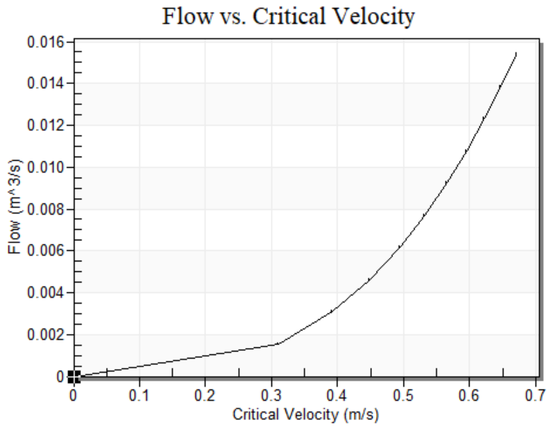
(Source: ERA 2013)

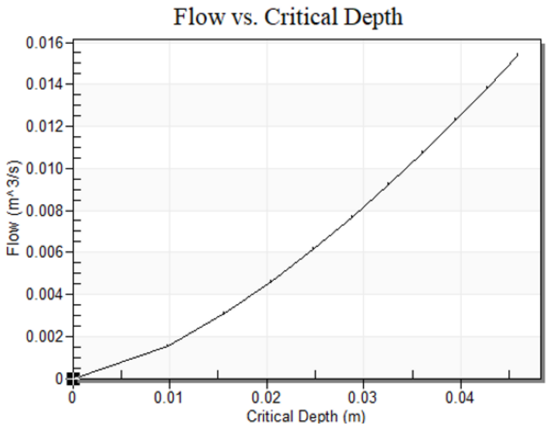
Appendix 5: On track discharge using rational method

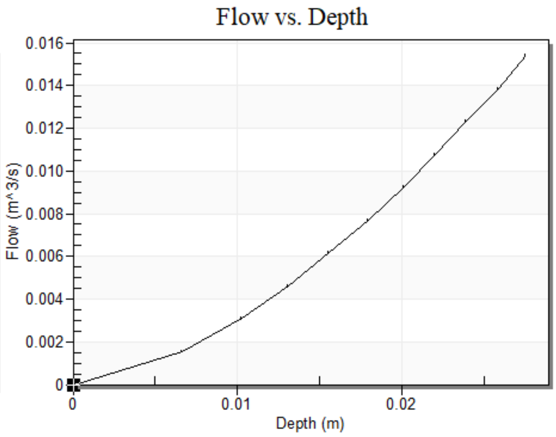
									Intensity (mm/hr)								
Chainage	**Length (m)**	**Slope (%)**	**Slope m/m**	**Width of the track (m)**	**Half width of the track (m)**	**Area (km^2)**	**Tc (min)**	**Adopted Tc (min)**	**5 Year**	**10 Year**	**25 Year**	**50 Year**	**Q_5**	**Q_{10}**	**Q_{25}**	**Q_{50}**	**Design flow**
250	250	1	0.01	6.9	3.45	0.00086	0.2975	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
500	250	1.1	0.011	6.9	3.45	0.00086	0.28678	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
750	250	2.4	0.024	6.9	3.45	0.00086	0.21238	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
1000	250	3.5	0.035	6.9	3.45	0.00086	0.18366	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
1250	250	2.4	0.024	6.9	3.45	0.00086	0.21238	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
1500	250	1.1	0.011	6.9	3.45	0.00086	0.28678	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
1750	250	0.9	0.009	6.9	3.45	0.00086	0.30982	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
2000	250	2.2	0.022	6.9	3.45	0.00086	0.21961	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
2250	250	2.2	0.022	6.9	3.45	0.00086	0.21961	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
2500	250	1.5	0.015	6.9	3.45	0.00086	0.2545	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
2750	250	2.5	0.025	6.9	3.45	0.00086	0.20906	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
3000	250	2.5	0.025	6.9	3.45	0.00086	0.20906	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
3250	250	3.4	0.034	6.9	3.45	0.00086	0.18572	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
3500	250	2.9	0.029	6.9	3.45	0.00086	0.19745	5	115	130	150	165	0.0107	0.0121	0.0154	0.0184	0.0221
3640	140	2	0.02	6.9	3.45	0.00048	0.22782	5	115	130	150	165	0.0060	0.0068	0.0086	0.0103	0.0124

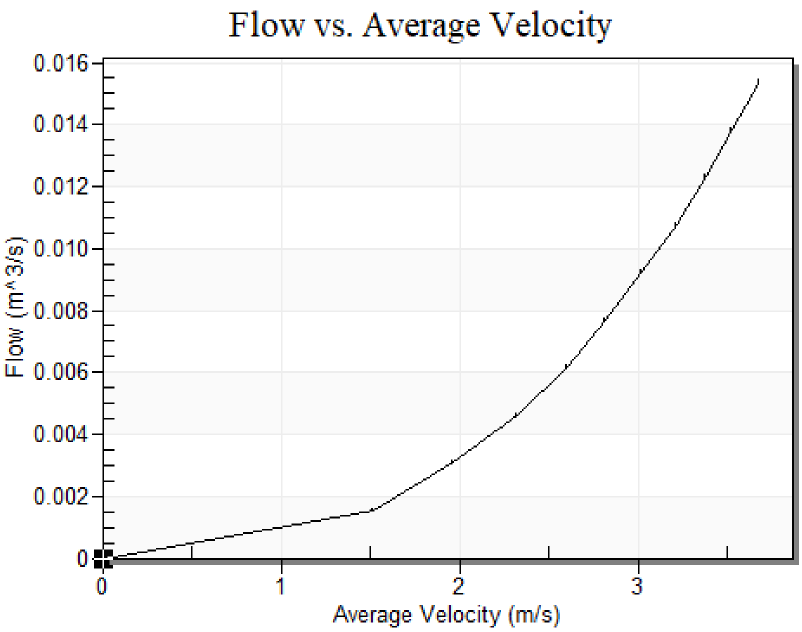
Appendix 6: Longitudinal side drains graphs from Hydraulic toolbox software











Appendix 7: Calculation sheet showing the different parameters like slope from google earth elevation profile in comparison to the developed contours

Roadway slopes			
Section	**Chainage**	**Length (m)**	**Slope (m/m)**
1	0+250	250	0.03
2	0+500	250	0.035
3	0+750	250	0.05
4	0+1000	250	0.042
5	0+1250	250	0.034
6	0+1500	250	0.03
7	0+1750	250	0.028
8	0+2030	280	0.034

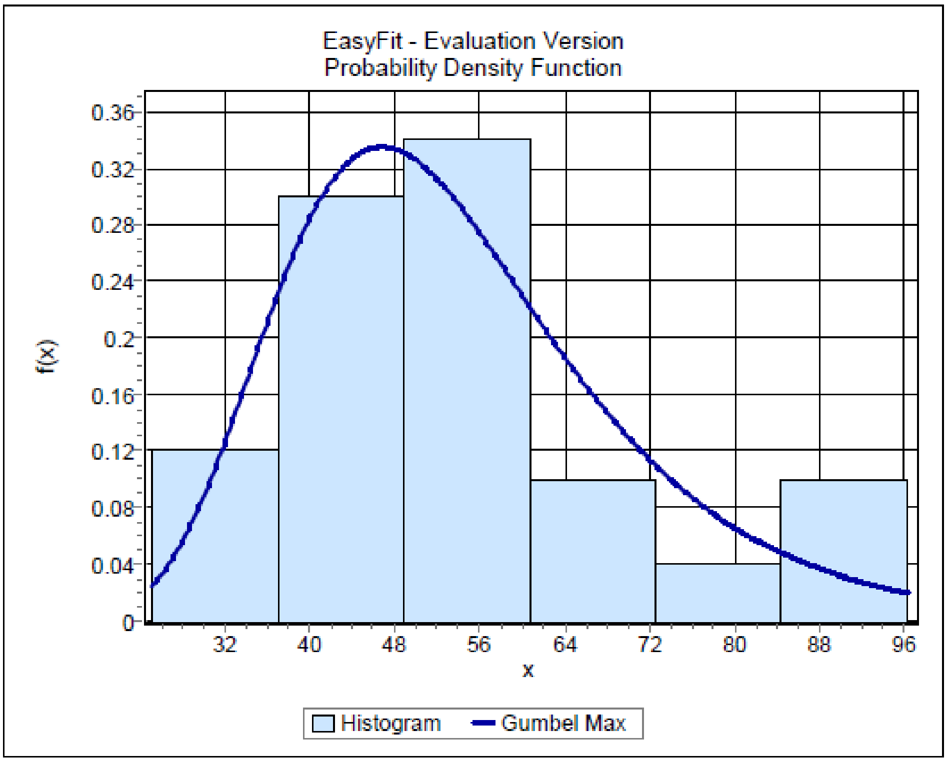
Ontrack slopes developed		
Chainage	**Length (m)**	**Slope m/m**
250	250	0.01
500	250	0.011
750	250	0.024
1000	250	0.035
1250	250	0.024
1500	250	0.011
1750	250	0.009
2000	250	0.022
2250	250	0.022
2500	250	0.015
2750	250	0.025
3000	250	0.025
3250	250	0.034
3500	250	0.029
3640	140	0.02

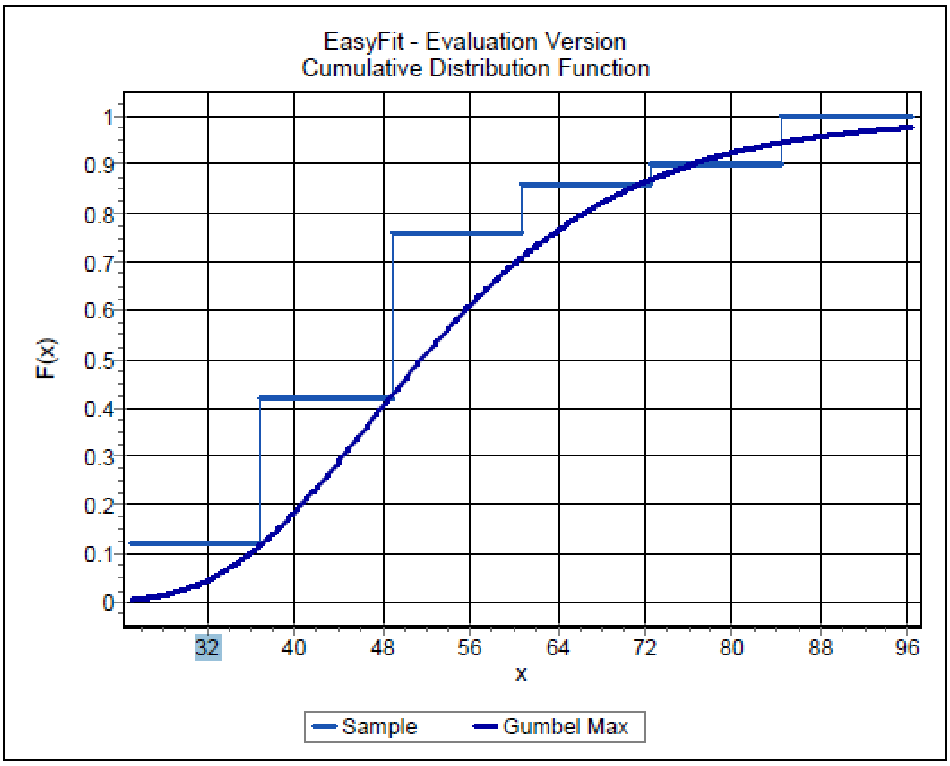
Appendix 8: Design Storm Frequency (yrs) by Geometric Design Criteria

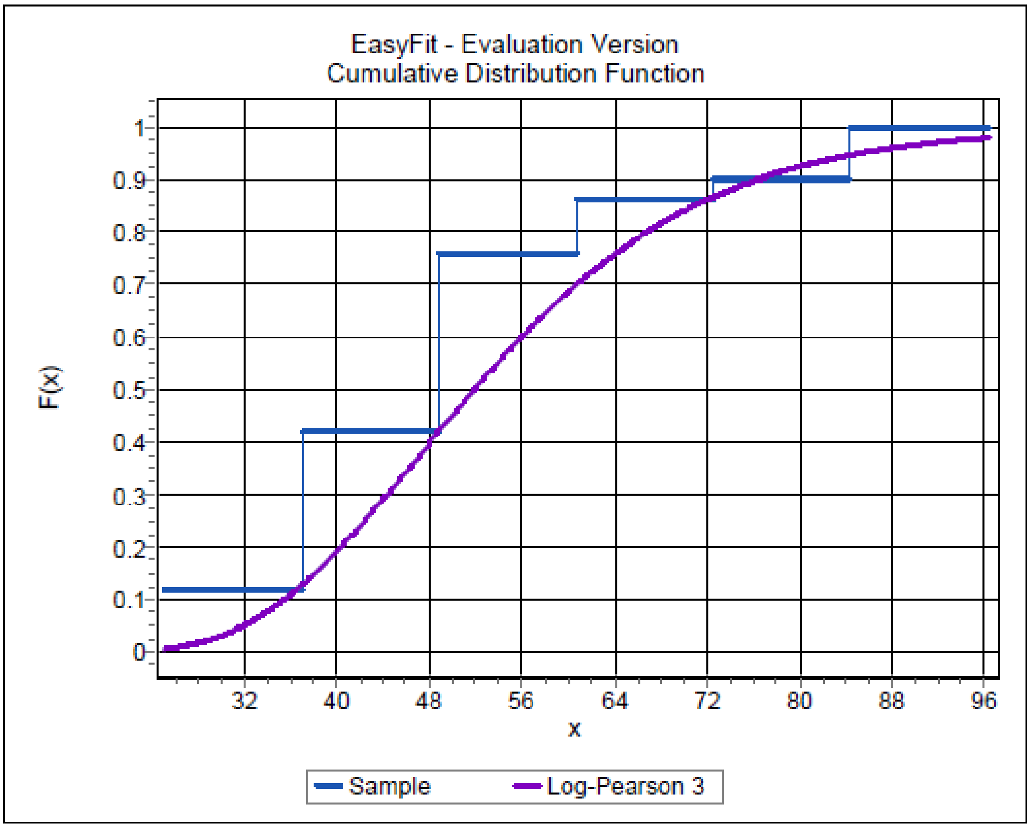
Structure Type	EW1/DC8/DC7		DC6/DC5		DC4/DC3		DC2/DC1/track	
	Design	Check	Design	Check	Design	Check	Design	Check
Gutters and Inlets*	5/5/5	10/10/10	5/5	10/10	5/2	10/5	---	---
Side Ditches	10/10/10	25/25/25	5/5	10/10	5/2	10/5		
Ford/Low-Water Bridge	---	---	----	----	---	---	5/5/5	10/10/10
Culvert, pipe (see Note) Span<2m	25/25/25	50/50/50	10/10	25/25	10/5	25/10	5/5/5	10/10/10
Culvert, 2m< span<6m	50/50/50	100/100/100	25/25	50/50	25/10	50/25	10/10/10	25/25/25
Short Span Bridges 6m< span<15m	50/50/50	100/100/100	25/25	50/50	25/10	50/25	10/10/10	25/25/25
Medium Span Bridges 15m< span<50m	100/100/100	200/200/200	50/50	100/100	50/25	100/50	50/25/25	100/50/50
Long Span Bridges spans>50m	100/100/100	200/200/200	50/50	100/100	50/25	100/50	50/25/25	100/50/50

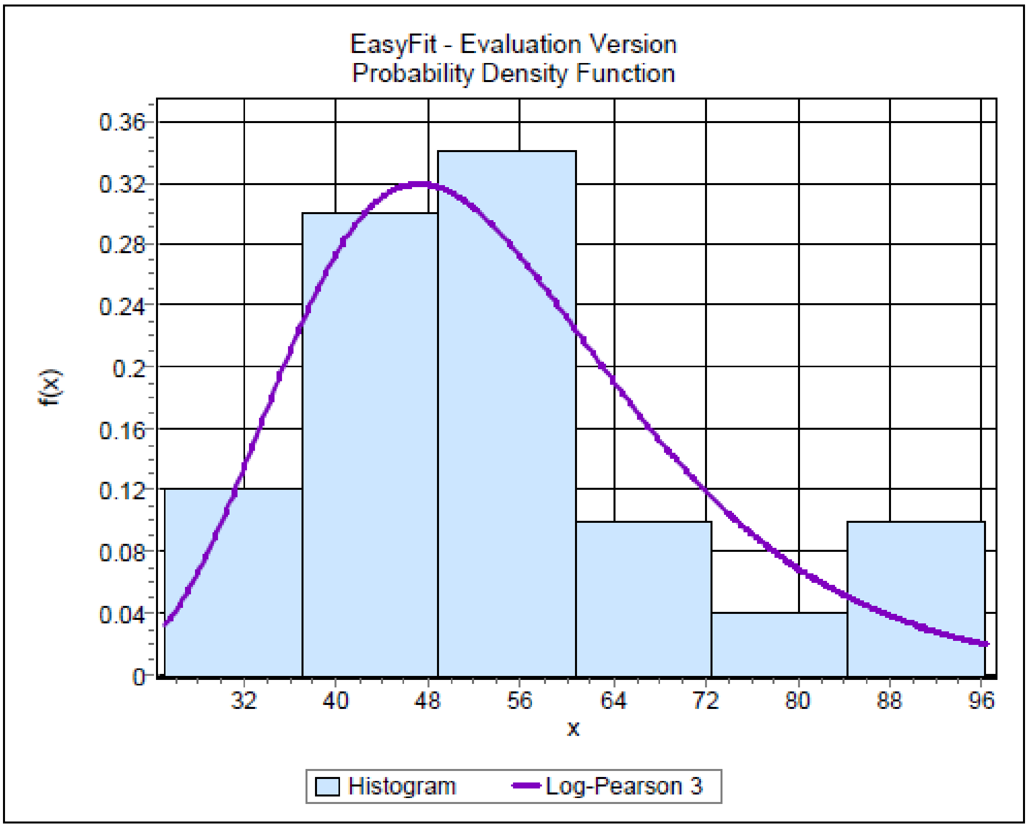
(Source: ERA, 2013)

Appendix 9: Graphs showing the goodness of fit test






EasyFit - Evaluation Version
Cumulative Distribution Function
F(x)
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
32
40
48
56
64
72
80
88
96
x
Sample
Log-Pearson 3


EasyFit - Evaluation Version
Probability Density Function
f(x)
0.36
0.32
0.28
0.24
0.2
0.16
0.12
0.08
0.04
0
32
40
48
56
64
72
80
88
96
x
Histogram
Log-Pearson 3

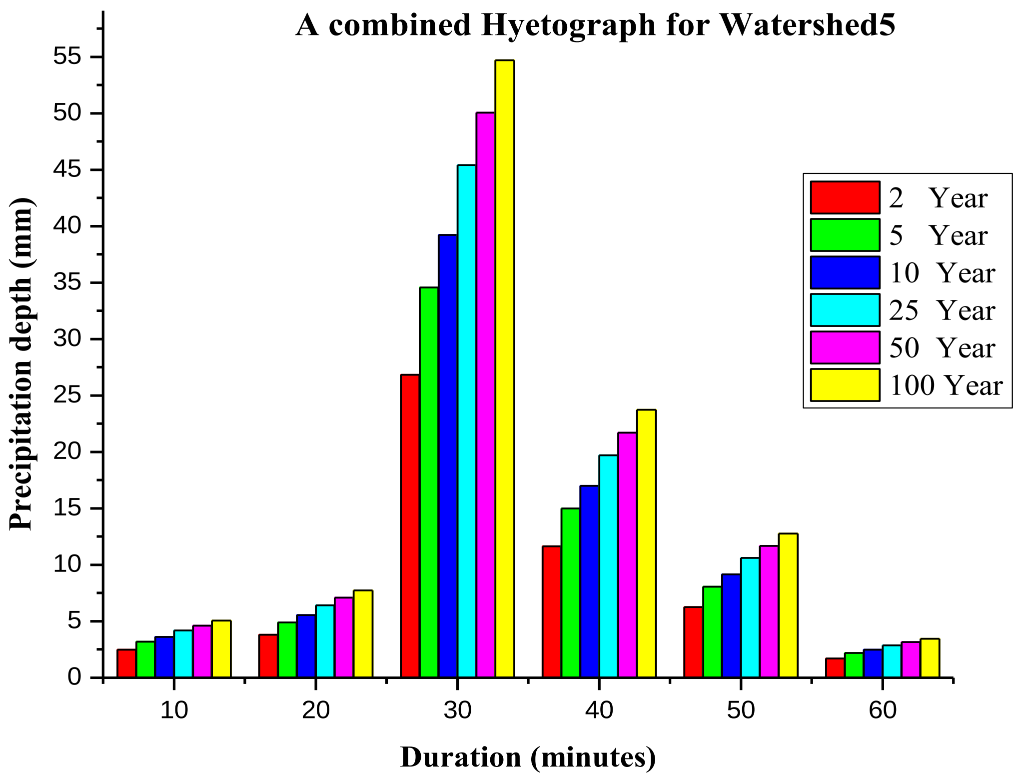
Appendix 10: 24hr Rainfall Depth Vs Frequency for Ethiopia

Return Period Years	24 hr Rainfall Depth (mm) vs Frequency (yr)							
	2	5	10	25	50	100	200	500
RR-A1	50.30	66.02	76.28	89.13	98.63	108.06	117.48	130.00
RR-A2	51.92	65.52	74.45	85.70	94.07	102.45	110.91	122.27
RR-A3	47.54	59.61	67.66	77.92	85.62	93.34	101.13	111.58
RR-A4	50.39	63.83	72.28	82.55	89.97	97.20	104.32	113.63
RR-B1	58.87	71.26	79.29	89.35	96.84	104.37	112.02	122.41
RR-B2	55.26	69.95	79.68	92.03	101.29	110.61	120.07	132.87
RR-C	56.52	71.04	80.54	92.52	101.48	110.50	119.66	132.06
RR-D	56.23	76.84	90.37	107.46	120.23	133.05	146.00	163.44

Note: RR- Rainfall Region

(Source: ERA 2013)

Appendix 11: Combined hyetographs for watershed 5



(Source: origin software)

Appendix 12: Calculated CN value for watershed 5

SOIL DATA FOR CATCHMENT 5			
SOIL NAME	**SOIL_TYPE**	**Area (sqkm)**	**HYDGRP**
EUVERTISOLS	Eutric Vertisols	0.002768	D
HUNITISOLS	Humic Nitisols	0.03544	B
EUVERTISOLS	Eutric Vertisols	0.82487	D

AREA, D	0.827638
AREA, B	0.035440
TOTAL AREA	**0.863078**

%AREA (D)	95.89
%AREA (B)	4.11
%PAVED	13
%ASPHALT	5
%FOREST	16
%RESIDENTIAL& COMMERCIAL	65

HYDROLOGIC SOIL D			
LAND USE	**% AREA**	**Curve Number, CN**	**PRODUCT**
13% Paved	12.47	98	1221.64
5% Asphalt	4.79	98	469.86
16% Forest	15.34	77	1181.36
65% Residential and commercial	62.33	89.5	5578.40
			8451.27

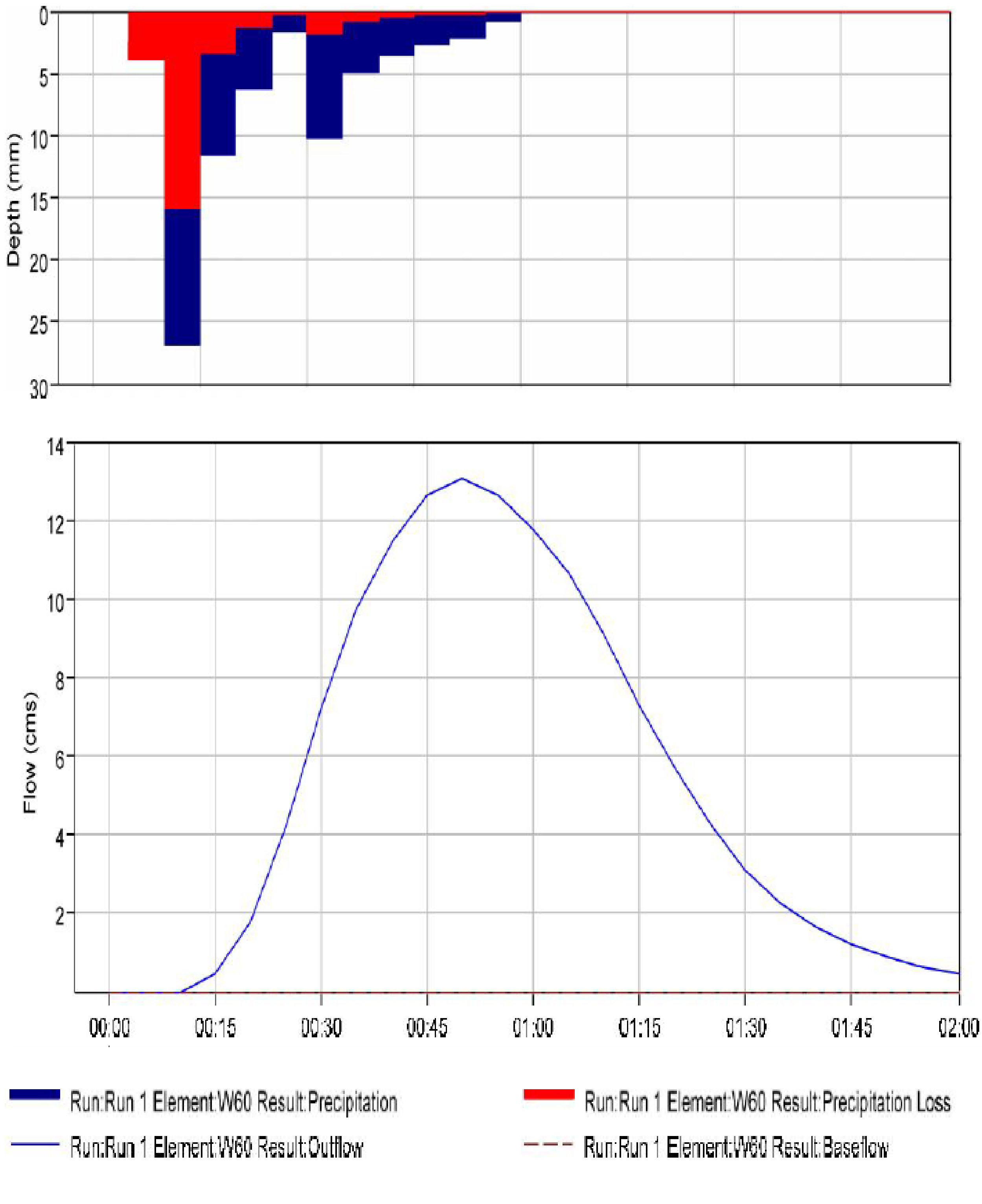
HYDROLOGIC SOILB			
LAND USE	**% AREA**	**Curve Number, CN**	**PRODUCT**
13% Paved	0.53	98	52.36
5% Asphalt	0.21	98	20.14
16% Forest	0.66	55	36.17
65% Residential and commercial	2.67	80	213.72
			322.39

AVERAGE CN	87.74
Potential, S (mm)	35.50
Initial Abstraction, I_a (mm)	7.10

Appendix 13: Results from HECHMS

Developed for 2year return period for watershed5

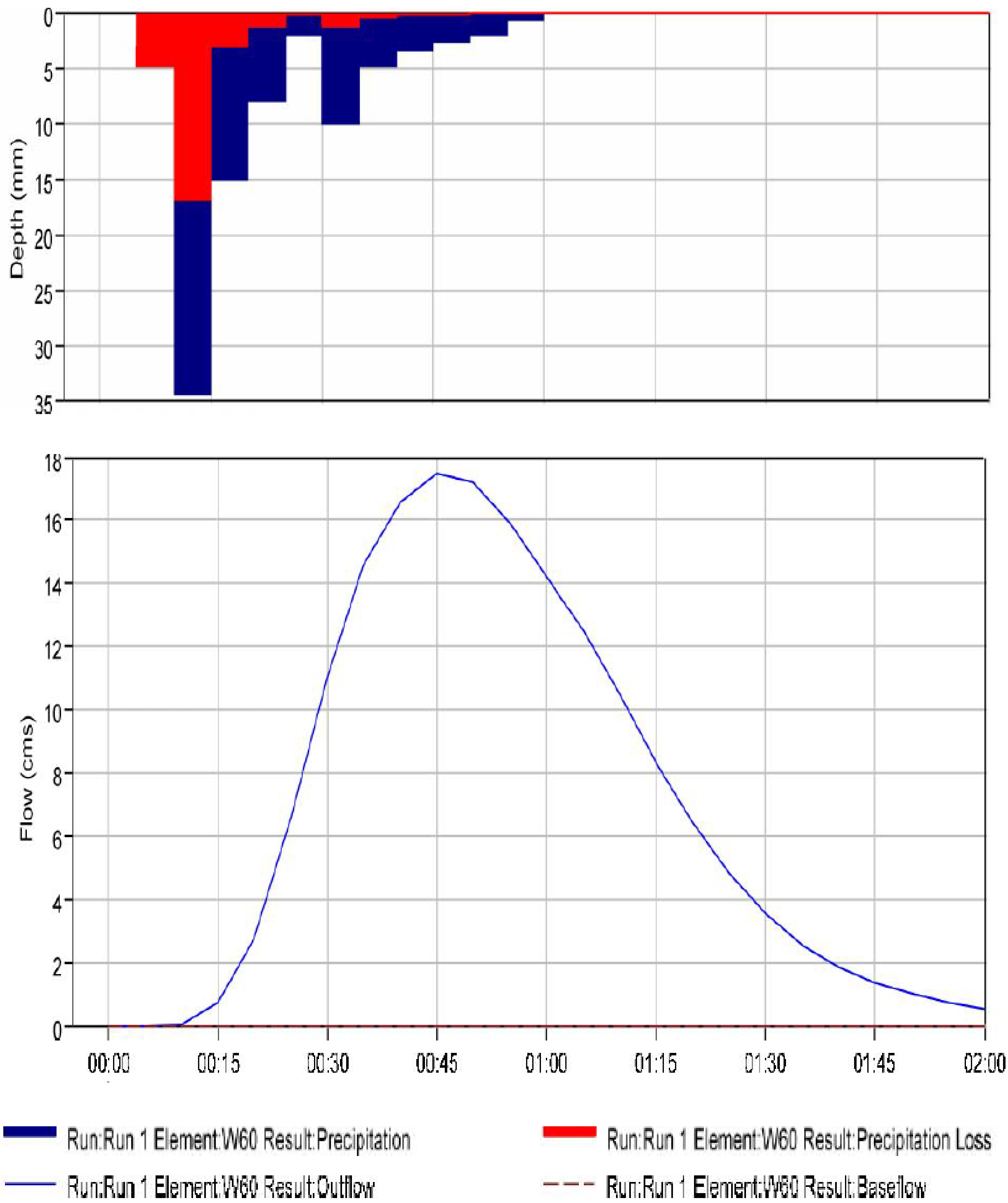
Time	Precip (MM)	Loss (MM)	Excess (MM)	Direct Flow (M3/S)	Baseflow (M3/S)	Total Flow (M3/S)
00:00				0.0	0.0	0.0
00:05	2.48	2.48	0.00	0.0	0.0	0.0
00:10	3.79	3.79	0.00	0.0	0.0	0.0
00:15	26.84	15.84	11.00	0.5	0.0	0.5
00:20	11.64	3.26	8.38	1.7	0.0	1.7
00:25	6.26	1.36	4.90	4.2	0.0	4.2
00:30	1.70	0.33	1.37	7.2	0.0	7.2
00:35	10.18	1.73	8.45	9.7	0.0	9.7
00:40	4.81	0.69	4.12	11.5	0.0	11.5
00:45	3.55	0.47	3.08	12.6	0.0	12.6
00:50	2.70	0.33	2.37	13.0	0.0	13.0
00:55	2.11	0.25	1.86	12.6	0.0	12.6
01:00	0.77	0.09	0.68	11.8	0.0	11.8
01:05	0.00	0.00	0.00	10.6	0.0	10.6
01:10	0.00	0.00	0.00	9.1	0.0	9.1
01:15	0.00	0.00	0.00	7.3	0.0	7.3
01:20	0.00	0.00	0.00	5.7	0.0	5.7
01:25	0.00	0.00	0.00	4.3	0.0	4.3
01:30	0.00	0.00	0.00	3.1	0.0	3.1
01:35	0.00	0.00	0.00	2.3	0.0	2.3
01:40	0.00	0.00	0.00	1.6	0.0	1.6
01:45	0.00	0.00	0.00	1.2	0.0	1.2
01:50	0.00	0.00	0.00	0.9	0.0	0.9
01:55	0.00	0.00	0.00	0.6	0.0	0.6
02:00	0.00	0.00	0.00	0.5	0.0	0.5



(Source: HEC HMS)

Developed hydrograph for 5year return period for watershed5

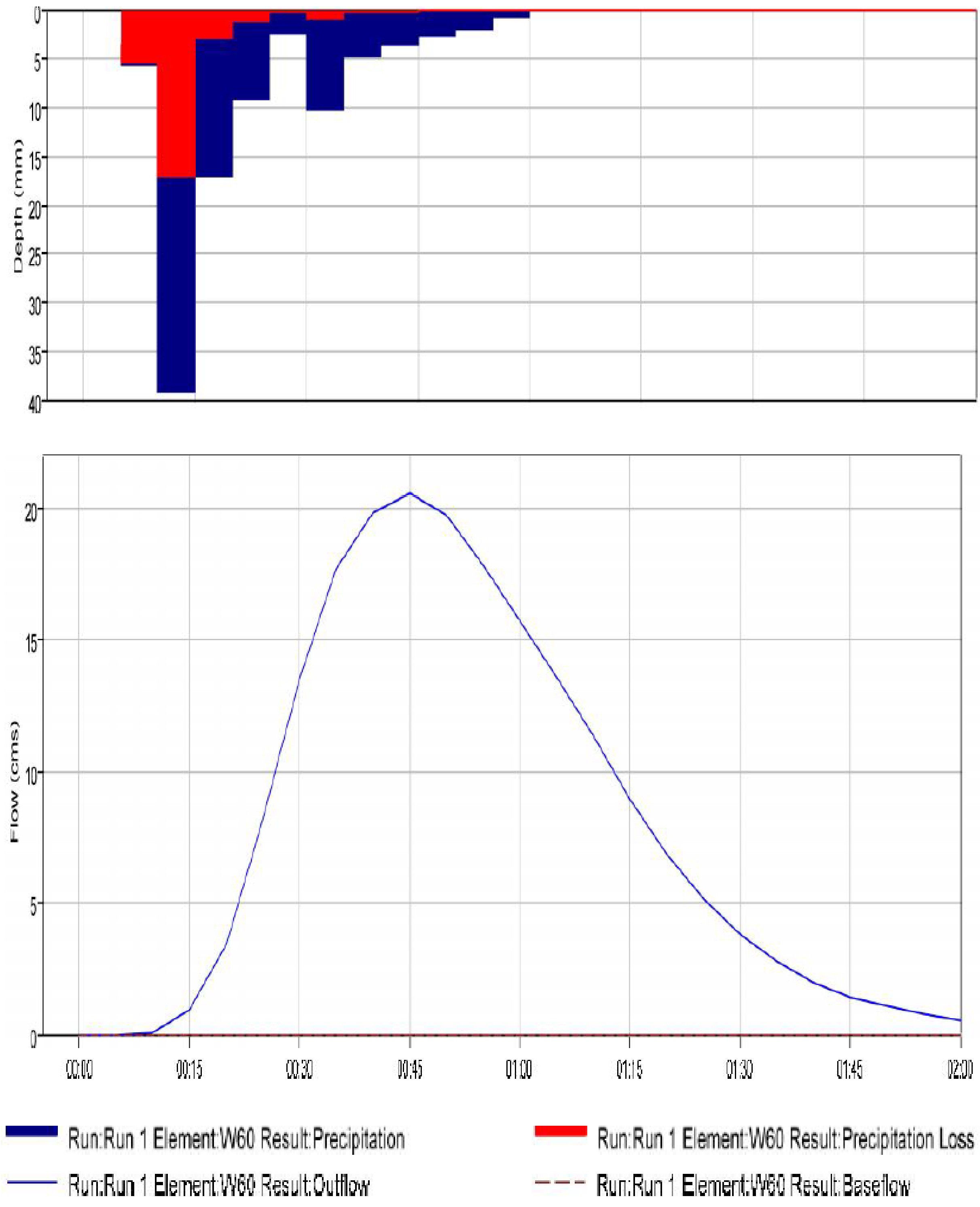
Time	Precip (MM)	Loss (MM)	Excess (MM)	Direct Flow (M3/S)	Baseflow (M3/S)	Total Flow (M3/S)
00:00				0.0	0.0	0.0
00:05	3.19	3.19	0.00	0.0	0.0	0.0
00:10	4.89	4.86	0.03	0.0	0.0	0.0
00:15	34.58	16.81	17.77	0.7	0.0	0.7
00:20	15.00	3.09	11.91	2.7	0.0	2.7
00:25	8.07	1.26	6.81	6.6	0.0	6.6
00:30	2.19	0.30	1.89	11.0	0.0	11.0
00:35	10.18	1.25	8.93	14.6	0.0	14.6
00:40	4.81	0.51	4.30	16.6	0.0	16.6
00:45	3.55	0.35	3.20	17.5	0.0	17.5
00:50	2.70	0.25	2.45	17.2	0.0	17.2
00:55	2.11	0.19	1.92	15.9	0.0	15.9
01:00	0.77	0.07	0.70	14.2	0.0	14.2
01:05	0.00	0.00	0.00	12.5	0.0	12.5
01:10	0.00	0.00	0.00	10.5	0.0	10.5
01:15	0.00	0.00	0.00	8.3	0.0	8.3
01:20	0.00	0.00	0.00	6.4	0.0	6.4
01:25	0.00	0.00	0.00	4.8	0.0	4.8
01:30	0.00	0.00	0.00	3.5	0.0	3.5
01:35	0.00	0.00	0.00	2.6	0.0	2.6
01:40	0.00	0.00	0.00	1.9	0.0	1.9
01:45	0.00	0.00	0.00	1.3	0.0	1.3
01:50	0.00	0.00	0.00	1.0	0.0	1.0
01:55	0.00	0.00	0.00	0.7	0.0	0.7
02:00	0.00	0.00	0.00	0.5	0.0	0.5



(Source: HEC HMS)

Developed hydrograph for 10year return period for watershed5

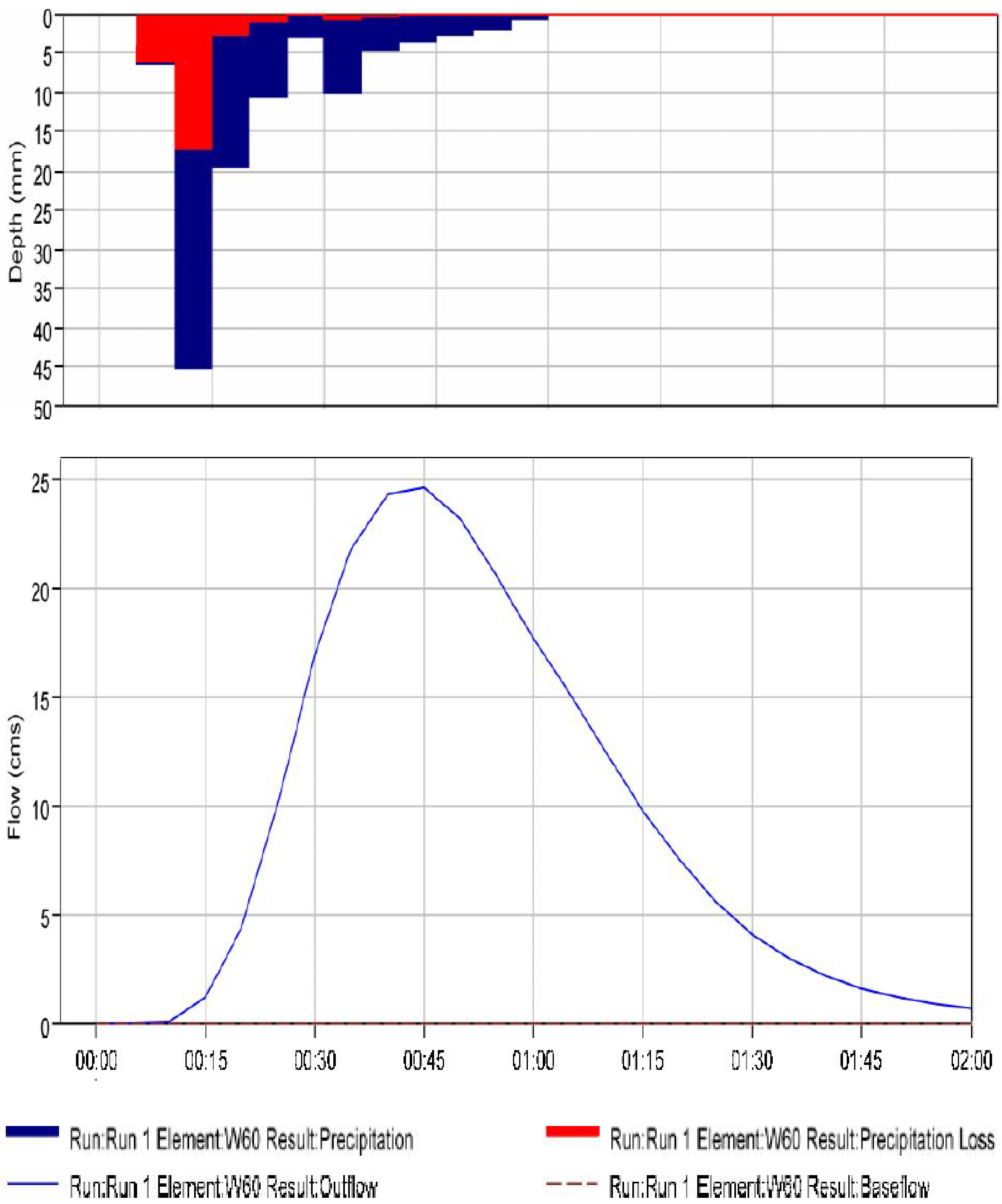
Precip (MM)	Loss (MM)	Excess (MM)	Direct Flow (M3/S)	Baseflow (M3/S)	Total Flow (M3/S)
			0.0	0.0	0.0
3.62	3.62	0.00	0.0	0.0	0.0
5.55	5.44	0.11	0.0	0.0	0.0
39.22	17.13	22.09	0.9	0.0	0.9
17.01	2.98	14.03	3.4	0.0	3.4
9.15	1.19	7.96	8.1	0.0	8.1
2.48	0.29	2.19	13.5	0.0	13.5
10.18	1.05	9.13	17.6	0.0	17.6
4.81	0.44	4.37	19.8	0.0	19.8
3.55	0.30	3.25	20.5	0.0	20.5
2.70	0.22	2.48	19.7	0.0	19.7
2.11	0.16	1.95	17.8	0.0	17.8
0.77	0.06	0.71	15.7	0.0	15.7
0.00	0.00	0.00	13.6	0.0	13.6
0.00	0.00	0.00	11.3	0.0	11.3
0.00	0.00	0.00	8.9	0.0	8.9
0.00	0.00	0.00	6.9	0.0	6.9
0.00	0.00	0.00	5.2	0.0	5.2
0.00	0.00	0.00	3.8	0.0	3.8
0.00	0.00	0.00	2.7	0.0	2.7
0.00	0.00	0.00	2.0	0.0	2.0
0.00	0.00	0.00	1.4	0.0	1.4
0.00	0.00	0.00	1.0	0.0	1.0
0.00	0.00	0.00	0.8	0.0	0.8
0.00	0.00	0.00	0.6	0.0	0.6



(Source: HEC HMS)

Developed hydrograph for 25year return period for watershed5

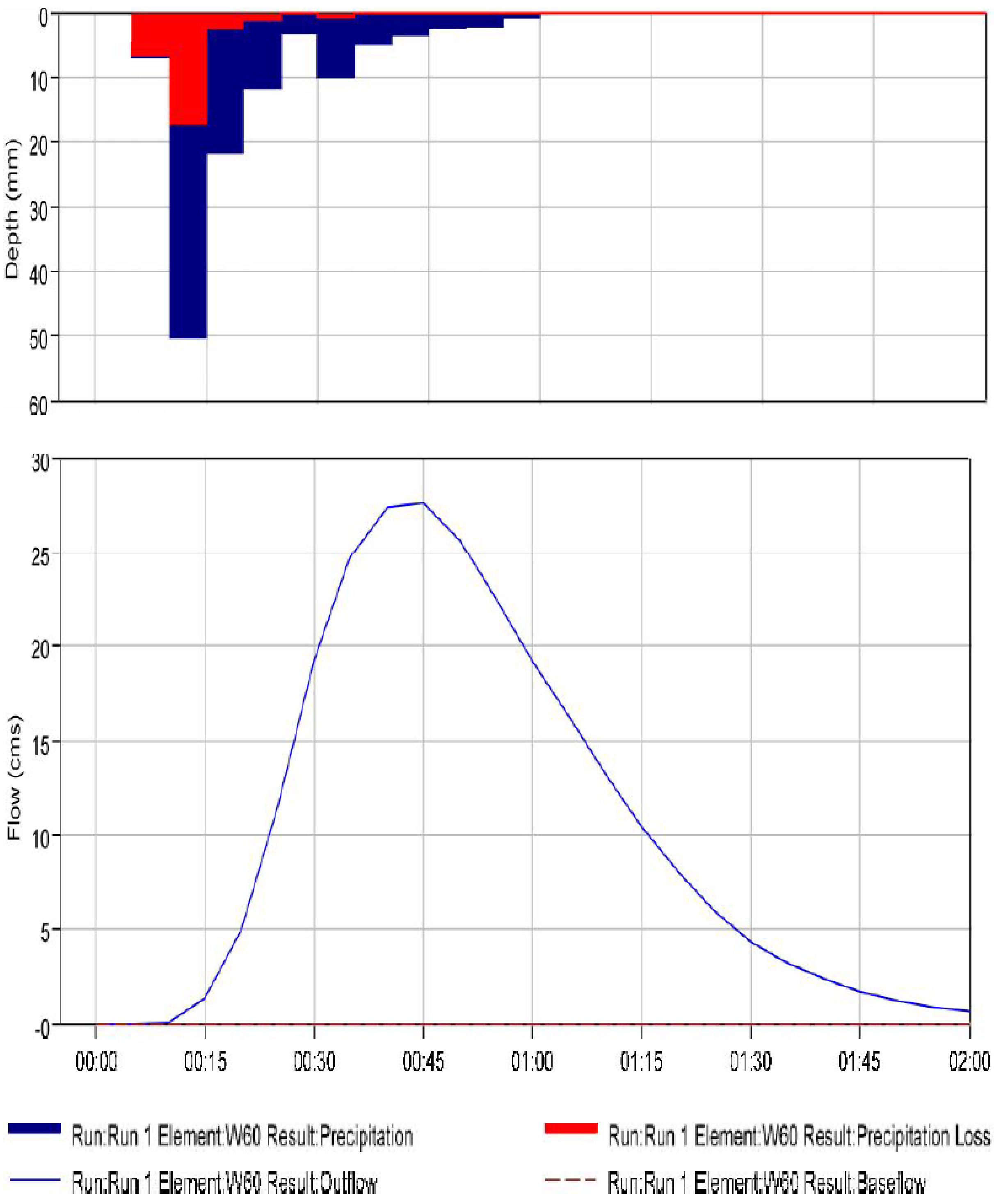
Time	Precip (MM)	Loss (MM)	Excess (MM)	Direct Flow (M3/S)	Baseflow (M3/S)	Total Flow (M3/S)
00:00				0.0	0.0	0.0
00:05	4.19	4.19	0.00	0.0	0.0	0.0
00:10	6.42	6.10	0.32	0.0	0.0	0.0
00:15	45.42	17.38	28.04	1.2	0.0	1.2
00:20	19.70	2.82	16.88	4.3	0.0	4.3
00:25	10.60	1.12	9.48	10.2	0.0	10.2
00:30	2.88	0.27	2.61	16.9	0.0	16.9
00:35	10.18	0.85	9.33	21.8	0.0	21.8
00:40	4.81	0.36	4.45	24.2	0.0	24.2
00:45	3.55	0.25	3.30	24.6	0.0	24.6
00:50	2.70	0.18	2.52	23.1	0.0	23.1
00:55	2.11	0.14	1.97	20.4	0.0	20.4
01:00	0.77	0.05	0.72	17.6	0.0	17.6
01:05	0.00	0.00	0.00	15.0	0.0	15.0
01:10	0.00	0.00	0.00	12.4	0.0	12.4
01:15	0.00	0.00	0.00	9.7	0.0	9.7
01:20	0.00	0.00	0.00	7.5	0.0	7.5
01:25	0.00	0.00	0.00	5.6	0.0	5.6
01:30	0.00	0.00	0.00	4.1	0.0	4.1
01:35	0.00	0.00	0.00	2.9	0.0	2.9
01:40	0.00	0.00	0.00	2.1	0.0	2.1
01:45	0.00	0.00	0.00	1.6	0.0	1.6
01:50	0.00	0.00	0.00	1.1	0.0	1.1
01:55	0.00	0.00	0.00	0.8	0.0	0.8
02:00	0.00	0.00	0.00	0.6	0.0	0.6



(Source: HEC HMS)

Developed hydrograph for 50year return period for watershed5

Time	Precip (MM)	Loss (MM)	Excess (MM)	Direct Flow (M3/S)	Baseflow (M3/S)	Total Flow (M3/S)
00:00				0.0	0.0	0.0
00:05	4.62	4.62	0.00	0.0	0.0	0.0
00:10	7.08	6.55	0.53	0.0	0.0	0.0
00:15	50.06	17.45	32.61	1.4	0.0	1.4
00:20	21.72	2.71	19.01	5.0	0.0	5.0
00:25	11.69	1.07	10.62	11.6	0.0	11.6
00:30	3.17	0.26	2.91	19.2	0.0	19.2
00:35	10.18	0.74	9.44	24.7	0.0	24.7
00:40	4.81	0.31	4.50	27.4	0.0	27.4
00:45	3.55	0.22	3.33	27.6	0.0	27.6
00:50	2.70	0.16	2.54	25.7	0.0	25.7
00:55	2.11	0.12	1.99	22.4	0.0	22.4
01:00	0.77	0.04	0.73	19.2	0.0	19.2
01:05	0.00	0.00	0.00	16.2	0.0	16.2
01:10	0.00	0.00	0.00	13.3	0.0	13.3
01:15	0.00	0.00	0.00	10.4	0.0	10.4
01:20	0.00	0.00	0.00	8.0	0.0	8.0
01:25	0.00	0.00	0.00	6.0	0.0	6.0
01:30	0.00	0.00	0.00	4.4	0.0	4.4
01:35	0.00	0.00	0.00	3.2	0.0	3.2
01:40	0.00	0.00	0.00	2.3	0.0	2.3
01:45	0.00	0.00	0.00	1.7	0.0	1.7
01:50	0.00	0.00	0.00	1.2	0.0	1.2
01:55	0.00	0.00	0.00	0.9	0.0	0.9
02:00	0.00	0.00	0.00	0.7	0.0	0.7



(Source: HEC HMS)

Appendix 14: Summary tables extracted from HY8 software

Culvert Summary Table: Culvert

Total Discharge (cms)	Culvert Discharge (cms)	Headwater Elevation (m)	Inlet Control Depth (m)	Outlet Control Depth (m)	Flow Type	Normal Depth (m)	Critical Depth (m)	Outlet Depth (m)	Tailwater Depth (m)	Outlet Velocity (m/s)	Tailwater Velocity (m/s)
0.00	0.00	2296.90	0.000	0.000	0-NF	0.000	0.000	2.070	0.000	0.000	0.000
5.02	5.02	2297.76	0.859	0.280	1-JS1t	0.171	0.543	2.568	0.498	0.488	1.438
10.03	10.03	2298.26	1.364	0.593	1-JS1t	0.264	0.862	2.846	0.776	0.881	1.847
15.05	15.05	2298.69	1.794	0.886	1-JS1t	0.342	1.130	3.081	1.011	1.221	2.126
20.06	20.06	2299.09	2.188	1.181	1-S2n	0.411	1.369	0.622	1.224	8.067	2.342
25.08	25.08	2299.45	2.552	1.477	1-S2n	0.475	1.588	0.751	1.422	8.352	2.519
30.10	30.10	2299.80	2.900	1.794	1-S2n	0.536	1.793	0.876	1.611	8.593	2.669
35.11	35.11	2300.14	3.242	2.127	1-S2n	0.594	1.987	0.997	1.792	8.806	2.800
40.13	40.13	2300.49	3.590	2.477	5-S2n	0.648	2.173	1.115	1.966	8.995	2.915
45.14	45.14	2300.85	3.951	2.846	5-S2n	0.701	2.350	1.231	2.136	9.168	3.019
46.33	46.33	2300.94	4.039	2.936	5-S2n	0.714	2.391	1.258	2.176	9.207	3.042