



ADDIS ABABA UNIVERSITY

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

Hydrologic and Hydraulic Adequacy Assessment of Drainage Structures in Bila Town

By

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Abstract

The objective of the study is to assess adequacy of Drainage structures of Bila town. The study employed both primary and secondary data collection. The primary data collection were observation (field visit), and photographs that show the existing drainage structure conditions and information that were gathered from the residents about the drainage structures during the rainy season. The secondary data sources were master plan of the town, rainfall data, and demography of the area, socio economy of the town, land use land cover information and climatic condition.

Daily rainfall data is obtained from nearby Nedjo rain gage station. Consistency of the rainfall was checked With Double mass curve analysis using rain gage stations Gimbi, Dengero, Deddessa, Ayira and Dembidollo and fitting distribution frequency is checked using log Pearson type III method.

The catchment is divided in to 5 small sub catchments, 13 conduits, with 13 junction nodes and 5 outfall. Tape meter and GPS is used to obtain dimension width, depth and elevation of each drainage structures. Sub catchment Imperviousness is obtained using land use land cover data and percentage of imperviousness for different land surface.

Hydrologic analysis and hydraulic analysis was carried out by EPA SWMM 5 in which used to simulate response of catchment to rainfall events in which runoff, water depth profile, node surcharge, outfall loading, node flooding ,link flow and conduit surcharge are obtained. Runoff is also obtained from rational formula for comparison purpose. Parameters needed for rational method are taken from ERA manual. And selected extreme daily event was disaggregated into hourly events using reduction formula. At point culvert found is where outfall loading obtained therefore maximum discharge and peak discharge is used to analysis culverts using HY-8 culvert analysis tool.

The results from the model show that there are nodes flooded, storm water network system has been not well planned and has not sufficient carrying capacity to satisfy the simulated rainfall event.

Then storage facilities Detention pond was proposed for reduction of flooding effect at downstream where coffee plant is affected

Finally appropriate recommendation is given in order to have a standardized and harmonized urban drainage systems

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Acronyms

AACRA: Addis Ababa City Roads Authority

AASHTO: American Association of State Highway and Transportation Officials

DEM: Digital Elevation Model

EMA: Ethiopian Mapping Agency

EPA: Environmental Protection Agency

ERA: Ethiopian Roads Authority

FAO: Food and Agriculture Organization

FHWA: Federal Highway Administration

GIS: Geographic Information System

GPS: Geographic positioning System

Ha: Hectare

HDS: Hydraulic Design Series

HEC: Hydraulic Engineering Circular (FHWA)

HEC: Hydrologic Engineering Center (USACE)

HSG: Hydrological Soil Group

IDF: Intensity-Duration-Frequency

Km² : Square kilometer

M: Meter

masl: Meter Above Sea Level

MOUDC: Ministry of Urban Development and Construction

NMSA: National Meteorological Service Agency

R²: Coefficient of Determination

SWMM: Storm Water Management Model

WMO: World Meteorological Organization

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1. INTRODUCTION

1.1 Back ground

During the design storm event drainage system design is to provide for safe passage of vehicles, to collect storm water runoff from the road surface and right-of-way, convey it along and through the right-of way, and discharge it to an adequate receiving body without causing adverse on- or off-site impacts. Storm water collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Rapid removal of storm water from the pavement minimizes the conditions which can result in the hazards of hydroplaning. storm water conveyance systems (storm drain piping, ditches and channels, pumps, etc.) is to provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without surcharging inlets or otherwise causing surface flooding. Erosion potential must also be considered in the design of open channels or ditches used for storm water conveyance [1]

Drainage problems in urban areas introduce flooding, deterioration of roads, land degradation, sedimentation, water logging and etc. With urbanization, impermeability of land increases with the increase in impervious surfaces (i.e. residential houses, commercial buildings, paved roads, parking lots, etc.) with this drainage pattern changes, overland flow gets faster, flooding and environmental problems such as land degradation increases. And it becomes a crucial problem facing the existing and future road infrastructure [1]

Lack of urban Storm water drainage management represent one of the most common sources of complaint from the residents in many urban centers of Ethiopia [1]From this Bila town was the one have complain on drainage structure management and in sufficient drainage structures.

As the case study at Ginjo Guduru Kebele of Jimma town, Getachew et al [2] based their research on the assessment of the effect of urban road surface drainage: They: assessed the pavement damage due to improper drainage, identified areas most prone to flooding problems, assessed the existing condition of road and surface drainage infrastructure, examine the impacts of road surface drainage structures integration on road performance and related social as well as environment issues and make recommendations on urban road and drainage structures integration. From the study made, generally, they observed that the road surface drainage found to be inadequate due to insufficient road profile, insufficient drainage structures provision, improper maintenance and lack of proper interconnections

between the road and drainage infrastructures there by resulting to the damages to road surface material and flooding in the area. Carried out an investigation into the adequacy of the drainage system on Narok-Mai Mahiu road. They employed a research survey in order to obtain the information that would describe the state of drainage infrastructure in Narok Maai Mahiu road and how poor drainage affected the surrounding environment and the road users. Various data collection techniques that were used include questionnaires, photographs, observation, and interviews. The results indicated that Narok Maai Mahiu road drainage system was not adequate to satisfactorily drain the runoffs and reduce the speed of water. As case study, in two major roads in Khartoum state , Magdi M. E. Zumrawi [3] .based on their research The Impacts of Poor Drainage on Road Performance were selected for field survey of the existing condition of the provided drainage system and its subsequent impacts on pavement performance. This paper aims to study the various impacts of inadequate drainage on road pavement condition. Poor drainage causes early pavement distresses lead to driving problems and structural failures of road. To prevent or minimize premature pavement failures and to enhance the road performance, it is imperative to provide adequate drainage. As a result of this investigation, it was found that most roads in Khartoum state suffered from poor drainage which causes severe distresses and damages of pavement. according to research done on Performance Assessment of Road Drainage Systems of Burayu Town, by Mulualem Bekele [4], In most part of the town, runoffs run over road surfaces this is due to soil erosion, lack of drainage lines along most of the roads in the town, lack of appropriate maintenance of existing drainage facilities, poor waste management system and the drainage channels are filled with or blocked by silt and garbage. For this reason, the road drainage structure impairs their ability to convey the runoff properly. Additionally based on research done on Performance Assessment of Road Drainage Structures and Proposed Mitigation Measures: The case of Daleti-Odagodere Gravel Road in Benishangul-Gumuz Region, Yifred Kassa [5]. improper site selection for drainage structures, drainage structures at critical points where failure of carriageway due to over saturation and overtopping is expected, improper alignment of drainage structures at sections where scouring due to stream crosscurrent occurs and poor construction workmanship contribute to the problem.

Generally from previous work we observe drainage management and improper drainage structure existence cause adverse impact on environment. Therefore to fill the gap of management and minimize the drainage impact the assessment of hydrological and hydraulic adequacy of drainage structure was

vital and to give input data for future works in selected area, which reduce the complaint and give good communication between kebele and towns.

1.2 Statement of problems

Proper drainage system is one of the most important factors to be considered in the road design, construction and maintenance projects. When a road fails, whether it is concrete, asphalt or gravel, inadequate drainage is often a major factor to be considered.

Rainfall causes minor flooding, erosion of unsealed roads, due to inadequate drainage and lack of green spaces to absorb excess runoff. During the rainy seasons, the streets are surrounded by rainfall blocking traffic and disrupting the movements of people. Local, regional, and/or Federal regulations often control the allowable quantity and quality of storm water discharges. This regulation is not available in Ethiopia at the present time.

In Bila town due to road drainage structures are not properly functioning there are insufficient capacity of road ditches, unavailability of drainage structures at proper place, small number of culverts, street flood, poor management and siltation.

Concerning the Major problem observed in Bila town related to drainage the following are some part: Absence of appropriate maintenance of the existing drainage facilities (existing drainage channels are partially filled with silts and causing the storm water to flow over the road surface) and formation of gullies and Absence of drainage lines along most of the roads in the town

Downstream of Bila town was occupied with traditional farm land practices such as coffee, maize, Potato and sugarcane but the earn expected from them are not gained because flooding comes from the town cause waterlogging and gullies, beside to this loss of land are observed.

Next the increased in urbanization from time to time would increase surface runoff that will cause impact continuously

1.3 Significance of the study

The advantage that will be draw from this thesis may contribute to governments and other concerning bodies to solve the problem. To understand problems of structures inadequate and for taking correct measures as well as to reduce any inconvenience and disruption to travel due to over flow of water in the

main road due to flooding. The result at end helps for filling the gaps by identifying problems for proper designing of Storm water drainage system and proper functioning of Drainage schemes in the town.

1.4 Objective

1.4.1 General objectives

- The general objectives of the study is to determine adequacy assessment of existing drainage structures for Bila Town

1.4.2 Specific objective

- To assess hydrologic and hydraulic properties of existing structure
- Proposing Detention pond for area affected by flooding
- To recommend suitable measures

1.5. Research question

- ✓ What is existing drainage structure problems?
- ✓ What are hydraulic and hydrologic adequacy of drainage structures?
- ✓ What to be learned from the result?

1.6. Scope and Limitation of the Thesis

This thesis specifically focused on assessing existing drainage structures specially area affected by flooding problems, to do assessment for drainage structures; rational methods and EPA SWMM model peak runoff output were used for comparison. Drainage structure culvert is analyzed using HY-8 culvert analysis tool and also Detention pond was proposed to decrease effect of flooding.

The study was limited to the some parts of Bila town because no more existing drainage structures and the whole area was not affected by flooding problems. Beside to the model EPA SWMM the software needs primary data with high quality to minimize the errors within the data This thesis models deals with overall drainage effectiveness and compare the results. This research also does not include structural design of all types of drainage structures except proposing the type and size of required detention pond structures in area prominently affected.

1.7 Outline of the Research

The thesis is organized into five chapters from introduction part to the conclusion and recommendation. The first chapter's deals with introduction, the objective of the research, research question, statement of the problem, scope of the study and limitation of the thesis, and the second chapter contain literature review. The third chapter is about description of the study area, materials, methodology and procedure to be applied in the research from data collection to the result of the analysis. The forth chapter includes assessing parts, result and discussion and the last chapter focused on the conclusion and recommendation parts which concludes the study and recommends the possible solutions.

2. LITERATURE REVIEW

2.1 Hydrology

Hydrology is generally defined as a science dealing with the interrelationship between water on and under the earth and in the atmosphere. For the purpose of this section, hydrology will deal with estimating flood magnitudes as the result of precipitation. In the design of roadway drainage structures, floods are usually considered in terms of peak runoff or discharge in cubic meters per second (cms) and hydrographs as discharge per time.

2.1.1 Precipitation

Water evaporated from exposed water surfaces like streams, rivers, ponds, and also from land and plants in the form of water vapour .this water vapour get collected in atmosphere and behave like a gas. As evaporation continues, the amount of atmospheric vapour goes on increasing but since atmospheric can hold only certain fixed amount of water vapour in the presence of solid or liquid surface .the vapour may condensed in different forms such as, midst, hail, snow, rain, sleet. etc. Then the water which comes back to the earth surface in its various forms such as midst, hail, snow, rain. Sleet, etc is known as precipitation [6]

2.1.2 Runoff

New developments have a direct impact on existing drainage infrastructure and the surrounding environment [7]. They increase the area of paved surfaces, thus reducing infiltration, while causing surface runoff to exhibit higher peak flows, larger volumes, and shorter times to peak and accelerated transport of pollutants and sediment from urban areas. This results in pollution of the receiving watercourses and increased flood risk within the development. Controlling surface runoff thus becomes a key element in working towards urban sustainability [8]

2.1.3 Flooding

Floods generally develop over a period of days, when there is too much rainwater to fit in the rivers and water spreads over the land next to it (the „floodplain“). However, they can happen very quickly when lots of heavy rain falls over a short period of time. These „flashfloods occur with little or no warning and cause the biggest loss of human life than any other type of flooding [9]

2.1.3.1 Flood Types

Flash Floods: rainfall with a very high intensity or sudden massive melting of snow are agents for formation of flash flood. the area covered by water in a flash flood is relatively small compared to other types of floods. The amount of water that covers the land is usually not very large, but is so concentrated on a small area that it can raise very high. Because of the sudden onset and the high travelling speed of the water, flash floods can be very dangerous. The water can transport large objects like rocks, trees and cars. When a dyke breaks along the sea or along a river, the water may flow in so suddenly and with such speed that you could compare it with a flash flood [10]

Coastal Floods: the causes of coastal flood is a severe storm. The storm wind pushes the water up and creates high waves. A storm is formed in a low pressure area. An interesting fact is that beneath a low pressure area the sea level is higher. This contributes to the high sea level, but the wind can have a larger effect. A flood starts when waves move inland on an undefended coast or overtop or breach the coastal defense works like dunes and dikes. The waves attack the shore time and again. When it is a sandy coast, each wave in a storm will take sand away. Eventually a dune may collapse that way. Very characteristic of a coastal flood is that the water level drops and rises with the tide. At high tide the water may flow in and at low tide it may recede again. When a sea defense is breached, low tide is the time to repair the breach. In the animation you see the build-up of force by the sea and how the sea floods the coast [10]

Urban Floods: urban flooding is specific in the fact that the cause is a lack of drainage in an urban area. As there is little open soil that can be used for water storage nearly all the precipitation needs to be transported to surface water or the sewage system. High intensity rainfall can cause flooding when the city sewage system and draining canals do not have the necessary capacity to drain away the amounts of rain that are falling. Water may even enter the sewage system in one place and then get deposited somewhere else in the city on the streets. Urban floods are a great disturbance of daily life in the city. Roads can be blocked, people can't go to work or to schools. The economic damages are high but the number of casualties is usually very limited, because of the nature of the flood. The water slowly raises on the city streets. When the city is on flat terrain the flow speed is low and you can still see people driving through it. The water rises relatively slow and the water level usually does not reach life endangering heights [10].

River floods: Rainfall over an extended period and an extended area can cause major rivers to overflow their banks. The water can cover enormous areas. Downstream areas may be affected, even when they didn't receive much rain themselves. With large rivers the process is relatively slow. The rain water enters the river in many ways. Some rain will fall into the river directly, but that alone doesn't make the river rise high. A lot of rain water will run off the surface when the soil is saturated or hard. It will flow to small rivers that flow to larger rivers and these rivers flow into even larger rivers. In this way all the rain that fell in a large area (catchment area) comes together in this one very large river. When there is a lot of rain over a long period, it takes time for all the rainwater to reach the river. While the water level slowly rises, officials can decide to evacuate people before the river overflows. The area that is flooded can be huge. Villages surrounded by large stretches of water where cattle would normally graze. Whole communities can become isolated from the rest of the world as roads are blocked and communications are down. When a dike or a dam breaks and a lot of water is released suddenly, the speed of the water at the breach can be compared with the speed of a flash flood. As a larger area gets covered the speed will be reduced. The water spreads out as much as possible flowing to the lower lying areas before slowly rising. A breach is very dangerous for the people living close to it. The strength of the water may carry cars, trees and even houses away and cause loss of life [10]

Pluvial Floods: Pluvial is a type of flooding that can happen in relatively flat areas. Rain water falling in an area is normally stored in the ground, in canals or lakes, or is drained away, or pumped out. When more rainwater enters a water system than can be stored, or can leave the system, flooding occurs. In this case, rain is the source of the flood: not water coming from a river, but water on its way to the river. That's why it is also called "pluvial flood". Puddles and ponds develop on the land, canals are filled to brim and spill over; gradually a layer of water covers the land. It is like urban flooding, but without the sewage systems and in more rural areas. Because of the gradual character people have time to go indoors or leave the area. The layer of water is no more than centimeters or perhaps decimeters high and causes no immediate threat to people's life. Depending on the economic activity and size of the area that is covered it may cause immense economic damage [10]

2.1.3.2 Causes of flooding

Floods are natural phenomena which cannot be prevented. Flooding is described as a condition where wastewater and/or surface water escapes from or cannot enter a drain or sewer system and either remains

on the surface or enters buildings". Flooding is often thought of as a result of heavy rainfall, but floods can arise in a number of ways that are not directly related to ongoing weather events. Thus, a complete description of flooding must include processes that may have little or nothing to do with meteorological events. Flooding, by its very nature, is usually a result of both meteorological and hydrologic processes; the character of a flood is determined both by the detailed behavior of the precipitation and by the nature of situation in which the event is likely to occur (soil conditions, amount of antecedent rainfall, and so on). It is not likely that precisely detailed forecasts of flooding events will ever be possible, although it is certainly well within our capability to anticipate the possibility of most flood events [11]

2.1.3.3 Effects of Flooding

Floodwater can seriously disrupt public and personal transport by cutting off roads and railway lines, as well as communication links when telephone lines are damaged. Floods disrupt normal drainage systems in cities, and sewage spills are common, which represents a serious health hazard, along with standing water and wet materials in the home. Bacteria mould and viruses, cause disease, trigger allergic reactions, and continue to damage materials long after a flood. Floods can distribute large amounts of water and suspended sediment over vast areas, restocking valuable soil nutrients to agricultural lands. In contrast, soil can be eroded by large amounts of fast flowing water, ruining crops, destroying agricultural land / buildings and drowning farm animals. Severe floods not only ruin homes / businesses and destroy personal property, but the water left behind cause's further damage to property and contents. The environment and wildlife is also at risk when damage when damage to businesses causes the accidental release of toxic materials like paints, pesticides, gasoline etc. Floodwater can severely disrupt public and personal transport by cutting off roads and railway lines, as well as communication links when telephone lines are damaged. Unfortunately, flooding not only disrupts many people's lives each year, but it frequently creates personal tragedies when people are swept away and drowned.(www.environmentagency.gov.uk/fun)

2.2 Drainage Structures

2.2.1 Culvert

A culvert is any structure not classified as a bridge that provides an opening under a roadway, and other type of access or utility.

A culvert is defined as the following:

- A structure that is usually designed hydraulically to take advantage of submergence to increase hydraulic capacity.
- A structure used to convey surface runoff through embankments.
- A structure, as distinguished from bridges, that is usually covered with embankment and is composed of structural material around the entire perimeter, although some are supported on spread footings with the streambed serving as the bottom of the culvert.

2.2.1.1 Necessity of Drainage Culverts

Construction of a road embankment unavoidably obstructs and interferes with the natural overland flow and flow through the natural channels e.g. rivers, canals, drains etc. Suitable bridge/culvert openings under the road should, therefore, be provided across these channels with a view to pass the peak discharge through the channels without causing harmful afflux and disturbing the natural flow regime. Provision of adequate numbers of culverts of appropriate size is a prerequisite for a healthy road.

Submergence and overtopping of road not only causes damage to the road and road structures, it results in disruption of traffic, loss of travel time and miseries to many of the poor people who take shelter on roads during floods in many parts of our country. Overland flow, which would otherwise meet the natural stream at some downstream point, must be intercepted in longitudinal drains and discharged back into the nearest natural drainage channel through culverts and bridges. The local drainage arrangements consisting of longitudinal drains and culverts shall have to be designed to carry the runoff from the road surface too. Where a road runs in an undulating terrain, causeways or dips are often provided in valleys to avoid road in high embankment. Frequent dipping down from high road levels to the ground produces a very undesirable road profile. Constructing bridges and culverts under road in high embankment is a better proposition than providing so many dips and causeways leading to disruption in traffic movement during flood season. Bridges, culverts and underpasses are often used by local people and livestock to cross the busy roads like national and state highways in high embankments. They also act as passage for up and down movement of fish and other aquatic animals. Sediments and debris carried by the stream, especially during floods, must freely move downstream through these openings to avoid aggradations and other interrelated problems [12] For existing roads, it is not uncommon to find silted barrels of existing culverts and culverts having inadequate capacities causing

overtopping of the road embankments. In a very flat terrain, most of the streams are shallow and the banks are spilled with flood water moving in wide flood plains. In the absence of road, the spill flow moving over the land surface constitutes a substantial amount of peak flood. When a road is built in such a terrain with wide flood plains, the entire flood water has to move across the road through the bridge opening of limited span, resulting in very high afflux and other problems [13] Usually, the spill water is found to move along the toe of the road causing scouring and damage to road embankment. Provision of relief culverts on either side of the bridges in such flood plains are very helpful in the quick disposal of spill flood across the road which results in less afflux and ensures safety of the road embankment.

2.2.1.2. Location of Culverts

To be most effective, locations of culverts have to be very carefully decided after studying the terrain and collecting relevant information from top sheets and other sources. Field visit and consultation with local people and local authorities conversant with local topography and drainage problem of the area is extremely useful. Culvert skews are not advisable unless conditions do not permit to install culverts normal to the natural streambed. Sharp changes in the direction of flows to force shorter culvert crossings are prone to scouring. The eroded material has potential to block the culvert opening. Sharp and small radius bends also reduce the hydraulic efficiency of a channel [14]

2.2.1.3. Types and Size of Culverts

Culverts can be classified into two based on their functional types, runoff management and stream crossing. Runoff management culvert strategically placed to manage and route roadway runoff along, under, and away from the roadway. Many times these culverts are used to transport upland runoff, accumulated in road ditches on the upland side of the roadway, to the lower side for disposal. Stream crossing culvert is a drainage structure installed on the stream with recommended skewed angle, 150 – 450 if conditions do not permit to install normal to the stream channel. Installing culverts normal to the stream channel decreases construction cost. Where large skew angles are required, consideration of the most appropriate road alignment is significant [15] Strategically placed culverts, along with road ditch turnouts, will help to maintain a stable velocity and the proper flow capacity for the road ditches by timely out letting water.

This will help to alleviate roadway flooding, reduce erosion, and thus reduce maintenance problems.

Culverts preserve the road base by draining water from ditches along the road, and keeping the sub base dry. Culverts may be of several types and geometry namely, pipe culverts (circular and elliptic), box culverts (square and rectangular), slab and arch culverts (with or without bottom slab) etc. Selection of type and geometry of culverts inter alias depends on the required width and area of opening, height and vertical clearance required, length of culvert and height of embankment decided from geometrics of road design. While it is easier to decide between a pipe culvert and box/slab culvert, selection between box and slab culvert is a matter of cost optimization. The more the carriageway width of the road more will be the length of culvert and consequently more will be the number of joints. For mountainous regions, the culverts are generally provided at frequent interval. Pipe culverts are, therefore, very common in hilly stretches of roads. However, in stretches where the streamlets carry large size cobbles and boulders, there is a fair possibility of pipes getting damaged. Pipe culverts are, therefore, avoided in such stretches and either slab type or box type culverts are preferred. Generally, drainage structures are designed to prevent road damage during the most usual floods such as annual, 10-year, 50-year or 100-year flood, depending on the importance of the road and the type of structures [1] and to minimize the modifications in the hydrology of the area.

2.2.1.4 Size of Culverts

The size of the culvert is designed on the basis of the following considerations from the points of view of: Peak flow and hydraulic conveyance requirement ease of maintenance and desilting operation. Permissible velocity for fish movement where the channel carries fish Movement of debris, gravels, boulders etc. The required size of the culvert is decided on the basis of hydrological and hydraulic analysis. However, the minimum size of the culvert is fixed on the basis of ease in maintenance, movement of fish, debris etc.

2.2.2 Drainage ditches.

Drainage ditches that are man-made with regular geometric cross sections, and unlined, or lined with artificial or natural material to protect against erosion. Ditch is a long narrow channel dug in the ground parallel to a road usually used for drainage and as a boundary marker. Good ditches make good roads. Properly designed and constructed ditches serve a number of essential purposes: → They collect road surface run-off and drain it away from the road → They store large amounts of rainfall → With proper turnouts and buffers, they keep pollution from reaching sensitive water resources → They collect and

drain subsurface water away from the road's base and sub grade soil materials. Proper ditching involves careful consideration of many factors, including watershed size, degree of slope, width of right-of-way, ditch size and shape, and native soil type. According to Dunham (2001), for unpaved roads, Ditches must be lined with stone and grass, and for paved or asphalt road, Ditches should be constructed from cement and concrete to control erosion. The primary purpose of roadside ditches is to serve as conveyance structures preventing water from pooling on the roadway surface. Effective roadway ditches prevent overland runoff from reaching the roadway, as well as drain water from the road surface. Economical ditches should convey the intended design storm efficiently and require minimum maintenance while serving their intended purpose of roadway drainage structures.

2.3 Functions of Storm water drainage system

One of the drainage system's functions is to collect surface water and/or ground water and direct it away, thereby keeping the ballast bed drained. The drainage system must also protect the substructure from erosion, from becoming sodden, and from losing its load-bearing capacity and stability. Another main objective of storm sewer is to protect;

- Public health and safety;
- Environmental protection;
- Sustainable development;
- Occupational health and safety.

Drain and Sewer systems are provided in order to prevent spread of disease by contact with fecal and other waterborne waste, to protect drinking water sources from contamination by waterborne waste and to carry runoff and surface water away while minimizing hazards to the public. Additionally, the impact of drain and sewer systems on the receiving waters shall meet the requirements of any national or local regulations or the relevant authority [16]

2.4 Hydrologic analysis consideration

For drainage facilities, which are designed to control volume of runoff, like detention facilities, or where flood routing through culverts is used, then the entire discharge hydrograph will be of interest. The analysis of the peak rate of runoff, volume of runoff, and time distribution of flow is fundamental to the

design of drainage facilities. Errors in the estimates will result in a structure that is either undersized and causes more drainage problems or oversized and costs more than necessary.

In the hydrologic analysis for a drainage facility, it must be recognized that many variable factors affect floods. Some of the factors which need to be recognized and considered on an individual site by site basis include the following:

- Rainfall amount and storm distribution;
- Drainage area size, shape and orientation;
- Ground cover;
- Type of soil;
- Type of terrain and stream(s);
- Antecedent moisture condition;
- Storage potential (over bank, ponds, wetlands, reservoirs, channel, etc.);
- Watershed development potential;
- Form of precipitation (rain, hail, or combinations thereof); and, Elevation.

The type and source of information available for hydrologic analysis will vary from site to site. It is the responsibility of the designer to determine the information required for a particular analysis [17].

2.5 Selection of method for hydrologic analysis

The following guidelines should be used to select the hydrology method for computing the design peak flow:

Table 1.1 hydrological methods and size of drainage area

Size of drainage area	Hydrologic Method
1 – 50 hectares	Rational Formula
50 - 1300 Hectares	NRCS* TR-55 / SCS Method and Other Unit Hydrograph Methods
Greater than 1300 Hectares	NRCS TR-20 or HEC-1 Method

* U.S. Natural Resources Conservation Service (NRCS), formerly the U.S. Soil Conservation Service (SCS)

Alternatively, Modified Rational Method could be used for size of drainage area greater than 50 hectares. The peak flow from a drainage basin is a function of the basin's physiographic properties such as size, shape, slope, soil type, land use, as well as climatological factors such as selected rainfall intensities. The methods presented should give acceptable predictions.

A) Rational Formula

The rational formula is an empirical formula relating runoff to rainfall intensity. It is expressed in the following form:

$$Q=0.00278CIA \dots\dots\dots (2.1)$$

Where, Q = peak flow in cubic meters per second (m³/s)

A = drainage area in hectares

C = runoff coefficient (weighted)

I = rainfall intensity in millimeters per hour (mm/hr)

Basic Assumptions:

- The peak rate of runoff (Q) at any point is a direct function of the average rainfall intensity (I) for the time of concentration (T_c) to that point.
- The recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity.
- The time of concentration is the time required for the runoff to become established and flow from the most distant point of the drainage area to the point of discharge.

The rational method provides the most reliable results when applied to small, developed

Watersheds and particularly to roadway drainage design. The validity of each assumption should be verified for the site before proceeding.

Procedure

1. Obtain the following information for each site:

- a. Drainage area;
 - b. Land use (% of impermeable area such as pavement, sidewalks or roofs);
 - c. Soil types (highly permeable or impermeable soils);
 - d. Distance from the farthest point of the drainage area to the point of discharge; and,
 - e. Difference in elevation from the farthest point of the drainage area to the point of discharge.
2. Select the appropriate runoff coefficient (C) value
 3. Determine the time of concentration (Tc).
 4. Determine the rainfall intensity rate (I) for the selected recurrence intervals.
 5. Compute the design flow ($Q = 0.00278 CIA$).

Value for C:

The runoff coefficient (C) accounts for the effects of infiltration, detention storage, evapotranspiration, surface retention, flow routing and interception.

The product of C and the average rainfall intensity (I) is the rainfall excess of runoff per hectare. The runoff coefficient should be weighted to reflect the different conditions that exist or expected to exist in the future within watershed [17]

2.6 Hydraulic equation

2.6.1 Manning equation

Discharge is determined for a known opening size of the drainage structure and bottom slope and/or the size of the drainage structure is determined for a known discharge and bottom slope by trial and error method. The Manning's equation can be used for uniform flow in a pipe, and stream channel, but the Manning's roughness coefficient needs to be considered variable, dependent upon the depth of flow.

The Manning's equation is used for calculating the cross-sectional area, wetted perimeter, and hydraulic radius for flow of a specified depth in a pipe of known diameter and/or stream channel cross-section. Manning's equation is applicable for a constant flow rate of water through a channel with constant slope, size & shape, and roughness.

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \dots\dots\dots (2.2)$$

Where, Q = the volumetric flow rate passing through the channel reach in m³/sec.

A = the cross-sectional area of flow normal to the flow direction in m²

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

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

R = the hydraulic radius = A/P.

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

2.7 Hydraulic analysis software

2.7.1 Culvert hydraulic analysis software (Hy-8)

The federal highway administration (FHWA) Culvert Hydraulic Analysis Program (HY-8) enables users to analyze culvert performance for a highway crossing that has multiple culvert barrels that share the same tail water and roadway overtopping.

HY-8 uses HDS-5 procedures and includes:

- Culvert configurations with constants in HDS-5
- Embedded or depressed invert design;
- Utah State University (USU) outlet transition loss;
- Energy dissipater analysis (HEC-14);
- Internal hydraulic jumps;
- broken-back barrel analysis [18]

2.7.1.1 Data required to analysis culverts

2.7.1.1.1 Discharge Data

Minimum, Design, and Maximum

HY-8 will perform culvert hydraulic calculations based on the input minimum, design, and maximum discharge values. Calculations comprising the performance curve are made for ten equal discharge

intervals between the minimum and maximum values. A user may input a narrower range of discharges in order to examine culvert performance for a discharge interval of special interest.

Minimum Discharge

Lower limit used for the culvert performance curve. Can be edited to a number greater than '0'.

Design Discharge

Discharge for which the culvert will be designed. Always included as one of the points on the performance curve.

Maximum Discharge

Upper limit used for the culvert performance curve.

2.7.1.1.2 Roadway data

When defining the roadway data for the culvert, the following parameters are required:

- Roadway Profile
- Roadway Station
- Crest Length
- Crest Elevation
- Roadway Surface
- Top Width

Source: HY-8 user manual

2.8 Hydrologic and hydraulic software

2.8.1 The EPA Storm Water Management Model (SWMM)

The EPA Storm Water Management Model (SWMM) is one of the most commonly used rainfall-runoff simulation model.

- i) SWMM is widely used in analysis and design of storm water drainage systems, especially on urban areas.

- ii) There have been previous studies carried out on the same geographical area using the same software.
- iii) The U.S. Environmental Protection Agency (EPA) provides the SWMM model and a related graphical user interface free of charge.

SWMM was first developed in 1971, and has since then been upgraded several times. The current version (number 5) was completely re-written by the U.S. EPA and a consulting firm of CDM, Inc [19]. SWMM can be used for a range of applications. It is suitable for modeling either single precipitation events or continuous modeling of multiple events. The simulation period consists of multiple time steps, and SWMM can track both runoff quantity and quality for each time step. The model is conceptually divided into four major environmental compartments:

(i) the atmosphere compartment,

Accounting for precipitation and pollutants from air;

(ii) the land surface compartment,

Modeling areas receiving precipitation and generating runoff;

(iii) the transport compartment,

Routing flow from runoff source areas through a network of pipes, channels, etc.; and

(iv) the ground-water compartment,

Receiving infiltration from the land surface and providing input to the transport compartment. Of these, (ii) and (iii) are discussed in more detail below. The objects and processes in these four compartments account for all the major component affecting the regional water balance (see Chapter 2.1.1) [19].

The land surface compartment

The processes occurring on the land surface are an essential part of SWMM. For modeling, the land surface is commonly divided into small, sufficiently homogeneous sub catchments, each draining to a single discharge point. All of the sub catchments have their own sets of hydrological parameters such as imperviousness and depression storage. Based on these, several hydrological phenomena are simulated during every time step [19].

Water input to the sub catchments is deducted from the atmosphere compartment. Each sub catchment is assigned a Rain Gage element, containing data on rainfall intensity or volume at certain time intervals [19].

If there is standing water on sub catchment surfaces, evaporation occurs. Evaporation rates (e.g. daily or monthly) can be defined by the user, or they can be computed from a data series of daily air temperatures. In case they are computed, the Hargreaves' method (see Equation 3) is used. In addition to daily air temperatures, the site's latitude must be known [19]

SWMM offers a selection of three different built-in infiltration models.

These are

- (i) the Horton's equation (Horton, 1933),
- (ii) the Green-and-Ampt method (Green and Ampt, 1911), and
- (iii) the Curve Number method.

Infiltration only takes place on the pervious fraction of the sub catchment, defined by the imperviousness parameter. To model the ground-water conditions more precisely, one can include optional Aquifer objects in the model. Aquifers simulate the vertical movements of ground water and may exchange water with the drainage system, e.g. through leaking pipes [19]

In cold conditions, precipitation may be stored in an assigned Snow Pack object. These objects have parameters for modeling snow accumulation and snowmelt within the sub catchment. In addition, snow removal can be accounted for using a special set of parameters [19]

SWMM conceptualizes the creation of surface runoff based on the same principles as presented in Chapter 2.1.6 [19].

Surface runoff generated at the source areas is defined to flow either into another sub catchment or into a drainage system entry point [19].

The transport compartment

The other major part of SWMM is the transport compartment. It describes the hydraulic routing of runoff and possible external inflows through a network of pipes and channels, also known as conduits.

These conduits are the links of the drainage network, joined together at junction nodes, which can represent manholes, pipe connection fittings, etc.

Typical conduit parameters include invert elevations at both ends, the conduit length, the Manning's roughness coefficient n , and cross-sectional geometry. Similarly, junction nodes have parameters such as invert elevation and depth from the ground surface.

There are also other possible types of nodes, e.g. flow dividers, storage units, pumps, and flow regulators [19].

SWMM offers three different options of flow routing:

- (i) steady flow routing,
- (ii) kinematic wave routing, and
- (iii) Dynamic wave routing.

These methods were briefly presented in Chapter 2.1.7. The choice of the routing method affects the accuracy of the results, as well as the time taken by running a simulation [19].

Flows exceeding the capacity of the drainage system may cause ponding. This means temporarily storing the excess volume at a certain junction, and allowing this pond to dry when system capacity is again available.

Ponding can also be disabled by the user, causing all excess volume to overflow the system and be lost [19].

2.9 Previous work on Comparison of EPA SWMM and rational method to decide design flow

According to research done on urban flood modeling by EPA SWMM 5, the EPA SWMM is used to simulate the response of catchment to rainfall events in which runoff, water depth profile, and outflow hydrograph are obtained. Runoff is also obtained from rational formula for comparison purpose. [20]

According to research done on title, Estimation of Storm Runoff Quantity Using Rational Method and SWMM the results obtained through Rational Method and SWMM software can be used to decide the design flow for design of drains by doing comparison [21]

2.10 Detention facilities

Detention ponds are the most common type of storage facility used for controlling storm water runoff peak discharges. The majority of these are dry ponds which release all the runoff temporarily detained during a storm

The following are design guidance and criteria for detention storage:

1. Design rainfall frequency, intensity, and duration must be consistent with highway standards and local requirements
2. Size, shape, and depth of a detention facility must provide sufficient volume to satisfy the projects' storage requirements.
3. Facility's outlet structure must limit the maximum outflow to allowable release rates.
4. System must be designed to release excess storm water expeditiously to ensure that the entire storage volume is available for subsequent storms and to minimize hazards.

Procedure

A general procedure for design of storage facilities is presented below.

1. Compute inflow hydrograph for runoff from the 2-, 5-, 10-, and 100-year design storms using the procedures outlined in the Hydrology Chapter. Both pre-and post-development hydrographs are required.
2. Perform preliminary calculations to evaluate detention storage requirements for the hydrographs from Step 1.
3. Determine the physical dimensions necessary to hold the estimated volume from Step 2, including freeboard. The maximum storage requirement calculated from Step 2 should be used. From the selected shape determine the maximum depth in the pond.
4. Select the type of outlet and size the outlet structure. The estimated peak stage will occur for the estimated volume from Step 2. The outlet structure should be sized to convey the allowable discharge at this stage. [22]

3. METHODOLOGY

3.1 Description of study area

3.1.1 History

Bila town is said to have an age of 130 years or more since it is assumed to have been founded in early 1880s. The town was said to have been found from a market place that had been found somewhere in the surrounding rural areas and managed to have come to the present place by the appeal of the people to the then governors in fear of robbers that had been repeatedly attacking the market from the surrounding forest areas. Established in this way the town has been administered under the rural kebeles until it be organized as urban/town kebele and assigned municipal administration in 1996. It soon grew to have two urban kebeles with areal coverage of 540 hectares and currently its area has grown to 1126 hectares [23]. The town has been serving as the administrative center of the district since then to date.

3.1.2 Location

The study area, Bila town, is currently one of the 23 towns in West Wollega zone or 369 towns in Oromia Regional State (CSA 2010). It is the administrative center of Bojji Dermaji district, one of the 20 districts in West Wollega zone. The town is generally located at about 50 kilometers west of Gimbi town, the administrative center of West Wollegga zone and at 475 kilometers from Addis Ababa, the capital of city of Ethiopia, due west on the main asphalt road to Asosa. Bila town is astronomically located between 9° 25' North latitude and 35° 35' East longitude. The town is surrounded by the rural kebeles of Bojji Dermeji district and shares boundaries with Lalisa Jeto in the north, LalisaJeto and Lalisa Babbo in the east, Burka Bojji in the south, and Burka Bojji and LalisaJeto in the west. The district in which the study area, Bila town, is located generally shares its boundaries with Kamashi zone of Benishangul Gumuz Regional State in the north, Lalo Assabi district in the east, Boji Chekorsa district in the south and Nedjo district in the west. [23]

Hydrologic And Hydraulic Adequacy Assessment Of Drainage Structures In Bila Town

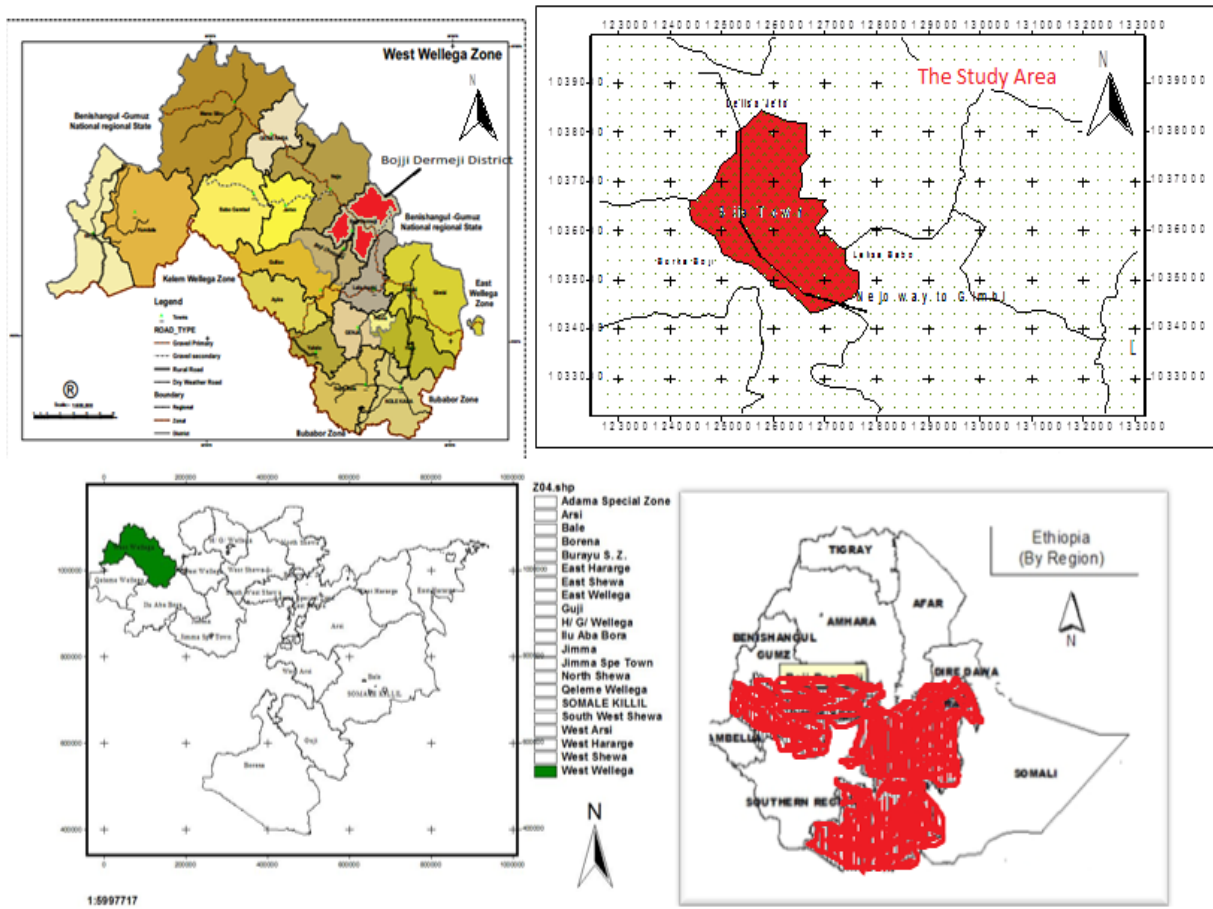


Figure 1 photos of Bila town from Bila Town administration office



Figure 2 photos of Bila town from google earth

3.1.3 Climate

The altitude of the town ranges from 1950masl in the western to 1990masl to the eastern part of it. The highest peak in the town is locally known as Karkarro hill. Climatically, the same as most parts of the district (except for the north eastern part of it which is lowland) the town is characterized by hot dry summer and wet cold winter, and its average temperature is 20°C . Generally a characteristic feature of a subtropical climatic region [23].

3.1.4. Demographic and Socio-economic Profiles

Population: According to 2007 Population and Housing Census of Ethiopia (CSA 2010) Bila town has a total population of 7,291 (3,466/3825) with the households of 1607. This urban population is 17% of the total population of the district. As the town shares its boundaries with three rural kebeles, there exists strong regular interaction between them. The current population is estimated to be 12,387 while the households to be 2,272 (as calculated below using the UN – Habitat 1996).The current figure for the population and is calculated using population projection formula devised by United Nations Manuals on Methods of Estimating Population (UN Population Studies 1974)

$$N_t = P e^{r * t} \dots\dots\dots (3.1)$$

Where,

N_t = number of people at a future time - the would be estimated/ calculated,

P = population at the beginning time - 7,291 (CSA 2010),

e = base of the natural logarithms – 2.71828 (given),

r = rate of increase (natural increase divided by100) – (in this case 5.53% = 0.0553,

(UN –HABITAT 1996)), and t = time period involved – 10 years, 2008 and 2017 (taking the result of 2007 Census year as a base/ bench mark); Thus $e^{r*t} = 1.6989$ (calculated).

Therefore, the current population of the town is estimated to be $N_t = 7,291 * 1.6989 = 12,387$.Applying the same formula to the households ($r = 3.46\%$ (UN – HABITAT 1996) and $e^{r*t} = 1.4134$), the total number of the households equals $1607 * 1.4134 = 2,272$.

Economy: There are group of people engaged in production of food and cash crops at a nearby rural neighbors of the town in which they reside since most of them were immigrants from there. Very few

people are engaged in craft trades such as metal, wood and textile (weaving) products at cottage level that provide services in economic production (agriculture) and household utensils, Others engaged in informal economies and retail trading while others in service rendering activities such as tea house, restaurants and small level hotels.

Accordingly, there are more than 35 different sizes of shops; 13 small scale flour mills, four coffee mills, one crusher, and five private and eight micro and small enterprises (MSEs) owned wood and metal works operating in the town. There are about 40 private and eight MSEs owned cafeteria/ tea rooms and six private owned restaurants and hotels as well as 10 butchers. On the other hand there are nine dadhi/tej (a local drink made of honey) houses and five groceries all private owned [23] There are many micro-level self-employed or family level officially unregistered activities that help many urban residents sustain on for their survival.

3.1.5 Social Services

Apart from government offices of the district clustered into six sites there are one TVET College, one Preparatory, one Secondary, three Primary (and other two at the edge of the boundary), Schools all governmental and three Kindergartens (run by religious institutions) in the town [23] With respect to health institutions there is one Health Center (governmental), one NGO clinic (religious institution), three private clinics and two private pharmacies in the town [23]. Two - one governmental and one private animal - clinics are also in place.

Potable water has been in service from earlier times under the committee and the work was overtaken by the office established under the town administration in 2012, but only for one or two days a week that enables to fetch very few, that may help only for drinking. Hydroelectric power has been nominally in service since 2007 even though it is characterized by extremely interruption in which it comes two or three days a week only for an hour or two.

3.1.6 Area

Currently the town has 1126 hectare of area coverage and its land use composition is as follows [23].

Refer table below Source:

Table 3.1 land use pattern of Bila town

S/No	Land use type	Area (ha)	% of total
1	Residential area	227.66	20.21
2	Business area	21.27	1.89
3	Industrial area	98.20	8.72
4	Administrative	8.01	0.71
5	Social Service	55.03	4.88
6	Recreational area	20.06	1.78
7	Road and transport	96.46	8.56
8	Mixed use	29.40	2.61
9	Greenery	261.25	23.19
10	Reserved area	219	19.44
11	Manufacturing and store	90.22	8.01
12	Forest	-	-
	Total	1126.56	100.00

3.2 Data availability and collection

3.2.1 Data availability

While conducting this thesis different data were required such as rainfall data, land use land cover, and dimension of structures. The rainfall data is obtained from National meteorology Agency Bole station and drainage structures dimension is measured using tape meter and GPS (Geographical positioning system).

3.2.2 Data collection types

Data collection was takes place into ways: primary and secondary data collection

Primary data collection includes:

- ✚ field observation,
- ✚ photographs that show the Existing drainage structure conditions,
- ✚ data collection for existing location drainage structure,

- ✚ elevation of each structures

The secondary data sources were:

- ✚ meteorological data
- ✚ different journals
- ✚ master plan of the town

3.3 Material were Used

For assessment of hydraulic and hydrological adequacy of drainage structure for study area the different material used were:

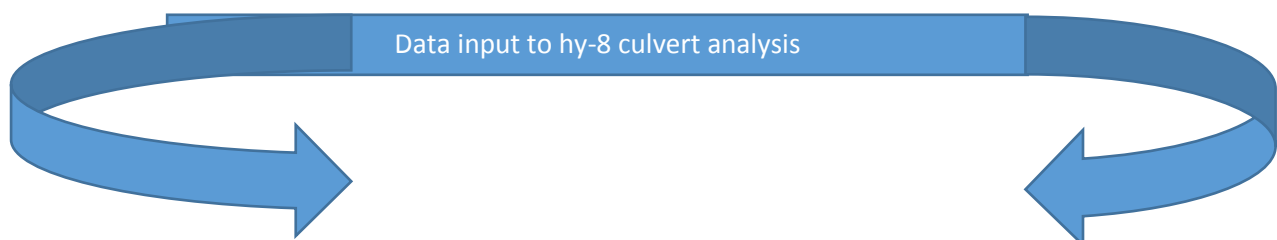
- ✚ SWMM (storm water management model)
- ✚ HY-8 culvert analysis tool,
- ✚ google earth,
- ✚ Geographical positioning system (GPS)device,
- ✚ measuring tape,
- ✚ digital camera,
- ✚ The Microsoft EXCEL was used to analyze data.

3.4 inputted data for the model

HY-8 culvert analysis tool (hydraulic analysis) and EPA SWMM (hydrologic and hydraulic analysis) are used while do analysis and to check the adequacy of the drainage structures

3.4.1 Data input for hy-8 culvert analysis

The data inputted to hy-8 culvert analysis where from field observation and from design manual and journals i.e. tail water data, roadway data, manning coefficient, culvert data and site data .input discharge data was determined using EPA SWMM model after comparison was takes place with rational method used to determine sub catchment runoff. Crossing properties for each culvert was discussed in figure below.





3.4.2 Data input for SWMM

- Precipitation, Soil Characteristics, Channel Characteristics, Elevations and temperature
- Elevation of different area is measured using Geographical position systems (GPS) and from master plan of the town
- Imperviousness, Slope ,Roughness, Width (a shape factor),Depression Storage, and Infiltration Parameter

3.4.2.1 Rainfall Data

Rainfall records were obtained from the National Meteorological Services Agency locate in Bole sub city.

□ Precipitation

- Hourly precipitation data in the form of intensity is generated and compared with that Ethiopian road authority recommend and selected the best.
- Rainfall data sets in one rain gage station Nedjo.

3.4.2.1.1 Filling missing rainfall data

The missed precipitation data results many problems in hydrologic analysis. Because of the cost associated with data collection and some natural and man-made conditions sometimes make it very difficult to have complete records of data at every stations clearly. Missed data sometimes prevent to obtain quantitative and qualitative data of the study area. For gauges that require periodic observation, the failure or absence of the observer to make the necessary visit to the gauge, destruction of recording gauges, and instrument failure because of mechanical or electrical malfunctioning can result in missing

data. Any such causes of instrument failure reduce the length and information content of the precipitation record. After selecting which station best matches with the records of the station in site using less percentage of missing data, the normal annual precipitation at any of the three stations differs by more than 10 percent from that of the interpolation station, the normal-ratio method is used. In this method, the precipitation at each of the three stations is multiplied by the ratio of the normal annual precipitation at the interpolation station to the normal annual precipitation at each station. The weighted precipitation of the three Stations is averaged to obtain the estimate for the interpolation station.

3.4.2.1.2 Consistency of Rainfall Data

Rainfall data reported from a station may not be consistent always. Over the period of observation of rainfall record, there could be (i) unreported shifting of the rain gauge site by as much as 8 km aerially or 30 m in elevation, (ii) significant construction work in the area might have changed the surroundings, (iii) change in observational procedure incorporated from a certain period or (iv) a heavy forest fire, earthquake or landslide might have taken place in that area. Such changes at any station are likely to affect the consistency of data from a station. Use of double mass-curve checks the consistency of the record and helps to correct the rain gauge data for the station. In this method, the accumulated annual rainfall of a particular station is compared with the concurrent accumulated values of mean rainfall of groups of 5 to 8 surrounding base stations.

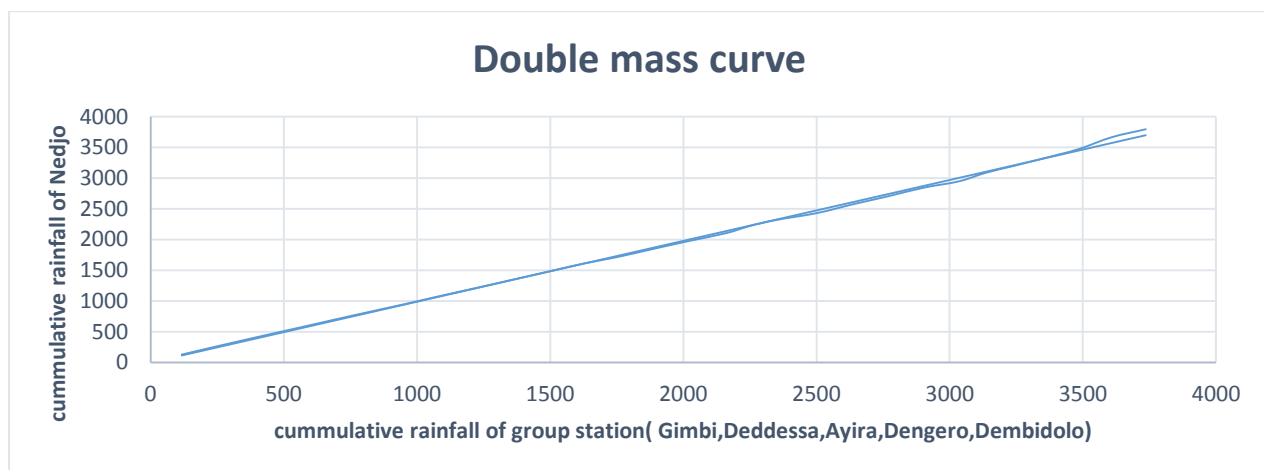
The procedure for double mass-curve analysis is discussed as follows:

- i) The doubtful station, say Nedjo , is marked and the group of stations surrounding it are identified.
- ii) A table is prepared in which the first column represents the year in decreasing order, i.e., it starts with the latest year of station Nedjo
- iii) Yearly precipitation values of station Nedjo are written in second column.
- iv) In the third column the cumulative rainfall of second column are entered.
- v) Mean yearly precipitation of the group of stations surrounding station Nedjo are computed and entered in the fourth column against the year of col. 1.
- vi) In column five, cumulative precipitation of the group of stations of column four are computed.

- vii) A graph is plotted taking the cumulative rainfall of the group of station as abscissa and cumulative rainfall of the station Nedjo as ordinate. Consecutive points are joined by straight line.
- viii) If the consistency of the station Nedjo has undergone changes from any year, then it can be noticed from the slope of the plot. The line joining the initial points of the graph station Nedjo are extended by a dotted line and correction is computed.
- ix) Rainfall of subsequent years from the year of deviation (marked x in the figure) are corrected by multiplying the correction factor. Corrections are to be applied to the data of station Nedjo only when the change of slope of the double mass-curve is observed for more than five years.

Therefore to apply correction factor for station Nedjo, only two year slope of change as observed from double mass curve drawn as shown below. Therefore no need of applying correction factor. Refer table below

Table 2.1 double mass curve



3.4.2.1.3 Fitting of frequency distributions

A common use of rainfall data is in the assessment of probabilities or return periods of given rainfall at a given location. Such data can then be used in assessing flood discharges of given return period through modelling or some empirical system and can thus be applied in schemes of flood alleviation or forecasting and for the design of bridges and culverts.

Frequency analysis usually involves the fitting of a theoretical frequency distribution using a selected fitting method, although empirical graphical methods can also be applied.

The fitting of a particular distribution implies that the rainfall sample of annual maxima were drawn from a population of that distribution. For the purposes of application in design it is assumed that future probabilities of exceedance will be the same as past probabilities. However there is nothing inherent in the series to indicate whether one distribution is more likely to be appropriate than another and a wide variety of distributions and fitting procedures has been recommended for application in different countries and by different agencies.

3.4.2.1.4 Rainfall Analysis

3.4.2.1.4.1 Rainfall depth-duration-frequency relationship

Rainfall records were obtained from the National Meteorological Services Agency located in Bole sub city.

Available rainfall data on these stations has been collected and analyzed in order to prepare the necessary depth or intensity input data for peak discharges computation. The analysis and processing is aimed at determination of appropriate intensity-duration relationship applicable for the thesis. Estimates of maximum rainfall depths for different return periods (T) are obtained by statistical technique of frequency analysis. Extreme value type I, Gumbel and Log Pearson Type III distributions can be used for modeling storm determination of desired return periods in areas where appropriate IDF curves are not available. Thus, the analysis consists of determining maximum rainfall depths associated with T value of interest. In the absence extreme rainfall values for periods less than 24 hours (12, 6, 3 or less than these hours) ,it is difficult to apply regression analysis to drive appropriate IDF curve for a given area, hence rainfall ratio method is used to estimate the rainfall depth to be distributed on a given duration based on a 24 hour rainfall. With this condition, the following the relationship adopted for IDF development at a given station.

$$R_t / R_{24} = (t / 24) [(b + 24)^n / (b + t)^n] \dots\dots\dots(3.2)$$

Where: R_t : R_{24} -Rainfall ratio, R_t : Rainfall in a given durations (hr) R_{24} : Rainfall in 24 hours, n: constant, b: constant, t: time (hr). Based on studies of a large number of rainfall gauges in East Africa, the average values of b and n are found to be 0.3 and 0.9 respectively. These values have been adopted for this thesis project IDF development [24]. Extreme rainfall depth at Nedjo sub city station for different return periods was determined using Log Pearson Type III distributions analysis. Consists of determining maximum rainfall depths associated with T value of interest. Using the daily maximum

rainfall from metrological agency, 24 hour design rainfall was calculated using Log Pearson type III distribution methods. The values are compared with Ethiopian Roads Authority recommended values and the maximum was taken, as it is recommended by ERA. Maximum rainfall for the station were located on appendix part.

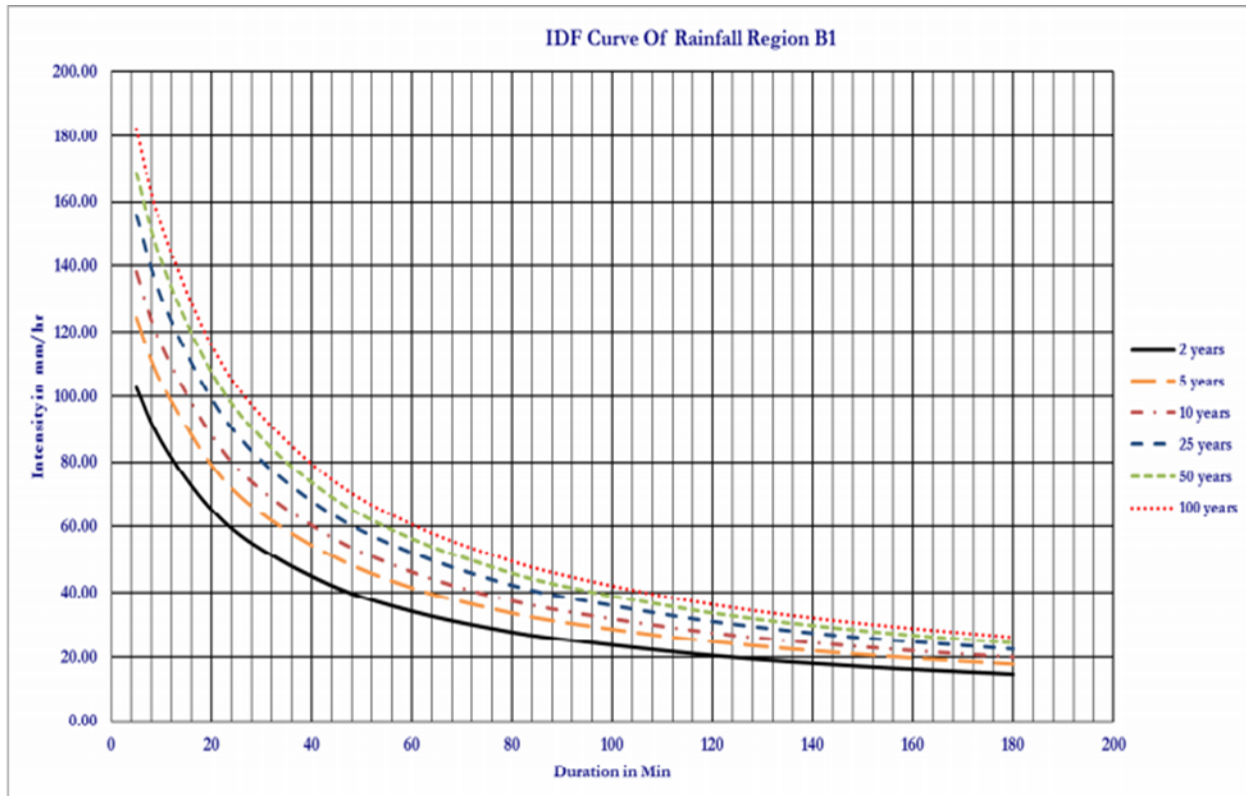


Figure 3 IDF curve of rainfall region B1

Design Storm Frequency

The modeling storm frequency (return periods) provided by Low Volume Road Modeling Manual for different structures corresponding to the road modeling standard which is reproduced as Table 3 below. Adopted accordingly on drainage structure modeling return period selection on this road project. To impart a sustainable transportation system, to promote and enhance the overall socio economic development along the project road corridor, the SNNP National Regional State Rural Roads Authority planned to construct The Road Project with a modeling Standard of DC1. Hence the modeling storm frequency stated under DC1 used to compute the modeling discharge and to modeling the respective drainage crossing structures. But for those Structures which have not stated or considered in DC1

standard like short, medium, and long span Bridges we use the next Geometric Modeling Standard which is DC2.

Table 3.2 design storm frequency(yrs.) by geometric design criteria

Structure type	Geometric design standard			
	DC4	DC3	DC2	DC1
Gutters and inlets	5	5	5	2
Side ditches	15	10	10	5
Ford	15	10	10	5
Drift	15	10	10	5
Culvert diameter <2m	25	20	20	10
Large culvert diameter >2m	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short span bridge (<10m)	50	50	25	-
Masonry arch bridge	50	50	25	
Medium span bridge (10 – 50 m)	100	100	50	-
Long span bridge >50m	100	100	100	-

3.4.2.2 Sub catchment parameterization

Sub catchment parameterization were used in SWMM are imperviousness, depression storage, width, slope

3.4.2.2.1 Imperviousness

For this study, no flow measurements were available, and therefore calibration was not an option. Other ways to define the values of imperviousness are to estimate them based on land use data

Imperviousness area percentage calculation

The amount and characteristics of runoff not only depends on the rainfall pattern, but also on the catchment properties. The catchments could be described as hydrological units where storm runoff

and infiltration are generated in the basis of a single set of model parameters and input data. They represent the level of spatial discretization of the hydrological model.

The imperviousness was calculated for each sub-catchment according to the percentage of different surfaces, see Equation below.

$$\varphi = \frac{(A1 * \varphi1 + A2 * \varphi2 + \dots + An * \varphin)}{(A1 + A2 + \dots + An)} \text{----- (3.3)}$$

Where φ =imperviousness of the whole sub-catchment,

φ_i =imperviousness of each type of surface,

A_i =area of each surface

Refer table 3.3 below for impervious coefficient for different types of surface [25]

Table 3.3 impervious coefficient for different types of surface

Type of surface	Imperviousness coefficient
Roofs	90%
Concrete and asphalt surfaces	80%
Paved surfaces with gravel joints	70%
Gravel road, sharply slope mountainous park area without significant vegetation	40%
Outcrop with not significant slope	30%
Gravel paths with undeveloped parts of soil	20%
Parks with lush vegetation and rugged mountainous woodland	10%
Cultivated land , lawn or grassland	0-10%
Flat woodland	0-10%

3.4.2.2.2 Depression storage

Pervious and impervious parts of a sub catchment threats separately by SWMM, and thus both may be given independent values of depth of depression storage. One can also define a portion of the impervious area to have no depression storage at all. This could be realistic on steep roofs, for example. The SWMM User’s Manual [19] suggests some literature values for the depression storage. For impervious areas, the values range from 1.3 to 2.5 mm and for lawns from 2.5 to 5.1 mm. The highest value is given for forest litter (7.6 mm). These values are not very exact, and depression storage actually is one of the common calibration parameters used for SWMM parameterization

Based on the values above, the depression storage for all sub catchments were set to 2.3 mm for impervious sub catchment fraction and to 5 mm for the pervious fraction.

3.4.2.2.3 Slope

The slope parameter tells the amount of inclination. In reality, the sub catchment shape and slope vary within the sub catchment. This is the case especially with large heterogeneous sub catchments. In SWMM sub catchments are conceptually represented as rectangular planes. These planes are inclined so that all surface flow is directed perpendicularly towards one of the edges of the rectangle

3.4.2.2.4 Manning’s roughness coefficient n for overland flow

For impervious areas the roughness coefficient n was set to the value of 0.011, which is the literature value for smooth asphalt [19]. For non-asphalt surfaces, this was considered a good compromise between smoother materials (e.g. rooftops), and slightly rougher materials (e.g. concrete)

3.4.2.2.5 Flow width

Flow width is one of the least tangible SWMM parameters. It is defined as the ‘characteristic width of the overland flow path for sheet flow runoff’ [19]

According to [19] the width parameter can be calculated by dividing the sub catchment area by the length of the longest overland flow path in the area. In case several flow paths exist, their maximum lengths should be averaged.

3.5 Discharge calculated using rational methods

3.5.1 Time of concentration

Time of concentration T_c is the sum of overland flow time of concentration (T_{c1}) and channel flow or watercourse time of concentration .they are determined based on empirical formula they have.

$$T_{c1 (hr)} = 0.604(RL_1/S^{0.5})^{0.467} \dots\dots\dots(3.4)$$

R = roughness coefficient of land use =0.02 from Appendix

L_1 = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

$$T_{c2(hr)} = (0.87 * L^2 / 1000 * Sav)^{0.385} \dots\dots\dots(3.5)$$

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service.

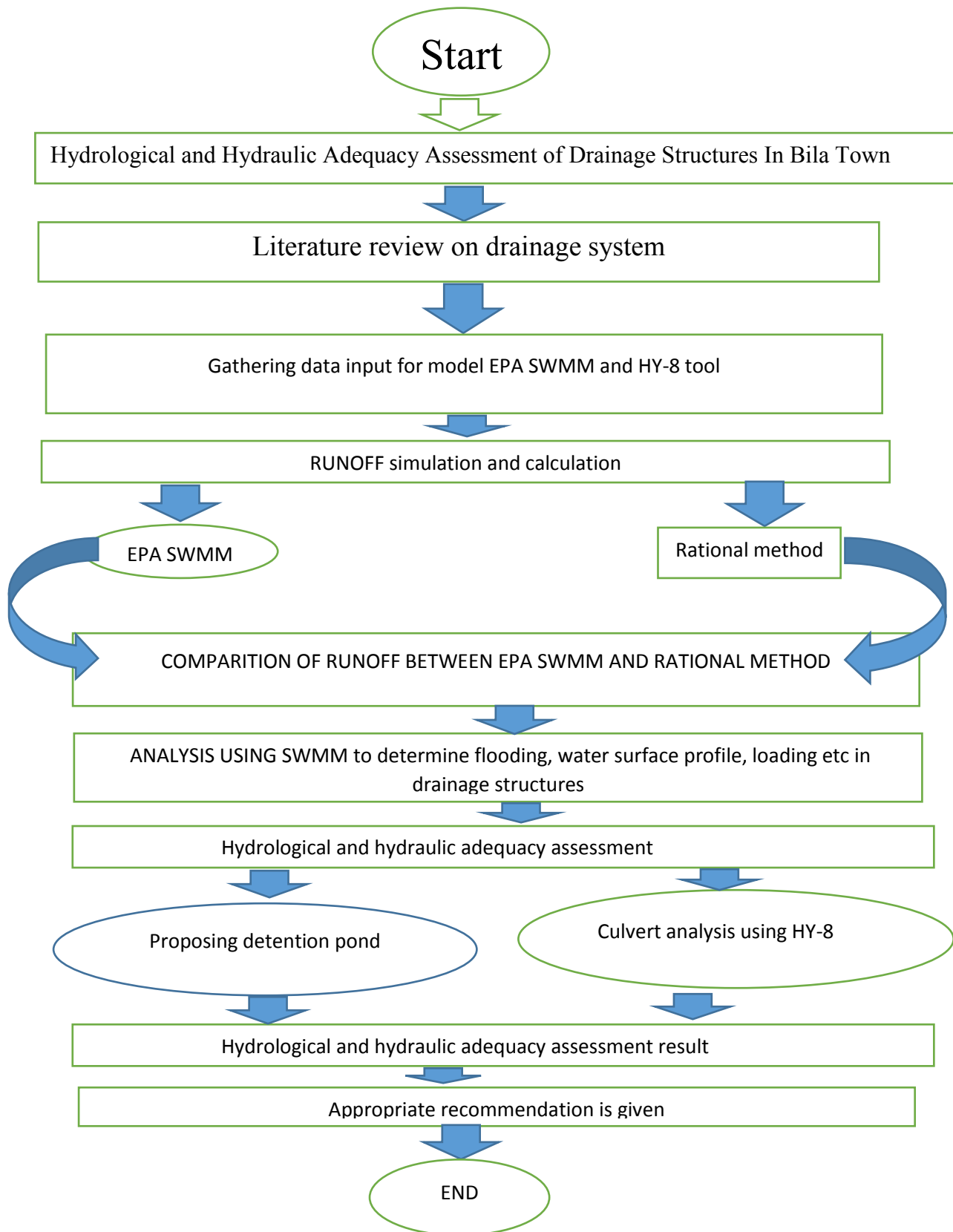
L = length of watercourse (km)

S_{av} = average slope (m/m)

Use a minimum t_c 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 [1].both time concentration of watercourse; overland flow computed and runoff coefficient shown on appendix respectively.

3.6 Progress in Hydraulic and Hydrological analysis of Drainage Structures

To do analysis for checking the adequacy of drainage structures for Bila town different process were undertaking .see chart below for more, the chart shows from starting to the end what it look like therefore any one can understand the work activity.



4. RESULT AND DISCUSSION

4.1 Rainfall input data preparation

The input rainfall data was selected after daily maximum rainfall from metrological agency, 24 hour design rainfall was calculated using Log Pearson type III distribution methods. Design rainfall calculated with log Pearson type III was compared with ERA recommended and the maximum was selected which ERA generated. The rainfall of ERA is attached on the appendix part.

Table 4.1 log Pearson type III distribution analysis

T(return periods)	KZ for cs=0.2	kz*sy	YT=Yavg+Kz*sy	XT=antilog YT	XT(ERA)
2	-0.033	-0.0033	1.7567	57.1	58.84
5	0.83	0.083	1.843	69.66	71.26
10	1.301	0.1301	1.8901	77.62	79.29
25	1.818	0.1818	1.9418	87.4	89.55
50	2.159	0.2159	1.9759	94.6	96.84
100	2.472	0.2472	2.0072	101.67	104.37
200	2.763	0.2763	2.0363	108.71	112.02

$Cs=0.2$, for $Kz=f(Cs,T)$,

Kz = Log Pearson Type III distribution frequency factor (taken from readily available table)

Y = logarithm of Rainfall depth (X)

n = Total number of X (individual data)=28

Y_{avg} = 1.76

Sy =0.1

Where: YT = Log XT –logarithm of Rainfall depth (XT) at return period T years [mm]

Y_{avg} = Mean value of logarithmic rainfall data (daily) [mm]

Sy = Standard deviation

4.2 Runoff calculation using rational formula.

The need of runoff calculation for each sub catchment was, I could not gained the runoff result calculated with rational method. This output runoff by rational method was used to make comparison with the model EPA SWMM 5

Table 4.2 Depth Duration frequency curve development for Bila town

Depth Duration frequency curve development for Bila town							
		Depth for given return periods(mm)					
duration(hr)	Duration(min)	Rainfall ratio	5 years	10 years	25 years	50 years	100 years
0.08	5	0.145	10.33	11.5	12.98	14.04	15.13
0.17	10	0.243	17.32	19.26	21.76	23.53	25.36
0.33	20	0.37	26.36	29.33	33.13	35.83	38.62
0.5	30	0.449	31.99	35.6	40.21	43.48	46.86
1	60	0.581	41.4	46.07	52.03	56.26	60.64
1.5	90	0.65	46.32	51.54	58.21	62.95	67.84
2	120	0.695	49.53	55.11	62.24	67.3	72.54

Rainfall intensity used was a minimum time of concentration value of 15 minutes for areas that are not developed and intercepting catchments [1].

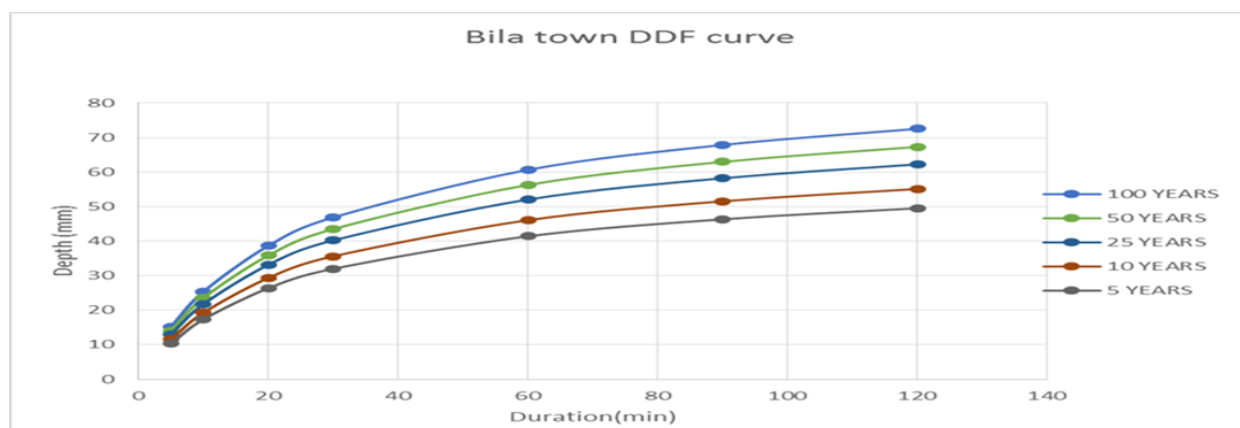


Figure 4 depth duration frequency curve development for Bila town

Table 4.3 intensity duration for different return periods (mm/hr)

Intensity duration for different return periods(mm/hr)				
5 years	10years	25years	50years	100yers
129.13	143.75	162.25	175.5	189.13
101.88	113.29	128	138.41	149.17
79.87	88.87	100.39	108.57	117.03
63.98	71.2	80.42	86.96	93.72
41.4	46.07	52.03	56.26	60.64
30.88	34.36	38.81	41.96	45.23
24.76	27.55	31.12	33.65	36.27

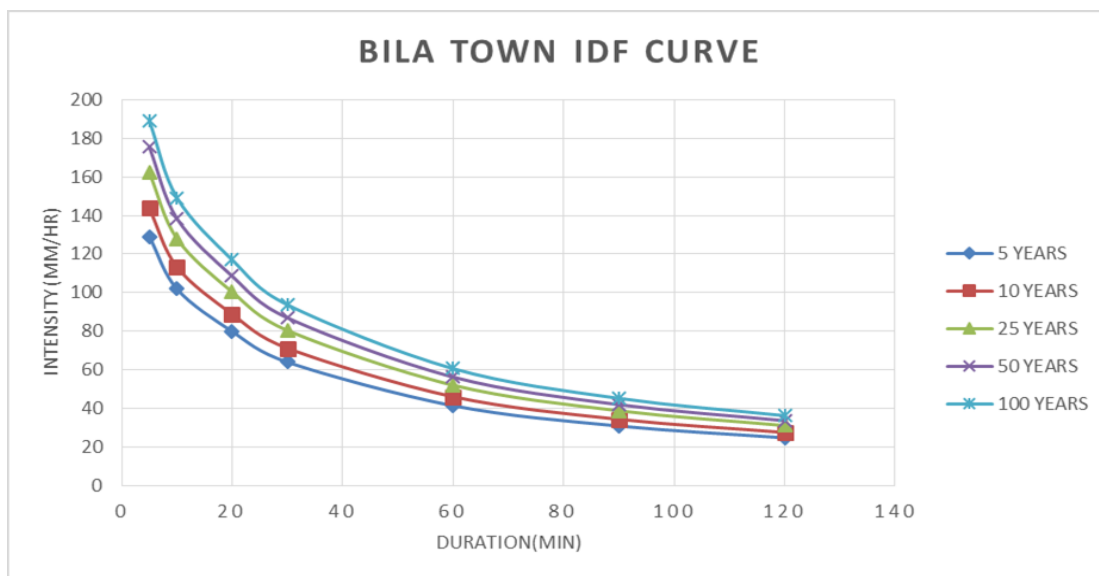


Figure 5 intensity duration curve for Bila town

Time of concentration calculation and weighted runoff coefficient was shown on appendix part. Since calculated time of concentration was less than 15 minutes, as ERA recommended 15 minutes time of concentration was used to calculate runoff amount for each sub catchment. Therefore for 15 minute time of concentration rainfall intensity was read from the graph.

Table 4.4 peak runoff calculation using rational formula

		10YRS RETURN PERIOD		
sub catchment	Area(ha)	C	I	$Q=0.00278CIA(m^3/s)$
1	3.89	0.55	101.08	0.601204635
2	5.23	0.63	101.08	0.925874808
3	10.03	0.62	101.08	1.747441525
4	1.7	0.52	101.08	0.248406122
5	4.91	0.51	101.08	0.70365811

4.3 Comparison between simulated and calculated runoff

Before checking adequacy of structures the simulated and calculated runoff for each sub catchment was compared to do assessment. See table 9

Table 4.5 peak runoff comparison using rational method and EPA SWMM

	10 yrs. return period		
sub catchment	Rational method	EPA SWMM	differences
1	0.6	0.64	-0.04
2	0.92	0.91	0.01
3	1.74	1.73	0.01
4	0.25	0.29	-0.04
5	0.7	0.73	-0.03
	25 yrs. return period		
1	0.679210163	0.73	-0.05078984

2	1.046005873	1.06	-0.01399413
3	1.974169815	1.97	0.004169815
4	0.280636496	0.33	-0.0493635
5	0.794956845	0.84	-0.04504316

4.5 EPA SWMM5 Model Result

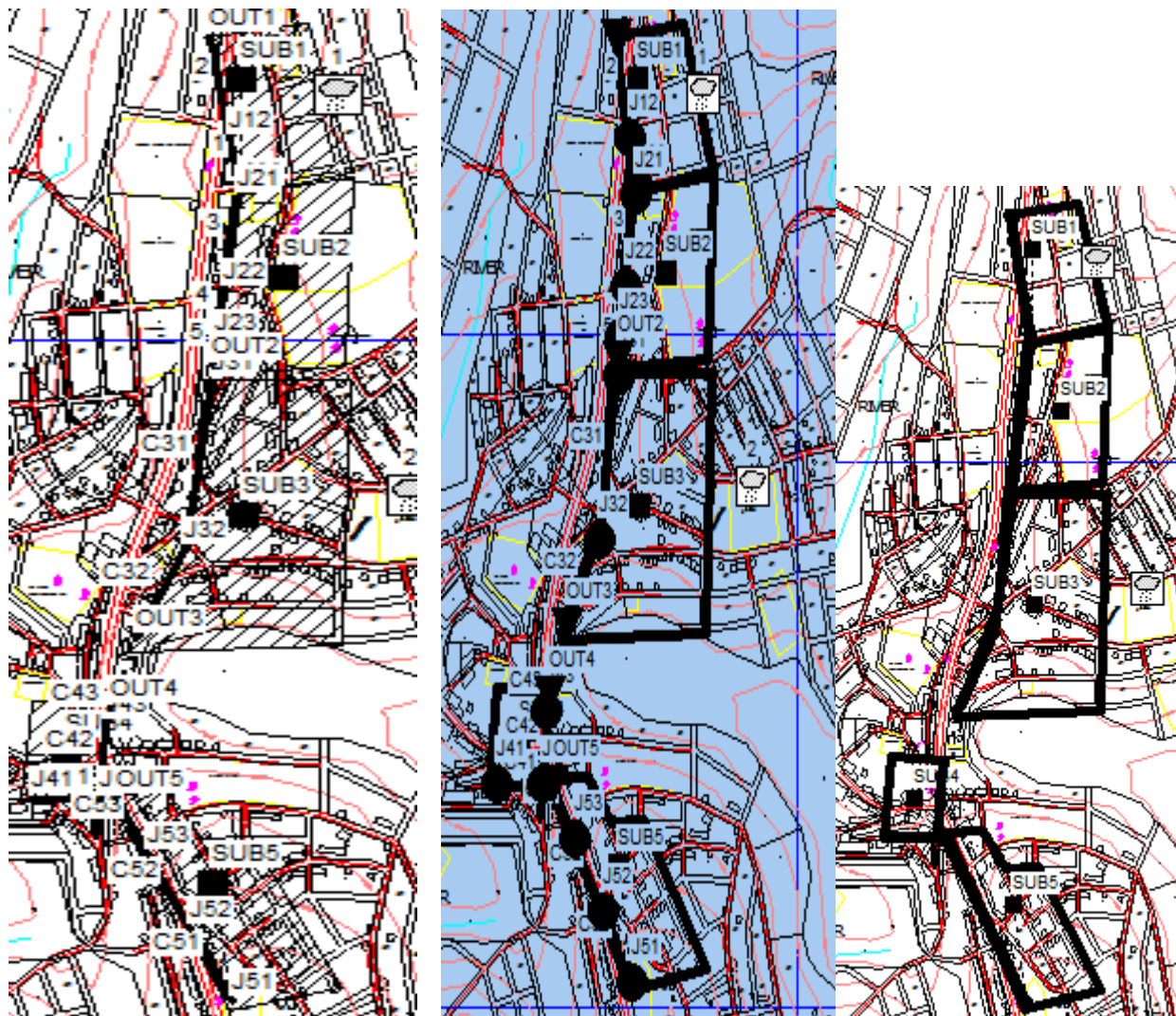


Figure 6 schematic used in SWMM for Bila town

Table 4.6 node flooding for 10 years return period.

Topic: Node Flooding Click a column header to sort the column.						
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10 ⁶ ltr	Maximum Poned Depth Meters
J11	0.80	0.180	0	00:45	0.344	0.000
J12	1.14	0.094	0	00:38	0.326	0.000
J21	2.09	0.790	0	00:30	3.076	0.000
J31	2.08	1.389	0	00:45	5.898	0.000
J32	1.93	0.029	0	00:30	0.022	0.000
J41	0.54	0.067	0	00:30	0.048	0.000
J42	1.70	0.169	0	00:35	0.475	0.000
J43	1.82	0.040	0	00:47	0.184	0.000

Junction found in sub catchment 1, 2, 3 and 4 were affected by flooding this shows the dimension fixed for the links and junctions were not adequate

For 25 years return period, intensity is 114.2 mm/hr. when using this intensity for design hour of flood is increased with maximum rate from each node. This shows existing drainage ditch is inadequate by using return period 10 years and 25 years. Refer table below.

Table 4.7 node flooding for 25 years return period

Topic: Node Flooding Click a column header to sort the column.						
Node	Hours Flooded	Maximum Rate CMS	Day of Maximum Flooding	Hour of Maximum Flooding	Total Flood Volume 10 ⁶ ltr	Maximum Poned Depth Meters
J11	0.91	0.270	0	00:45	0.562	0.000
J12	1.35	0.094	0	00:36	0.376	0.000
J21	2.11	0.941	0	00:30	3.519	0.000
J31	2.11	1.629	0	00:45	6.844	0.000
J32	1.96	0.030	0	00:29	0.023	0.000
J41	0.64	0.096	0	00:30	0.124	0.000
J42	1.76	0.169	0	00:34	0.547	0.000
J43	1.86	0.044	0	00:46	0.188	0.000

Addition to this water surface profile shows too flooding effect. Water surface profile from node J31-out3; the simulation status report shows that sections between these junctions are surcharged (flooded). Water surface profile for others shown on appendix part.

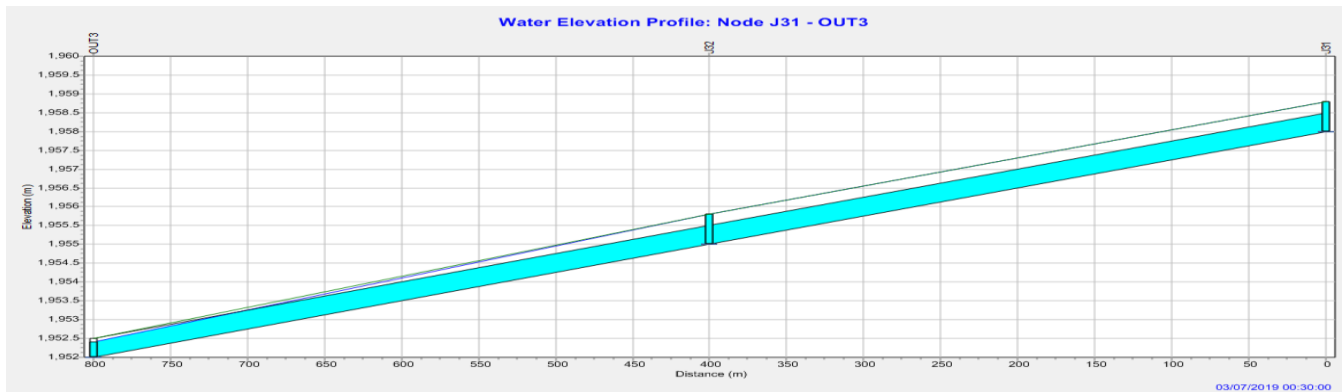


Figure 7 water elevation profile from Junction 31 to outfall 3 for 25 years return period

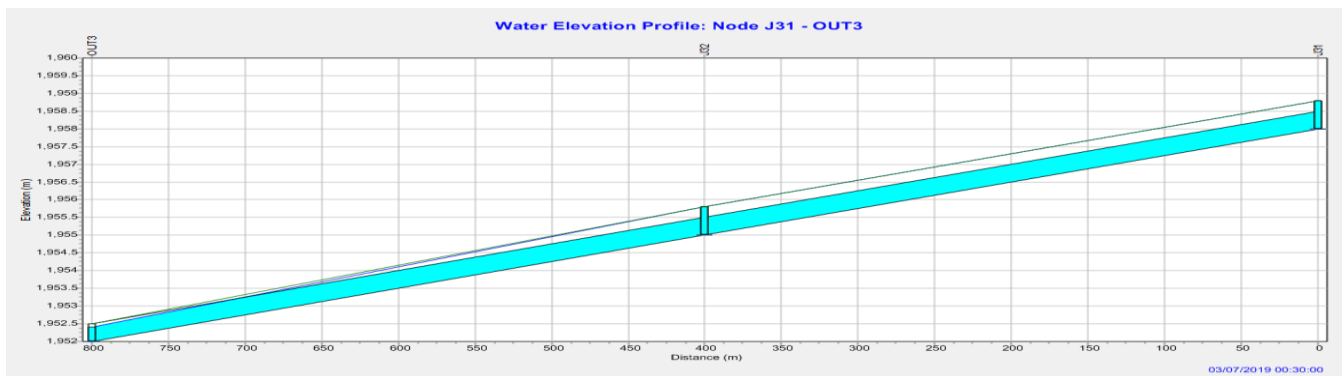


Figure 8 water elevation from Junction 31 to outfall for 10 years return period

4.6 Structures Affected By Sedimentation

Some of drainage structures at junction affected by sedimentation. For example at junction 12 there where flooding effect found at maximum flooding time 00:30 minute due to inadequate structure. Addition to this there was sedimentation accumulation that increase flooding because no manual clearance. Refer figure below.



Figure 9 photographs of Junction 12 affected by sedimentation

4.7 Effect of erosion at downstream of outfall 3 and 4

Downstream of outfall 3 and 4 were affected by erosion of soil due to flooding refer figure 4.5 below. Which cause washing of soil and destroy land where coffee planted on it.



Figure 10 photos of downstream outfall 3 and 4

4.8 Proposing detention pond

4.8.1 Design Discharge

Peak discharges from the 5-and 25-year design storm events are as follows

- Pre-developed 5-year peak discharge = 1.31 CMS
- Pre-developed 25-year peak discharge = 1.74 CMS

Hydrologic And Hydraulic Adequacy Assessment Of Drainage Structures In Bila Town

□ Post-development 5-year peak discharge = 1.54 CMS

□ Post-development 25-year peak discharge = 1.97 CMS

Since the post-development peak discharge must not exceed the pre-development peak discharge, the allowable design discharges are 1.31 and 1.74 for the 5- and 25-year storms, respectively. See table below that EPA SWMM out put

Table 4.8 runoff hydrographs

pre development runoff				Post development runoff			
H:M	5years	10years	25years	H:M	5years	10years	25years
0:15	0	0	0	0:15	0	0	0
0:30	0.9	1.06	1.28	0:30	1.41	1.64	1.94
0:45	1.31	1.5	1.74	0:45	1.54	1.73	1.97
1:00	1.3	1.55	1.7	1:00	1.35	1.6	1.72
1:15	1.12	1.28	1.42	1:15	1.09	1.22	1.36
1:30	1.01	1.1	1.17	1:30	0.97	1.04	1.11
1:45	0.86	0.94	1.04	1:45	0.81	0.89	0.99
2:00	0.71	0.83	0.9	2:00	0.77	0.79	0.85
2:15	0.66	0.74	0.82	2:15	0.66	0.71	0.79
2:30	0.33	0.37	0.4	2:30	0.25	0.26	0.28
2:45	0.2	0.21	0.23	2:45	0.12	0.13	0.13
3:00	0.13	0.14	0.14	3:00	0.07	0.07	0.08
3:15	0.09	0.09	0.1	3:15	0.05	0.05	0.05
3:30	0.06	0.07	0.07	3:30	0.03	0.03	0.03
3:45	0.05	0.05	0.05	3:45	0.02	0.02	0.02
4:00	0.04	0.04	0.04	4:00	0.02	0.02	0.02
4:15	0.03	0.03	0.03	4:15	0.01	0.01	0.01
4:30	0.02	0.02	0.02	4:30	0.01	0.01	0.01
4:45	0.02	0.02	0.02	4:45	0.01	0.01	0.01
5:00	0.02	0.02	0.02	5:00	0.01	0.01	0.01

5:15	0.01	0.01	0.01	5:15	0.01	0.01	0.01
5:30	0.01	0.01	0.01	5:30	0	0	0
5:45	0.01	0.01	0.01	5:45	0	0	0
6:00	0.01	0.01	0.01	6:00	0	0	0

4.8.2 Preliminary Volume Calculations

For runoff from the 5-and 25-year storms, the required storage volumes V_s , are computed using equation

$$V_s = 0.5 t_i (Q_i - Q_o) \dots\dots\dots (4.1)$$

$$5 \text{ years storm: } V_s = 0.5 * 6 * 3600 (1.54 - 1.31) = 10800 * 0.23 = 2484 \text{ m}^3$$

$$25 \text{ year storm : } V_s = 0.5 * 6 * 3600 (1.97 - 1.74) = 2504 \text{ m}^3$$

4.8.3 Rectangular Basins

Underground storage tanks are often rectangular. The volume of a rectangular basin can be computed by dividing the volume into triangular and rectangular shapes and using Equation 4.2. The variables in Equation 4-2 are illustrated in Figure 4.9.

Box:

Triangle:

$$V = L W D$$

$$V = 0.5 W (D^2 / S) \dots\dots\dots (4.2)$$

Where: V = Volume at a specific depth, m^3

D = Depth of ponding for that shape, m

W = Width of basin at base, m

L = Length of basin at base, m

S = Slope of basin, m/m . S value to pass flow easily to outlet is 5 percent

If the basin is not on a slope, then the geometry will consist only of rectangular shaped boxes

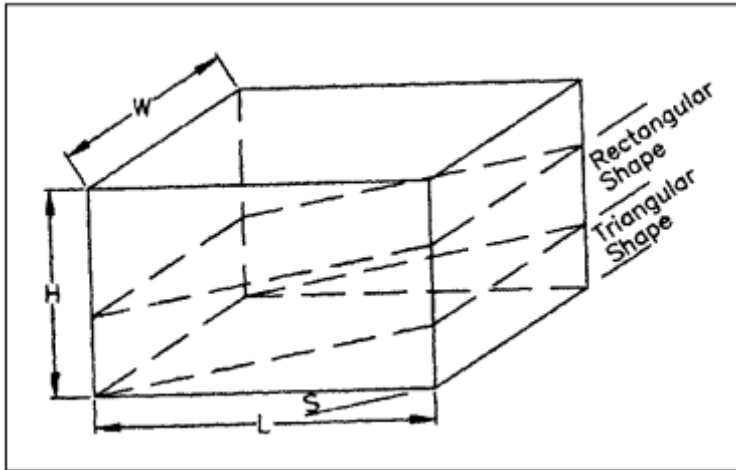


Figure 4.9 photos of rectangular basin

Estimating the trial dimensions of a basin for a given basin storage volume can be accomplished by equation below.

$$L = \{ -Z D (r + 1) + [(Z D)^2 (r + 1)^2 - 5.33 (Z D)^2 r + ((4r V) / D)]^{0.5} \} / 2r \dots \dots \dots (4.3)$$

Where

$V_s=2504 \text{ m}^3$, Depth available for storage 25 years is 1.6m, Available free board =0.6m

Basin side slope (Z)=3, Width to Length Ratio of Basin (r) = $\frac{1}{2}$

Dimension of the basin at outlet

$$L = \{-Z D (r + 1) + [(Z D)^2 (r + 1)^2 - 5.33 (Z D)^2 r + ((4r V) / D)]^{0.5} \} / 2r$$

$$= \{-3*1.6(0.5+1)+[(3*1.6)^2(0.5+1)^2-5.33(3*1.6)^2*0.5+((4*0.5*2504)/1.6)]^{0.5}\} / 2*0.5$$

$$=48.66\text{m use } 50\text{m}$$

$$W=0.5L=0.5*50=25\text{m}$$

$$\text{Volume for box (} V) =L*W*D=50*25*1.6=2000 \text{ m}^3$$

$$\text{Volume for triangle (} V)=0.5*W*(D^2/S)$$

$$=0.5*25*(1.6^2/0.05)= 640\text{m}^3$$

Table 4.9 depth-stage-storage relationship

depth	stage	storage
0.1	1930.1	125.0244
0.2	1930.2	250.1049
0.3	1930.3	375.2529
0.4	1930.4	500.4794
0.5	1930.5	625.7956
0.6	1930.6	751.2128
0.7	1930.7	876.7422
0.8	1930.8	1002.395
0.9	1930.9	1128.182
1	1931	1254.115
1.1	1931.1	1380.205
1.2	1931.2	1506.463
1.3	1931.3	1632.9
1.4	1931.4	1759.528
1.5	1931.5	1886.357
1.6	1931.6	2504

4.8.4 Weir

Stage-discharge and stage-storage characteristics of a storage facility that should provide adequate peak flow attenuation for runoff from both the 5-and 25-year design storms are presented below. The storage-discharge relationship was developed by requiring the preliminary storage volume estimates of runoff for both the 5-and 25-year design storms to be provided when the corresponding allowable peak discharges occurred. Storage values were computed by solving the broad-crested weir equation for head, H, assuming a constant discharge coefficient of 1.8, weir length of 4m and no tail water submergence. The capacity of storage relief structures was assumed to be negligible.

Table 4.10 stage-discharge

$H^{1.5}$	H	C	$Q=CLH^{1.5}$
0.031623	0.1	1.45	0.1834121
0.058095	0.15	1.47	0.34159713
0.089443	0.2	1.48	0.5295009
0.164317	0.3	1.46	0.95960992
0.252982	0.4	1.47	1.48753541
0.353553	0.5	1.45	2.05060967
0.464758	0.6	1.45	2.69559641
0.585662	0.7	1.45	3.39683971
0.715542	0.8	1.45	4.15014217
0.853815	0.9	1.45	4.95212682
1	1	1.45	5.8
1.15369	1.1	1.45	6.69140045
1.314534	1.2	1.45	7.624298
1.482228	1.3	1.45	8.59692271
1.656502	1.4	1.45	9.60771357
1.837117	1.5	1.45	10.6552804
2.023858	1.6	1.45	11.7383747

- Height corresponding to coefficient of discharge was read from table see appendix part

4.9 Hy-8 Culvert Analysis Result

Loading of outfall was basis for culvert analysis because at outfall 1, 2 and 4 culvert was found .therefore the data gained using SWMM model; average flow and maximum flow was input for HY-8 analysis. The other was gathered from field observation and available table.

Table 4.11 outfall loading

Summary Results				
Topic: Outfall Loading		Click a column header to sort the column.		
Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10 ^{^6} ltr
OUT1	96.40	0.186	0.368	2.804
OUT2	95.13	0.067	0.117	1.095
OUT4	93.86	0.049	0.086	0.808

Source: SWMM output result

4.9.1 Existing culvert analysis

There were three culverts at outfall of different sub catchment. Culvert 1 was overtopped when design discharge was 1.69 m³/s which was greater than **0.16 m³/s** which was adequate. See table below, for others shown in appendix part. Culvert 2 surcharged when design discharge become 1.69 but the used design discharge for design was **0.07 m³/s**.so it was adequate to path the discharge safely. Also culvert 3 was adequate to path design discharge **0.05 m³/** refer the table below for more information for culvert 1A for others shown in appendix part.

Table 4.12 flow summary

Headwater Elevation (m)	Total Discharge (cms)	Culvert 1A Discharge (cms)	Roadway Discharge (cms)	Iterations
1963.50	0.00	0.00	0.00	1
1963.64	0.04	0.04	0.00	1
1963.70	0.07	0.07	0.00	1
1963.75	0.11	0.11	0.00	1
1963.79	0.15	0.15	0.00	1
1963.83	0.19	0.19	0.00	1
1963.84	0.20	0.20	0.00	1
1963.89	0.26	0.26	0.00	1
1963.92	0.30	0.30	0.00	1
1963.95	0.33	0.33	0.00	1
1963.98	0.37	0.37	0.00	1
1965.00	1.69	1.69	0.00	Overtopping

Table 4.13 culvert summary table

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	1963.50	0.00	0.0	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
0.04	0.04	1963.64	0.14	0.0*	1-S2n	0.05	0.11	0.05	0.08	1.53	0.08
0.07	0.07	1963.70	0.20	0.0*	1-S2n	0.09	0.15	0.09	0.11	2.61	0.10
0.11	0.11	1963.75	0.25	0.0*	1-S2n	0.10	0.19	0.11	0.13	2.36	0.11
0.15	0.15	1963.79	0.29	0.0*	1-S2n	0.12	0.22	0.13	0.14	2.56	0.11
0.19	0.19	1963.83	0.33	0.0*	1-S2n	0.13	0.25	0.15	0.15	2.73	0.12
0.20	0.20	1963.84	0.34	0.0*	1-S2n	0.14	0.25	0.15	0.16	2.80	0.12
0.26	0.26	1963.89	0.39	0.0*	1-S2n	0.16	0.29	0.17	0.17	3.03	0.13
0.30	0.30	1963.92	0.42	0.0*	1-S2n	0.17	0.31	0.19	0.18	3.02	0.13
0.33	0.33	1963.95	0.45	0.0*	1-S2n	0.18	0.33	0.20	0.19	3.09	0.14
0.37	0.37	1963.98	0.48	0.0*	1-S2n	0.19	0.35	0.21	0.20	3.18	0.14

Table 4.14 water surface profile

Total Discharge (cms)	Culvert Discharge (cms)	Headwater Elevation (m)	Inlet Control Depth(m)	Outlet Control Depth(m)	Flow Type	Length Full (m)	Length Free (m)
0.00	0.00	1963.50	0.00	0.0	0-NF	0.00	0.00
0.04	0.04	1963.64	0.14	0.0*	1-S2n	0.00	15.00
0.07	0.07	1963.70	0.20	0.0*	1-S2n	0.00	15.00
0.11	0.11	1963.75	0.25	0.0*	1-S2n	0.00	15.00
0.15	0.15	1963.79	0.29	0.0*	1-S2n	0.00	15.00
0.19	0.19	1963.83	0.33	0.0*	1-S2n	0.00	15.00
0.20	0.20	1963.84	0.34	0.0*	1-S2n	0.00	15.00
0.26	0.26	1963.89	0.39	0.0*	1-S2n	0.00	15.00
0.30	0.30	1963.92	0.42	0.0*	1-S2n	0.00	15.00
0.33	0.33	1963.95	0.45	0.0*	1-S2n	0.00	15.00
0.37	0.37	1963.98	0.48	0.0*	1-S2n	0.00	15.00

5. Conclusion and recommendation

5.1 Conclusion

This study analyzed the adequacy assessment of drainage structures of Bila town using EPA SWMM and hy-8 culvert analysis. During the assessment primary data collection and secondary data collection had taken. Primary data collection are elevation, photographs and dimension of existing structures. Secondary data were rainfall data, master plan of the area and background of Bila town Such that climatic information, socio economic, Demography and land use land cover. Rainfall filling done using normal ratio methods and consistency of rainfall checked using double mass curve analysis. The fitting distribution was using log Pearson type III and compared with Ethiopian road authority (ERA) recommend. The ERA Recommendation was used for model SWMM. The SWMM model was successfully used to model the quantity of runoff in an urbanized area of Bila town by comparing runoff quantity calculated with rational method.

The drainage structures analysis using SWMM result shows some parts of junction and links (conduit) were affected by flooding which were due to inadequate hydrological (runoff amount) and hydraulic (size of drainage structures).

To reduce impact of flooding storage facilities detention pond is proposed which comprises rectangular basin and weir at outlet.

Finally drainage structures found in Bila town is not enough to carry surface runoff, additional drainage structures should construct, for area sedimentation effect found manual maintenance should practice, hydrological and hydraulic analysis should takes properly before construction of drainage structures, integrated storm water should practice to manage drain and protect environment from impact caused due to flooding such as erosion of bank, gullies formation.

5.2 Recommendation

At study area Bila town road drainage structures are inadequate: insufficient capacity of drainage structures, unavailability of drainage structures at proper place, lack of good management found. This factor cause gullies, flooding and erosion of banks etc. in order to minimize the problem the following recommendation were drawn.

- Use of the research study results for further study of other sub catchment of the Bila town in order to have a standardized and harmonized urban drainage systems.
- Integrated storm water management should practice to minimize impact comes from flooding
- Appropriate dimension for drainage structure should implemented in order to path design discharge.
- Sedimentation affect should analyzed for Bila town for further drainage structures.
- Slope alignment and velocity effect on scour depth formation should evaluated
- the municipality should be solve the debris/silt problem by conduct these maintenance activities: stop debris upstream by using a barrier; clean the ditch frequently; making sure debris can pass through the ditch; steepen the ditch grade to promote self-cleaning and should arrange the mechanisms by which the drainage channel at the station will be cleaned properly and meant for the right use. And also proactive measures should be taken to reduce and manage flooding hazards (like clearing of drains before rain season begins.
- In general the skilled and knowledgeable workers should use appropriate hydrological analysis, hydraulic design for road drainage structures.
- For drainage filled and alignment problem, periodic cleaning and adjustment of the slope were recommended.
- Finally for community creates awareness concerned the effects of disposing solid materials in to drainage facility by the municipality and other concerned body

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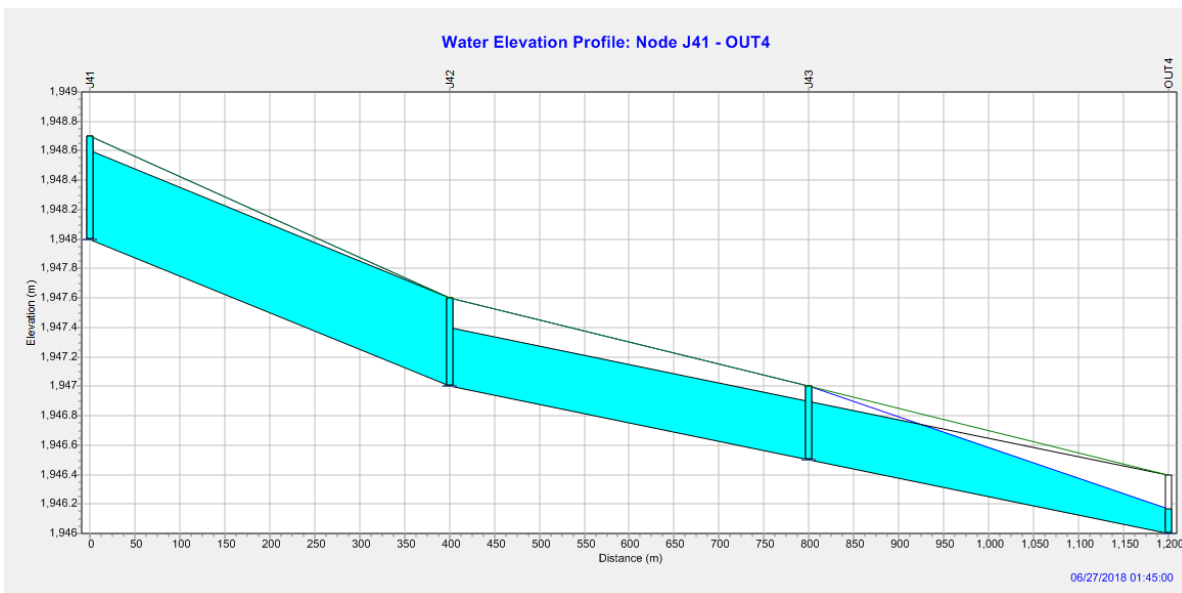
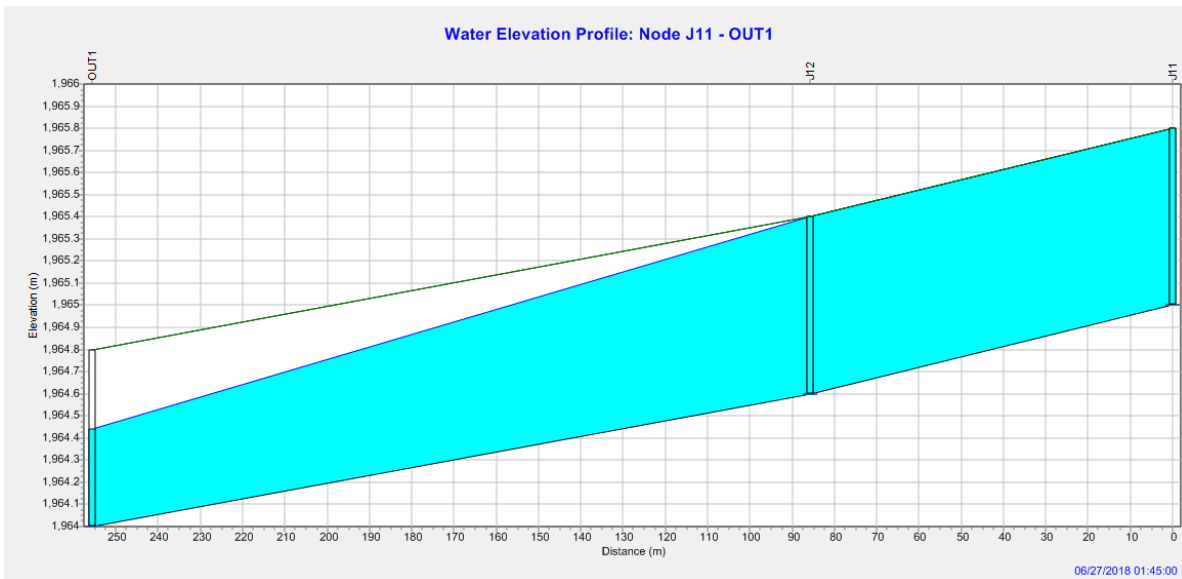
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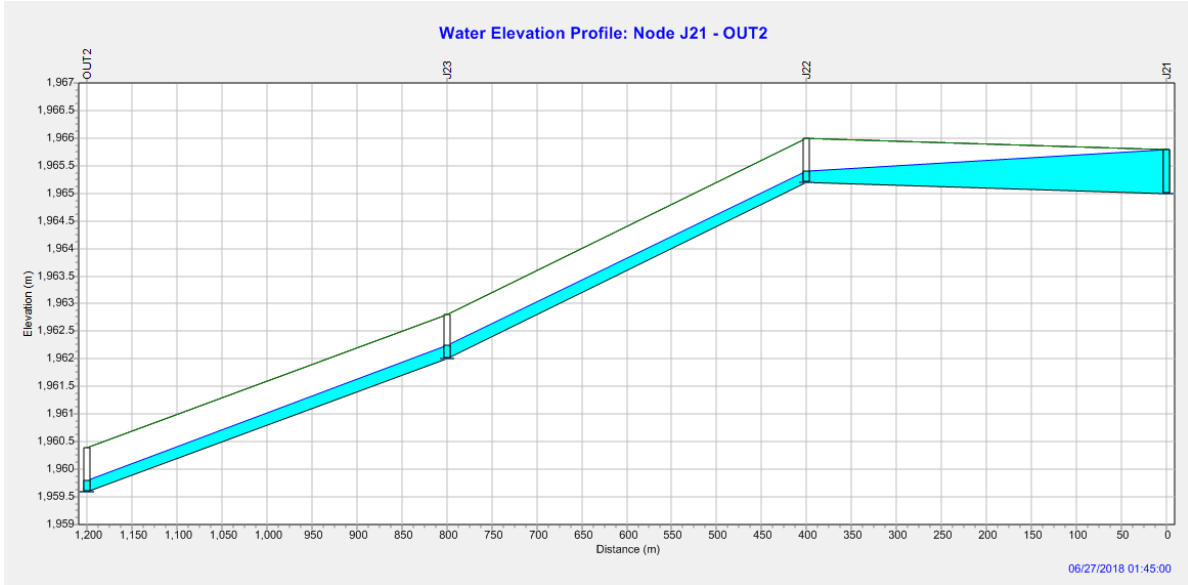
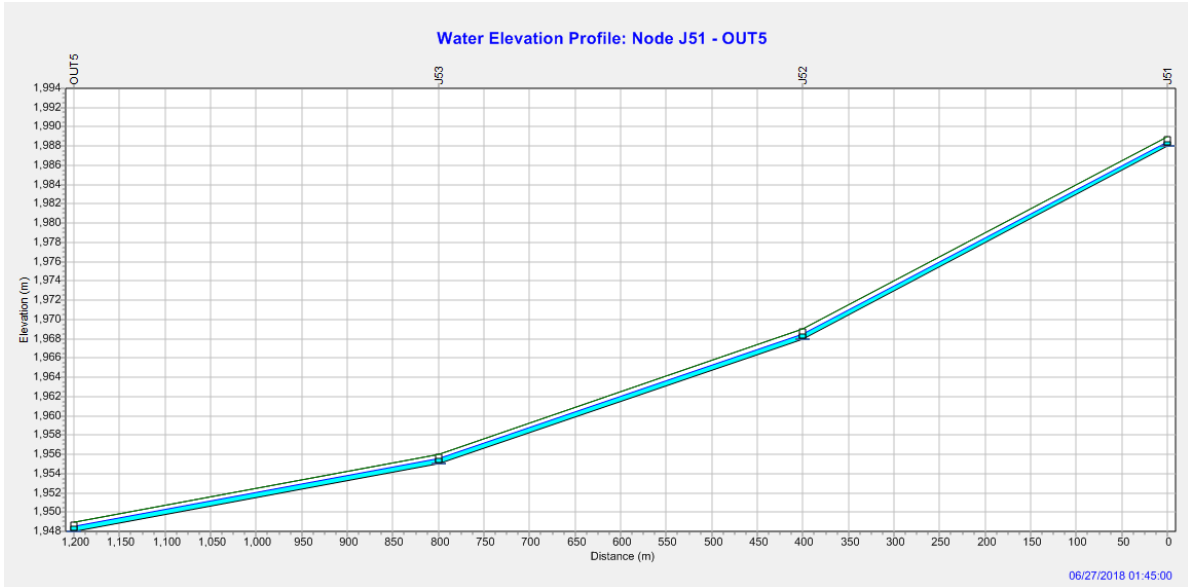
Hydrologic And Hydraulic Adequacy Assessment Of Drainage Structures In Bila Town

Appendices

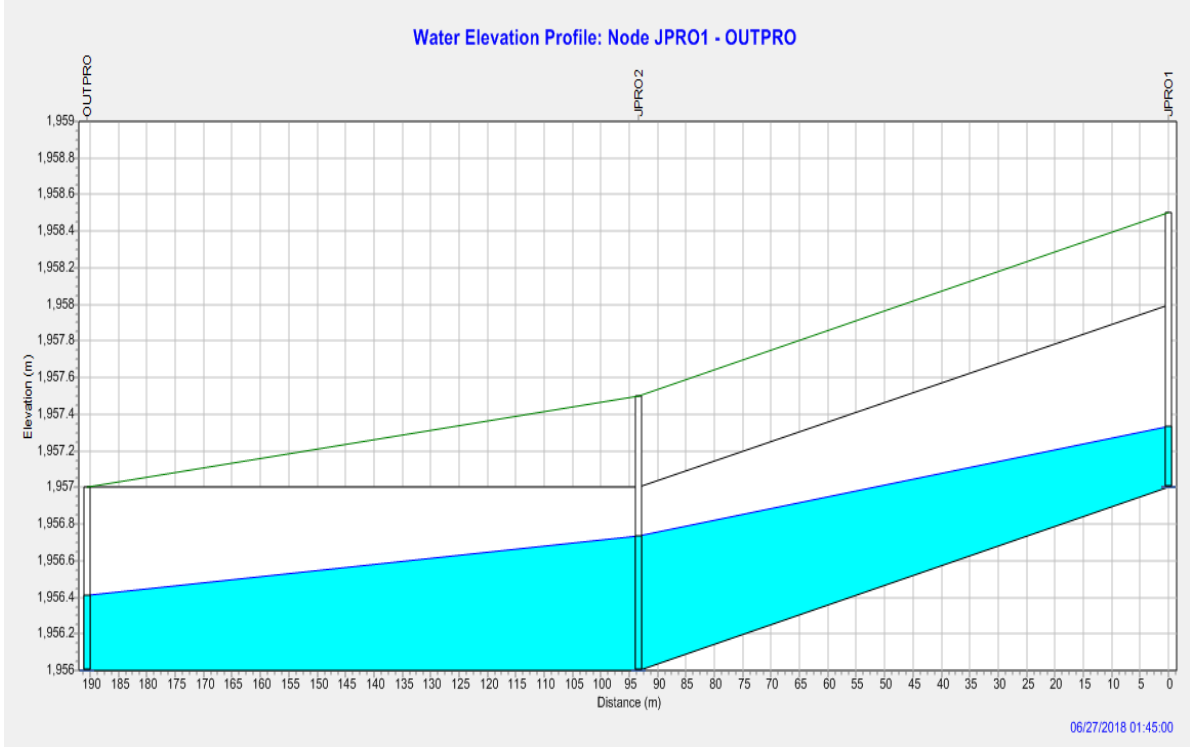
Appendix 1: water flow for different node



Hydrologic And Hydraulic Adequacy Assessment Of Drainage Structures In Bila Town



Hydrologic And Hydraulic Adequacy Assessment Of Drainage Structures In Bila Town



Appendix 2: Rainfall for all classified region

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

Note: RR- Rainfall Region

Source: Ethiopian road authority, 2013.drainage design manual

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Appendix 3:24 hour maximum rainfall

years	24 hr. maximum rainfall of Nedjo
1990	72.5
1991	64.5
1992	48.9
1993	51.6
1994	64.6
1995	75.3
1996	78.2
1997	57.6
1998	92
1999	47.9
2000	41.4
2001	36.4
2002	67.5
2003	76.5
2004	54.4

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2005	67.5
2006	53.5
2007	55.2
2008	84.6
2009	46.5
2010	44.7
2011	47
2012	65
2013	49.5
2014	61
2015	68.1
2016	50
2017	75.5

Hydrologic And Hydraulic Adequacy Assessment Of Drainage Structures In Bila Town

Appendix 4: runoff coefficient values under different surface conditions

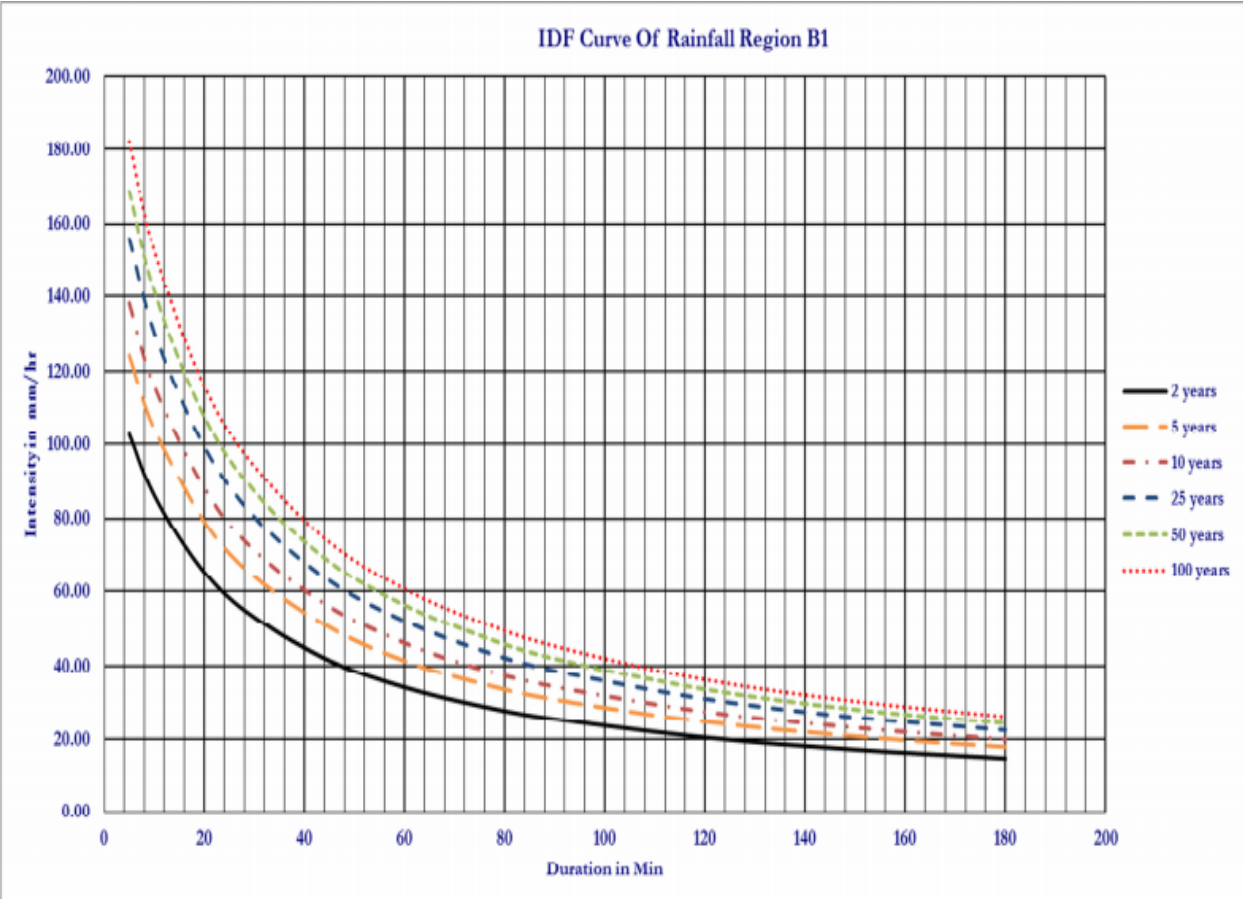
Type of Drainage Area	Runoff Coefficient, C*
Business:	
Downtown areas	0.70 - 0.95
Neighborhood areas	0.50 - 0.70
Residential:	
Single-family areas	0.30 - 0.50
Multi-units, detached	0.40 - 0.60
Multi-units, attached	0.60 - 0.75
Suburban	0.25 - 0.40
Apartment dwelling areas	0.50 - 0.70
Industrial:	
Light areas	0.50 - 0.80
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
Lawns:	
Sandy soil, flat, 2%	0.05 - 0.10
Sandy soil, average, 2 - 7%	0.10 - 0.15
Sandy soil, steep, 7%	0.15 - 0.20
Heavy soil, flat, 2%	0.13 - 0.17
Heavy soil, average, 2 - 7%	0.18 - 0.22
Heavy soil, steep, 7%	0.25 - 0.35
Streets:	
Asphaltic	0.70 - 0.95
Concrete	0.80 - 0.95
Brick	0.70 - 0.85

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Appendix 5: Simulated Sub catchment runoff

Topic: Subcatchment Runoff Click a column header to sort the column.								
Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10 ^{^6} ltr	Peak Runoff CMS	Runoff Coeff
SUB1	50.54	0.00	0.00	6.43	40.81	1.59	0.74	0.807
SUB4	50.54	0.00	0.00	8.90	39.52	0.67	0.34	0.782
SUB5	50.54	0.00	0.00	6.92	37.23	1.83	0.74	0.737
SUB2	50.54	0.00	0.00	6.92	39.06	2.04	0.90	0.773
SUB3	50.54	0.00	0.00	5.93	39.60	3.97	1.71	0.784
newproposed	50.54	0.00	0.00	7.21	41.13	1.70	0.87	0.814

Appendix 7: IDF Curve of Rainfall Region B1



Appendix 8: Coefficient of skewness for different return period

K_T values for Pearson Type III distribution (positive skew)

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

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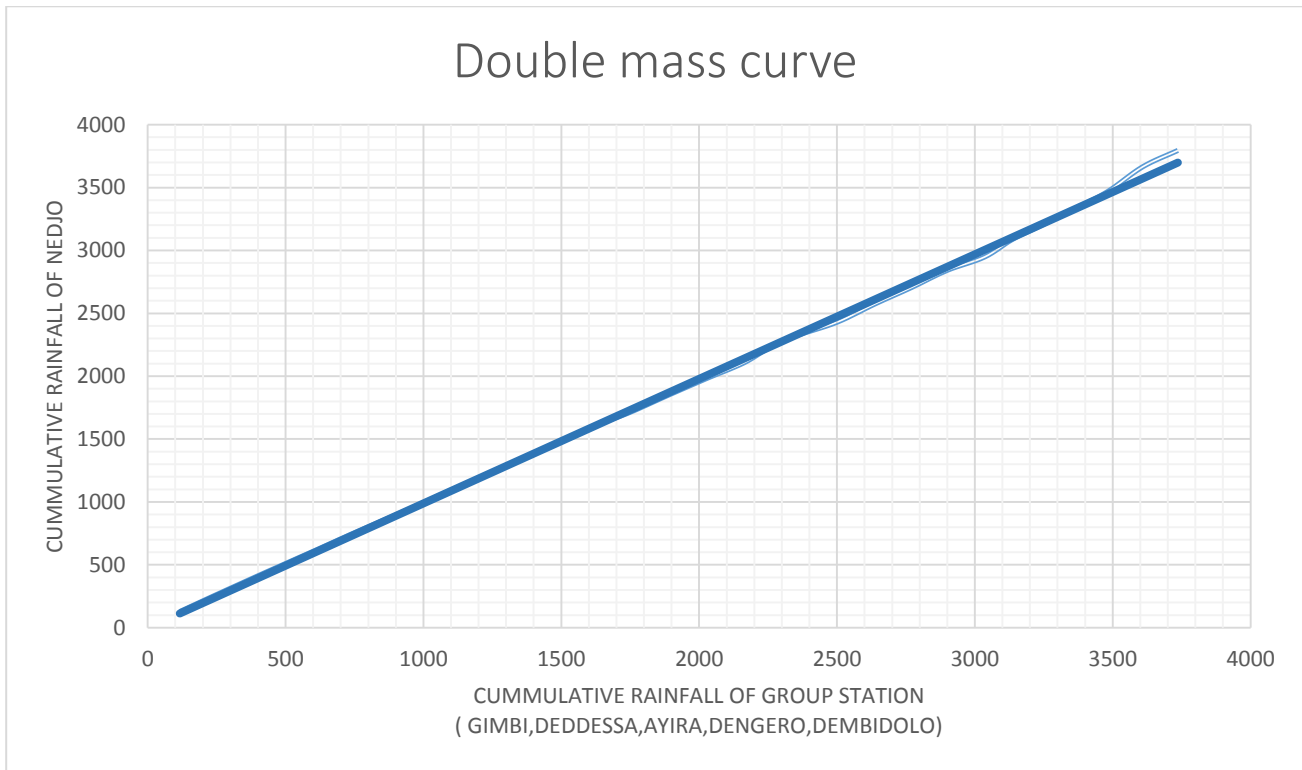
Appendix 9: Weighted runoff coefficient for different sub catchment.

S.NO	Catchment	Surface condition	Area proportion (%)	C	C*A	Weighted C
		Description				
1	SUB 1	Un improved area	2.95	0.3	0.012	
		concrete	4	0.9	0.036	
		Asphalt street	5.2	0.83	0.043	0.55
		Single family area	85.7	0.5	0.44	
		Multiunit attached	2	0.75	0.015	
2	SUB 2	Un improved area	9.3	0.3	0.027	
		Asphalt street	7.36	0.83	0.1	0.63
		Single family area	82.26	0.5	0.411	
3	SUB 3	Un improved area	3	0.3	0.09	
		Asphalt street	8.72	0.83	0.072	0.62
		Single family area	90.9	0.5	0.454	
4	SUB 4	Un improved area	17.86	0.3	0.054	
		Asphalt street	13.1	0.9	0.1179	0.52

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		Single family area	69.02	0.5	0.345	
5	SUB 5	Un improved area	5.58	0.3	0.02	
		Asphalt street	4.49	0.9	0.4	0.51
		Single family area	89.9	0.4	0.35	

Appendix 10: double mass curve



Appendix 11: time of concentration calculation

Sub catchment	Area (ha)	L₁(KM)	L₂(KM)	T_{c1}(min)	T_{c2}(min)	T_c=T_{c1}+T_{c2}(min)
SUB 1	3.89	0.153	0.265	3.93	8.5	12.43
SUB 2	5.23	0.1767	0.2929	4.25	5.54	9.79
SUB 3	10.03	0.24	0.466	6.812	7.8	14.62
SUB 4	1.7	0.1075	0.1553	3.77	4.89	8.66
SUB 5	4.91	0.130	0.389	7.79	5.08	12.87

Appendix 12: Mean annual rainfall of Ethiopia.

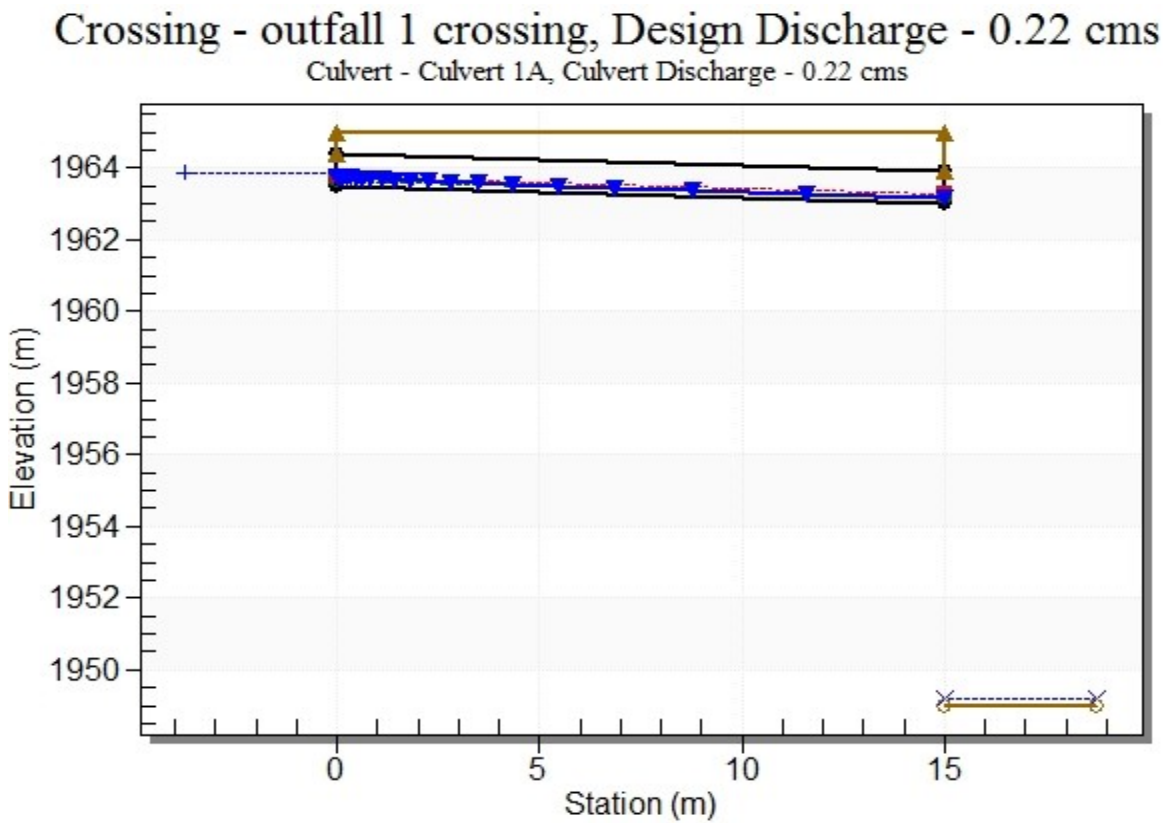


Appendix 13: conduit surcharge

Summary Results					
Topic: Conduit Surcharge Click a column header to sort the column.					
Conduit	Hours Both Ends Full	Hours Upstream Full	Hours Dnstream Full	Hours Above Normal Flow	Hours Capacity Limited
1	0.80	0.80	1.14	0.01	0.01
2	0.01	1.14	0.01	0.01	0.01
3	0.01	0.01	2.09	0.01	0.01
C31	1.95	2.10	2.00	0.10	0.01
C32	0.01	2.00	0.01	0.01	0.01
C41	0.52	0.61	1.71	0.54	0.52
C42	1.79	2.03	1.90	1.82	1.69
C43	0.01	1.90	0.01	0.01	0.01

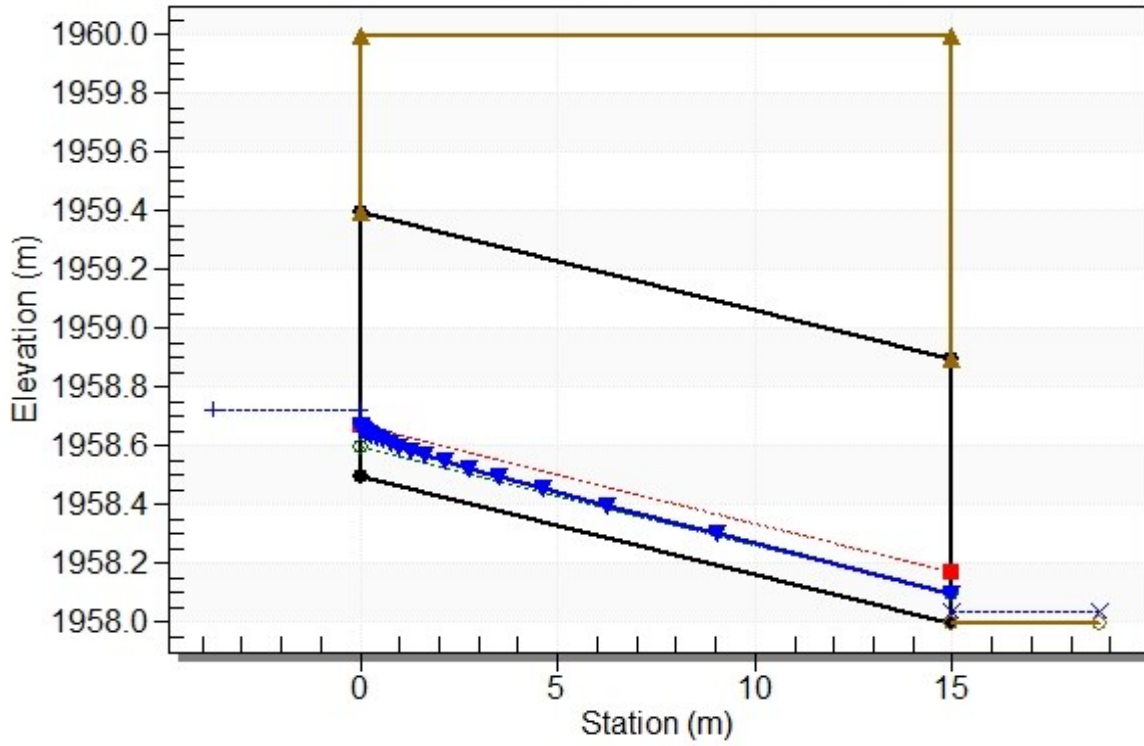
Appendix 14: existing culverts water flow profile

Water Surface Profile Plot for Culvert: Culvert 1A



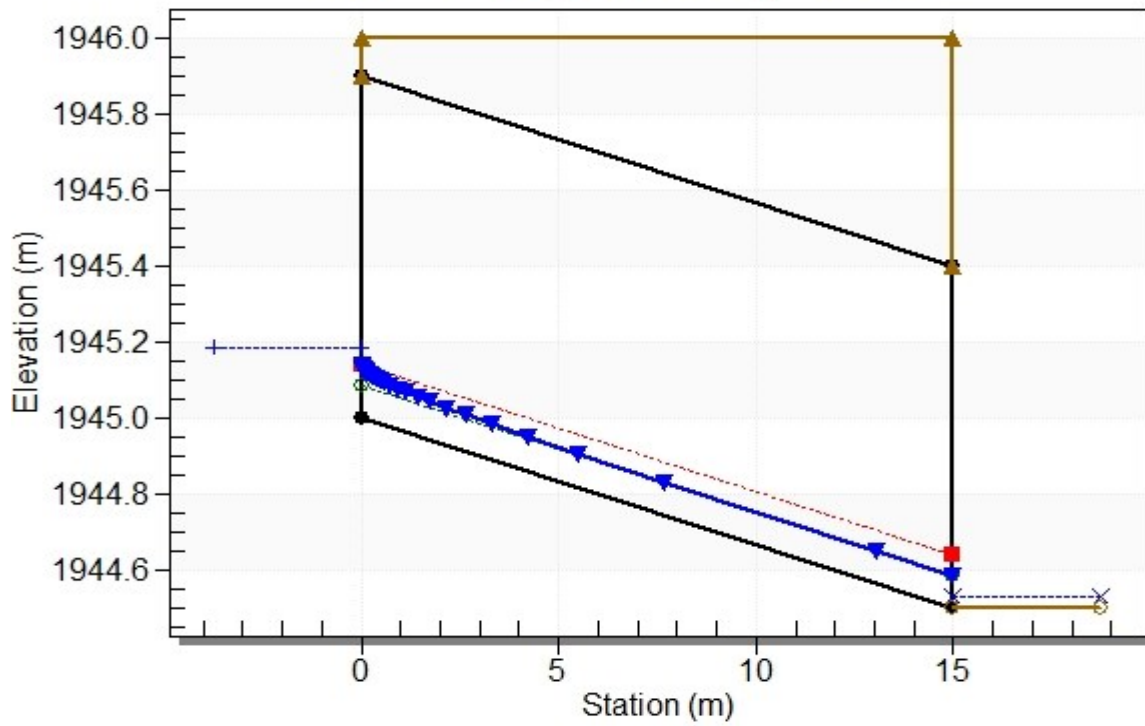
Water Surface Profile Plot for Culvert: Culvert 2A

Crossing - Crossing 2, Design Discharge - 0.09 cms
Culvert - Culvert 2A, Culvert Discharge - 0.09 cms



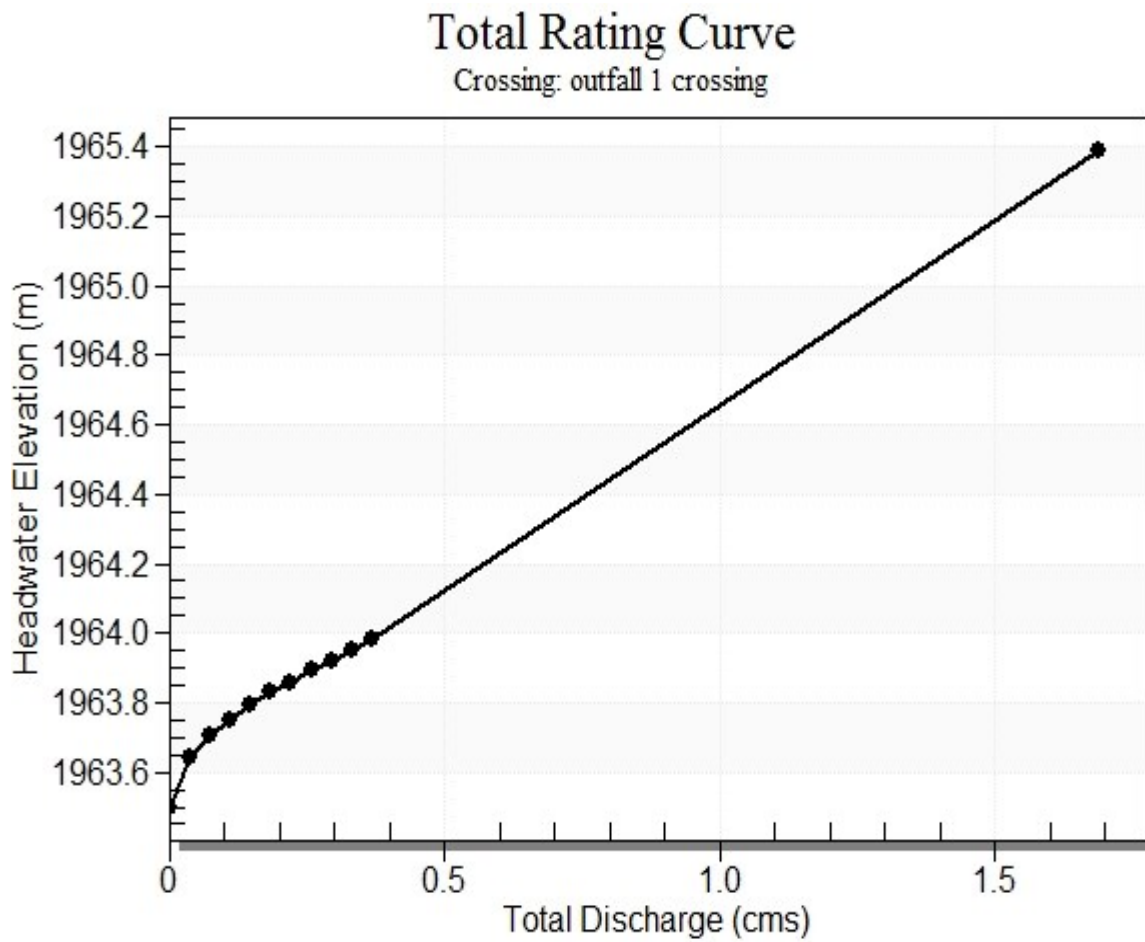
Water Surface Profile Plot for Culvert: Culvert 3A

Crossing - Crossing 3, Design Discharge - 0.06 cms
Culvert - Culvert 3A, Culvert Discharge - 0.06 cms

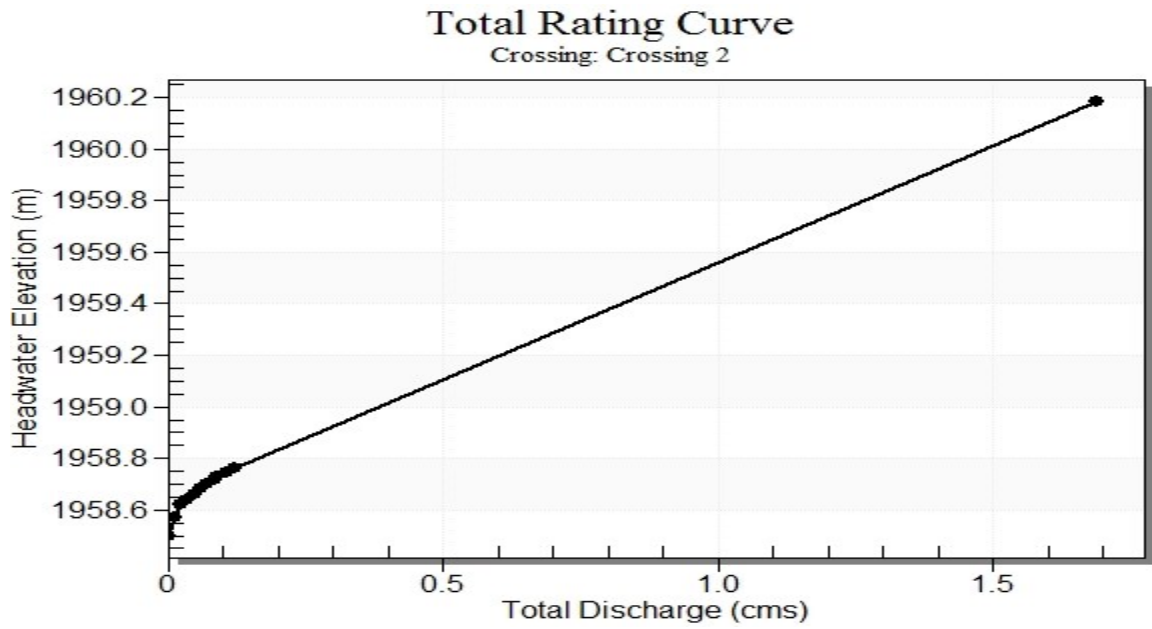


Appendix 15: Culvert total rating curve

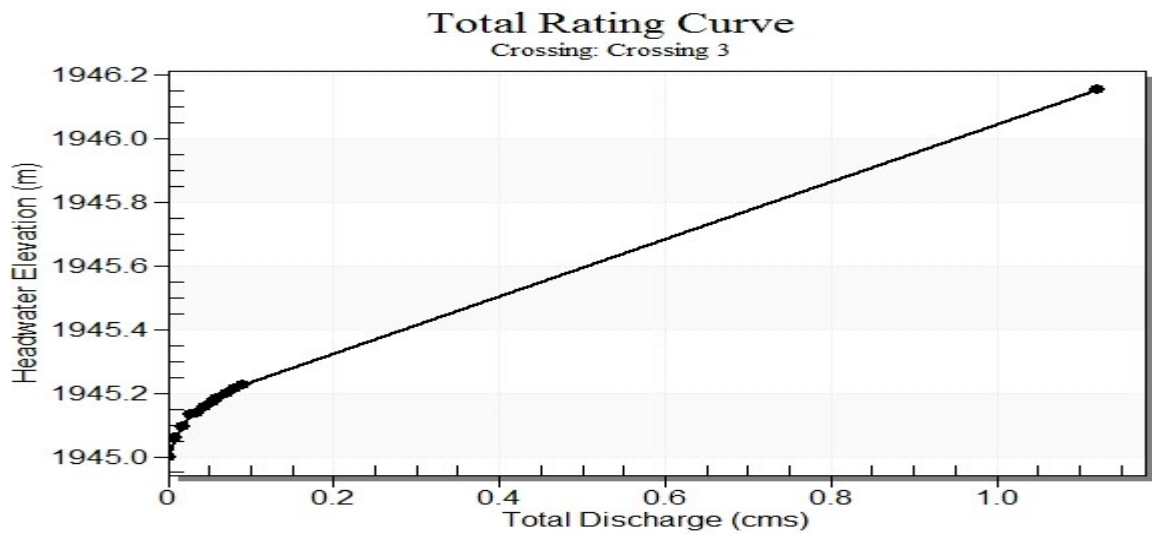
Culvert **Rating Curve Plot for Crossing: outfall 1 crossing**



Rating Curve Plot for Crossing: Crossing 2

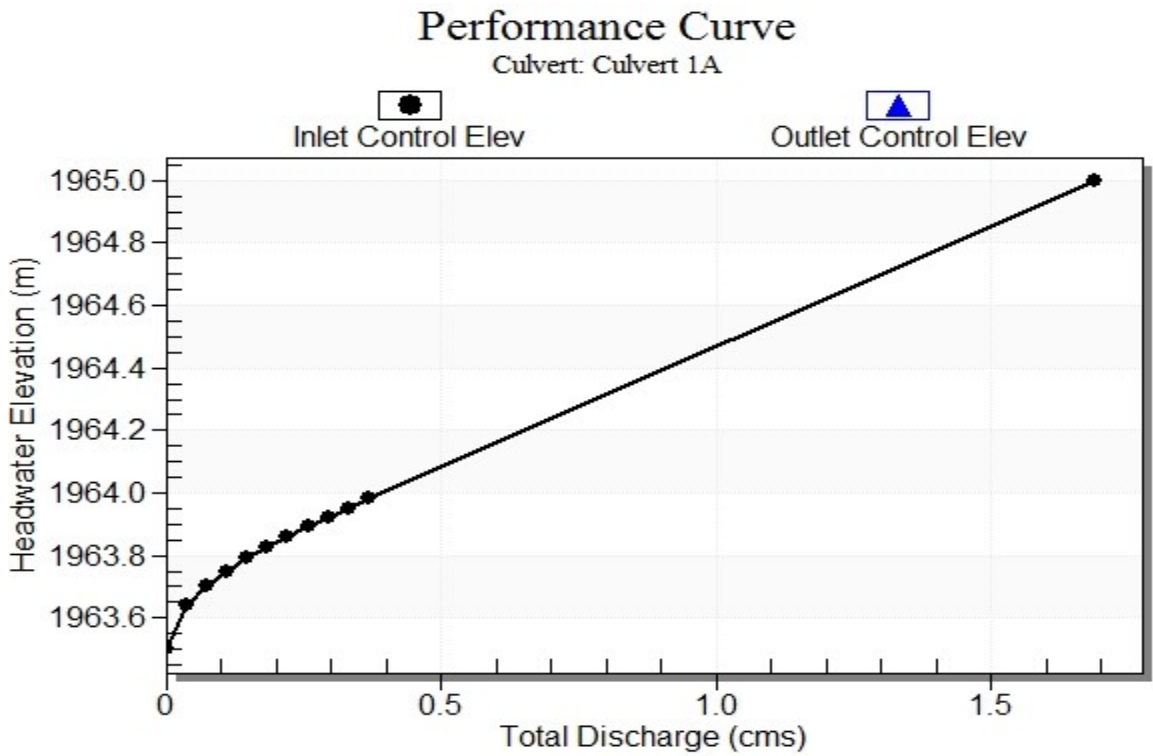


Rating Curve Plot for Crossing: Crossing 3

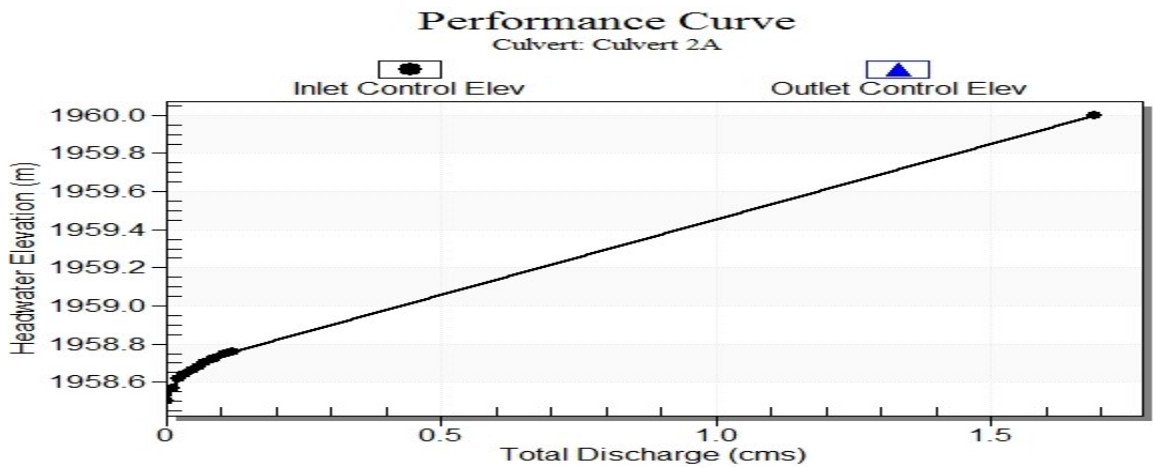


Appendix 16 culvert performance curve.

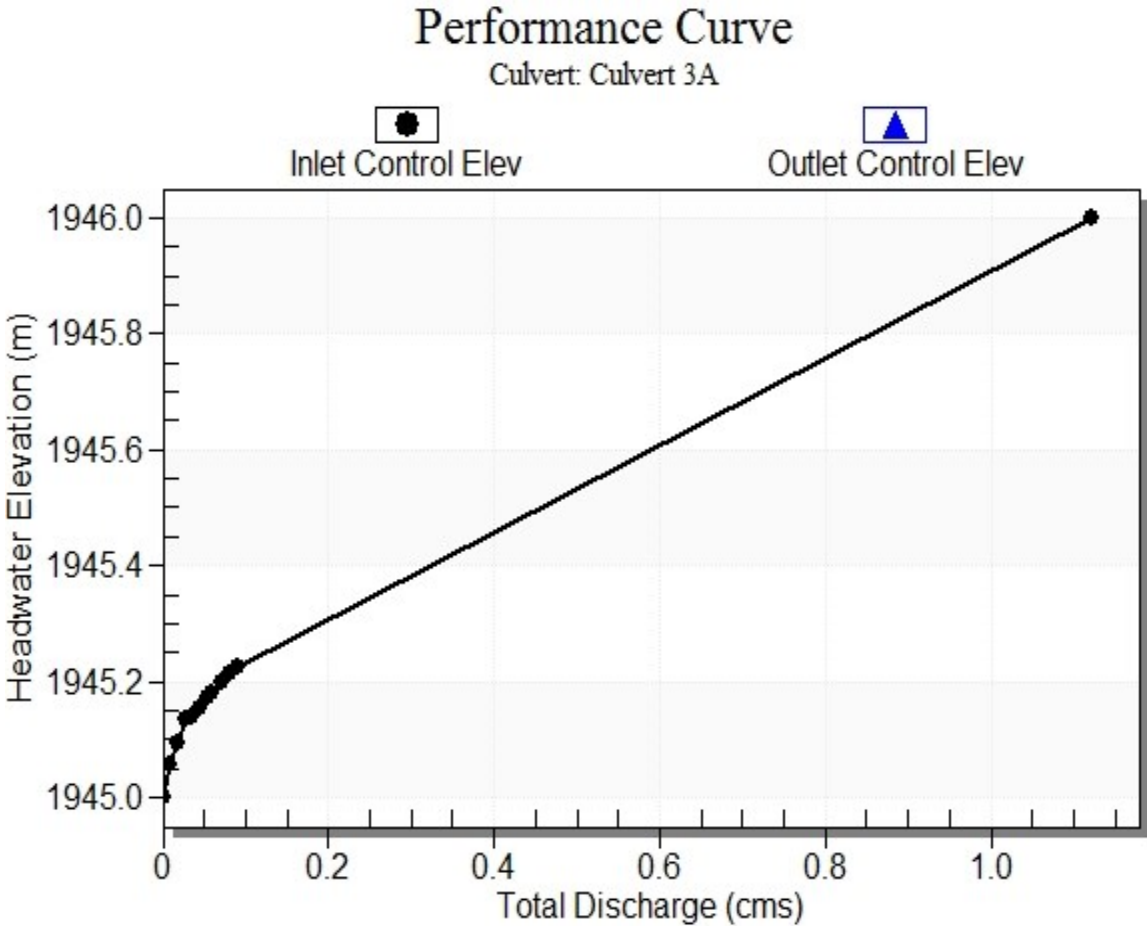
Culvert Performance Curve Plot: Culvert 1A



Culvert Performance Curve Plot: Culvert 2A



Culvert Performance Curve Plot: Culvert 3A



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Appendix 18 consistency checking table (for double mass curve)

	nedjo	gim	nek	dedes	deng	ayira	avg group	AVG GROUP						
	nedjo	gim	nek	dedes	deng	ayira	avg group	AVG GROUP	year	station nedjo	cumulative rainfall	arg perce	cum group	
2017	97.7	61.05	62	102	45.7	68.2	67.79	73.69						
2016	63	60	62.3	58.2	52.9	67.4	60.16	59.28	2017	52.3	87.7	51.4	96.5	
2015	97.8	74.9	77.1	32	76.5	122	76.5	71.66	2016	41	128.7	38.7	135.2	
2014	62.3	72.3		165	118.2	137.5	98.6	83.56	2015	32.6	161.3	35	170.2	
2013	127.2	49.41	50.76	153	49.8	56.2	71.834	86.034	2014	33.6	194.9	31.8	202	
2012	74.6	74.5	77.07	65	76.9	63.5	71.394	73.614	2013	29.4	224.3	22.5	224.5	
2011	49.5	48.9	48.5	49.9	49.8	52.2	49.86	49.32	2012	34.6	258.9	37.4	261.9	
2010	62.1	61.5	61.48	62	62.1	59.7	61.356	61.836	2011	33.5	292.4	49.86	311.76	
2009	45.2	44.85	58.96	44.9	46.9	44.3	47.982	48.162	2010	28	320.4	32.2	343.96	
2008	58.9	77.2	77.5	67.8	53.5	63.2	67.84	66.98	2009	45.2	365.6	47.982	391.942	
2007	52.2	57.26	72.9	47.1	53.7	50.4	56.272	56.632	2008	42.3	407.9	40.3	432.242	
2006	37.5	70.6	68.08	57.4	67.6	28.6	58.456	60.236	2007	31.4	439.3	35.6	467.842	
2005	52	58.3	72.5	48.9	57.9	42	55.92	57.92	2006	37.5	476.8	58.456	526.298	
2004	51.6	60.4	57.8	47.49	61	44.6	54.258	55.658	2005	34	510.8	36	562.298	
2003	49.3	62.78	59.27	50.7	49.7	46.8	53.85	54.35	2004	36	546.8	37.5	599.798	
2002	46.2	53.25	55.03	29.2	60.2	68.5	53.236	48.776	2003	49.3	596.1	53.85	653.648	
2001	36.6	51.72	62.6	40.8	48.1	27.4	46.124	47.964	2002	46.2	642.3	53.236	706.884	
2000	42.9	65.4	68.9	59.24	68.7	29.5	58.348	61.028	2001	36.6	678.9	46.124	753.008	
1999	56.1	63.89	61.5	47.8	60.6	55	57.758	57.978	2000	42.9	721.8	39	792.008	
1998	50.9	60.56	82.3	56.2	57.3	32.6	57.792	61.452	1999	42.3	764.1	40.5	832.508	
1997	50.1	59.9	70.64	43.06	56.7	44.6	54.98	56.08	1998	41	805.1	38	870.508	
1996	41.4	50.8	74.86	60.5	57.2	36.8	56.032	56.952	1997	45	850.1	46.2	916.708	
1995	53.1	44.38	66.4	33.6	52.3	51.4	49.616	49.956	1996	41.4	891.5	56.032	972.74	
1994	45.4	52	42	44	41.5	43.2	44.54	44.98	1995	48.2	939.7	49.616	1022.356	
1993	54.9	55.5	54.5	52.3	58	53.4	54.74	55.04	1994	45.4	88.7	44.98	97.5	
1992	47.4	51.2	38.2	46.2	55.3	47.4	47.66	47.66	1993	54.9	143.6	55.04	152.54	
1991	74.6	86.2	64.5	65.3	73.6	71.5	72.22	72.84	1992	47.4	191	47.66	200.2	
1990	50.2	52.4	54.6	56.8	60.8	46.9	54.3	54.96	1991	74.6	265.6	72.84	273.04	
									1990	50.2	315.8	54.96	328	

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Appendix 18 flow summary at crossing culvert

Headwater Elevation (m)	Total Discharge (cms)	Culvert 2 Discharge (cms)	Roadway Discharge (cms)	Iterations
1958.50	0.00	0.00	0.00	1
1958.57	0.01	0.01	0.00	1
1958.62	0.02	0.02	0.00	1
1958.64	0.04	0.04	0.00	1
1958.66	0.05	0.05	0.00	1
1958.68	0.06	0.06	0.00	1
1958.70	0.07	0.07	0.00	1
1958.72	0.08	0.08	0.00	1
1958.73	0.09	0.09	0.00	1
1958.75	0.11	0.11	0.00	1
1958.76	0.12	0.12	0.00	1
1960.00	1.69	1.69	0.00	Overtopping

Headwater Elevation (m)	Total Discharge (cms)	Culvert 3 Discharge (cms)	Roadway Discharge (cms)	Iterations
1945.00	0.00	0.00	0.00	1
1945.06	0.01	0.01	0.00	1
1945.09	0.02	0.02	0.00	1
1945.13	0.03	0.03	0.00	1
1945.14	0.04	0.04	0.00	1
1945.15	0.04	0.04	0.00	1
1945.17	0.05	0.05	0.00	1
1945.19	0.06	0.06	0.00	1
1945.20	0.07	0.07	0.00	1
1945.21	0.08	0.08	0.00	1
1945.23	0.09	0.09	0.00	1
1946.00	1.12	1.12	0.00	Overtopping

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Appendix 20: water surface profile table

Total Discharge (cms)	Culvert Discharge (cms)	Headwater Elevation (m)	Inlet Control Depth(m)	Outlet Control Depth(m)	Flow Type	Length Full (m)	Length Free (m)
0.00	0.00	1958.50	0.00	0.0	0-NF	0.00	0.00
0.01	0.01	1958.57	0.07	0.0*	1-S2n	0.00	15.00
0.02	0.02	1958.62	0.12	0.0*	1-S2n	0.00	15.00
0.04	0.04	1958.64	0.14	0.0*	1-S2n	0.00	15.00
0.05	0.05	1958.66	0.16	0.0*	1-S2n	0.00	15.00
0.06	0.06	1958.68	0.18	0.0*	1-S2n	0.00	15.00
0.07	0.07	1958.70	0.20	0.0*	1-S2n	0.00	15.00
0.08	0.08	1958.72	0.22	0.0*	1-S2n	0.00	15.00
0.09	0.09	1958.73	0.23	0.0*	1-S2n	0.00	15.00
0.11	0.11	1958.75	0.25	0.0*	1-S2n	0.00	15.00
0.12	0.12	1958.76	0.26	0.0*	1-S2n	0.00	15.00

Total Discharge (cms)	Culvert Discharge (cms)	Headwater Elevation (m)	Inlet Control Depth(m)	Outlet Control Depth(m)	Flow Type	Length Full (m)	Length Free (m)
0.00	0.00	1958.50	0.00	0.0	0-NF	0.00	0.00
0.01	0.01	1958.57	0.07	0.0*	1-S2n	0.00	15.00
0.02	0.02	1958.62	0.12	0.0*	1-S2n	0.00	15.00
0.04	0.04	1958.64	0.14	0.0*	1-S2n	0.00	15.00
0.05	0.05	1958.66	0.16	0.0*	1-S2n	0.00	15.00
0.06	0.06	1958.68	0.18	0.0*	1-S2n	0.00	15.00
0.07	0.07	1958.70	0.20	0.0*	1-S2n	0.00	15.00
0.08	0.08	1958.72	0.22	0.0*	1-S2n	0.00	15.00
0.09	0.09	1958.73	0.23	0.0*	1-S2n	0.00	15.00
0.11	0.11	1958.75	0.25	0.0*	1-S2n	0.00	15.00
0.12	0.12	1958.76	0.26	0.0*	1-S2n	0.00	15.00

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Appendix 21: culvert summary

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	1958.50	0.00	0.0	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
0.01	0.01	1958.57	0.07	0.0*	1-S2n	0.02	0.06	0.04	0.01	0.58	0.38
0.02	0.02	1958.62	0.12	0.0*	1-S2n	0.03	0.09	0.05	0.02	0.96	0.50
0.04	0.04	1958.64	0.14	0.0*	1-S2n	0.05	0.11	0.06	0.02	1.31	0.58
0.05	0.05	1958.66	0.16	0.0*	1-S2n	0.06	0.12	0.06	0.02	1.75	0.65
0.06	0.06	1958.68	0.18	0.0*	1-S2n	0.08	0.14	0.08	0.03	2.09	0.72
0.07	0.07	1958.70	0.20	0.0*	1-S2n	0.09	0.15	0.09	0.03	2.53	0.77
0.08	0.08	1958.72	0.22	0.0*	1-S2n	0.09	0.16	0.09	0.03	2.96	0.82
0.09	0.09	1958.73	0.23	0.0*	1-S2n	0.09	0.17	0.10	0.04	2.26	0.84
0.11	0.11	1958.75	0.25	0.0*	1-S2n	0.10	0.19	0.11	0.04	2.34	0.90
0.12	0.12	1958.76	0.26	0.0*	1-S2n	0.11	0.20	0.12	0.04	2.41	0.94

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	1945.00	0.00	0.0	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
0.01	0.01	1945.06	0.06	0.0*	1-S2n	0.01	0.05	0.01	0.01	1.15	0.51
0.02	0.02	1945.09	0.09	0.0*	1-S2n	0.02	0.07	0.05	0.01	0.76	0.66
0.03	0.03	1945.13	0.13	0.0*	1-S2n	0.04	0.09	0.06	0.02	1.05	0.78
0.04	0.04	1945.14	0.14	0.0*	1-S2n	0.05	0.11	0.06	0.02	1.31	0.87
0.04	0.04	1945.15	0.15	0.0*	1-S2n	0.05	0.11	0.07	0.02	1.43	0.91
0.05	0.05	1945.17	0.17	0.0*	1-S2n	0.07	0.13	0.08	0.03	1.88	1.03
0.06	0.06	1945.19	0.19	0.0*	1-S2n	0.08	0.14	0.08	0.03	2.19	1.09
0.07	0.07	1945.20	0.20	0.0*	1-S2n	0.09	0.15	0.09	0.03	2.53	1.15
0.08	0.08	1945.21	0.21	0.0*	1-S2n	0.09	0.16	0.09	0.03	2.84	1.20
0.09	0.09	1945.23	0.23	0.0*	1-S2n	0.09	0.17	0.10	0.04	2.26	1.25

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Appendix 22: crossing data for culvert

Crossing Data - outfall 1 crossing

Crossing Properties
Name: outfall 1 crossing

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	0.00	cms
Design Flow	0.22	cms
Maximum Flow	0.37	cms
TAILWATER DATA		
Channel Type	Irregular Channel	
Irregular Channel	Define...	
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	m
Crest Length	0.90	m
Crest Elevation	1965.00	m
Roadway Surface	Paved	
Top Width	15.00	m

Culvert Properties

Culvert 1A

Add Culvert
Duplicate Culvert
Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 1A	
Shape	Circular	
Material	Concrete	
Diameter	900.00	mm
Embedment Depth	0.00	mm
Manning's n	0.0120	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	m
Inlet Elevation	1963.50	m
Outlet Station	15.00	m
Outlet Elevation	1963.00	m
Number of Barrels	1	

Crossing Data - Crossing 2

Crossing Properties
Name: Crossing 2

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	0.00	cms
Design Flow	0.09	cms
Maximum Flow	0.12	cms
TAILWATER DATA		
Channel Type	Rectangular Channel	
Bottom Width	3.00	m
Channel Slope	0.0200	m/m
Manning's n (channel)	0.0180	
Channel Invert Elevation	1958.00	m
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.00	m
Crest Length	0.90	m
Crest Elevation	1960.00	m
Roadway Surface	Paved	
Top Width	15.00	m

Culvert Properties

Culvert 2A

Add Culvert
Duplicate Culvert
Delete Culvert

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 2A	
Shape	Circular	
Material	Concrete	
Diameter	900.00	mm
Embedment Depth	0.00	mm
Manning's n	0.0120	
Culvert Type	Straight	
Inlet Configuration	Square Edge with Headwall	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.00	m
Inlet Elevation	1958.50	m
Outlet Station	15.00	m
Outlet Elevation	1958.00	m
Number of Barrels	1	

Appendix 23: broad-crested weir coefficient C as function of weir crest breadth and head

Broad-Crested Weir Coefficient C Values as a Function of Weir Crest Breadth and Head (coefficient has units of $m^{0.5}/sec$) ⁽¹⁾															
Head ⁽²⁾ (m)	Breadth of Crest of Weir (m)														
	0.15	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.25	1.50	2.00	3.00	4.00
0.10	1.59	1.56	1.50	1.47	1.45	1.43	1.42	1.41	1.40	1.39	1.37	1.35	1.36	1.40	1.45
0.15	1.65	1.60	1.51	1.48	1.45	1.44	1.44	1.44	1.45	1.45	1.44	1.43	1.44	1.45	1.47
0.20	1.73	1.66	1.54	1.49	1.46	1.44	1.44	1.45	1.47	1.48	1.48	1.49	1.49	1.49	1.48
0.30	1.83	1.77	1.64	1.56	1.50	1.47	1.46	1.46	1.46	1.47	1.47	1.48	1.48	1.48	1.46
0.40	1.83	1.80	1.74	1.65	1.57	1.52	1.49	1.47	1.46	1.46	1.47	1.47	1.47	1.48	1.47
0.50	1.83	1.82	1.81	1.74	1.67	1.60	1.55	1.51	1.48	1.48	1.47	1.46	1.46	1.46	1.45
0.60	1.83	1.83	1.82	1.73	1.65	1.58	1.54	1.46	1.31	1.34	1.48	1.46	1.46	1.46	1.45
0.70	1.83	1.83	1.83	1.78	1.72	1.65	1.60	1.53	1.44	1.45	1.49	1.47	1.47	1.46	1.45
0.80	1.83	1.83	1.83	1.82	1.79	1.72	1.66	1.60	1.57	1.55	1.50	1.47	1.47	1.46	1.45
0.90	1.83	1.83	1.83	1.83	1.81	1.76	1.71	1.66	1.61	1.58	1.50	1.47	1.47	1.46	1.45
1.00	1.83	1.83	1.83	1.83	1.82	1.81	1.76	1.70	1.64	1.60	1.51	1.48	1.47	1.46	1.45
1.10	1.83	1.83	1.83	1.83	1.83	1.83	1.80	1.75	1.66	1.62	1.52	1.49	1.47	1.46	1.45
1.20	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.70	1.65	1.53	1.49	1.48	1.46	1.45
1.30	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.82	1.77	1.71	1.56	1.51	1.49	1.46	1.45
1.40	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.77	1.60	1.52	1.50	1.46	1.45
1.50	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.79	1.66	1.55	1.51	1.46	1.45
1.60	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.83	1.81	1.74	1.58	1.53	1.46	1.45

(1) Modified from Reference 49

(2) Measured at least 2.5 H_c upstream of the weir