Addis Ababa University

Institute of Technology

School of Civil and Environmental Engineering

Postgraduate Program in Hydraulic Engineering

Performance Assessment of Road Drainage Systems of Burayu Town

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By

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A Thesis Submitted to the School of Civil and Environmental Engineering Presented in Partial Fulfillment of the Requirements for the Degree of Master of Science in School of Civil and Environmental Engineering under the Stream of Hydraulic Engineering

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ADDIS ABABA INSTITUTE OF TECHNOLOGY

School of Civil and Environmental Engineering

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Approval by Board of Examiners

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Abstract

The objective of the study is to assess the Performance of Drainage Systems of Road at Burayu Town Administration. The study employed both primary and secondary data collection. The primary data collection were observation (field visit), Interview, and photographs that show the existing drainage structure conditions and information that were gathered from the residents about the performance of the drainage structures during the rainy season. The secondary data sources were land cover map and geological map. Hydrological analysis was carried out by using Rational and SCS equations. Hydraulic parameters were determined from the design documents data during construction and from the construction sites by measuring the parameters where the structures are constructed by using Manning's equation and GIS was used for watershed delineation. The findings of the study that were investigated includes: In most part of the town, runoffs run over road surfaces were due to soil erosion, flatness of the ditch channel, poor waste management system and lacking of the drainage structure at some station. As there is no adequate provision for garbage container in the town, residents dispose wastes in to the drainage channels, culvert, open spaces, and rivers/ streams. The drainage channels are filled with or blocked by silt and garbage, the blockage of drainage structures is due to flat areas and siltation of drainage system which leads to failures on roadways. The computed discharge values which were determined by both SCS methods and rational method were compared by dividing the watershed area according to their area of application. At station 16+200 the discharges were computed by SCS method for 60ha and also, computed by rational method by divided 60ha into two parts 30ha/0.3km^2 and compared the results. The hydraulic calculation was carried out at station 16+200 by using Manning equation and checked by trial and error. Therefore, the design and check discharges at station 16+200 are 17.0m^3/sec and 21.5m^3/sec respectively. Therefore, new drainage structure will be needed at station 16+200. Therefore, this study recommends improvement in the integration of road and drainage structures serve for the intended purposes sustainably to the stakeholders.

Keywords: Drainage Structure, Urban Road, Flooding, Blockage, Maintenance

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Acronyms

AACRA: Addis Ababa City Roads Authority

AASHTO: American Association of State Highway and Transportation Officials

ACPA: American Concrete Pipe Association

ADOT: Arizona Department of Transportation

AMC: Antecedent Moisture Content

ASCE: American Society of Civil Engineers

CN: Curve Number

DDM: Drainage Design Manual

DEM: Digital Elevation Model

DS3/4: Design Standard Three/four

EMA: Ethiopian Mapping 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

Km^2: Square kilometer

m: Meter

masl: Meter Above Sea Level

MoUDC: Ministry of Urban Development and Construction

NMSA: National Meteorological Service Agency

R^2 : Coefficient of Determination

UDA: Urban Dwellers Association

USACE: United States Army Corps of Engineers

USAID: United States Agency for International Development

USGS: United States Geological Survey

US NRCS: United States Natural Resources Conservation Service

USSCS: United States Soil Conservation Service

Yrs: years

1. INTRODUCTION

1.1 General Background of the Drainage Structures

Though water is very essential for all life on earth, it can also cause devastation through erosion and flooding. Due to the development of infrastructures as a result of urbanization, the surface runoff water greatly flow over road surface in town and damages the road section. The contributed runoff water, thus, need to be safely disposed to the rivers/outlet channels so that the functional utility of the road infrastructure maintained and there by avoid the damages which otherwise occurred to the road and property (Liang, 2001).The objective of roadway drainage is to prevent on site water standing on the surface and convey the offsite storm runoff from one side of the roadway to the other.

In the design of highway, highway drainage structures are extremely important components. Adequate drainage is essential in the design of highways since it affects the highways serviceability and usable. Drainage of road is one of the many components of a road project. The objective of road drainage is to remove the storm water as rapidly as possible so that traffic may move safely and efficiently without any loss of time.

Roadway drainage involves the collection, conveyance, removal, and disposal of surface water runoff from the traveled way, shoulders, and adjoining roadside areas (FHWA, 1996).A complete drainage system design includes consideration of both major and minor drainage systems. The minor system, sometimes referred to as the "Convenience" system, consists of the components that historically considered as part of the "storm drainage system". These components include curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, detention basins, and water quality control facilities (Alderson, 2006).

Provision of culverts of adequate size and numbers in a road drainage scheme whether the road is a new one or an up-gradation of an existing one is intimately related to the health and safety of the road. Provision of adequate drainage is an important factor in the location and geometric design of highways.

Adequate level of service can be acquired by properly designing them. Initial cost, design life, and the risk of loss of use of the road way for a time due to runoff exceeding the capacity of the drainage structure, need to be considered in the design. Adequate drainage facilities should be provided for the flow of water away from the surface and subsurface of the road to properly designed channels and then discharge to the natural waterways.

1.2 Background of the Study Area

Burayu Town was founded in 1938 by a land lord of that area named “Grazmatch” RobiKelecha. The name “Burayu” was derived from one of the indigenous trees of the region. The term “Burayu” is an Afanoromo word, which means “TiqurInchet”in Amharic. During Imperial era, Burayu used to be administered within “Menagesha Awuraja”of Shoa Province. During the Derg regime Burayu used to have two different entities namely: Urban Dwellers Association (UDA) and Peasant Association (PA) until 1991, at that time Burayu was one kebele of Wolmera district under the West Shoa Administration Zone. In 1996, the municipality of Burayu town was established over the area between Ketta and Gefersa.

Burayu town is one of the corridor routes that link the central part of the country to the Oromia region specially Asko&Oromia National Administration and Regional States. The route was constructed by single surface treatment pavement structure in early years.

According to Burayu Town Urban Infrastructure Management Report (2008-2010), the town lies at a relatively gentle slope where runoff runs and drains towards the nearest natural water body. Most parts of the town are covered by reddish soil that has low infiltration rate. There are a number of dry streams in the town, which collect runoff from their watersheds and drain it towards river. Due to soil erosion and poor waste management system, the drainage channels are filled with or blocked by silt and garbage.

These reduce the carrying capacity of drainage channels and create overflow of runoffs. A major problem observed during data collection is disposal of solid and liquid waste in the storm water drainage line, which results in the blockage of the existing drainage channels and hence impairs their ability to convey the runoff properly.

A drainage structure like ditches, culverts and bridges are available in Burayu town road. Burayu is located in Oromia National Regional State, Oromia Special Zone surrounding Finfine at a distance of 15km from Addis Ababa (Source:MoUDC,2010). Nevertheless, sever damage is noticed in most of the road segment. The causes of the failure are anticipated to be various. This failure results in serious damage to highway structure and traffic operation problems. These problems results in loss of a road section or even many road sections greatly hampering the traffic flow that negatively contributes to the mobility of the road users in general and the socio-economic well-being of the people in particular. The absence of proper drainage negatively affects the economy of the community (car crash, delay in time or time wastage, wastage of gasoline) as well as social activities such as facilitation of trade and businesses, access to basic services, safe transportation.

1.3 Statement of the Problem

Road construction without adequate provision of drainage is a major cause of gully erosion. The road that has no proper drainage systems which results in gully erosion may occur anywhere in the world. The problem is particularly severe in developing countries due to neglect in maintenance and lack of provision of safe outlets to the excess runoff.

In Oromia region at Burayu town, road drainage structures are not properly functioning due to the following. The main causes for improper functioning of the road are insufficient capacity of road ditches, unavailability of drainage structures at proper place, small number of culverts, street flood, poor management and siltation.

The major drainage related problems that are observed in the town includes: lack of drainage lines along most of the roads in the town; lack of appropriate maintenance of the existing drainage facilities (existing drainage channels are either partially or fully filled with silts and causing the storm water to flow over the road surface) and formation of gullies and rills across the existing drainage channels and roads.

The presence of lined drainage lines are limited to some parts of the town and in most parts of the town runoff flows over the surface of roads. Due to soil erosion and poor waste management system, the drainage channels are filled with or blocked by silt and garbage. These inadequacy results in the blockage of the existing drainage channels and hence impairs their ability to convey the runoff properly.

At road at stations (16+900, 17+200), the road side ditch were loaded or silted by many things such as debris, silt, waste materials, and disposal of solid and liquid waste in the storm water drainage line. At station 17+600, the road becomes flooded or there was a flood across the road (street flood) and at station 18+200, the ditch is broken. Those problems created the road to malfunction during the rainy season every year.

To improve this problem, culvert and ditch drainage structures performance should be evaluated and improvement measures should be proposed for sustainable and proper functioning.

1.4 Research Questions

The study addressed the following basic research questions:

1. What are the types of drainage structures failures on Burayu town road?
2. What are the major causes of drainage structures failures on Burayu town road?
3. Are the hydraulic capacities of the different drainage structures of the road adequate?

1. 5 Objectives of the Study

1.5.1 General Objective

The general objective of the study is to assess the performance of the existing road drainage structures of Burayu town road.

1.5.2 Specific Objectives

The following are the specific objectives of the study:

1. To assess the condition of the existing drainage system
2. To evaluate the hydraulic capacity of the existing road drainage system
3. To examine the impacts of storm water drainage infrastructure on road performance and related environment issues

1.6. Significance of the Study

The study of road drainage structures needs considerable amount of skilled work force, money, energy and time. The road that has no proper drainage systems fails to serve its function and it requires a high investment in maintaining the road to avoid traffic interruption. Therefore, this study, generally, contributes the following significances to Ethiopia Road Authority (ERA), Burayu Town Administration, academicians, researchers, and other stake holders who will conduct similar researches on other road drainage structures. Some of them include:

- preserving the road network and save the asset value
- identifying the existing situation in order to know the real problems of the drainage systems
- finding possible solutions based on the recommendations to avoid the problems

1.7. Scope of the Study

This study is geographically limited to Burayu Town Administration. Generally, the research focuses on the performance assessment of the existing drainage systems in Burayu Town Administration road.

2. LITERATURE REVIEW

The primary purpose of road drainage structure is to serve as conveyance structures preventing water from pooling on the roadway surface. Effective drainage structures prevent overland runoff from reaching the roadway, as well as drain water from the road surface. Economical drainage structure should convey the intended design storm efficiently and require minimum maintenance while serving their intended purpose of roadway drainage structures (ERA, 2002).

2.1General Description of Road Drainage Structures

Road drainage structures that cross the rivers and valleys are vital components of the road network that contributes greatly to the national development and public daily life. Any damage or collapse of these structures can cause the risk of the lives of road users as well as create serious influence to the entire country economic development (Liang, 200). Furthermore, the reconstruction of these road drainage structures needs considerable amount of skilled work force, money and time. Road drainage structures are essential components during the design development of road infrastructures. Drainage structures intended to allow the runoff of any flow of water with limited damages and disturbances to the road and to the surrounding areas.

Two types of drainage systems, surface and sub-surface, are commonly used to conduct water away from the area surrounding the road and to evacuate extra water from the road structure. The surface drainage system controls surface water caused by direct rainfall, melted snow, or surface runoff (Dawson, 2009). Surface drainage involves collecting the water from the road surface, road shoulders, side slopes and adjacent areas and carrying it away via downhill slopes, roadside ditches and pipes. The design of road drainage systems varies with factors such as road importance and age, traffic load and rural/urban area (Faisca et al., 2009).A surface drainage system (ditch) collects and diverts storm water from the road surface and surrounding areas to avoid flooding. It also prevents damage to sub-surface drains, water supplies (wells) and other sensitive areas adjacent to roads. It decreases the possibility of water infiltration into the road and retains the road bearing capability (Faisca et al., 2009).

Subsurface drainage systems drain water that has infiltrated through the pavement and the inner slope, but also groundwater. Subsurface drainage systems usually comprise culverts and have a direct linkage to surface drainage systems (Dawson, 2009). The subsurface drainage system drains subsurface water from in our roads or from the subsurface areas surrounding our roads.

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

A complete drainage system design includes consideration of both major and minor drainage systems. The minor system, sometimes referred to as the "Convenience “system, consists of the components that historically considered as part of the "storm drainage system". These components include curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, detention basins, and water quality control facilities (Alderson, 2006). Inadequate urban storm water drainage problems represent one of the most common sources of compliant from the citizens in many towns of Ethiopia (GTZ-IS, 2006), and this problem is getting worse and worse with the ongoing high rate of urbanization.

The pattern of urbanization and modernization Ethiopia has meant increase densification along with urban infrastructure development. This has led to deforestation, use of corrugated roofs and paved surfaces. The combined effect of this results in higher raindrop intensity and consequently accelerated and concentrated runoff. Due to inadequate integration between road and urban storm water drainage, infrastructure provision and poor management significant proportion of the area is exposed to flooding hazards/risks. Appropriate design of the surface drainage system is an essential part of road design (Kalantari, 2011).

The basic design techniques in roadway drainage system should be developed for economic design of surface drainage structures including ditches, culverts and bridges (ERA, 2002). A hydraulic investigation and analysis of both the upstream and downstream reaches of the watercourse is necessary to determine the best location, size, and elevation of the proposed crossroad structure, whether a culvert or a bridge.

The investigation should ensure that any roadway structure or roadway embankment that encroaches on or crosses the flood plain of a watercourse will not cause significant adverse effect to the flood plain and will be capable of withstanding the flood flow with minimal damage. It is significant to provide attention during design of the magnitude, frequency and appropriate water surface elevations for the design flood, the 100-year flood, and the overtopping or 500-year flood for all structures (ADOT, 2007).

Culverts are usually, designed to operate with the inlet submerged if conditions permit. This allows for a hydraulic advantage by increasing discharge capacity. Bridges are usually, designed for non-submergence during the design flood event, and often incorporate some freeboard.

Providing significant amount of freeboard is important for bridges to allow passage of drift, debris, and ice at high water levels, as well as to accommodate uncertainty in the design of high water elevation or the possibility of an event more than the design event. The impact of sediment and other floating materials can attribute the damage of bridge deck (Melville and Coleman, 2000). A freeboard of 1.5m should be provided for bridges, for smaller streams of expected less size of debris, a freeboard of less than 1.5m is provided, however, according to ERA draft drainage design manual, the minimum freeboard must not be less than 1.0m (ERA, 2001).

The two main types of water flows that can be considered are the flows that usually crossing the area that could be diverted by the presence of the road, and the flows generated by the runoff of the rainwater falling on the carriageway and its surroundings. The basic design techniques in roadway drainage system should be developed for economic design of surface drainage structures including ditches, culverts and bridges (ERA, 2002).

2.1.1. Road Drainage Culverts

2.1.1.1. 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, nallas, 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 (MOWR, 2004). 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 (Mazumder et al, 2002). 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.

2.1.1.2. 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 (AACRA, 2004).The minimum grade for a culvert should generally be 0.5 (ACT Government, 1994).Flatter grades may be prone to siltation and are difficult to construct. The maximum grade for a culvert should be chosen to limit the pipe full flow velocity to a value less than or equal to 6m/sec to avoid scour (ACT Government, 1994).Installing culverts without wing walls and head walls will decrease the hydraulic efficiency of the culvert. As a result, scouring and potential of diversion of water will be created.

2.1.1.3. 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 (Austroads, 1994).

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 (ERA, 2002) and to minimize the modifications in the hydrology of the area.

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.

For upgrading projects, hume pipe culverts having diameter less than 900mm are to be replaced with a minimum diameter 1200mm as recommended by IRC: SP:13 (2004).

2.1.2 Road Drainage Ditches

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.

Roadside ditch design is an important aspect of roadway structure and safety. Their design may be influenced by many factors, including motorists' safety, aesthetics, economy of construction and maintenance . A stable ditch should provide adequate capacity for the intended design storm and be resistant to erosion failures. Ideally, the ditch design should also provide recoverable slopes to enhance driver safety in errant vehicles. Inadequate design of ditch capacity can result in compromised driver safety conditions, including water overtopping the ditch line and flooding the intended path of travel (VODT, 1998).

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.

In addition, erosion failures can result in steeper ditch side slopes, and possibly failure of the roadway shoulder and structure. Adequate right-of-way should be acquired to accommodate the required ditch side slopes and capacity.

The successful design of roadside ditches must satisfy two requirements: ditch capacity and stability. The logic implied in the selection of storm return period is that the initial period after ditch construction, before vegetation is developed, constitutes the most critical period regarding ditch stability. After vegetation is fully developed, the channel is considered stable and channel capacity becomes more critical (VODT, 1995).

2.1.2.1. Ditch Shape

Channel shapes are generally determined for a location by considering the terrain, flow regime and the quantity of flow to be conveyed. Typically, ditch geometry is either rectangular, Vshaped and trapezoidal. Roadway channels should provide recoverable slopes, thereby minimizing the impact of errant vehicles. This can be accomplished by designing ditch cross-sections with mild side slopes. Depending on side slopes used, both V-shaped and trapezoidal ditches can provide driver safety and be economical to construct. When mild side slopes are used, the shape tends to approach a parabolic shape, which is recognized as being the most hydraulically efficient shape (AASHTO, 1992).

Because V-shaped ditches are more susceptible to erosion trapezoidal ditches may be preferred on certain soil conditions, such as fill sections and highly erodible soils.

The side slopes of the ditch/channel should not exceed the angle of repose of the soil comprising the ditch line, and should generally be 3:1 or flatter (Brown et al, 1996, AASHTO 1991). Where local conditions dictate the use of some type of rigid lining, the use of steeper slopes (>2:1) may be more economical.

2.1.2.2. Ditch Slope

Channel slope is one of the major parameters in determining shear stress exerted by the flow on the boundary. Particle entrainment will occur when the shear stress exerted on the boundary by the flow exceeds the resisting shear stress provided by the boundary. Because particle entrainment is to be minimized, flow in roadside ditches is usually designed to be subcritical (Chen and Cotton, 1988).

Special design features, such as drop structures, check dams, etc., should be considered to minimize shear stresses exerted on the ditch boundary/lining, and avoid the occurrence of supercritical flow. Carefulness should be exercised when designing ditch gradients in excess of roadway gradients. When this practice is necessary, for example when draining the ditch to a natural stream, proper safety measures should be installed to insure driver safety along the design reach.

2.1.2.3. Lining Considerations

During the construction phase, when the bare ditch line is fully exposed to weathering processes, the chance for significant erosion is the highest (Gray and Sotir, 1996). Consequently, a net lateral movement down to the flow line would be expected on embankments, such as ditch side walls. When loosely compacted particles, such as particles displaced by raindrops, enter the flow line, they can become easily entrained during storm events. Once entrained by the flow, sediment will be carried downstream, eventually being deposited within the ditch or in a receiving stream. The receiving stream water quality may become impaired, depending on the significance of the erosion/deposition event, and consequently the stream ecosystem may be adversely affected.

Because erosion can become a significant consequence from roadway ditch construction, protective linings, either temporary and/or permanent, should be applied when necessary.

Temporary linings are expected to provide erosion protection through the establishment of vegetation and then degrade over time, typically a 2-year period (WSDOT, 1990). When the predicted velocity of a ditch exceeds the stability criteria for the soil comprising the ditch lining, temporary linings are used. When flow conditions exceed the stability of these lining, permanent linings will be specified. Permanent linings can be either flexible or rigid. Typically, rigid linings include concrete, paved or other low permeability, linings.

Because these types of linings tend to have low permeability, concrete and pavement linings may inhibit infiltration where infiltration may be desirable. High velocities are generated, which may cause erosion problems at ditch outlets.

In addition, rigid linings have been associated with water seeping beneath the structure, at the sidewalls and structure inlet, causing soil piping under the structure and consequent failure. When rigid linings fail, the failure is usually abrupt with little forewarning, and, consequent repairs are expensive.

Virginia Department of Transportation hydraulic designers and discussions with other states' hydraulic engineers, it was learned that concrete and pavement linings are used less frequently in modern ditch design because of high failure rates associated with piping under the structure. Instead, riprap is being used more frequently as a rigid lining.

Riprap provides a less rigid boundary and the ability to mold with changing conditions in the ditch line while providing continued erosion protection. When used with a geo fabric underlying the rock, fine particle entrainment can be minimized.

When stability criteria permit, flexible permanent linings, typically grass, are preferred over rigid linings. Flexible linings are generally less costly to construct and have reduced maintenance costs. In addition, flexible linings, such as grass, offer high hydraulic resistance, which promotes lower velocities and increased infiltration. While the grass blades serve to reduce flow velocity and thereby lowering shear stress, the root structure reinforces the shear resistance of the soil. As a natural permanent lining, vegetated linings have the ability re-establish themselves seasonally, provided proper environmental conditions. Typically, states have a variety of seed mixes to accommodate seasonal and geographical changes across their state. Generally, a period of about 2-years is observed to ensure full establishment of a vegetated lining (VDOT, 1989).

2.2. Alignment of Drainage Structures

Culverts that have internal diameter less than or equal to 1.22m are minor drainage structures. The vertical alignment of a culvert with respect to the stream channel is important to its hydraulic performance, to stream stability, to construction and maintenance costs, and to the safety and integrity of the roadway. Proper alignment is also particular importance to prevent outlet scour or excessive sediment buildup in the culvert barrels.

A culvert placed too low in relation to the channel bottom may lose hydraulic performance if the channel aggrades. In addition, a culvert placed at a slope different from the channel slope may have problems related to both sediment deposition and bed scour, and this affects hydraulic performance.

The horizontal alignment of culverts and bridges should match the natural streambed alignment, as close as practicable. This is often possible when installing an original culvert at a new crossing or when removing the existing culvert and replacing it with another at exactly the same location. A culvert invert slope should match the streambed slope. Placing culvert on a flatter or steeper gradient from the natural streambed can cause sediment deposition in the barrel. It can also scour that removes sediment from the barrel.

2.3. Flow Velocity in Road Drainage Structures

The introduction of a culvert to convey the stream flow beneath a roadway can cause an increase in flow velocity downstream of the structure. The increased flow velocity may be sufficient to cause erosion and degradation of the channel profile. This effect can be detrimental to downstream land users and to the culvert itself. If the natural stream velocity, then the increased velocity at the culvert outfall will accelerate this naturally occurring process. Erosive velocity must be avoided to protect lower lands and the roadway embankment. The flow velocity at the outlet of the roadway drainage woks shall not exceed the erosive velocity of the channel or the natural velocity of the channel, which is greater.

2.4. Design Flood for Road Drainage Structures

Drainage works designed for storms having a recurrence interval of at least that are presented in Appendix A of table 3. Moreover, all bridges and major culverts checked for performance under a storm event less frequent than the design storm events that are presented in Appendix A of table 3.

All the other drainage structures checked for the storm having the next lower frequent than the design storm event. Minor culverts designed for a 10-year storm and checked for adequate performance with a 25-year interval storm event (FHWA, 2001).

2.4.1. The Criteria for Roadside Channels

In Ethiopia, the design discharge frequency for permanent roadside ditch linings should be according to the values and channel side slopes should not exceed the angle of repose of the soil and/or lining should be 2:1 or flatter in the case of rock riprap lining. Stone pitching or grouted riprap must be used for channel side slopes steeper than 2:1.

2.5. Description and Function of Road Drainage Structures

Storm drainage facilities consist of storm drains, gutters, ditches, curbs, inlets and culverts.

The placement and hydraulic capacities of storm drainage structures and conveyances should be designed to avoid/minimize damage to adjacent property and secure a low degree of risk of traffic interruption by flooding. Different types of structures are employed in the drainage structure:

- Open channels whether artificial or natural convey the flows of water
- Culverts and bridges convey flows under road cross-section
- Energy dissipaters, used to control the velocities of flows, especially at culvert outlets
- Storm drainage facilities, used to collect the runoff of the carriageway and surrounding areas and direct it to the channels (ERA, 2002).

2.5.1. Description of Road Drainage Structures

There are two different types of drainage structures commonly used to direct water from the area surrounding the road and to evacuate extra water from the road structures. These are surface and sub-surface structures.

A surface drainage system collects and diverts storm water from the road surface and adjoining areas to avoid flooding. It decreases the possibility of water infiltration into the road and retains the road bearing capacity. Appropriate design of the surface drainage system is an essential part of road design (Kalantari, 2011). Sub-surface drainage systems drain water that has infiltrated through the pavement and the inner slope but also ground water.

In flat terrain, where there is a high risk of silting, a factor of safety of two allowed in the design of the culvert. Moreover, all pipes should have a minimum diameter of 0.60m to ensure that they can be cleaned manually. It is important to install energy dissipating structures and/or armor at the outlet where scour and erosion are likely to occur.

These structures are required where high exit velocity due to steep culvert installation, near proximity to channel banks, and drops at the end of the culvert.

Culverts are drainage structures that have the span length of less than or equal to 6-meters otherwise it is major drainage structure (ERA, 2002). However, ERA BMS considers those drainage structures that have span length of 4-meters and above as bridge. In this research, drainage structures are considered bridges that have span length of greater than 6-meters. Bridges are major roadway drainage structures, which are used in runoff drainage systems where stream span is large, for which special designs are made almost in every case greater than 6-meters (USNBIS, 1990).

The sizing of minor drainage structures is of considerable economic importance, as these structures can comprise a significant cost of total road construction costs. The selection of the appropriate design flood and good practice in the design of these structures determines the initial costs, the provision of the desired level of serviceability to traffic, and the safety of the road users.

With this respect, the most important parameters for the design of major and minor drainage structures are the design flood, hydraulics analysis and selection of construction materials.

2.5.2 Functions of Road Drainage Structures

The function of road drainage structures are to collect, transport, and dispose of surface/sub-surface water originating on or near the roadway right of way or flowing in streams crossing bordering the right of way. It prevents erosion of the back slope by runoff from the hill above. It intercepts water, not allowing it to enter side drain that may cause greater discharge inside drains. Pipe roughness, outlet conditions including tail water level do not influence flow capacity of culverts operating under inlet control. When the culvert barrel is not capable of conveying as much flow as the inlet opening will accept the outlet, control occurs (FHWA, 2001).

2.6 Road Drainage Failure

In principle, there are two different natural causes for the clogging or blockage of drainage structures, namely erosion in the stream caused by high water flows and landslides on the river banks resulting in soil and vegetation being transported with the stream. The manmade ditches are inherently different from natural channels and often much smaller, erosion mechanisms from natural streams/rivers are applicable to roadway ditches.

While natural streams tend to be larger than roadway ditches, the mechanisms of flow conveyance and erosion are similar (Richardson *et all*, 1990).

A large number of variables are involved in the erosion process. It may be difficult to determine which variable is the dominate erosion mechanism in the field. When a stream is in equilibrium with its environment, its slope will be an independent variable. That is, the stream has adjusted so that the flow is capable of transporting only the amount of sediment supplied at the upper end of the stream and by the tributaries (Richardson *et all*, 1990).When a change occurs upstream, the stream will respond by a change in slope to accommodate the change by increasing or decreasing the slope downstream. An increase/decrease in channel slope corresponds to erosion /deposition of soil comprising the ditch line.

The most common reason for the occurrence of damage is blockage of the ditch inlet with materials such as fine soil, gravel, stone and tree branches. Since that is normally the result of high water flows, there is commonly not enough time to remove the barrier before the water flows over the road and the whole embankment is destroyed (Magnusson et al., 2009). Hence, insufficient maintenance can often be the cause of road drainage failure (Hansson et al., 2010).

On Burayu town road at station 17+200, reason for the occurrence of damage is blockage of the ditch with materials such as fine soil, gravel, stone, plastics, tree branches silt, waste materials and garbage (Source: own survey, 2016).

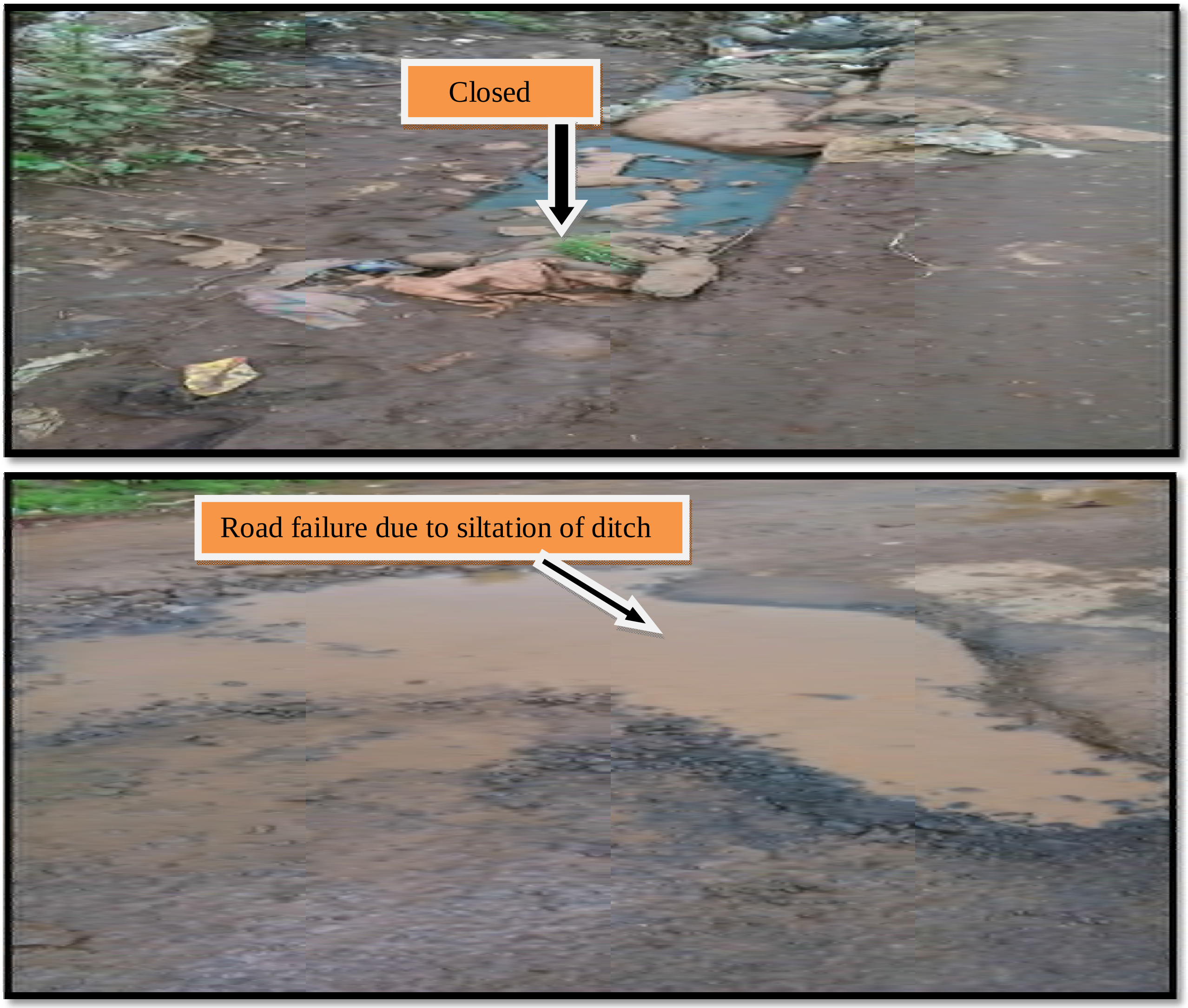


Figure 2.1: Typical Road Failure Source: own survey, 2016

2.6.1 Clogs and Silting of Ditch

Ditch gets clogged because debris accumulates at in inlet. They become silted when the grade is too flat and the flow is restricted. To solve the debris/silt problem, 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 (Source: own field survey, 2016).

Figure 2.2: Typical silt loaded ditch Source: Own field survey, June 2016

2.7. Requirements to Construct Drainage Structures

A complete drainage system design includes consideration of both major and minor drainage systems. The minor system, sometimes referred to as the "Convenience “system, consists of the components that historically considered as part of the "storm drainage system". These components include curbs, gutters, ditches, inlets, access holes, pipes and other conduits, open channels, detention basins, and water quality control facilities (Alderson, 2006).

According to HEC No. 22, the minor system normally designed to carry runoff from 10-year frequency storm events (FHWA, 2001). Minor culverts designed for a 10-year storm and checked for adequate performance with a 25-year interval storm event. The culvert skew shall not exceed 45^0 as measured from a line perpendicular to the roadway centerline.

Avoiding of improper alignment of drainage structures is significant in order to avoid hazardous problems of traffic and damage of foundations, abutments and piers of structures. Crosscurrents of stream and river flows are the causes of damage foundations, abutments and piers of drainage structures. Narrow sections and hard basement are important during construction of drainage structures in order to minimize the cost of construction with the exception of excavation cost.

2.8 The Effect of Neglected Minor Drainage Structures on Roadways

Drainage structures are important elements of the road structure to be accessible throughout the year without traffic interruption (Liang, 2001). The effect of neglected minor drainage structures on roadways makes the carriageway to be weak due to poundage of water that infiltrated into the carriageway. The infiltrated water oversaturated the carriageway-wearing course, and sub-grade. As a result, the carriageway could not carry traffic as intended. Due to this effect, catastrophic problem was created for the proper functioning of the road. When drainage structures are neglected to be constructed at appropriate locations:

- Surface water can pond at the edge of the road and weakens the road surface
- Silt can accumulate at the edge of the road i.e. the silt cannot be washed away through the drainage structure due to unconstructed drainage structure
- The visibility for road users is reduced, with increased risk of accidents on persons or animals

In order to serve a road properly for the road users, drainage structures should be constructed by considering where the location of the crossing in the watershed is required and how can water, sediment, and wood be transported at that location and how is the catchment configured.

3. MATERIALS AND METHODOLOGY

3.1. Description of the Study Area

The study area, Burayu town, is located in the Western Shoa part of Ethiopia, in Oromia National Regional State, Oromia Special Zone Surrounding Finfine at a distance of 15 km from Addis Ababa. Its astronomical location is 9º02’30’’North Latitude and 38º03’30’’- 38º41’30’’ East Longitude. (Source: Ministry of Urban Development and Construction).

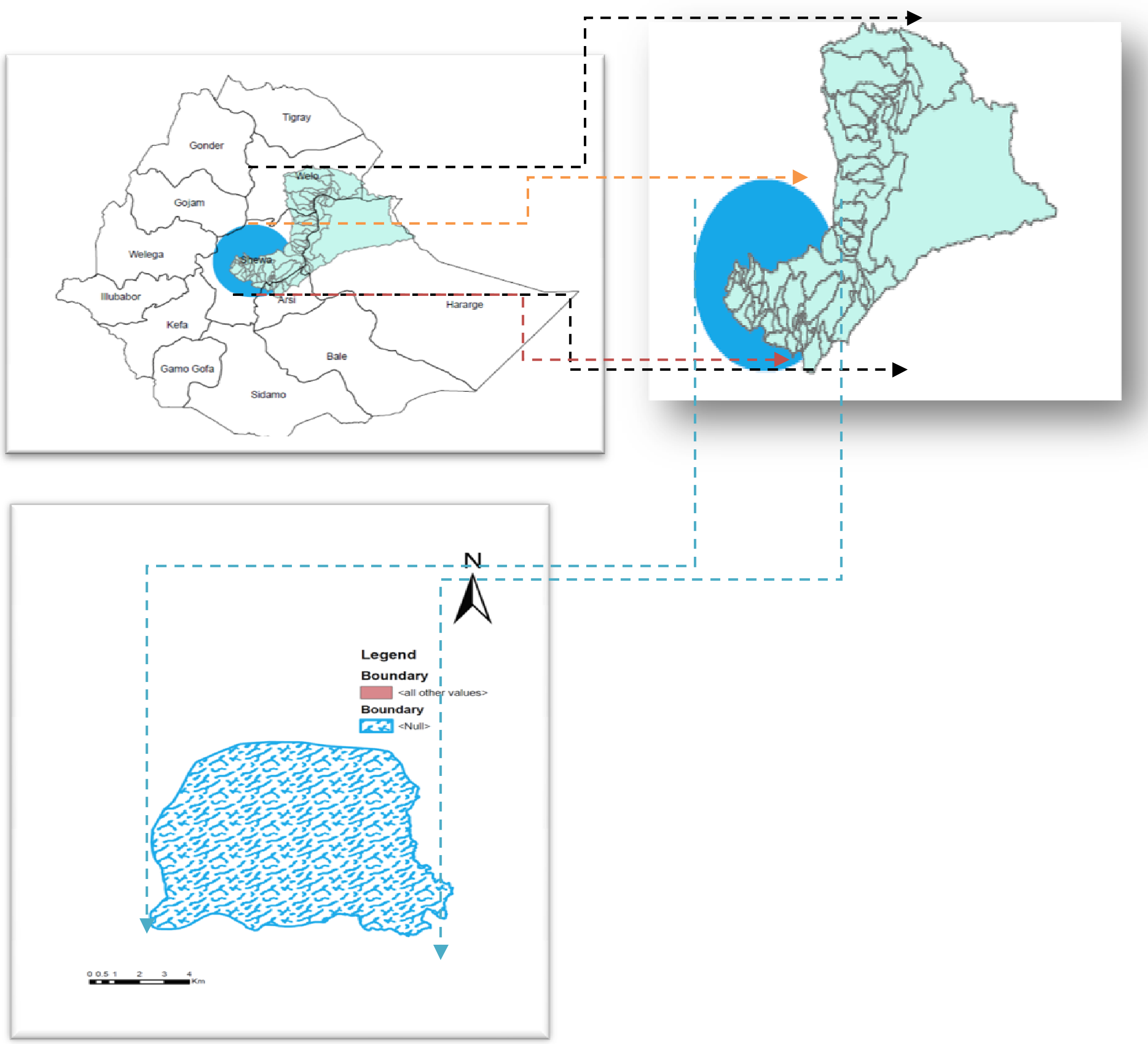


Figure 3.1: Location Map of the Study Area

3.1.1. Topography

The built-up area of Burayu town is 6361 ha excluding Guje kebele and it has linear shape. Its altitude is 2450–2560 meter above sea level. According to Ministry of Urban Development and Construction, the principal natural constraints for the physical expansion of the town is the steep topography of the land and flooding while manmade constrains are existence of informal settlement, high tension lines, rugged and steep slopes in Guje kebele. The major or the dominant soil is red soil. The overall catchment area largely dominated by Lithosols (D)major soil and bare land to residential area land forms.

3.1.2. Climate

It has mean annual temperature of 14^{oC} and the prevailing wind direction is from East to West. The mean annual rainfall in the study area is 1,188 mm.

3.1.3. Soil Characteristics

The development of soils depends primarily on geologic and climatic conditions. In Ethiopia, 17 major soil units have been identified (EMA, 1988). The FAO Soil Map of Ethiopia classifies 19 soil units, which do not all coincide spatially with the EMA soil map. For this research, since the FAO classification system is recent, the FAO classification (FAO, 1998) is selected. The type of soil at the study area is Lithosols of hydrologic soil group D.

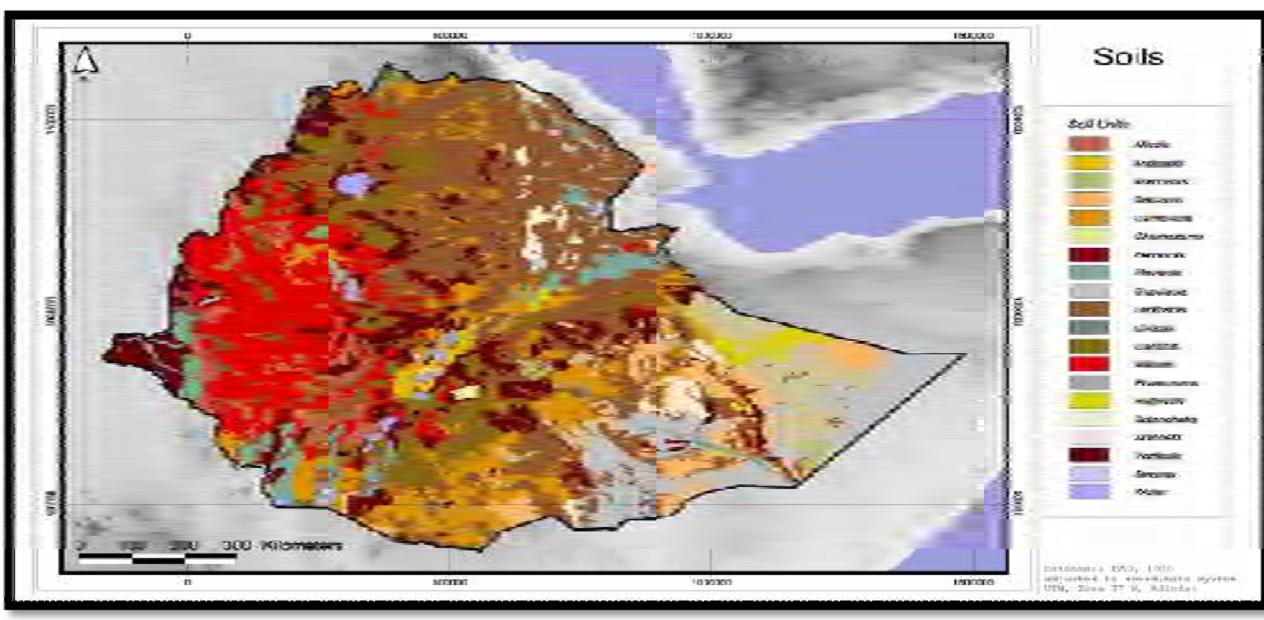


Figure 3.2: Soils in Ethiopia (FAO, 1998) Source (FAO, 1998)

3.1.4. Demography

According to the National Population and Housing Census carried out in 2007, the population of the town was 48,876. Out of these, 24,003 (49.11%) were males and 24,873 (50.89%) were females. Regarding age distribution, 15,857 (32.4%) were within the age group of 0-15 years, 31,728 (64.9%) were within the age group of 16-60 years and 1,291 (2.6%) were 60 years and above. The average household size in the town is calculated to be 4.2.

3.1.5. Socio-Economy Information

Urban Economy and Role of the Town

The town serves as administration and industrial center. There are 105 manufacturing, 12 wholesale, 631 retail trade, 347 service trade, 4 fuel stations and 3 garages in the town. There are also three banks and three micro financial institutions giving financial services in the town. Economic rate activity for both sex is 59.7% while unemployment rate is 16%. The average annual revenue of the municipality with in the 2005-2010 periods was 43,902,029.14 Birr and the major sources of revenue were taxes and service charges. The major investment opportunities in the town are agro-industry processing and construction.

Burayu town has economic linkages with the surrounding areas, towns, and Addis Ababa. The town gets grain products, livestock supply, natural resources (fuel wood, charcoal) and surplus labor from surrounding areas. The town gets agricultural inputs from Adama and Addis Ababa, manufacturing and commercial products and some construction materials from Addis Ababa and the town itself.

3.1.6. Social services

Education

There are 58 private kindergartens; six government and 32 private primary schools; five government and 32 junior secondary schools; two high schools including preparatory; one TVET, one private and one government college in Burayu town.

Health

Regarding health services in the town, there are two government health centers, eight private clinics and three government health posts.

Municipal services

Burayu town has meeting hall, public library, two abattoirs giving slaughtering service for Christians and Muslims and one daily market. There are eight forest areas (Malka Atetee, Haro Gafarsa, Galma Anfo and Menagesha forest) covering a total area of 29 ha forest and street trees are planted along 2km of the road. Recreation and related function including three parks, one stadium and dam called Gefersa in Burayu. There are 12 churches for Orthodox Christians, 15 churches for Protestants and 9 mosques for Muslims in the town.

3.1.7. Infrastructure, Housing and Service Information

Infrastructure

Regarding infrastructure, the town has asphalt and gravel roads connecting it to different woredas, zones, regions and Addis Ababa. The distribution of roads as per the type of construction materials shows that, there are 250km gravel, 35km asphalt, 250km earth pressed and 26.79 km cobble stone roads. In addition to these, the town gets 24 hours electric supply from the national grid, mobile, fixed telephone lines, and internet services. The main water supply source is potable underground water distributed with pipe network system and 18 public water points. The town has open ditch system to discharge storm water (Source: Ministry of Urban Development and Construction).

Housing

There are 27,726 private and government owned houses built of mud and wood while wood, stone, sand, and Hollow Concrete Block are widely available construction materials in the town.

Transport

There is daily inter-urban transport service to and from five towns. 11 Bajaj and 84 minibus taxis and 85 horse-drown carts are the main intra-urban transport service.

(Source: Ministry of Urban Development and Construction)

3.2. Methodology

3.2.1. Data Collection

Before using and processing of any research, the primary task is collecting/getting relevant information or data of the study area. This section identifies and discusses the types and source of data used for the study.

Material Used

The materials used for this study were GPS device, measuring tape, digital camera, Arc view GIS tool to obtain the physical parameters of the study area, DEM data and Shape file were used as an input data for ARC-GIS software for catchment delineation and estimation of catchment characteristics. The Microsoft EXCEL was used to analyze data.

3.2.2. Watershed Delineation

After data collection the boundary of the study area was delineated. The watershed was delineated from the Digital Elevation Model (DEM) and the Shape file of the town as shown in figure 3.3.

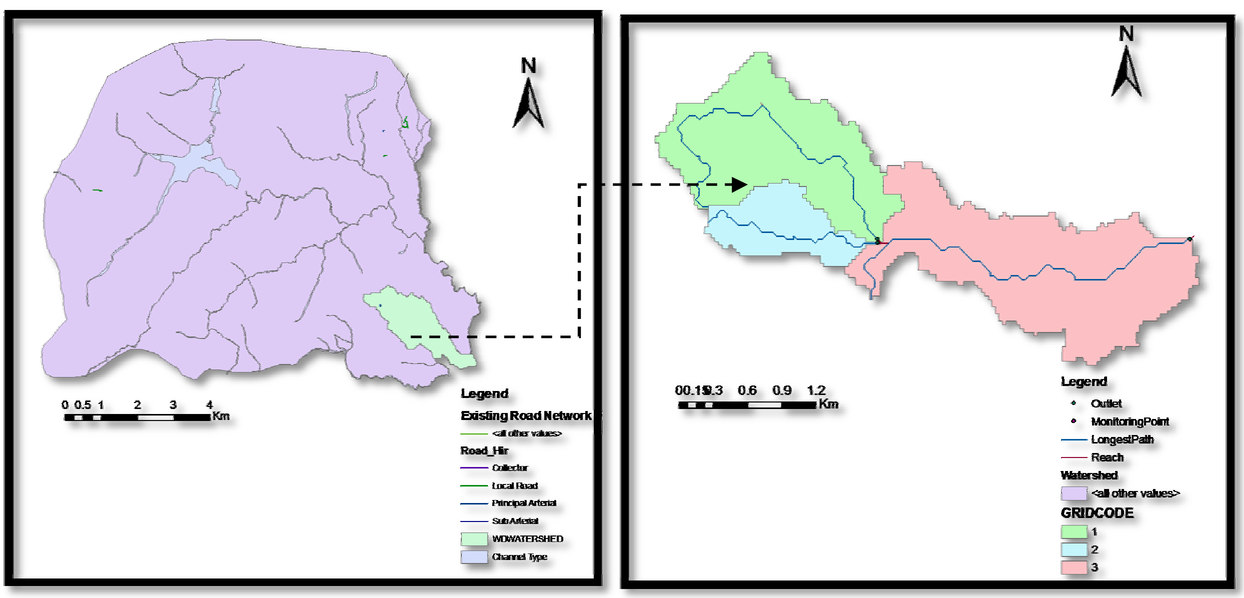


Figure 3.3: Catchment Area for Drainage Structure at Station 16+200

3.2.3. Hydrological Information

The hydrological information required for estimating the design floods were obtained from ERA drainage design manual IDF curves and topographic map of the study area. ERA classified rainfall regions in to four major rainfall regions and eight sub-rainfall regions in the country and developed IDF curves. ERA developed four IDF curves for rainfall regions in the country for A1&A4, A2&A3, B, C & D and Bahir Dar & Lake Tana rainfall regions.

The maximum peak flood computed using the return period recommend in ERA Drainage Design Manual 2011 considering the road standard and the design life span of the structure,10 years for design and 25 year check (review) return period were adopted. See Appendix A of table 3 design storm frequency (yrs) by Geometric Design Criteria - (ERA DDM, 2011).

The ERA regionalized drainage map of Ethiopia was used to derive the rainfall intensities that were utilized in the Rational and SCS method. Accordingly the project area hydrologic region as per the ERA Drainage Manual 2011 is rainfall region A2 and the parameters in the hydrologic analysis are obtained from the ERA Manual.

To compare the generated IDF curve rainfall data with rainfall data of the study area, local rainfall data are required. But due to shortage of rainfall data at the project location I have used the parameters from the ERA Drainage Design Manual 2011 for rainfall region A2.

But there is local rainfall data near the study area in order to compare with the rainfall which is generated from ERA regionalized IDF curve. After the rainfall data were compared, the already developed regionalized IDF curve by ERA is used to determine rainfall intensity. The study area lies on sub-region A2 and the IDF curve constructed for A2 rainfall region was used. Therefore, the rainfall intensity from the IDF curve for the corresponding return period was used in the study.

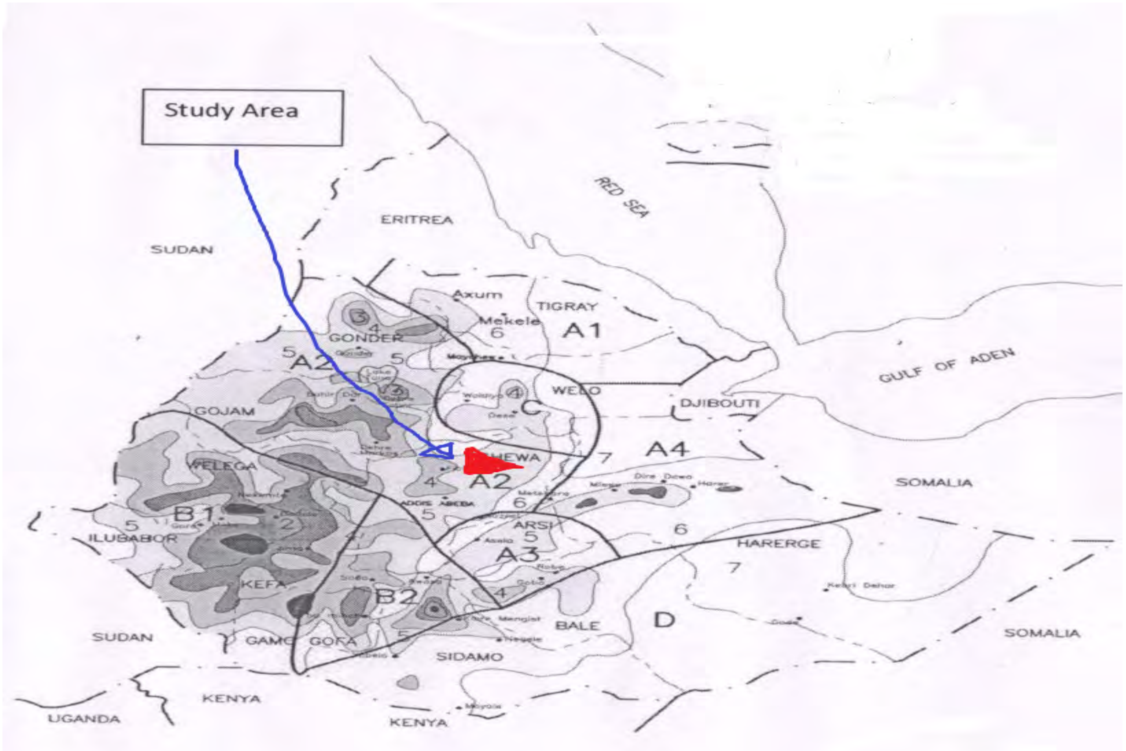


Figure 3.4: Rainfall regions of Ethiopia (ERA DDM, 2002)

3.2.4. Meteorological data

At present there are several meteorological stations, which were installed by National Meteorological Agency (NMA) of Ethiopia. In spite of the fact that sufficient numbers of meteorological stations have been established throughout the country, but at the study area metrological station is not installed in order to get the meteorology data. The quality of the studies is dependent on the quality of required elements and quantity or long term record of available data. The most commonly observed problem was related to insufficient and incomplete basic data. In this study, there was a problem of absence of gauging station at the study area for meteorology data. Meteorological data of this study was mainly based on rainfall data obtained/generated from the ERA developed IDF curves for the study area. In order to calibrated and validated the rainfall data at the study area, the near gauging stations/records data obtained from National Meteorological Service Agency of Ethiopia (NMAE) were used. The summary of the selected stations is presented in table 3.1 blow.

Table 3.1 Summary of selected rainfall stations nearby the study area

S/No	Stations	Latitude	Longitude	Altitude	Years of data used	(%) Missed data	Sub-Basin
1	AddisAlem	9.0420	38.3830	2372	1980-2014	14.7	Awash
2	Holota	9.00	38.50	2400	1980-2014	17.2	Awash

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, hence interpolation filling method was used to compute the missed data for analyzing rainfall data of Holota and AddisAlem stations. Then performing interpolation between them and calculated to get the estimated records of the missing data for the corresponding time period.

Homogeneity test

Homogeneity analysis was used to separate a change in the statistical properties of the time series data. The causes can be either natural or man-made. These include alterations to land use and relocation of the observation gauging station. Therefore in order to select the representative meteorological station for the analysis of areal rainfall estimation, checking homogeneity of group stations is essential, the homogeneity of the selected gauging stations daily rainfall records were carried out by non-dimensional equation:

$$Pi = \frac{\overline{Pi}}{\overline{P}} \qquad \text{.....1}$$

Where: Pi = Non dimensional Value of precipitation for the month i

$\overline{Pi}$ = Over years averaged monthly precipitation for the station i

$\overline{P}$ = Over year's average yearly precipitation of the station

Consistency test

Consistency of time series data analyzed based on theory that a plot of two cumulative quantities that are measured for the same time period should be straight line and their proportionality unchanged, which is represented by slop. Therefore, inconsistency of the record was done by the double-mass curve technique. This technique is based on the principle that when each recorded data comes from the parent population, they are consistent. The double mass curve technique was used to adjust precipitation records to take account of non-representative factors such as change in location or exposure of rain gauge. The accumulated totals of the gauge in question are compared with the corresponding totals for a representative group of nearby gauge.

If significant change in the system of the curve is observed, it should be corrected by:

$$Px' = Px * \frac{M'}{M} \quad \text{……………………………………………………………………………..2}$$

Where: - Px' = Corrected precipitation at station x

P_x = Original recorded precipitation at station x

M' = Corrected slope of the double mass curve

M = Original slope of the double mass curve

3.3. Data evaluation

The performance evaluations of these metrological data (rainfall) were tested with time series performances against generated rainfall data from IDF curves of ERA of the study area. The time series based metrics is the correlation coefficient (R^2).

$$R^2 = \frac{\sum_{i=1}^{n}\left(p^i obs - \overline{p}\, i_{obs}\right)^{\wedge}2 \; - \sum_{i=1}^{n}\left(p^i gen - \overline{p}\, i_{gen}\right)^{\wedge}2}{\sum_{i=1}^{n}\left(p^i obs - \overline{p}\, i_{obs}\right)^{\wedge}2} \quad \text{…...3}$$

Where: P^i obs and P^i gen are the i^{th} observed and generated variables, and n is the total number of observations.

3.3.1. Calibration

Identifying and addressing large discrepancies between observed and generated behavior is critical in the calibration effort. Calibration is an iterative exercise used to establish the most suitable parameter in the studies. It is very important because reliable values for some parameters can only be found by calibration (Reuben, 2007). It involves the identification of the most important parameters and changing the parameter set. Parameters changed during calibration were classified into physical and process parameters. Physical parameters represent physically measurable properties of the watershed; while the process parameters are those not directly measurable. Calibration can be manual, automatic and a combination of the two methods (Tigist, 2009).Manual calibration use trial and error techniques in parameter adjustment through a number of simulation runs. It is subjective to the modeler's assessment and can be time consuming.

Computer based automatic calibration involves the use of a numerical algorithm which finds the extreme of a given numerical objective function. Thus, in this study data performance is assessed statistically by comparing the perception/rainfall data of near gage station of the study area and generated rainfall data values from IDF curves of ERA which were developed for the study area. The statistical measure commonly used is the coefficient of determination (R^2).

3.3.2. Validation

Validation is the process of representing that a given site specific data is capable of making accurate predictions. This was done by applying the calibrated data using a different data set out of the range of calibration without changing the parameter values. The data is said to be validated if its accuracy and predictive capability in the validation period have been proven to lie within acceptable limits(Reuben, 2007).Observed/gauged precipitation and generated rainfall values were again compared as in the previous calibration procedure. If the resultant fit is acceptable then the data prediction as valid.

3.4. Hydrological Equations for Determining Peak Flood

Many comprehensive hydrologic equations have been developed in the past decades due to advances in hydrologic sciences and Geographical information system (GIS). Among them the SCS method of peak discharge estimation (Runoff estimation), developed by the U.S soil conservation service (1972), and Rational method has been used for this study. This is because it is applicable for areas which do not have sufficient rainfall and stream flow records. In addition ERA drainage design manual recommended using SCS and Rational method for the peak discharge estimation and most the consultants in the country uses ERA drainage design manuals for the hydrology and hydraulic analysis of drainage structures.

Generally, the peak flood determination methods can be grouped into two broad categories viz., deterministic and statistical.

- In deterministic methods, the physical aspects of the rain fall-runoff process either conceptually or empirically, where the relationship between rainfall and runoff is quantified based on measured data and experience. Deterministic methods often require a large amount of judgments and experience to be used effectively.
- Statistical methods apply the techniques and procedures of modern statistical analysis to actual or synthetic data and fit the required design parameters directly. Statistical methods do not require much objective judgments and experience to apply.

In most cases, rational and soil conservation service, (SCS) methods of flood estimation are applied for minor drainage structures due to unavailability of gauged data at the study area. Based on the aforementioned concepts, Rational and SCS mathematical equations are used for this thesis according to the area of the catchment.

3.4.1. Rational Method

Rational formula is particularly useful if local stream flow data do not exist (Keller and Sherar, 2003).It was the most commonly used method for the design of side ditch when a peak flow rate is desired. For hydraulic designs on very small watersheds, a complete hydrograph of runoff is not always required (David, 2007). The maximum, or peak, of the hydrograph is sufficient for design of the structure in question. Among a number of methods for estimating a design discharge, the rational formula is an empirical formula relating runoff to rainfall intensity.

According to ERA drainage design manual 2002 and AASHTO 1990, the rational formula is most accurate for estimating the design peak runoff for small catchment areas of up to 50 hectares ($0.5km^2$). Runoff coefficient, time of concentration, runoff generating area and rainfall intensity are the major design parameters for momentary peak discharge estimation under this method.

The rational formula estimates the peak rate of runoff at any location in a catchment area as a function of the catchment area, runoff coefficient, and means rainfall intensity for duration equal to the time of concentration (the time required for water to flow from the most remote point of the basin to the location being analyzed).

Due to this fact, for this research, the rational method was not used to determine the rate of runoff for large drainage areas. Therefore, the maximum value of the catchment area, 50 hectares, is considered.

The rational formula is expressed as:

$Q = 0.00278CIA$ …..equation (3.1)

Where, Q= Peak flow in cubic meter per second (m^3/sec)

C= Dimensionless weighted runoff coefficient

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

A= Drainage area in hectares (ha) (source: VDOT, 1995)

The basic assumptions in rational method to determine peak flood are:

i. The peak rate of runoff at any point is a direct function of the average rainfall intensity for the time of concentration to that point.
ii. The recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity.
iii. The time of concentration is the time required for the runoff established and flow from the most distant point of the drainage area to the point of discharge.

The main reason that is required to limit the use of rational method for small watersheds pertains to the assumption that rainfall is constant throughout the entire watershed. Severe storms, say a 100-year return period, generally cover a very small area. Applying the high intensity corresponding to a 100-year storm to the entire watershed could produce greatly exaggerated flows, as only a fraction of the area maybe experiencing such intensity at any given time.

The variability of the runoff coefficient also favors the application of the rational method to small and developed watersheds. Although the coefficient is assumed to remain constant, it actually changes during a storm event. The greatest fluctuations take place on unpaved surfaces as in rural settings. Moreover, runoff coefficient values are much more difficult to determine and may not be as accurate for surfaces that are not smooth, uniform and impervious. Generally, the rational method provides the most reliable results when applied to small, developed watersheds and particularly to roadway drainage design. According to the aforementioned facts, I considered the runoff coefficient constant throughout the catchment area that is encompassed the study area.

Steps to Peak Flood Estimation using the rational method:

The following procedure outlines the rational method for estimating peak discharge:

- ✓ Determine the watershed area;
- ✓ Determine the time of concentration;
- ✓ Assure consistency with the assumptions and limitations for application of the rational method;
- ✓ Select the appropriate runoff coefficient; and
- ✓ Compute the peak discharges for the watershed for the desired frequency using equation (Q=0.00278CIA)

3.4.1.1 Runoff Coefficient

The runoff coefficient, C, in equation (4.1) is a dimension less value representing characteristics of the water contributing area that affect how much of the rainfall will become runoff. It is a function of the ground cover and a host of other hydrologic abstractions.

$$\boxed{C_{weighted} = \frac{\sum (A_i * C_i)}{A_T}}$$ …………………………………..…………………. equation (3.2)

Where C_i - runoff coefficient for each drainage area

A_i - area under each type of drainage area

A_T -total drainage area considered

3.4.1.2 Rainfall Intensity

The rainfall intensity (I) is the average rainfall rate in mm/hr for duration equal to the time of concentration for a selected return period. Once a particular return period has been selected for design and a time of concentration calculated for the catchment area, the rainfall intensity can be determined from Rainfall-Intensity-Duration curves. Rainfall-Intensity-Duration curves for use in Ethiopia. Rainfall intensity is a function of geographic location, design exceedence frequency (or return interval), and storm duration.

It is true that the greater the return interval (hence, the lower the exceedence frequency), the greater the precipitation intensity for a given storm duration. Quantification of rainfall is generally carried out using Return Period maps and intensity-duration-frequency (IDF) curves (Chow et al, 1988).The IDF relationship is a mathematical relationship between the rainfall intensity, the duration, and the return period (the annual frequency of exceedance). For this research, ERA regionalized IDF curves would be used to quantify rainfall. The study area is found in the rainfall region of Ethiopia, in rainfall sub-region A2 as shown on Appendix A at figure 2.

3.4.1.3 Time of Concentration

The time of concentration was defined as the period required for water to travel from the most hydraulically distant point of the water contributing area to the drainage system under consideration. Use of the Rational Method requires the time of concentration (t_c) for each design point within the catchment area. The duration of rainfall is then set equal to the time of concentration and is used to estimate the design average rainfall intensity (I). Peak discharge generated from pavement surface and small cross streams has been determined through rational model application.

To determine time of concentration for over land flow there are many formulae. Among these the Kerby and Kirpich formulae are presented and for defined flow (Channel flow), U.S. SCS formula is presented.

$T_c = 0.604(RL/S^{0.5})^{0.467}$ ………………………………………………………. (3.3) Kerby Formula

Where: T_c - Time of concentration in hours

L - Length of overland flow in kilometers

S - Slope in m/m (Source: ERA, DDM -2013)

R - Roughness coefficient

$$T_c = 0.0013 * m * \left(\frac{L^{0.77}}{S^{0.385}} \right)$$ …………………………………………… (3.4) Kirpich Formula

Where, T_c-Time of concentration in hours

L - Length of overland flow in kilometers

S - Slope in m/m

M- Earth type coefficient

(m=one for bare earth, m=two for grass and m=0.4 for asphalt)

(Source: ERA, DDM -2013)

3.4.1.4 Catchment Area

In general, the catchment area can be determined from topographic maps DEM data and field surveys. However, for large catchment areas, it is necessary to divide the area into sub-catchment areas to account for major land use changes.

3.4.2. Soil Conservation Services (SCS) method

This method is developed by the U. S. Soil Conservation Service for calculating rates of runoff and requires the same basic data as the Rational Method: catchment area, a runoff factor, time of concentration, and rainfall. The SCS approach, however, is more sophisticated in that it considers also the time distribution of the rainfall, the initial rainfall losses to interception and depression storage, and an infiltration rate that decreases during the course of a storm. With the SCS method, the direct runoff can be calculated for any storm, either real or fabricated, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. The SCS approach, however, also considers the time distribution of the rainfall, the initial rainfall losses to interception and depression storage and an infiltration rate that decreases during the course of a storm.

With the SCS method, the direct runoff can be calculated for any storm, either real or synthetic, by subtracting infiltration and other losses from the rainfall to obtain the precipitation excess. It relates rainfall intensities to catchment parameters and uses a standard unit hydrograph to calculate the time of distribution of the runoff.

It is therefore, potentially more accurate than the rational method and is applicable when the catchment area is larger than 50 hectares (ERA, 2011).

Characteristics

Characteristics of the dimensionless hydrograph vary with the size, shape, and slope of the tributary drainage area. The most significant characteristics affecting the dimensionless hydrograph shape are the basin lag and the peak discharge for a given rainfall. Basin lag is the time from the center of mass of rainfall excess to the hydrograph peak. Steep slopes, compact shape, and an efficient drainage network tend to make lag time short and peaks high; flat slopes, elongated shape, and an inefficient drainage network tend to make lag time long and peaks low.

Catchment Area

A catchment area was determined from topographic maps DEM data and field surveys. For large catchment areas, it might be necessary to divide the area into sub-catchment areas to account for major land use changes, obtain analysis results at different points within the catchment area, or locate storm water drainage structures and assess their effects on the flood flows. A field inspection of existing or proposed drainage systems shall be made to determine if the natural drainage divides have been altered. These alterations could make significant changes in the size and slope of the sub catchment areas.

3.4.2.1 Curve Numbers

In hydrograph applications, runoff is often referred to as rainfall excess or effective rainfall - all defined as the amount by which rainfall exceeds the capability of the land to infiltrate or otherwise retain the rainwater. The principal physical watershed characteristics affecting the relationship between rainfall and runoff are land use, land treatment, soil types, and land slope. Land use is the watershed cover, and it includes both agricultural and nonagricultural uses. Items such as type of vegetation, water surfaces, roads, roofs, etc. are all part of the land use.

Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops.

The SCS uses a combination of soil conditions and land use (ground cover) to assign a runoff factor to an area. These runoff factors, called runoff curve numbers (CN), indicate the runoff potential of an area when the soil is not frozen.

The higher the CN, the higher is the runoff potential. Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration.

The SCS has divided soils into four hydrologic soil groups based on infiltration rates (Groups A, B, C and D). Soil type A has the highest infiltration and soil type D has the least amount of infiltration. Consideration should be given to the effects of urbanization on the natural hydrologic soil group. If heavy equipment can be expected to compact the soil during construction or if grading will mix the surface and subsurface soils, appropriate changes should be made in the soil group selected.

Also, runoff curve numbers vary with the antecedent soil moisture conditions, defined as the amount of rainfall occurring in a selected period preceding a given storm. In general, the greater the antecedent rainfall, the more direct runoff there is from a given storm (Source: ERA, DDM - 2013).

3.4.2.2 Rainfall-Runoff Equation

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. Data for land treatment measures, such as contouring and terracing, from experimental watersheds were included. The equation was developed mainly for small watersheds for which only daily rainfall and watershed data are ordinarily available. It was developed from recorded storm data that included the total amount of rainfall in a calendar day but not its distribution with respect to time.

The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall. The SCS method is based on a 24-hour storm event which has a Type II-time distribution. The Type II storm distribution is a ’typical ‟ time distribution which the SCS has prepared from rainfall records”. It is applicable for interior rather than the coastal regions and should be appropriate for Ethiopia.

The Type II rainfall distribution will usually give a higher runoff than a Type I distribution. To use this distribution, it is necessary for the user to obtain 1) the 24-hour rainfall value for the frequency of the design storm desired, and then 2) multiply this value by 24 to obtain the total 24-hour storm volume in millimeters.

The basic rainfall-runoff relationship in the SCS methodology is equated as**:**

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$ ……………………………………………………………………equation (3.5)

For P>0.2S

Q =0 for P≤0.2S (Source: VDOT, 1995)

Where:

Q = accumulated direct runoff, mm

P = accumulated rainfall (i.e., the potential maximum runoff), mm

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

S = potential maximum retention, mm

S is a site index defined as the maximum possible difference between P and Q as P⇒∞,P-I_a is called "effective rainfall". It is related to the soil and cover conditions of the catchment area through the curve numbers. The curve number is a transformation of potential maximum retention (NRCS, 2004).

$$S = \frac{25400}{CN} - 254$$

Where:

S= potential maximum retention, mm

CN = SCS runoff curve number

The relationship between I_a and S was found to be; I_a= 0.2S

The initial abstraction consists of interception, infiltration and surface storage, all of which occur before runoff begins.

3.4.2.3 Time of Concentration

Travel time (Tt) is the time it takes water to travel from one location to another in a catchment area. Tt is a component of time of concentration (Tc), which is the time for runoff to travel from the hydraulically most distant point of the catchment area to a point of interest within the catchment area.

Tc is computed by summing all the travel times for consecutive components of the drainage conveyance system. It is a combination of three values in SCS method of determining peak flow rate.

A. Sheet flow,

B. Shallow concentrated flow, and

C. Open channel flow

The type that occurs is a function of the conveyance system and is determined by field inspection. It is often a combination of these flows so that the total travel time is the sum of the time taken for the water to pass through all of the segments of the catchment.

Travel Time

Water moves through a catchment area as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

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

T_t = L/ (3600V) ……………………………………………………………………………….. (3.6)

Where :(Source: VDOT, 1995)

T_t= travel time, hr

L = flow length, m

V = average velocity, m/s

3600 = conversion factor from seconds to hours

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.

$$T_c = \left(\frac{0.87L^2}{100Sav}\right)^{0.385} \quad \text{……………………………………………………………………… (3.7)}$$

L = length of watercourse (km) (Source: ERA, DDM- 2002)

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

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

$T_c = T_{t1} + T_{t2} + \ldots T_{tm}$ ……………………………….…………………………….………. (3.8)

Where:

T_c= time of concentration, hr

m= number of flow segments (Source: ERA, DDM - 2002)

1. **Sheet Flow**

In sheet flow, travel time is determined by Manning's kinematic solution. The Manning's kinematic solution is expressed as:

For sheet flow of less than 100 meters, use Manning's kinematic solution (Overton and Meadows 1976) to compute T_t:

$T_t = [0.091(nL)^{0.8} / (P2)^{0.5} S^{0.4}]$ ………………………………………………………….. (3.9)

Where, T_t=travel time, hr

n= Manning's roughness coefficient (table 3)

L=flow length, m

P2 = 2-year, 24-hour rainfall, mm

S = Slope of hydraulic grade line (land slope), m/m

According to ERA DDM 2002, the Manning's kinematic solution is based on the following criteria:

i. Shallow steady uniform flow

ii. Constant intensity of rainfall excess

iii. Rainfall duration of 24-hours

iv. Minor effect of infiltration on travel time

2. **Shallow Concentrated Flow**

After a maximum of 100 meters, sheet flow usually becomes shallow concentrated flow (ERA DDM, 2002). The average velocity for this flow can be determined from equations (3.10) and (3.11) in which average velocity is a function of watercourse slope and type of channel.

Unpaved $V = 4.9178 (S)^{0.5}$…………………………………………………………….…….... (3.10)

Paved $V = 6.1961 (S)^{0.5}$………………..……………...……………………………….……..... (3.11)

Where:

V = average velocity, m/s

S = slope of hydraulic grade line (watercourse slope), m/m

According to ERA, DDM-2002 these two formulae are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and R (hydraulic radius, meter). According to the ERA, DDM 2002 for unpaved areas, the value of n is 0.05 and R is 0.12; for paved areas, the value of n is 0.025 and R is 0.06. After determining average velocity, equation (4.6) is used to estimate travel time for the shallow concentrated flow segment.

3. **Open Channels**

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on Ethiopian Mapping Authority (EMA) topographic maps (1:50,000). Average flow velocity is usually determined for bank-full elevation. Manning's equation or water surface profile information can be used to estimate average flow velocity. When the channel section and roughness coefficient (Manning's n) are available, then the velocity can be computed using the Manning Equation. For this thesis, topographic map of the study area was used that was produced in 1:50,000 scale.

$V = (R^{2/3}S^{1/2})/n$ …………………………………………………..………………….. (3.12)

After average velocity is calculated, T_t is calculated by using equation (4.6)

$Tc = T_{t1} + T_{t2} + T_{t3}$…………………………………………..…………………………. (3.13)

Where, T_{t1}=travel time for sheet flow

T_{t2}=travel time for shallow concentrated flow

T_{t3} =travel time for open channel flow (Source: ERA, DDM- 2002)

Using the calculated time of concentration, unit peak discharge is obtained from Appendix A on Table (1). After unit peak discharge is obtained, design peak discharges determined using the formula:

Design Peak Discharge, Qp =Qu*Q*A ……………………………….………. equation (3.14)

Where:

Qp= Design Peak Discharge, m^3/sec

Qu=Unit Peak Discharge, m^3/sec/100ha/mm

Q= Direct Runoff, mm

A= Area of the catchment, ha

The unit peak discharge is obtained from the following equation, which requires the time of concentration (tc) in hours and the initial abstraction rainfall (Ia/p) ration as input:

$$Qu= \propto * 10^{C0 + C1\ logtc + C2\ (log\ tc)\ ^\wedge 2}$$

Where C0, C1 and C2 = regression coefficients given in appendix B in table 4 for various Ia/p ratios: $\propto$ = unit conversion factor equal to 0.000431 in SI unit.

Ia/p Parameter

Ia/p is a parameter that is necessary to estimate peak discharge rates. Ia denotes the initial abstraction and p is the 24 hour rainfall depth for a selected return period. The 24 rainfall depth is taken from ERA drainage design manual for rainfall region A2. For a given 24 hour rainfall distribution Ia/P represents the fraction of rainfall that must occur before runoff begins.

3.4.2.4 Runoff and Curve Number

The physical catchment area characteristics affecting the relationship between rainfall and runoff (i.e. the CN values) are land use, land treatment, soil types, and land slope. Land use is the catchment area cover and it includes agricultural characteristics, type of vegetation, water surfaces, roads and roofs.

Land treatment applies mainly to agricultural land use, and it includes mechanical practices such as contouring or terracing and management practices such as rotation of crops. The SCS method uses a combination of soil conditions and land-use to assign a runoff factor (curve number) to an area. These runoff factors or curve numbers (CN), indicate the runoff potential of an area. The higher the CN, the higher is the runoff potential.

The potential maximum retention S has been converted to the Curve Number CN in order to make the operations of interpolating, averaging, and weighting more nearly linear.

This relationship is

$$S = \frac{25400}{CN} - 254$$

Zero potential maximum retention (S=0 or CN=100) represents an impermeable watershed; CN = 0 represents a mathematical upper bound to the potential maximum retention, which is an infinitely abstracting watershed. As the potential maximum retention (S) can theoretically vary between zero and infinity, it shows that the Curve Number, CN, can range from one hundred to zero. For highly permeable, flat-lying soils, S will go to infinity and CN will be zero; all rainfall will infiltrate and there will be no runoff. (Source: VDOT, 1998)

3.4.2.5 Hydrological Soil Groups

Soil properties influence the relationship between rainfall and runoff by affecting the rate of infiltration. The SCS has divided soils into four hydrologic soil groups (HSGs) based on infiltration rates Groups A, B, C, and D.

Group A: Sand, loamy sand or sandy loam. Soils having a low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels.

Group B: Silt loam, or loam. Soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C: Sandy clay loam. Soils having a moderately high runoff potential due to slow infiltration rates. These soils primarily consist of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D: Clay loam, silty clay loam, sandy clay, silty clay or clay. Soils having a high runoff potential due to very slow infiltration rates. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material (ERA, 2002).

3.4.3. Hydrology and Hydraulics

The designs of highway drainage structures do require adequate hydrological analysis which assesses the flood potential of the watershed/road surface area at the road crossing point on a highway corridor. Hydrological analysis is the most important step prior to the hydraulic design of any drainage structure. Assessment of hydrological condition involves study of landscape characteristics of the watershed area including topographic conditions, soil characteristics, land use and rainfall. Hydrological parameters calculation should be completed by making it familiar with the local conditions and stream flows. The calculation of hydrology was carried out using equations such as rational and SCS method and Hydraulic parameters calculations are carried out using Manning’s equation.

3.5. Hydraulic Analysis

The chief aim of this task was to determine the opening sizes of the drainage structures from the rate of flood runoff (discharge) and the volume of runoff that will pass through the drainage structures.

This method deploys the hydraulic characteristics of the stream influencing the maximum discharge, such as velocity of flow, slope of the stream, cross sectional area of the stream and shape and roughness of the stream. This method will be used for major streams to compute the design flood levels at crossing sites after the design discharges have been estimated by the hydrological methods of either the Rational or SCS Methods .Cross-Sections of the crossing sites are being determined by the survey.

3.5.1 Manning’s Formula of Hydraulic Analysis

This method deploys the hydraulic characteristics of the stream influencing the maximum discharge, such as velocity of flow, slope of the stream, cross sectional area of the stream and shape and roughness of the stream. This method is used for the design flood levels at crossing sites after the design discharges have been estimated by the hydrological methods of the Rational and SCS method. Cross-Sections of the crossing sites have been determined by the field survey.

Channel discharge quantity (Q) is the product of the mean channel velocity (V) and the area (A) of the channel. To determine the discharge (Q) in natural drainages, canals, and non-pressure pipes, the following formula is used:

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \quad \text{..(3.15)}$$

Where:

Q= discharge, in cubic meters per second [m^3/s]

S = channel slope [m]

R = hydraulic radius [m] = A/P

A= cross sectional area, in square meters [m^2]

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

n = Manning roughness coefficient

Manning's Formula can be used to compute the average flow velocity (V) in any channel or natural stream with uniform flow. Manning's formula can be readily solved for a given channel when the known or assumed depth of flow is used. However, to determine the depth that a given discharge will produce in a channel, a trial and error solution is required.

Discharge will be determined for a known opening size of the drainage structure and bottom slope and/or the size of the drainage structure will be 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 would also be 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. It will be applicable for constant flow rate of water through a channel with constant slope, size & shape, and roughness.

To calculate average flow velocity:

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

Where:

V = average flow velocity [m/s])

n = Manning roughness coefficient

S = channel slope [m]

R = hydraulic radius [m] = A/P

Where A and P is:

A = cross-sectional area in [m^2]

P = wetted perimeter in [m]

Roughness Coefficient (n) varies considerably, depending on the characteristics of a channel or the smoothness of a canal, pipe, etc. Roughness coefficients represent the resistance to flood flows in channels and flood plains (USGS, 2009). Manning's "n" values for various natural and manmade channels are found in many hydraulics manuals and handbooks. Smooth, open stream channels with gravel bottoms have values around 0.035-0.055. Very winding, vegetated, or rocky channels have values around 0.055 to 0.075. Smooth earth or rock channels have values of 0.020 to 0.035.

Roughness values typically increase as channel vegetation and debris increase, as channel sinuosity increases, and as the mean size of channel materials increases. The value decreases slightly as flow depth increases. For this research, the Manning's roughness coefficients were used for different materials that are presented in ERA DDM, 2002.

3.6. Data Types and Sources

Quantitative as well as qualitative data sources were used for the study. The secondary data sources were used to strengthen the primary data.

Data Sources

The study used both primary (field) and secondary (documented) data for the evaluation of the drainage structures.

1) **Primary Data**

The primary data were collected from the practical construction sites where the structures are constructed. All data such as the material, structural form, span and diameter collected for the research purpose.

2) **Secondary data**

The secondary data were collected from the previously documented structures data used during the road construction. The data collected are analyzed to the research purpose.

3.6.1 Data Analysis

The research focuses on the performance assessment of road drainage systems. Analysis of the collected data was carried out by rational and SCS methods based on the following factors that affect flood.

- Drainage basin characteristics including: size, shape, slope, land use, geology, soil type, surface infiltration, and storage;
- Stream channel characteristics including: geometry and configuration, slope, hydraulic resistance, natural and artificial controls, channel modification, aggradations, ice and debris;
- Floodplain characteristics;
- Meteorological characteristics such as precipitation amount and type (rain, snow, hail, or combinations thereof), rainfall intensity and pattern, areal distribution of rainfall over the basin, and duration of the storm event

These parameters were obtained from long-term climatic data, hydrological data, and geological data, soils, land use / land cover maps prepared at medium and large scales for general purposes and hydrographic and topographic survey and geotechnical investigations along the road route (Source: ERA, DDM- 2002).

3.6.2 Data Processing

The continuity of a recorded data may be broken with missing due to many reasons such as damage or fault in gauging station during a measuring period. So, before starting any procedures, it is important to check whether the data were homogenous, consistence and complete with no missing data. If the missing data existing it should be estimated using the data filling methods. Because incorrect data leads to inconsistency and ambiguous results that may contradict to the actual value.

3.6.2.1 Homogeneity test

The selected stations were plotted for comparison with each other; for illustration, from Appendix C, figure C-1 below show the result of homogeneity analysis result plotted to check similarity of the two selected group stations. Hence the group stations are homogenous since all the values of Pi are less than 0.3.

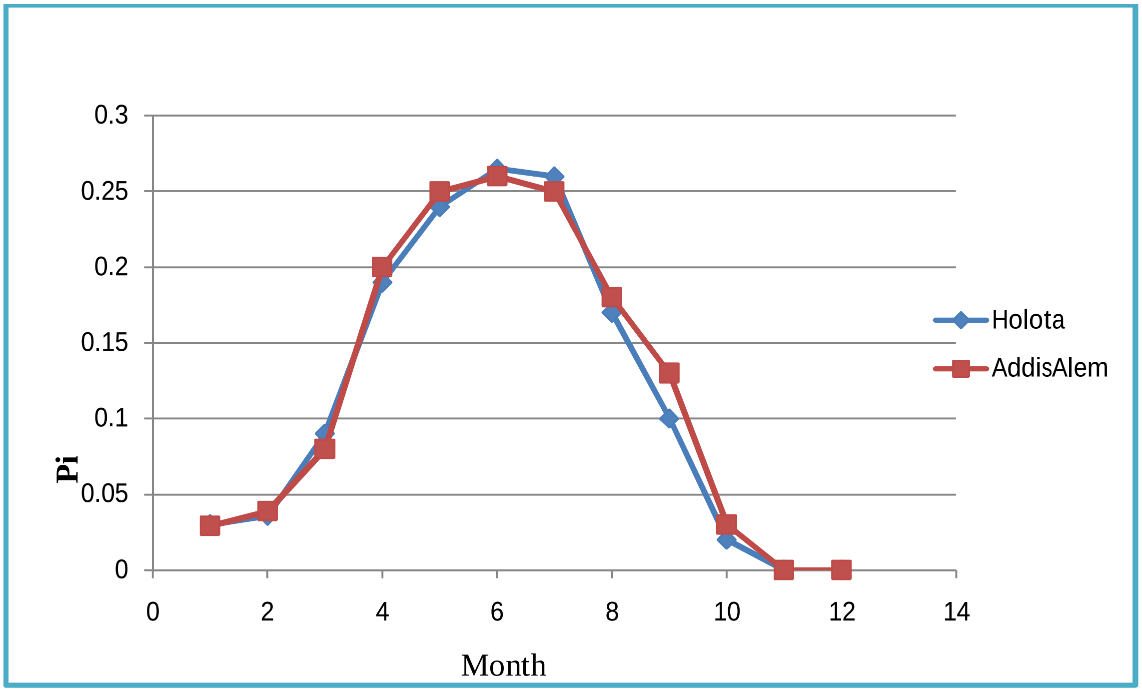


Figure 3.5: Homogeneity test for Holota and AddisAlem stations

3.6.2.2 Consistency test

According to the double mass curves analysis, all the stations were consistent. For illustration the double mass curves for some selected stations are presented figure 4.4 below.

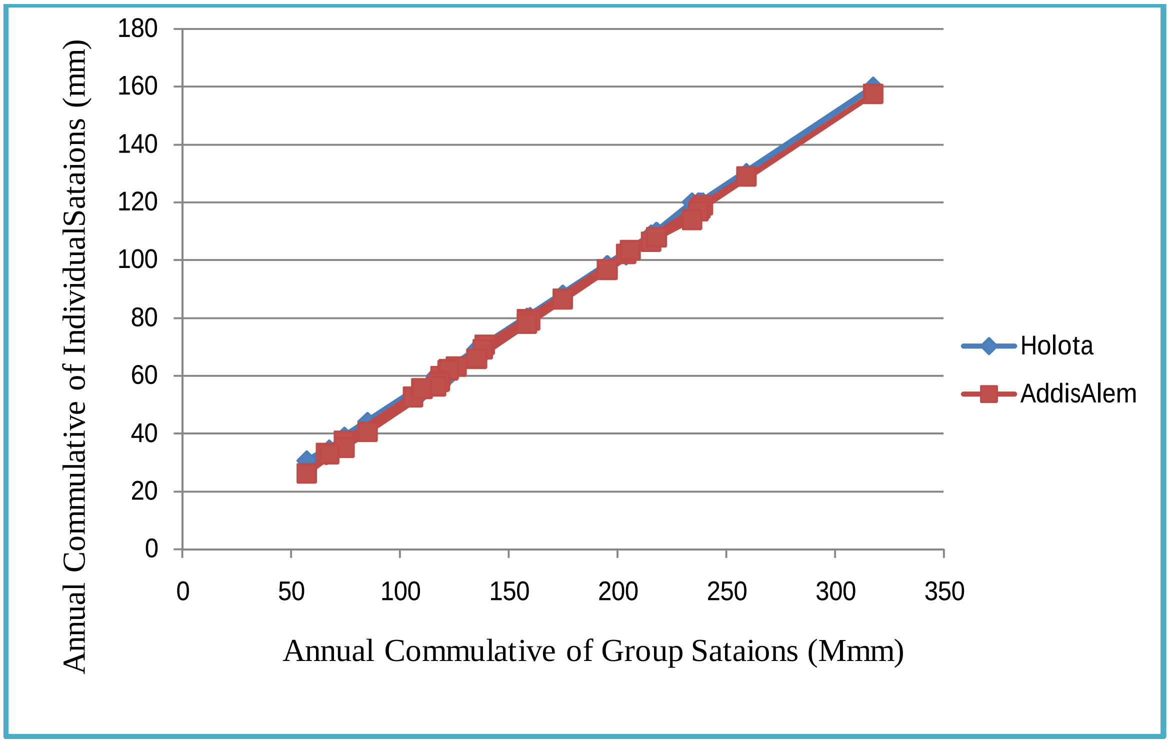


Figure 3.6: Consistency test for Holota and AddisAlem stations

4. RESULTS AND DISCUSSION

4.1. Evaluation of Data

4.1.1. Calibration and Validation of Results

In this section, generated rainfall intensity from IDF curves results were compared with the observed/gauged precipitation near the study area. These comparisons are carried out taking account the statistical parameters mentioned in previous sections. For this research, gauged precipitations from two stations located near study area were compared with the generated rainfall intensity.

The calibration data range for rainfall gauging station is as follow. For the Holota gauging station, data for the period (1980-2003) were used for calibration. And the validation data range for rainfall gauging station is as follow. For the Holota gauging station, data for the period (2004-2014) were used for validation.

4.1.2. Statistical Analysis

Figure 4.1 shows the statistical summary of the comparison between generated and observed rainfall values for the calibration period. Little differences were observed on the correlation mean value of rainfall between Holota and the study area stations in the calibration period. Then, as it was mentioned above, the correlation coefficient (R^2) was used to check the results. This behavior shows that the small differences between generated and observed/gauged values of rainfall data.

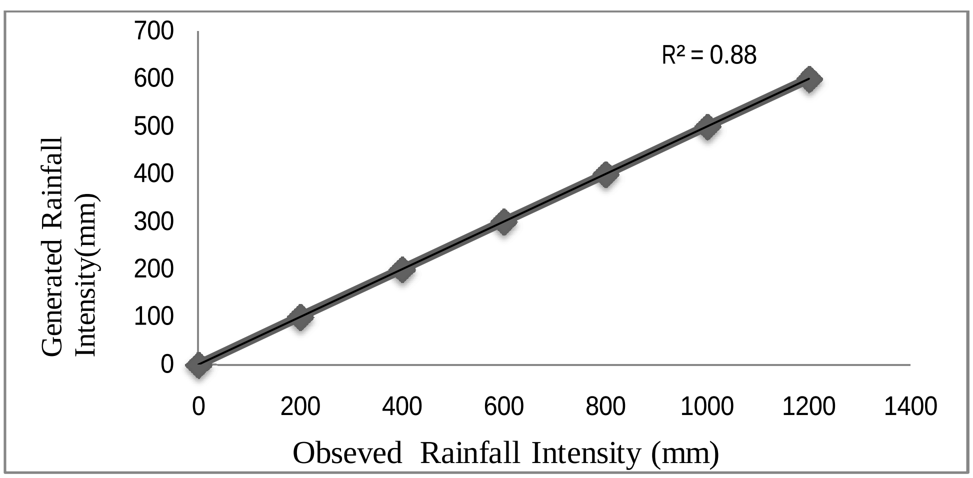


Figure 4. 1: Comparison of observed/gauged and generated annual rainfall for calibration period

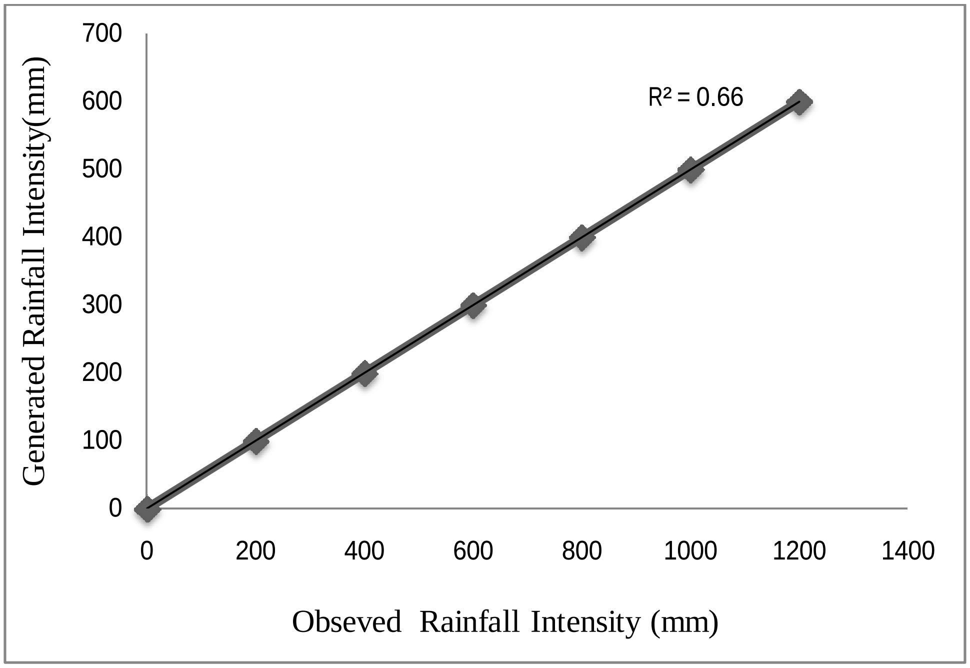


Figure 4.2: Comparison of observed/gauged and generated annual rainfall for validation period

4.2. Hydrologic and Hydraulic Analysis

On Burayu town road, drainage structures performance assessment, the peak flood were computed by the rational method and SCS methods according to their area of application and Manning's equation was used for hydraulic analysis for existing drainage structures.

4.2.1. Computation of Catchment Parameters at Stations 16+200 Drainage Structure

On Burayu road, drainage structures performance assessment, the maximum peak flood is computed. After the watershed areas delineation, watershed properties like soil type and curve number are computed.

i. Return Periods and Rainfall Intensities

The maximum peak flood computed using the return period recommend in ERA Drainage Design Manual 2011 considering the road standard and the design life span of the structure,10 years for design and 25 year check (review) return period were adopted. See Appendix A of table 3 design storm frequency (yrs) by Geometric Design Criteria - (ERA DDM, 2002).

ii. Runoff Curve Number

A hydrological characteristic of soil groups, the region is a wet antecedent moisture condition (AMC) region. After the watershed areas delineation, watershed properties like the land use coverage, soil type and curve number are computed. From the Appendix A in table 4, the runoff curve number for impervious area CN= 98.

iii. 24-Hour Rainfall Depth

Since the drainage structure at station, 16+200 is culvert that has diameter less than 2-meter. Based on the ERA Drainage Design Manual 2011 for region A2 data, the 24 hour rainfall depth P is taken as 79 mm for 10 yrs return period and 95mm for the 50 yrs return period were used.

iv. Direct Runoff Depth

Direct runoff (Q) is determined from Appendix A table 3, by using rainfall depth of 79mm for design is Q_{10}=73.14mm and for rainfall depth of 95mm is Q_{25}=89.09mm.

4.2.2. Runoff Computation by SCS Method at Station 16+200

Step 1: Determine Catchment Area

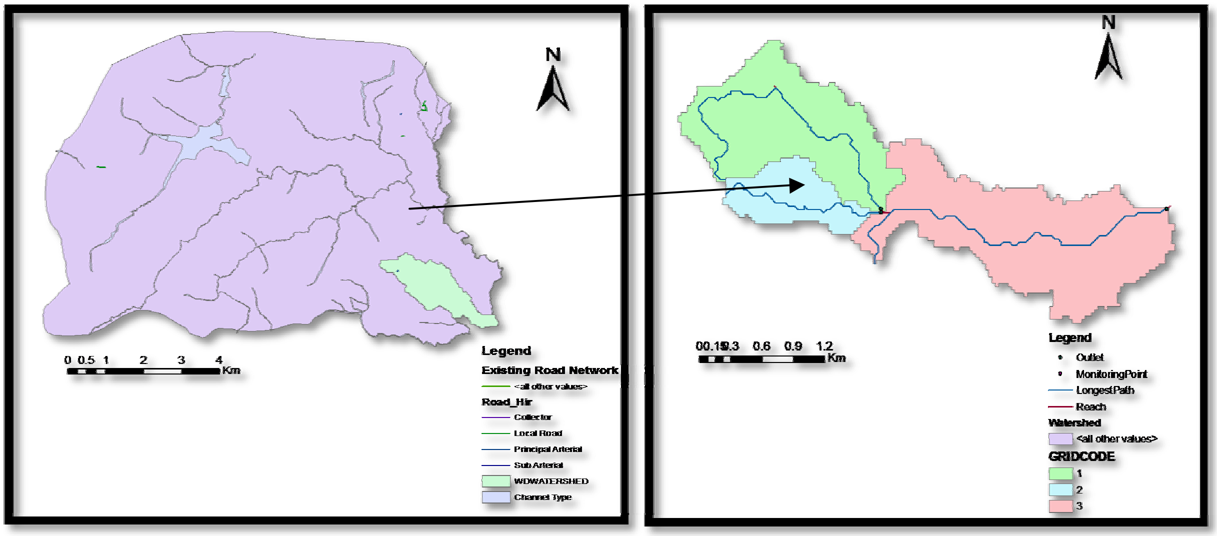


Figure 4.3: Catchment Area for Drainage Structure at Station 16+200

Step 2: Determine longest flow path and elevation

Name	Catchment Area ha	Stream length (m)	Elevation (m)
C_1	60	1800	2510

Step 3: Determine Catchment Area Property

Land Cover	Soil Type	Hydrologic Soil Group	Rainfall Region
Impervious area	Lithosols	D	A2

Step 4: Calculate Time of concentration

Time of Concentration

i. Sheet flow

Sheet flow, natural range, slope of 0.025 m/m, and the sheet flow length of 100m and from Appendix B, Table 3, for Smooth surfaces (concrete, asphalt, gravel, or bare soil) manning's roughness coefficient is 0.011.

The 2-year, 24-hour rainfall depth is determined from Appendix A table 3 to be 52mm. Hence, from Equation (4.9), travel time for sheet flow is determined as:

$Tt = [0.091(nL)^{0.8} / (P2)^{0.5} S^{0.4}]$

$= 0.06hr$

ii. Shallow Concentrated Flow

After a maximum of 100 meters, sheet flow usually becomes shallow concentrated flow (ERA DDM, 2002). For shallow concentrated flow, paved watershed slope is approximated 0.0028m/m and length from topography map is 900m.

The average velocity for this flow can be determined from equation (4.11), $V = 6.1961 (S)^{0.5}$ in which average velocity is a function of watercourse slope and type of channel.

Paved $V = 6.1961 (S)^{0.5}$

$= 0.32m/sec$

From equation (4.6), $T_t = L/ (3600V)$ travel-time is determined as:

$T_t = L/ (3600V)$

$= 900/ (3600 \times 0.32)$

$= 0.78hr$

iii. Channel flow

For channel flow, natural stream channel, winding with weeds and pools, slope is 0.01 m/m, and length is 800m. By direct measuring the average bottom width of the stream channel is 2.5m, side slopes are 1V:1.5H, 25-year storm depth is 1m. From Appendix B in Table 3, Manning's roughness coefficient for fallow (no residue) channels is 0.05.

A= Cross-sectional flow area = $by+zy^2$

$= (2.5 \times 1) + 1.5(1)^2 = 4m^2$

Pw= wetted perimeter = $b+2y\,(1+z^2)^{0.5} = 6.1m$

R = Hydraulic radius = A/Pw = 4/6.1 = 0.65m

From Equation (4.12), $V= (R^{2/3}S^{1/2})/n$

$V = (R^{2/3}S^{1/2})/n = 1.50m/s$

From Equation (4.6), Tt = L/ (3600V)

$T_t = L/(3600V)$

$= 0.14$ hr

Total Time of Concentration = 0.06 + 0.78 + 0.14 = 0.9hr= 54min

By the same procedures, catchment parameters at stations 15+300 and 17+600 are determined.

Step 5: Calculate peak flood

The peak flood estimation was done by using the SCS method flood estimation formula by

1) Rainfall runoff equation

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$ ……………………………………………………………equation (3.5)

For $P > 0.2S$

$Q = 0$ for $P \leq 0.2S$

Where:

Q = accumulated direct runoff, mm.

P = accumulated rainfall (i.e., the potential maximum runoff), mm.

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

Then I_a =1.0mm, for CN=98

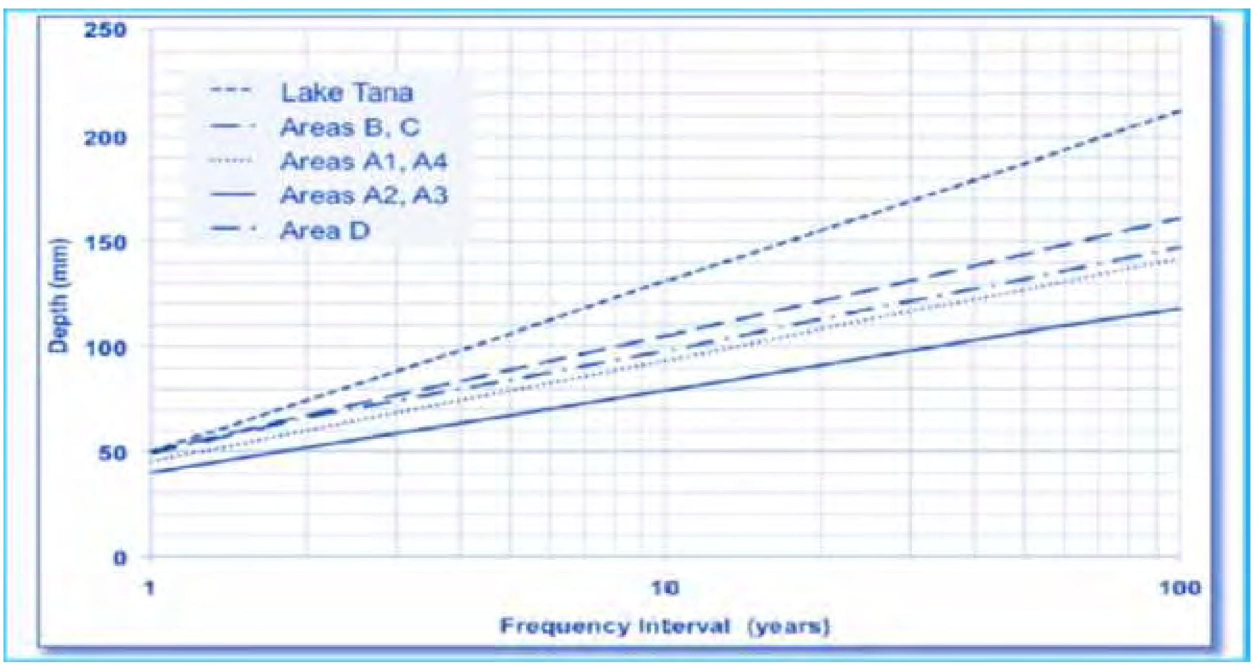


Figure 4.4: 24-hour Depth-Frequency Curve (ERA DDM, 2011)

Table 4.1: 24-hour Rainfall Depth

Return period	2 years	5 years	10 years	25 years	50 years	100 years
P (mm)	52	67	79	95	107	118
Q (mm)	46.29	61.19	73.14	89.09	101.06	112.03
I_a/P	0.019	0.014	0.012	0.010	0.009	0.008

2) Peak Discharge Computation

The following equation were used for the estimation of the peak discharge in SCS method

Design Peak Discharge, Qp =Qu*Q*A ………………………………equation (3.14)

For Curve Number (CN) =98, Ia = 1.0mm

Table 4.2: Peak Discharge Results for Drainage Structure at Station 16+200 by SCS method

Return period	2years	5years	10years	25years	50 years	100years
Qu	0.004104	0.004094	0.00409	0.004086	0.004084	0.004082
Qp(m^3/s)	11.4	15.0	17.9	21.8	24.7	27.4

Burayu-town road as described on geometric design report, categorized under DS3 road design standard. Hence the design storm frequency stated under DS3/4 used to compute the design discharge and to design the respective drainage crossing structures, which are presented in Appendix A of table 2.

Case I-during the diameter of culvert is less than 2 meter the design storm frequency (yrs) by geometric design criteria DS3/DS4 is used. Therefore, the anticipated peak discharges for catchment area at station 16+200 is 21.8m^3/s for check. The average flow velocity is 0.36m/sec.

The above result is computed by the SCS method for the area of 60ha. In order to check/compare the result in table (4.2) above, Rational method is used by dividing the watershed area as the following.

4.2.3. Runoff Computation by Rational Method at Station 16+200

Step 1: Determine Catchment Area

Burayu Town Catchment

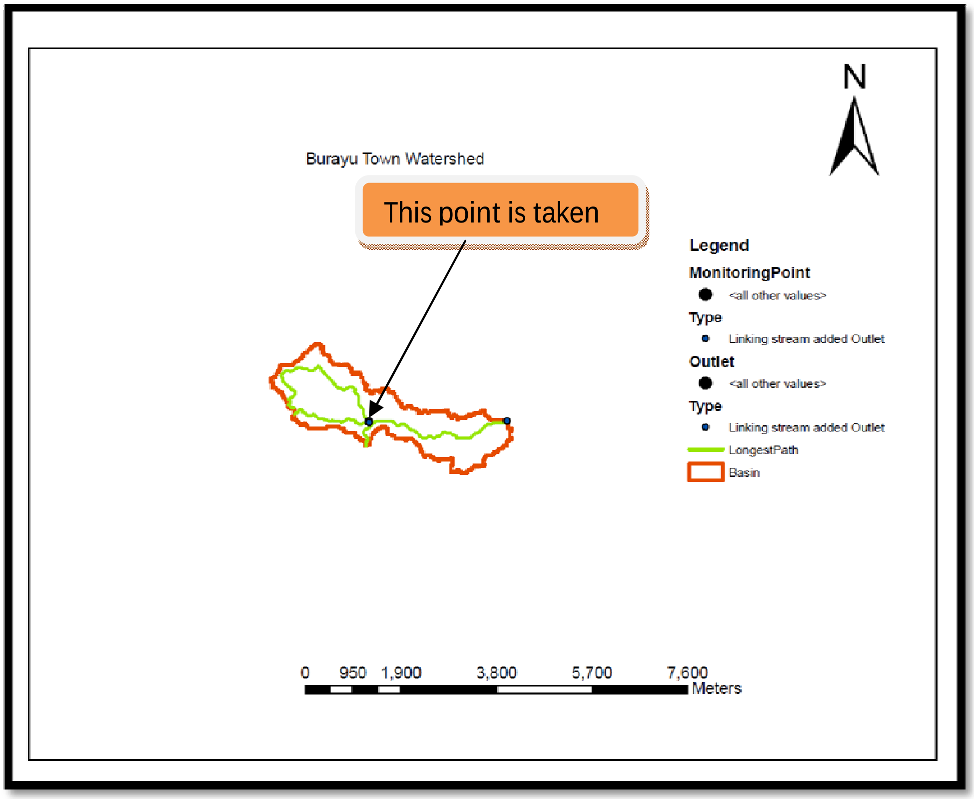


Figure 4.5: Catchment Area for Drainage Structure at Station 16+200

Step 2: Determine longest flow path and elevations

Name	Catchment Area	Shape length (m)	Min. Elevation (m)	Max. Elevation (m)
C1	30	900	2460	2510

Step 3: Determine Catchment Property

Land Cover	Soil Type	Hydrologic soil group	Rainfall Regional
Impervious area	Lithosols	D	A2

Step 4: Calculate the Time of Concentration

(i) Calculation of time of concentration for over land flow

$T_c = 0.604(RL/S^{0.5})^{0.467}$ ………………………………………………………………………. (3.4)

R = roughness coefficient of land use (concrete) from Appendix B of table 1 = 0.02

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) =0.33, S= 0.15

T_c= time of concentration (hours) $=0.604(RL/S^{0.5})^{0.467}$

$$Tc=0.604(0.02*0.33/0.15^{0.5})^{0.467}$$

$$Tc = 0.090hr$$

(ii). Calculation of time of concentration for defined watercourses 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.

$$= \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

L = length of watercourse (km) = 0.9

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

S_{av}=0.07

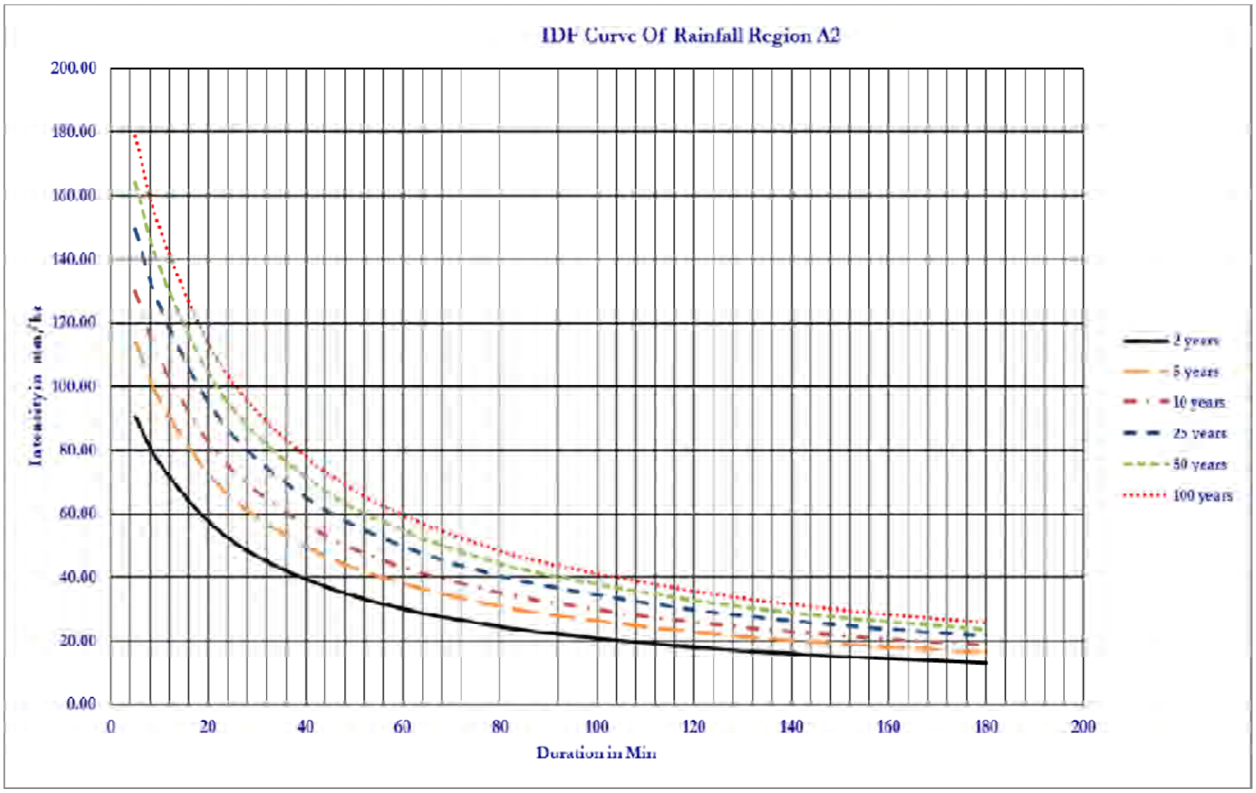
$$= \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385} = \left(\frac{0.87*0.9^2}{1000*0.07}\right)\text{^}0.385$$

T_c=0.170hr

$T_{ct} = T_{c1} + T_{c2}$ =0.26hr =15min

Step 5: Determine rainfall intensity

Table 4.3: Intensity Duration Frequency curve for region A2 (ERA DDM, 2002)



The catchment area were found in rainfall region A2, used the IDF curve of rainfall region A2 and find the rainfall intensity for different return periods.

I_2=65mm/hr; I_5=82mm/hr; I_{10}=95mm/hr; I_{25}=105mm/hr; I_{50}=118mm/hr; I_{100}=130mm/hr

Step 6: Determine runoff coefficients

Table 4.4: Runoff coefficient values for different drainage areas

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

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

Runoff coefficient (C) under streets for concrete type drainage area, C_{av} = 0.87

Step 7: Calculate the Peak flood

Q=0.00278CIA

Where, Q= Peak flow in cubic meter per second (m^3/sec)

C= Dimensionless weighted runoff coefficient

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

A= Drainage area in hectare (ha) which is 30ha

Table 4.5: Peak Discharge Results for Drainage Structure at Station 16+200 by rational method

Return period	2 year	5year	10 year	25 year	50 year	100year
I(mm/hr)	67	83	95	105	118	130
A(ha)	30	30	30	30	30	30
Q (m^3/s)	5.035	6.202	7.078	7.808	8.757	9.486

The above table (4.5) is the computed peak discharge at station 16+200 by using rational method for the area of 30ha/0.3km^2. From the above table (4.5), it is confirmed that the peak discharge at road station 16+200 was computed by dividing the watershed into two parts which has the area of 60ha/ 0.6km^2. Therefore, if we double the above result it approximately becomes equal with the result which was computed by using the SCS method. The results which computed by two methods (Rational and SCS method) were compared. Rational method is used for catchment area less than 50 hectares (0.5 km^2) and SCS method used for catchment areas greater than 50 hectares. The table below (4.5) that shows the double result of the computed peak discharge using the rational method is shown at table (4.6) below.

Table 4.6: Peak Discharge Results for Drainage Structure at Station 16+200 by rational method

Return period	2 year	5year	10 year	25 year	50 year	100year
I(mm/hr)	67	83	95	105	118	130
A(ha)	30+30	30+30	30+30	30+30	30+30	30+30
Q (m^3/s)	11.2	14.9	17.8	21.5	24.8	27.5

In order to check the computed peak discharges (check discharges) with the design discharges of the drainage structure (culvert) at station 16+200 is as following.

Hydraulic calculation was carried out for drainage structures at station 16+200 using by Manning's Equation is as following.

4.2.4. Hydraulic Calculation for Drainage Structure at Station 16+200

The drainage structure at station 16+200 is culvert as shown on figure (4.6) below. The design diameter of the culvert is 1.4m, the existing culvert was installed at a slope of 0.2m/m and the wetted perimeter (p) is 4.4meter, the roughness coefficient value for concrete (n) 0.020 from appendix B of table 1. The hydraulic calculation was carried out using equation (3.15).Therefore, the design and review discharges are $17.0m^3$/sec and $21.5m^3$/sec respectively.

Proper design and construction of drainage structures are vital components for road structure to function without traffic interruption. Appropriate hydrological analysis of the catchment area where the drainage structure will be constructed and appropriate hydraulic parameters should be determined. If proper hydrological analysis and hydraulic calculation were not well practiced, either overdesign or under design would occur that both involve excessive costs on a long-term basis.

A drainage structure designed to carry a short recurrence interval flood would have a low first cost, but the maintenance cost would be high because the drainage structure and roadway may be damaged by storm runoff almost every year. On the other hand, a drainage structure designed to carry the long recurrence interval flood would be high in initial cost, but low in maintenance cost.

Design of the drainage structure at station 16+200 on Burayu town road is under design that costs much every year for maintenance as shown on figure (4.6) below. Therefore, appropriate design, and construction should be carried out in order to the road to function properly as intended for the road users.

Therefore, additional pipe culvert is needed at this station in order to pass or accommodate the coming flood. The size and type of drainage structure at this station is recommended in the recommendation part.

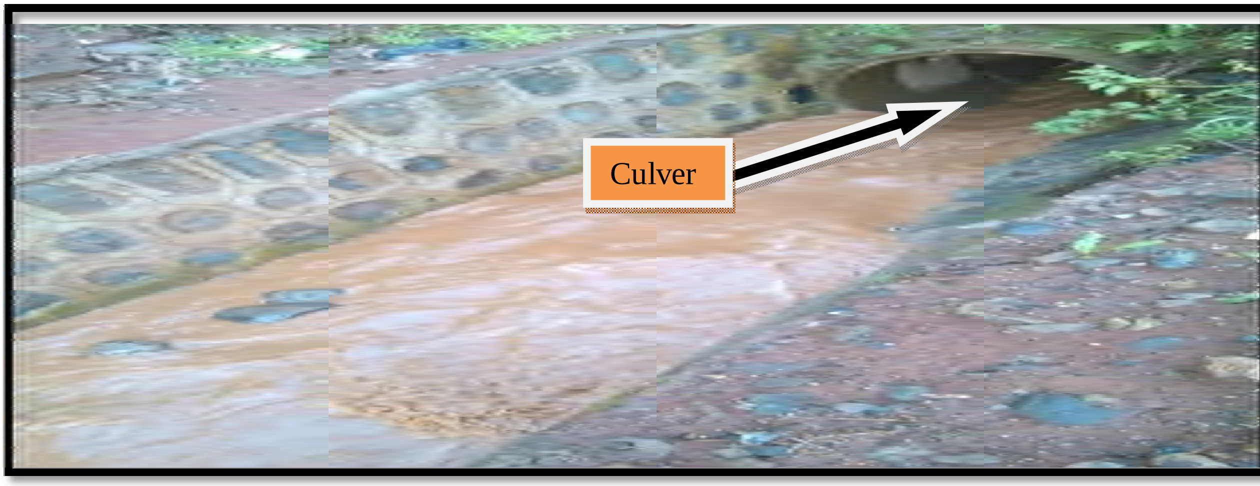


Figure 4.6: Road section at station 16+200

4.2.5. Runoff Computation by SCS Method at Station 15+300

Step 1: Determine Catchment Area

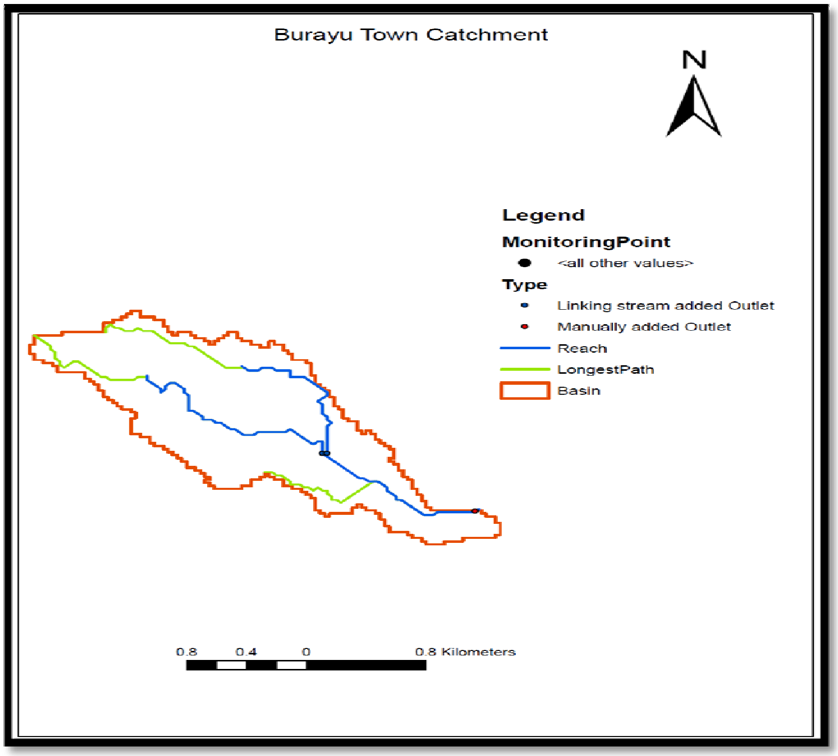


Figure 4.7: Catchment Area for Drainage Structure at Station 15+300

Step 2: Determine longest flow path and elevation

Name	Catchment Area (ha)	Stream length (m)	Elevation (m)
C_2	100	2003	2500

Step 3: Determine Catchment Area Property

Land Cover	Soil Type	Hydrologic Soil Group	Rainfall Region
Street and roads: paved, open ditches	Lithosols	D	A2

Step 4: Calculate Time of concentration

(i) $T_c = 0.604(RL/S^{0.5})^{0.467}$………………………………………………………………. (3.4)

R = roughness coefficient of land use (concrete) from Appendix B of table 1 = 0.02

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

T_c= time of concentration (hours)

(ii) Calculation of time of concentration for defined watercourses 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.

$$= \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

L = length of watercourse (km), S_{av}= average slope (m/m)

T_c = 0.98hr

The peak flood estimation was done by using the SCS method flood estimation formula

1) Rainfall Runoff equation

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous hydrologic and vegetative cover conditions.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$ ……………………..……………………………………………equation (3.5)

For P > 0.2S

Q =0 for $P \leq 0.2S$

Where:

Q = accumulated direct runoff, mm.

P = accumulated rainfall (i.e., the potential maximum runoff), mm.

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

CN= Curve Number, for impervious area: Paved; open ditches (including right-of-way) = 93

Then I_a =3.8mm

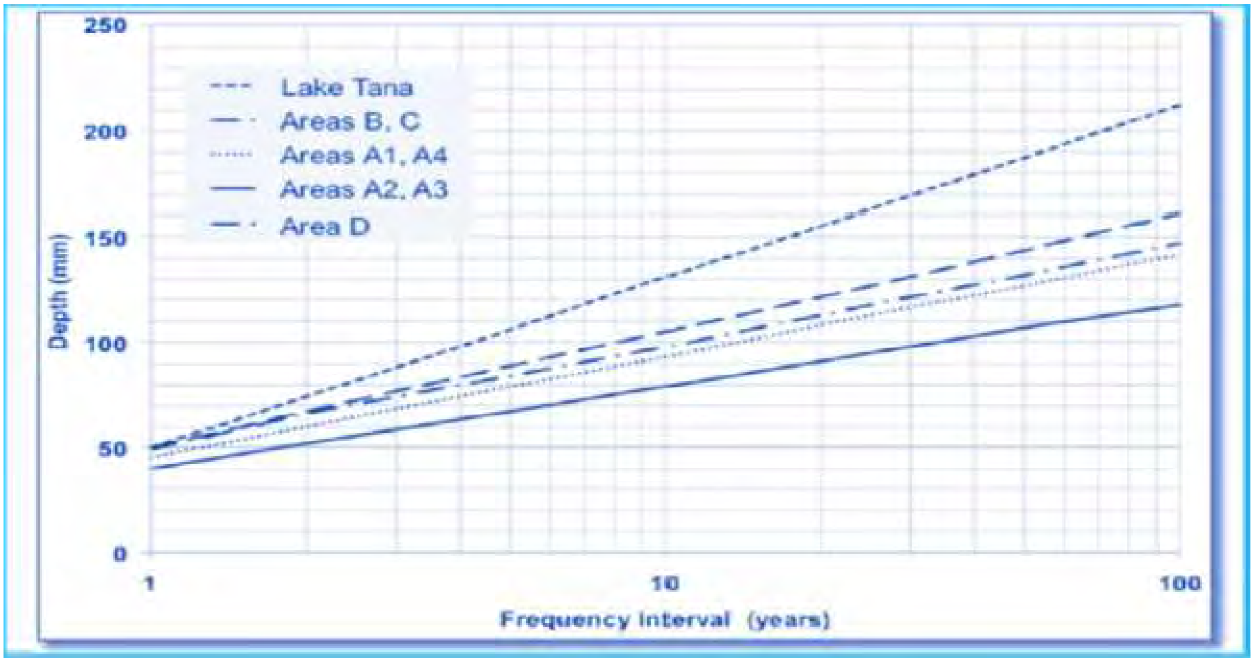


Figure 4.8: 24-hour Depth-Frequency Curve (ERA DDM, 2011)

Table 4.7: 24-hour Rainfall Depth

Return period	2 years	5 years	10 years	25 years	50 years	100 years
P (mm)	52	67	79	95	107	118
Q (mm)	34.12	48.52	59.96	75.39	87.07	97.82
I_a/P	0.073	0.057	0.048	0.040	0.036	0.032

2) Peak Discharge Computation

The following equation were used for the estimation of the peak discharge in SCS method

Design Peak Discharge, Qp =Qu*Q*A ………………………………………equation (3.14)

For Curve Number (CN) = 93, Ia = 3.8mm

Table 4.8: Peak Discharge Results for Drainage Structure at Station 15+300 by SCS method

Return period	2 years	5years	10years	25years	50 years	100years
Qu	0.003405	0.003375	0.003359	0.003344	0.00337	0.00333
Q	34.12	48.52	59.96	75.39	87.07	97.82
Qp(m^3/s)	11.61	16.37	20.14	25.21	29.34	32.57

Burayu town road was categorized under DS3/4 road design standard. Hence the design storm frequency stated under DS3/4 used to compute the design discharge and to design the respective drainage crossing structures, which are presented in Appendix A of table 2. The diameter of culvert is less than 2 meter this station (15+300). Therefore, the anticipated peak discharges for catchment area at station 16+200 is $25.21m^3/s$ for (frequency for 25year) for check. The average flow velocity is 0.25m/sec. The above result is computed by the SCS method for the area of 100ha. In order to checked/compared the result in table (4.8) above, rational method is used by dividing the watershed area as the following.

4.2.6. Runoff Computation by Rational Method at Station 15+300

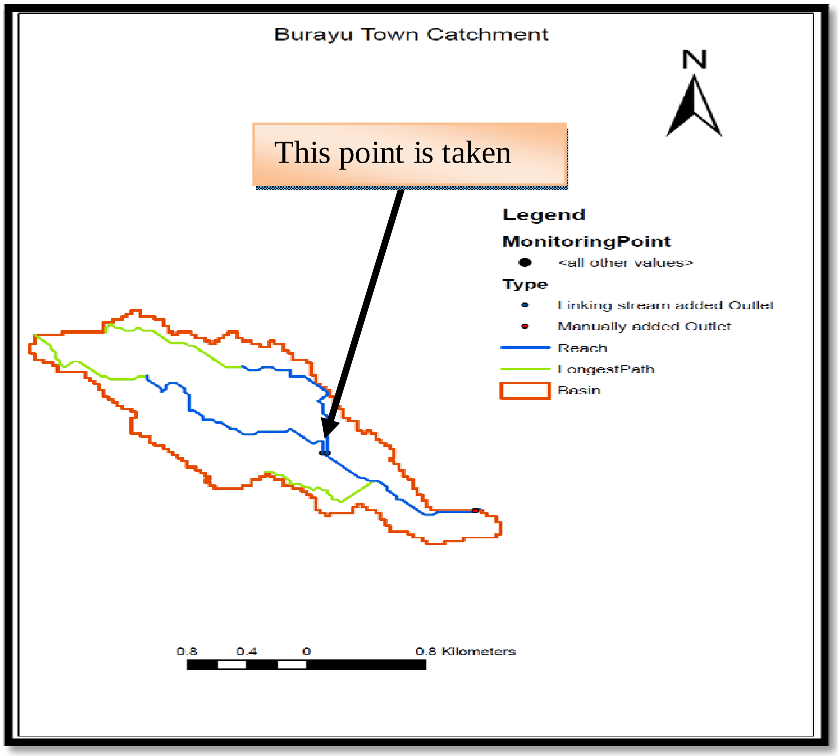


Figure 4.9: Catchment Area for Drainage Structure at Station 15+300

Step 2: Determine longest flow path and elevations

Name	Catchment Area (ha)	Shape length (m)	Min. Elevation (m)	Max. Elevation (m)
C1	50	2003	2460	2500

Step 3: Determine Catchment Property

Land Cover	Soil Type	Hydrologic soil group	Rainfall Regional
Street and roads: paved, open ditches	Lithosols	D	A2

Step 4: Calculate the Time of Concentration

(i) Calculation of time of concentration for over land flow

$T_c = 0.604(RL/S^{0.5})^{0.467}$ …………………………………………………………………. (3.4)

R = roughness coefficient of land use =0.02

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km) =0.25km, Slope = 0.16

T_c= time of concentration (hours) =$0.604(RL/S^{0.5})^{0.467}$

$Tc=0.604(0.02*0.25/0.16^{0.5})^{0.467}$

Tc = 0.07hr

(ii) Calculation of time of concentration for defined watercourses

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.

$$= \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

L = length of watercourse (km) = 2km

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

H =height of most remote point above outlet of catchment (m), Sav = 0.027

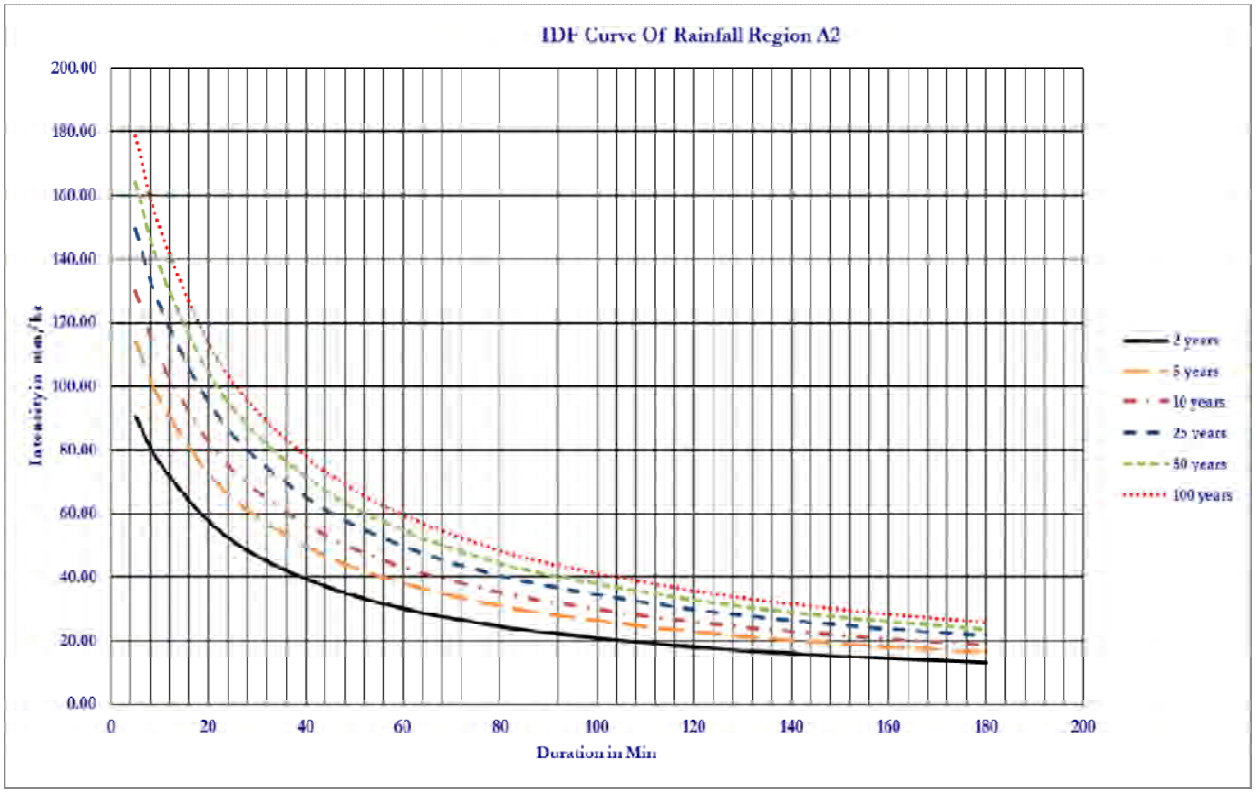
$$= \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385} = \left(\frac{0.87*2^2}{1000*0.027}\right)\text{^}0.385$$

T_c=0.45hr

$T_{ct} = T_{c1} + T_{c2}$ = 0.07+0.45= 0.52hr =31min

Step 5: Determine rainfall intensity

Table 4.9: IDF Curve of Rainfall Region A2



The catchment area was found in rainfall region A2, used the IDF curve of rainfall region A2 and finds the rainfall intensity for different return periods.

I_2=46mm/hr; I_5=58mm/hr; I_{10}=65mm/hr; I_{25}=75mm/hr; I_{50}=84mm/hr; I_{100}=92mm/hr

Step 6: Determine runoff coefficients

Table 4.10: Runoff coefficient values for different drainage areas

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

Runoff coefficient (C) under street for concrete type drainage area, $C_{av} = 0.87$

Step 7: Calculate the Peak flood

Q=0.00278CIA

Where, Q= Peak flow in cubic meter per second (m3/sec)

C= Dimensionless weighted runoff coefficient

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

A= Drainage area hectare (ha) which is 50ha

Table 4.11: Peak Discharge Results for Drainage Structure at Station 15+300 by rational method

Return period	2year	5year	10 year	25year	50 year	100 year
I(mm/hr)	47	58	66	75	84	92
Q (m^3/s)	5.71	7.05	8.02	9.12	10.21	11.18

The above table (4.11) is the computed peak discharge at station 15+300 by using rational method for the area of 50ha. From the above table, it is confirmed that the peak discharge at road station 15+300 was computed by dividing the watershed into two parts which has the area of 100ha. Therefore, if we double the above result it approximately becomes equal with the result computed using the SCS method. The results were compared by Rational and SCS methods according to their area of application. Rational method is used for catchment areas less than 50 hectares (0.5 km^2) and SCS method for catchment areas greater than 50 hectares. The table that shows the double result of the computed peak discharge using the rational method is shown in table (4.12) below.

Table 4.12: Peak Discharge Results for Drainage Structure at Station 15+300 by rational method

Return period	2 year	5year	10 year	25 year	50 year	100year
I(mm/hr)	40	49	56	65	72	79
A (ha)	50 +50	50 +50	50 +50	50 +50	50 +50	50 +50
Q (m^3/s)	11.4	16.0	19.99	25.4	29.0	32.34

Hydraulic calculation is carried out for drainage structures at stations 15+300 using by Manning's Equation as follows.

Therefore, the computed peak discharge at station 15+300 is $25.4m^3/s$ for review (check) and the average flow velocity is 0.254m/sec. Hydraulic calculation is carried out for drainage structures at station 15+300 using by manning's Equation as followed below.

4.2.7. Hydraulic Calculation for Drainage Structure at Station 15+300

Hydraulic calculations are carried out for drainage structures at stations 15+ 300.The drainage structures at station 15+300 is pipe culvert as shown on figure (4.10) below. The design diameter of the culvert is 1.5meter, the existing culvert was installed at a slope of 0.17m/m, and the wetted perimeter (p) is 4.7meter, the roughness coefficient value for concrete (n) 0.020 from appendix B in table (1) blow.

Therefore, the design discharge was $19.0m^3/sec$. The hydraulic calculation was carried out using equation (3.1). Therefore, the design and review discharges are $19.0m^3/sec$ and $25.4m^3/sec$ respectively. The drainage culvert was checked by trial and error by primary data approximation. Therefore, additional pipe culvert is needed at this station in order to pass or accommodate the coming flood.

Figure 4.10: Drainage Structure (culvert) at Road Station 15+300

4.2.8. Runoff Computation by Rational Method at Station 17+600

Step 1: Determine Catchment Area

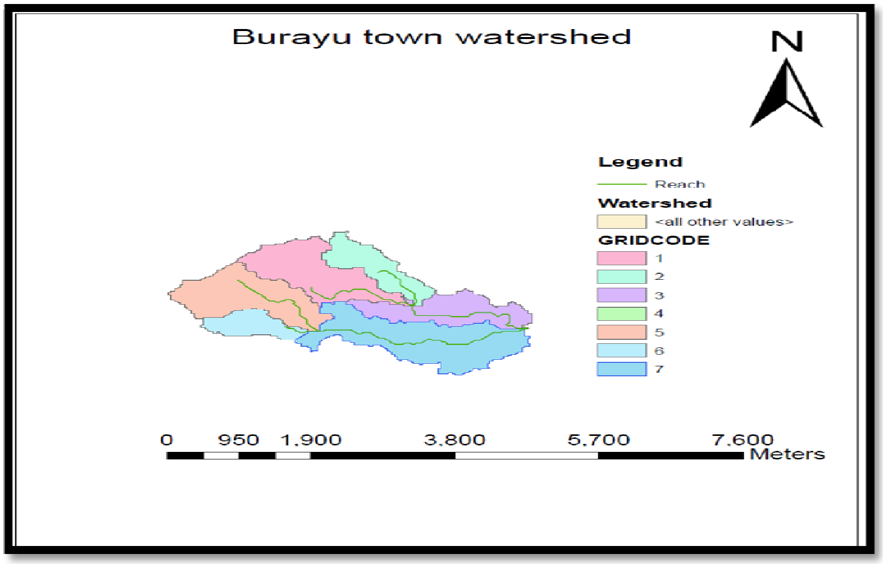


Figure 4.11: Catchment Area at Station 17+600

Step 2: Determine longest flow path and elevations

Name	Catchment Area (ha)	Shape length (m)	Max. Elevation (m)	Min. Elevation (m)
C1	50	800	2520	2480

Step 3: Determine Catchment Property

Land Cover	Soil Type	Hydrologic soil group	Rainfall Regional	AMC
Concrete	Lithosols	D	A2	Normal

Step 4: Calculate the Time of Concentration

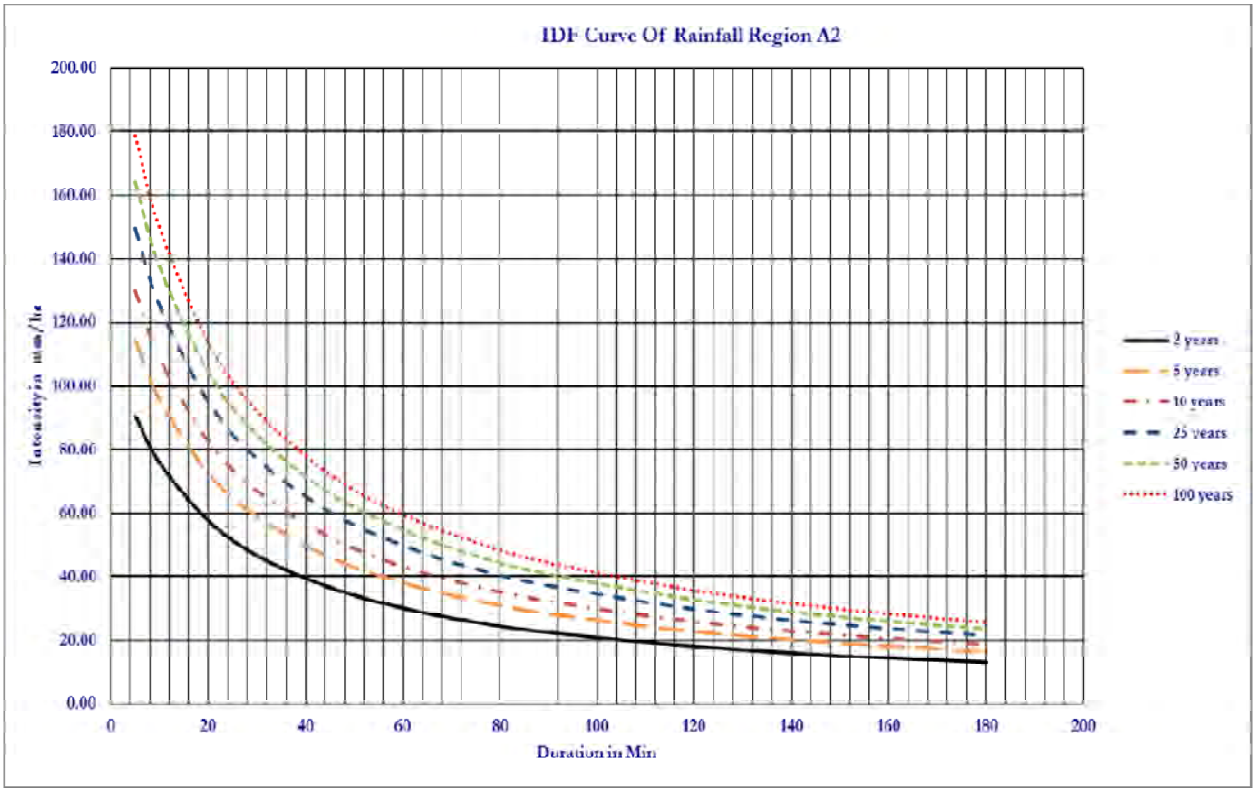
(i) Calculation of time of concentration for over land flow

$T_c = 0.604(RL/S^{0.5})^{0.467}$ …………………………………..………..………….. (4.4), for roughness coefficient of land use (R) = 0.02

$T_{ct} = 0.30hr = 18min$

Step 5: Determine rainfall intensity

Table 4.13: IDF curve of Rainfall Region A2



The catchment area were found in rainfall region A2, used the IDF curve of rainfall region A2 and find the rainfall intensity for different return periods.

I_2=64mm/hr; I_5=78mm/hr; I_{10}=90mm/hr; I_{25}=105mm/hr; I_{50}=116mm/hr; I_{100}=124mm/hr

Step 6: Determine runoff coefficients

Runoff coefficient (C) under street for concrete type drainage area, C_{av} = 0.87 from Appendix A of table 2.

Step 7: Calculate the Peak flood

Q=0.00278CIA, where, Q= Peak flow in cubic meter per second (m^3/sec)

Table 4.14: Peak Discharge Results for Drainage Structure at Station 17+600 by rational method

Return period	2year	5year	10year	25year	50year	100year
I(mm/hr)	64	78	90	105	116	124
Q (m^3/s)	7.73	9.43	10.88	12.69	14.02	14.99

Peak discharge is calculated using equation (3.1) at station 17+600 in the above table (4.14).Therefore, the computed peak discharge at station 17+600 is 10.88 $m^{3/}$s for (10 years frequency) and the average flow velocity is 0.22m/sec hence, Burayu-town road as described on geometric design report, categorized under DS3/4 road design standard. Therefore, the design storm frequency stated under DS3/4 used to compute the design discharge and to design the respective drainage crossing structures.

At this station, there is no drainage structure in order to convey/pass the coming discharge during rainy season. Drainage structures are important elements of the road structure to be accessible throughout the year without traffic interruption. Lack of drainage structure on roadways makes the carriageway to be weak due to poundage of water that infiltrated into the carriageway. The infiltrated water oversaturated the carriageway-wearing course, and sub-grade. As a result, the carriageway could not carry traffic as intended. When drainage structures are neglected to be constructed at appropriate locations: Surface water can pond at the edge of the road and weakens the road surface, Silt can accumulate at the edge of the road i.e. the silt cannot be washed away through the drainage structure due to unconstructed drainage structure and the visibility for road users is reduced, with increased risk of accidents on persons or animals.

As the results shown on the above table (4.14), Side ditch can safely accommodate the anticipated peak discharge 10.88m^3/s (10 years frequency). And the length of the road is 200m. Therefore, in order to convey/pass the coming discharge at this station bed width of 1m and total height 1m is enough to accommodate peak discharge generated from one way road surface. As a result, new drainage structure will recommended on the recommendation part.

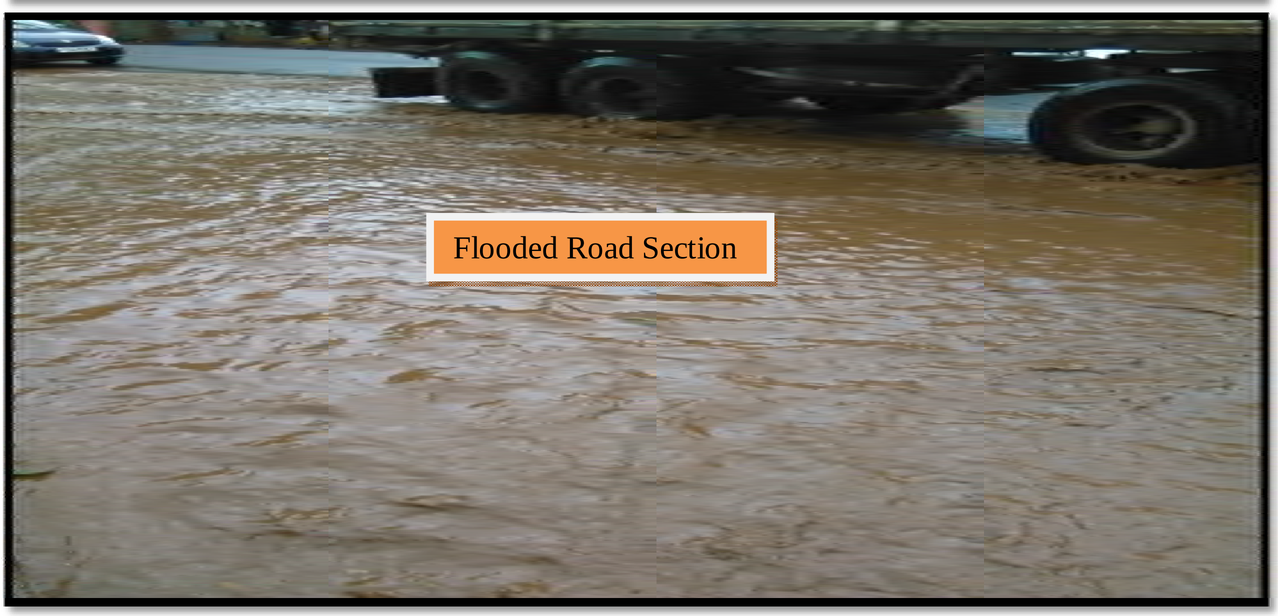


Figure 4.12: Road Section at Station 17+600 Source: Own field survey, 2016

4.3. Drainage Structure at Station 16+900

On Burayu town road, the road side ditch at station 16+900 were full closed by many things like silt, waste materials and garbage etc. A major problem observed at this station (16+900) during data collection is due to disposal of solid waste in the storm water drainage line and silt which comes from the agricultural activities, results the blockage of the existing drainage channels and hence impairs their ability to convey the runoff properly.

The main sources of solid wastes are households (residential units), commercial establishments, industries and hotels. As there is no adequate provision for garbage container in the town, residents dispose wastes in to the drainage channels, open spaces, and rivers/ streams. The conditions are more sever at flat areas. As it was observed during field survey the majority of the road drains are blocked by solid wastes of various types and many residents illegally connect their sewerage system in to the existing drains. Besides, this idea is reinforced from respondents' response.

Different construction activities are being carried out in the town by private individuals, governmental and non-governmental organizations. The leftover of the construction materials are disposed on roads and into drainage lines. The deposits sometimes stay for years and block the flow of runoff during peak rains.

Therefore, cleaning or removing these waste materials from this road channel should be carried out in order to the road to function properly as intended for the road users. Proactive measures should be taken to reduce and manage flooding hazards (like clearing of drains before rain season begins).The drainage structure at station 16+900 is channel/ditch as shown on the figure (4.13) below.

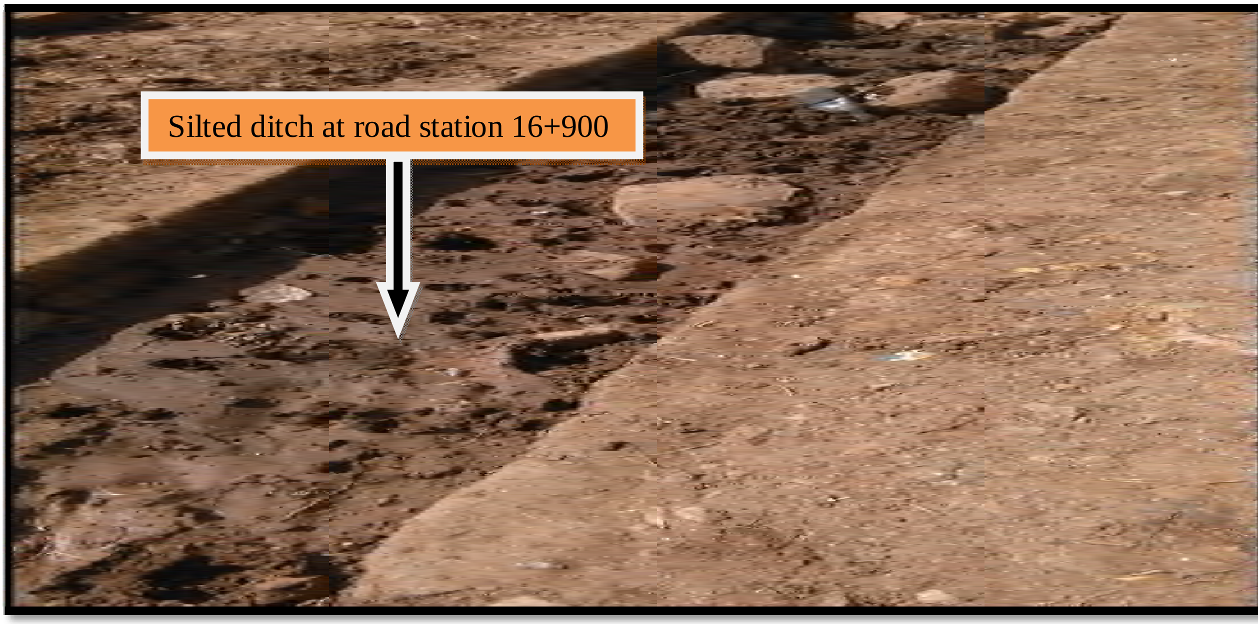


Figure 4.13: Road ditch at station 16+ 900 Source: Own field survey, 2016

4.4. Drainage Structure of Road Section at Station 17+200

The primary function of road drainage channel is to remove the storm water as rapidly as possible so that traffic may move safely and efficiently without any loss of time. On Burayu town road, the road side ditch at station 17+200 were full closed by many things like silt, waste materials and garbage etc. Inadequate integration between road and drainage lines followed by blockage of drains by solid wastes is the major causes of flooding in the study area (interviews from respondents, (Source: own field survey, 2016). A major problem observed at this station (17+200) during data collection is disposal of solid and liquid waste in the storm water drainage line, which results in the blockage of the existing drainage channels and hence impairs their ability to convey the runoff properly.

So that the ditch is not accommodate the flood safely and exposed the roads surface to potholes because of the drainage channel was closed by silt, plastic, garbage and other materials etc (Source: Own field survey, 2016).To solve the debris/silt problem, 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. Therefore, cleaning or removing these waste materials from this road channel should be carried out in order to the road to function properly as intended for the road users.

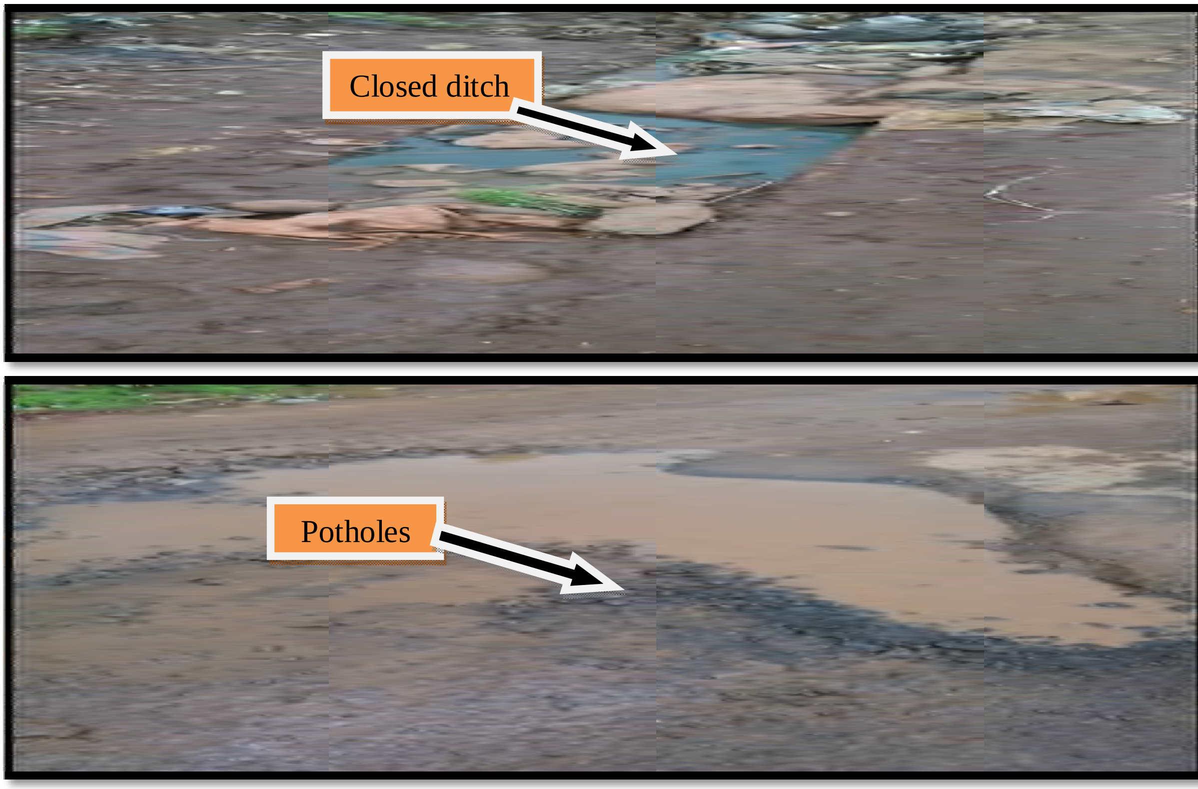


Figure 4.14: Road ditch at station 17+200 Source: Own field survey, 2016

4.5. Drainage Structure at Station 18+200

On Burayu town road ditch at station 18+200 becomes broken down due to soil erosion. This causes storm water to overflow on the carriageway and clogging of road side channels by silts. This has resulted potholes on road surface and accumulation of silts in the drainage structures.

During the field survey, it was observed that the town has serious problem related to lack of an appropriate maintenance for existing drainage facilities (storm water drainage system) which was broken down due to soil erosion at this station. Lack of an appropriate maintenance of the existing drainage facilities leads to existing drainage channels are either partially or fully filled with silts, plastic and causing the storm water to flow over the road surface (formation of gullies and rills across the existing drainage channels and roads). This problem ultimately reduces the carrying capacity of drainage channels and creates the overflow of runoff over the road surface and weakens the road surface. Therefore, the road ditches at this station needs an appropriate maintenance in order to function properly for the users.

Figure 4.15: Road ditch at station 18+200 Source: Own field survey, 2016

4.6. Proposing New Drainage Structures

Designing highway drainage structures involves many factors including estimating flood peaks, hydraulic performance, structural adequacy, and overall construction and maintenance costs. Drainage structures should be constructed at locations where they are required. Unless drainage structures constructed at appropriate locations, traffic interruption will be serious problem concern and the economic activities can be endangered.

4.6.1. Proposing Drainage Structure at Station 16+200

From equation (3.4), the design and check discharges are 17.0m^3/sec and 21.5m^3/sec at station 16+200 respectively. Therefore, new drainage structure will be added. Therefore, the proposed drainage structure is a pipe culvert that has length of seven-meters and diameter of 0.5meter for discharge (Q=4.5m^3/sec).

4.6.2. Proposing Drainage Structure at Station 15+300

The design and check discharges are 19.0m^3/sec and 25.4m^3/sec at station 15+300 respectively. There for additional new drainage structure will be needed at this station. Therefore, the proposed drainage structure is a pipe culvert that has length of seven-meters and diameter of 0.6meter for discharge (Q=6.4m^3/sec).

4.6.3. Proposing Drainage Structure at Station 17+600

4.6.3.1The effect of neglected drainage structures on roadways

Drainage structures are important elements of the road structure to be accessible throughout the year without traffic interruption. On Burayu town road at station 17+600, drainage structures were not constructed. This makes the carriageway to be weak due to pondage of water that infiltrated in to the carriageway.

The infiltrated water oversaturated the carriageway wearing course, and sub-grade. As a result, the carriageway could not carry traffic as intended. Due to this effect catastrophic problem was created for the proper functioning of the road. There is lacking of drainage structure at this station. New drainage structure will be needed at this station. From equation (3.1), the computed peak discharge is $10.88m^3$/sec. Therefore, the proposed drainage structure could safely accommodate the anticipated peak discharge of $10.88m^3$/s.

When drainage structures are neglected to be constructed at appropriate locations:-

- Surface water can pond at the edge of the road and weakens the road surface
- Silt can accumulate at the edge of the road i.e. the silt cannot be washed away through the drainage structure due to unconstructed drainage structure
- The visibility for road users is reduced, with increased risk of accidents on persons or animals

In order to serve a road properly for the road users, drainage structures should be constructed by considering where the location of the crossing in the watershed is required and how can water, sediment, and wood be transported at that location and how is the catchment configured. Therefore, based on these considerations the construction of ditch was required in order to use for intended purposes as aforementioned.

Figure 4.16: Road section at station 17+600

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This study analyzed the performance assessment of road drainage structure of Burayu town by using SCS method and rational for hydrologic analysis and Manning's equation for hydraulic analysis. The rainfall values which were obtained by generated rainfall from IDF curves of ERA and gauged rainfall from near station of the study area were checked and evaluated using R^2. First the meteorological missed data due to misreading and failed of the gauging instruments were filled by interpolation method. The filled data consistencies were then checked by double mass curve. The data also calibrated and validated using generated rainfall intensity and gauged precipitation data of nearby study area Holota station. The result of both tests showed that there was a good relationship between the observed and generated rainfall intensities by the statistical measure parameters.

Manning's equation was used to assess the adequacy of existing drainage structures at different road stations. Therefore, at station16+200 the hydraulic calculation was carried out and the design and check discharges are 17.0m^3/sec and 21.5m^3/sec respectively. Hence propose new drainage structures where required.

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.

At road at stations (16+900, 17+200), the road side ditch were loaded or silted by many things such as debris, silt, waste materials, and disposal of solid and liquid waste in the storm water drainage line. At station 17+600, the road becomes flooded or there was a flood across the road (street flood) and at station 18+200, the ditch is broken. Those problems created the road to malfunction during the rainy season every year.

These observed problems ultimately result in loss of a road section or even many road sections greatly hampering the traffic flow that negatively contributes to the mobility of the road users in general and the socio economic well-being of the people in particular.

5.2. Recommendations

At the study area, Burayu town, road drainage structures are not properly functioning. The main causes are the small number of culverts, insufficient capacity of road ditches, unavailability of drainage structures at proper place, street flood, poor management and siltation, etc. These factors cause gulling. Therefore, in order to minimize and/or avoid these problems, the following recommendations were drawn and/or suggested.

- ❖ At station (16+200, 15+300) the pipe culvert could not accommodate the peak discharge during the rainy season. Therefore, a pipe culvert of 0.5meter internal diameter should be constructed for station 16+200 and construction of pipe culvert of 0.6meter internal diameter is important at station 15+300.
- ❖ At two stations (16+900, 17+200), the road side ditches were loaded or silted by many things such as debris, silt, garbage, waste materials, and disposal of solid waste in the drainage line therefore, this impairs their ability to convey the runoff properly. Thus, 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 this 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).
- ❖ At station17+600, the road became flooded or there was a flood across the road (street flood) because of lack of drainage structures. Therefore, side ditch drainage structure of 1meter width and height of 1meter should be constructed by the municipality.
- ❖ At station 18+200, the ditch is cracked and broken. Hence, it should be maintained and repaired for its purpose.

In general, it is important to use appropriate hydrological analysis, hydraulic design, and stream morphological study for road drainage structures. This has to be done by skilled and knowledgeable workers.

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Appendix A: Time of Flow, Unit Peak Discharge, Velocity of flow and SCS CN Charts

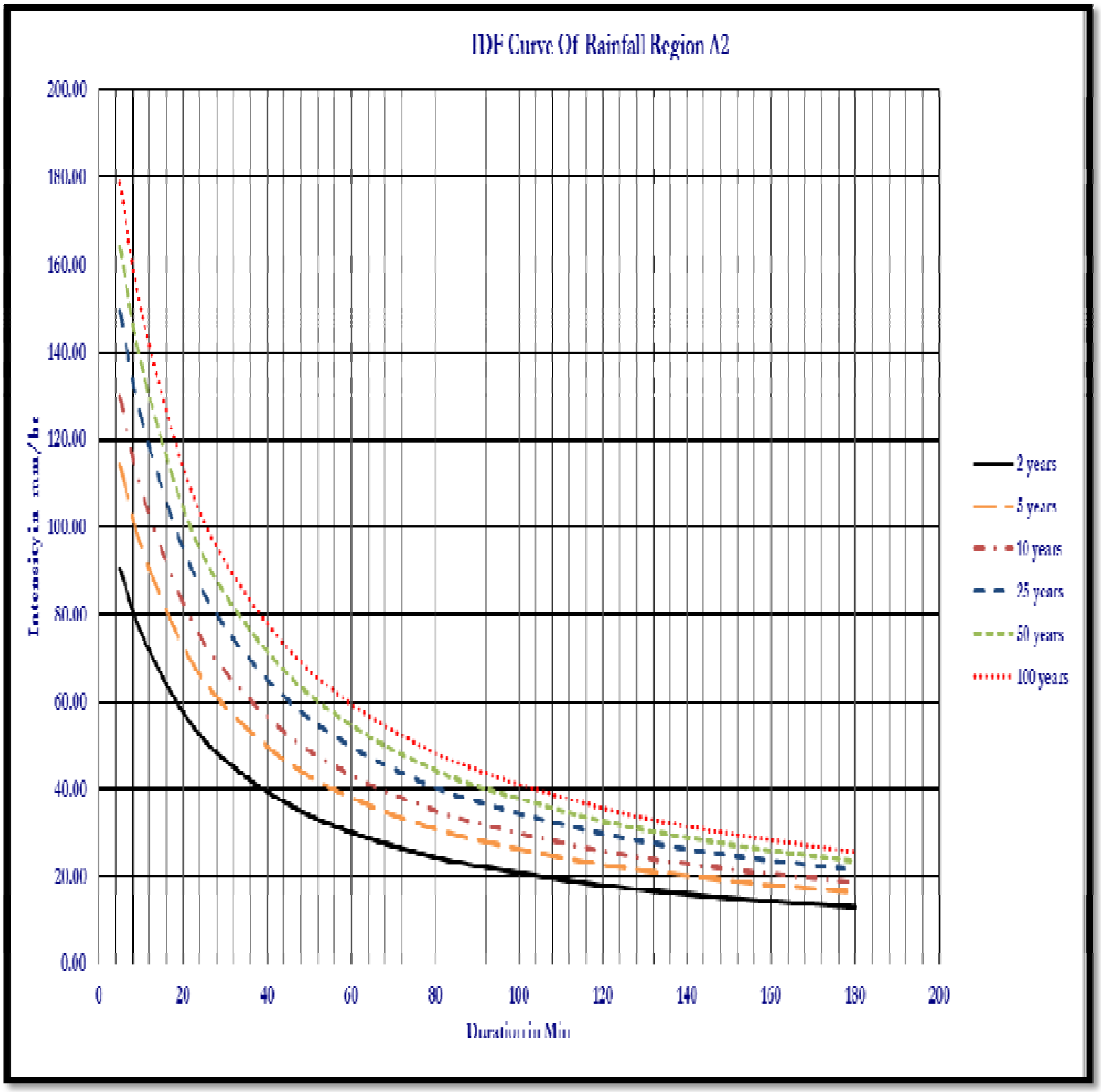


Figure1: Intensity duration frequency curve for region A2 (ERA DDM 2002)

Figure 2: Rainfall Regions of Ethiopia

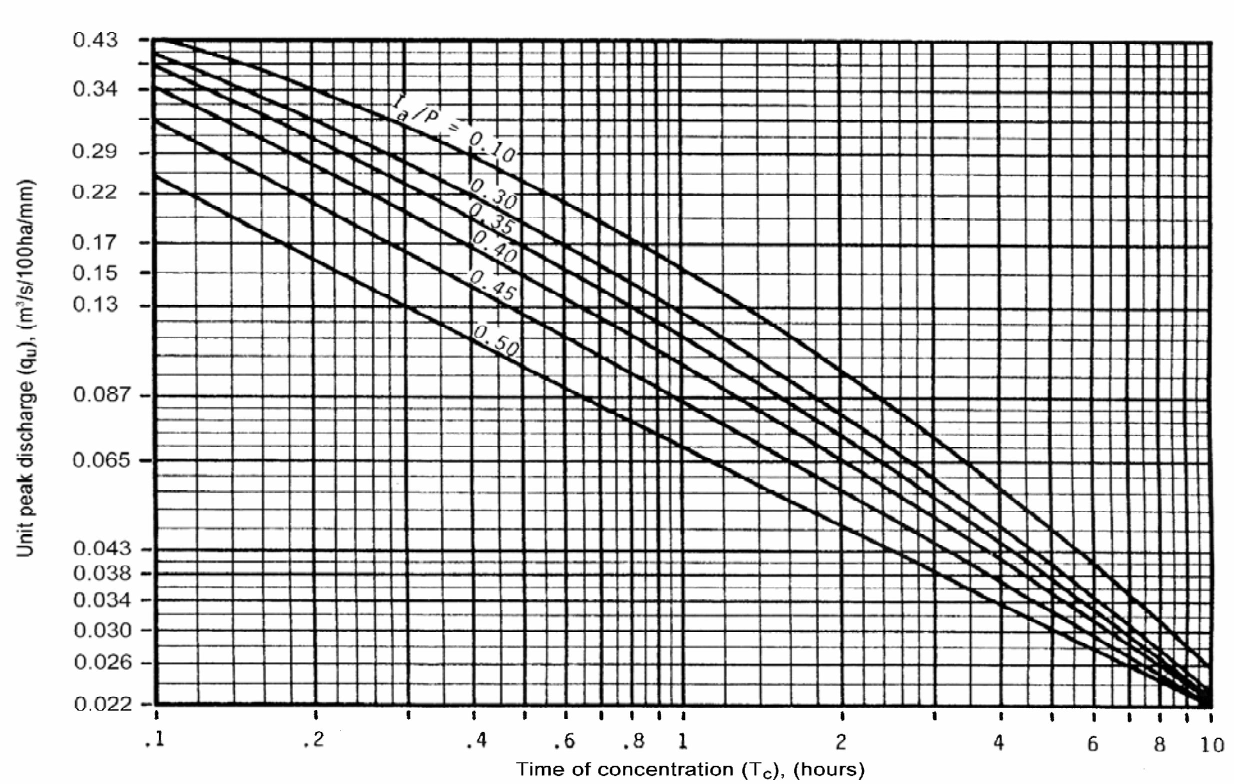


Figure 3: Unit Peak Discharge, Type II rainfall

Table1: Storm Design Return Period-years for Severe Risk Situations (ERA DDM, 2011)

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	15	10	5
Culvert diameter <2meter	25	20	20	10
Large culvert diameter >2meter	50	25	20	10
Gabion abutment bridge	50	25	20	-
Short span bridge(<15meter)	50	50	25	-
Masonry arch bridge	50	50	25	-
Medium span bridge (15-50 meter)	100	100	50	-
Long span bridge(>50meter)	100	100	100	-

Table 2: Table 5.19: Runoff coefficient values for different drainage areas

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

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

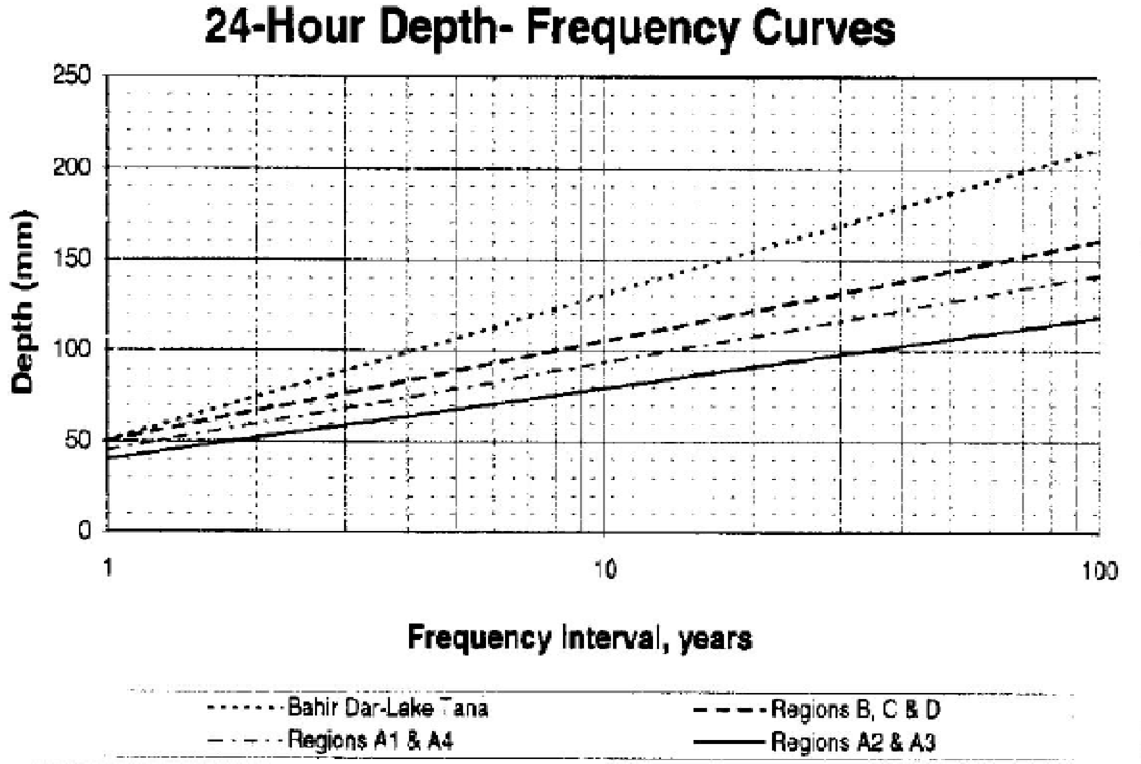
Table 3: Design Storm Frequency (yrs) by Geometric Design Criteria - (ERA DDM, 2002)

Structure Type	Geometric Design Standard			
	DS1/DS2	DS3/DS4	DS5/DS6/DS7	DS8/DS9/DS10
Gutters and Inlets	10 or 5	2	2	
Side Ditches	10	10	5	5
Ford/low water bridge				5
Culvert, pipe (span<2m)	25	10	5	5
Culvert (2m<span<6m)	50	25	10	10
Short span bridges (6m<span<15m)	50	50	25	25
Medium span bridges(15m<span<50m)	100	50	50	50
Long span bridges (span>50m)	100	100	100	100
Check/review flood	200	100	100	100

Table 4: Runoff Curve Numbers (ERA DDM 2011)

Land use			A	B	C	D
Cultivated land	Without conservation treatment		72	81	88	91
	With conservation treatment		62	71	78	81
Pasture land	Poor condition		68	79	86	89
	Good condition		39	61	74	80
Meadow			30	58	71	78
Wood or forest	Thin stand, poor cover, no mulch		45	66	77	83
	Good cover		25	55	70	77
Open spaces, lawns, parks	Good condition, grass cover >75% of area		39	61	74	80
	Fair condition, grass on 50-75%		49	69	79	84
Urban districts	Commercial and business areas, 85% impervious		89	92	94	95
	Industrial districts, 70% impervious		81	88	91	93
Residential	Average lot size	Average % impervious				
	< 0.05 hectares	65	77	85	90	
	0.1 hectares	38	61	75	83	
	0.2 hectares	25	54	70	80	
	0.4 hectares	20	51	68	79	
	0.8 hectares	12	46	65	77	
Paved roads with curbs and storm drains, paved parking areas, roofs.			98	98	98	98
Gravel roads			76	85	89	91
Earth roads			72	82	87	89
Open water			0	0	0	0

Table 5: 24 hour Depth-Frequency Curve (ERA DDM, 2011)



Region	24 HOUR DEPTH (mm) vs. FREQUENCY (yrs) TABLE					
	2	5	10	25	50	100
A1, A4	60	79	93	113	127	142
A2, A3	52	67	79	95	107	118
B and C	65	84	98	118	132	147
D	67	89	105	127	144	161
Bahir Dar	74	106	131	163	187	211

Table 6: Runoff Curve Numbers- Urban Areas

Cover description	Curve numbers for hydrologic soil groups				
Cover type and Hydrologic condition	**Average % impervious area [2]**	**A**	**B**	**C**	**D**
Open space (lawns, parks, cemeteries, etc.)[3]					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50 % to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm drains (excluding right-of way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Desert urban areas:					
Natural desert cover		63	77	85	88
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
0.05 hectare or less	65	77	85	90	92
0.1 hectare	38	61	75	83	87
0.135 hectare	30	57	72	81	86
0.2 hectare	25	54	70	80	85
0.4 hectare	20	51	68	79	84
0.8 hectare	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)		77	86	91	94

Appendix B: Roughness and Runoff Coefficient Values

Table1: Values of Roughness Coefficient (n) for Uniform Flow

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense Weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Backhoe-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2 Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1 Minor streams (top width at flood stage < 30 m)			
a. Streams on Plain			
1. Clean, straight, full stage, no rims or deep pools	0.025	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but some weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8 Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravel, cobbles, and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2 Flood Plains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050

Type of Channel and Description	Minimum	Normal	Maximum
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of spouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3 Major Streams (top width at flood stage > 30 m). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	--	0.060
b. Irregular and rough section	0.035	--	0.100
4 Various Open Channel Surfaces			
a. Concrete	0.012-	0.020	
b. Gravel bottom with:			
Concrete	0.020		
Mortared stone	0.023		
Riprap	0.033		
c. Natural Stream Channels			
Clean, straight stream	0.030		
Clean, winding stream	0.040		
Winding with weeds and pools	0.050		
With heavy brush and timber	0.100		
d. Flood Plains			
Pasture	0.035		
Field Crops	0.040		
Light Brush and Weeds	0.050		
Dense Brush	0.070		
Dense Trees	0.100		

Table 2: Ia Values for Runoff Curve Number

Curve Number	I_a (mm)	Curve Number	I_a (mm)	Curve Number	I_a (mm)
40	76.2	60	33.9	80	12.7
41	73.1	61	32.5	81	11.9
42	70.2	62	31.1	82	11.2
43	67.3	63	29.8	83	10.4
44	64.6	64	28.6	84	9.7
45	62.1	65	27.4	85	9.0
46	59.6	66	26.2	86	8.3
47	57.3	67	25.0	87	7.6
48	55.0	68	23.9	88	6.9
49	52.9	69	22.8	89	6.3
50	50.8	70	21.8	90	5.6
51	48.8	71	20.6	91	5.0
52	46.9	72	19.8	92	4.4
53	45.1	73	18.8	93	3.8
54	43.3	74	17.9	94	3.3
55	41.6	75	16.9	95	2.7
56	39.9	76	16.1	96	2.1
57	38.3	77	15.2	97	1.6
58	36.8	78	14.3	98	1.0
59	35.3	79	13.5	99	0.4

Table 3: Roughness Coefficient Values (Manning's n) for sheet flow

Surface Description	n[1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grasses:	
Short grass	0.15
Dense Grasses	0.24
Range (natural)	0.13
Woods:[2]	
Light underbrush	0.40
Dense underbrush	0.80

[1] The n values are a composite of information compiled by Engman (1986).
[2] When selecting n, consider cover to a height of about 0.03 m. This is the only part of the plant cover that will obstruct sheet flow.

Table 4: Coefficients for SCS Peak Discharge Method (highway hydrology manual)

Rainfall Type	Ia/P	C_0	C_1	C_2
I	0.1	2.3055	-0.5143	-0.1175
	0.2	2.23537	-0.5039	-0.0893
	0.25	2.18219	-0.4849	-0.0659
	0.3	2.10624	-0.4570	-0.0284
	0.35	2.00303	-0.4077	0.01983
	0.4	1.87733	-0.3227	0.05754
	0.45	1.76312	-0.1564	0.00453
	0.5	1.67889	-0.0693	0.00000
IA	0.1	2.03250	-0.3158	-0.1375
	0.2	1.91978	-0.2822	-0.0702
	0.25	1.83842	-0.2554	-0.0260
	0.3	1.72657	-0.1983	0.02633
	0.5	1.63417	-0.0910	0.0000
II	**0.1**	**2.55323**	**-0.6151**	**-0.1640**
	0.3	**2.46532**	**-0.6226**	**-0.1166**
	0.35	**2.41896**	**-0.6159**	**-0.0882**
	0.4	**2.36409**	**-0.5986**	**-0.0562**
	0.45	**2.29238**	**-0.5701**	**-0.0228**
	0.5	**2.20282**	**-0.5160**	**-0.0126**
III	0.1	2.47317	-0.5185	-0.1708
	0.3	2.39628	-0.512	-0.1325
	0.35	2.35477	-0.4974	-0.1199
	0.4	2.30726	-0.4654	-0.1109
	0.45	2.24876	-0.4131	-0.1151
	0.5	2.17772	-0.3680	-0.0953

Appendix C: Meteorological Data

Station name: Holota

Element: Monthly total rainfall (mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980	80.5	79.2	90.9	103.9	117.5	95.6	140.6	136.8	119.7	83.8	68.2	70.4	1187.1
1981	80.5	79.2	90.9	103.9	117.5	95.6	140.6	136.8	119.7	83.8	68.2	70.4	1187.1
1982	84.8	83.2	89.2	80.9	97.0	80.5	127.0	149.0	89.4	95.0	89.5	70.4	1136.0
1983	81.7	76.5	95.2	128.4	142.3	104.3	151.5	132.1	134.3	85.9	69.3	80.3	1281.9
1984	74.9	65.9	77.2	70.4	108.4	127.4	128.3	110.1	97.9	71.6	73.0	74.6	1079.7
1985	72.0	63.6	78.8	104.5	121.2	85.1	163.4	164.1	121.8	72.7	70.0	71.7	1189.0
1986	70.4	94.4	102.7	97.0	114.8	152.7	156.7	144.4	117.9	91.6	68.2	70.4	1281.3
1987	6.9	55.9	141.6	142.3	181.9	103.8	182.0	261.8	142.2	60.6	60.5	94.8	1434.4
1988	30.3	80.7	26.0	99.6	27.4	108.0	291.6	283.7	239.9	31.9	0.0	15.9	1235.1
1989	57.1	121.0	78.0	69.8	8.3	74.9	242.7	279.3	117.5	3.0	59.4	186.8	1297.8
1990	64.1	169.7	74.9	97.7	55.5	141.5	262.4	333.2	155.4	64.8	69.3	69.1	1557.6
1991	21.1	74.8	117.8	21.3	61.9	87.9	232.1	201.3	109.0	2.6	61.7	5.8	997.2
1992	47.4	33.5	58.8	95.0	34.6	115.4	183.2	315.2	118.9	36.3	0.6	77.8	1116.7
1993	77.6	91.1	80.8	117.3	118.4	111.2	135.8	187.5	150.9	77.8	68.2	70.4	1286.9
1994	70.4	63.6	101.2	92.4	99.2	111.5	137.6	183.4	120.4	74.3	71.9	70.4	1196.4
1995	71.5	73.1	75.1	155.9	96.7	87.5	124.1	136.2	101.6	70.4	68.2	79.1	1139.6
1996	79.2	66.8	107.0	96.5	104.4	115.5	150.1	159.6	127.9	76.6	68.5	70.4	1222.5
1997	90.6	63.6	70.4	109.2	85.2	105.1	132.9	129.3	100.0	102.0	91.6	71.3	1151.3
1998	92.8	69.5	101.5	76.2	95.4	122.0	146.3	153.9	125.3	91.3	68.2	70.4	1213.0
1999	75.5	63.6	78.7	119.3	93.0	115.2	141.2	131.7	99.9	126.3	69.5	70.4	1184.5
2000	70.4	65.9	73.0	108.6	97.3	118.0	114.7	144.3	113.9	84.7	79.7	71.5	1142.3
2001	72.7	76.2	107.5	78.7	105.8	152.8	144.9	117.7	98.3	80.2	69.0	70.4	1174.2
2002	105.7	87.5	98.2	80.8	82.4	125.1	135.9	119.9	93.2	70.4	68.2	89.6	1157.0
2003	76.6	82.8	103.9	119.0	75.3	114.3	141.7	135.2	105.0	70.4	68.8	75.1	1168.0
2004	104.2	68.0	77.1	135.1	88.6	106.8	132.5	133.1	123.8	76.4	69.7	70.8	1186.1
2005	91.0	64.4	107.8	82.4	97.2	104.6	151.9	146.0	126.3	74.4	72.1	70.4	1188.5
2006	70.4	68.4	170.6	128.1	112.2	142.5	206.2	267.2	117.3	76.5	68.2	74.9	1502.6
2007	76.7	71.2	82.1	78.1	119.7	145.8	143.1	127.5	106.4	81.5	68.2	70.4	1170.5
2008	70.4	67.7	70.4	73.8	101.9	115.9	162.7	144.4	102.9	83.5	87.7	70.4	1152.0
2009	88.4	65.3	74.0	84.1	84.1	99.8	136.3	156.2	113.5	70.4	68.2	83.3	1123.5
2010	2.3	42.5	90.2	74.3	79.2	96.7	280.1	250.6	188.6	2.3	26.4	19.1	1152.1
2011	2.3	41.1	44.9	34.9	78.1	146.8	242.2	272.7	170.4	2.3	17.2	2.3	1055.0
2012	2.3	2.7	8.2	99.0	46.1	59.3	285.3	199.5	226.1	2.3	2.3	8.0	941.0
2013	496.0	63.6	70.4	100.3	107.8	113.7	143.1	151.1	113.4	115.9	68.2	70.4	1613.9
2014	70.4	81.3	87.1	83.0	123.4	103.1	119.2	143.0	97.4	83.3	68.2	70.4	1130.0
Mean	78.0	71.9	85.8	95.5	93.7	111.0	168.9	178.2	125.9	67.9	61.9	68.0	1206.6

Station name: AddisAlem

Element: Monthly total rainfall (mm)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1980	33.0	48.0	63.0	89.0	71.0	168.0	223.5	242.0	138.0	1.7	8.1	6.0	1091.3
1981	33.0	48.0	63.0	89.0	71.0	168.0	223.5	242.0	138.0	1.7	8.1	6.0	1091.3
1982	52.0	49.0	42.5	65.5	69.0	72.2	237.0	267.0	72.0	64.0	112.0	40.0	1142.2
1983	18.0	65.0	29.0	102.0	151.2	110.3	190.1	272.0	140.6	29.6	14.5	42.5	1164.8
1984	2.5	0.0	32.0	12.0	138.0	157.6	155.4	150.5	107.5	3.0	2.0	8.0	768.5
1985	10.0	0.0	8.0	59.2	50.0	98.5	170.6	168.5	80.9	61.1	11.0	0.0	717.8
1986	2.5	64.3	105.0	96.0	155.0	138.0	193.0	179.0	121.0	16.0	3.0	4.5	1077.3
1987	0.0	44.0	106.0	59.0	132.0	89.0	129.0	124.0	90.0	13.0	0.0	5.5	791.5
1988	11.0	35.5	9.0	55.0	13.9	117.0	259.5	252.0	133.7	21.0	0.0	0.0	907.6
1989	32.0	61.0	80.0	104.0	38.9	126.2	250.2	256.0	210.0	10.5	0.0	24.0	1192.8
1990	9.6	131.6	55.0	55.8	99.1	111.5	189.9	342.8	196.5	13.0	0.0	0.0	1204.8
1991	0.0	79.8	102.6	43.9	16.8	144.6	370.4	353.8	193.6	0.0	0.0	24.1	1329.6
1992	41.0	50.7	72.4	85.5	32.7	75.1	323.1	385.4	234.3	36.3	5.4	8.2	1350.1
1993	10.2	12.5	13.4	116.8	88.9	168.6	359.7	367.3	168.1	44.3	0.0	0.0	1349.8
1994	0.0	0.0	34.5	46.1	43.0	218.0	250.4	228.1	155.6	0.0	17.5	8.0	1001.2
1995	0.0	39.5	47.1	144.9	94.5	245.1	402.5	352.3	136.8	6.4	0.0	7.6	1476.7
1996	46.1	7.9	129.5	70.1	160.4	163.2	371.9	326.6	123.0	6.2	0.0	2.2	1407.1
1997	23.5	0.0	17.6	70.1	23.7	88.1	243.1	164.1	86.1	58.8	81.5	0.0	856.6
1998	61.1	86.8	107.4	67.3	101.1	208.1	289.5	251.0	154.0	53.9	21.9	0.0	1402.1
1999	29.5	3.3	28.6	13.5	74.6	197.5	255.0	262.2	51.8	116.6	0.0	0.0	1032.6
2000	0.0	0.0	12.5	103.1	135.4	168.1	266.6	238.2	186.8	33.6	17.4	29.2	1190.9
2001	0.0	0.0	169.4	28.2	127.9	233.9	309.7	265.0	43.3	21.4	0.0	0.0	1198.8
2002	51.6	84.1	77.8	26.4	35.0	174.5	333.1	264.6	38.8	0.0	0.0	62.4	1148.3
2003	0.0	65.8	109.6	84.1	0.0	173.1	226.0	232.1	165.3	11.0	6.6	6.3	1079.9
2004	17.5	27.6	70.5	140.6	62.9	143.1	190.4	212.5	172.5	33.6	1.3	38.8	1111.3
2005	14.2	15.2	148.2	45.1	96.9	90.4	142.7	224.0	99.0	21.9	12.7	0.0	910.3
2006	0.0	83.5	101.0	61.1	83.2	148.9	334.3	265.4	135.5	75.7	0.0	12.1	1300.7
2007	12.1	59.3	120.1	94.9	123.4	170.1	237.3	204.3	75.0	23.1	0.0	2.3	1121.9
2008	1.9	12.5	14.6	36.9	125.6	145.1	291.5	268.1	85.6	46.9	85.4	70.3	1184.4
2009	43.2	2.8	48.6	11.3	51.5	96.1	297.6	146.5	108.3	70.3	68.0	81.7	1026.0
2010	83.0	84.2	95.5	106.0	89.2	113.0	143.2	129.7	96.8	75.2	68.0	80.8	1164.6
2011	72.7	66.9	87.1	71.2	96.6	118.7	110.4	121.8	82.7	70.3	82.6	70.3	1051.2
2012	70.3	66.2	71.7	86.9	76.6	96.3	131.8	122.8	127.8	70.3	68.0	70.3	1059.1
2013	496.0	63.5	70.3	96.6	103.5	108.5	134.9	142.0	108.2	110.8	68.0	70.3	1572.6
2014	70.3	79.2	85.2	81.2	117.4	99.1	113.7	134.8	94.0	81.8	68.0	70.3	1095.0
Mean	38.5	43.9	69.4	71.9	84.3	141.2	238.6	233.1	124.3	37.2	23.7	24.3	1130.6

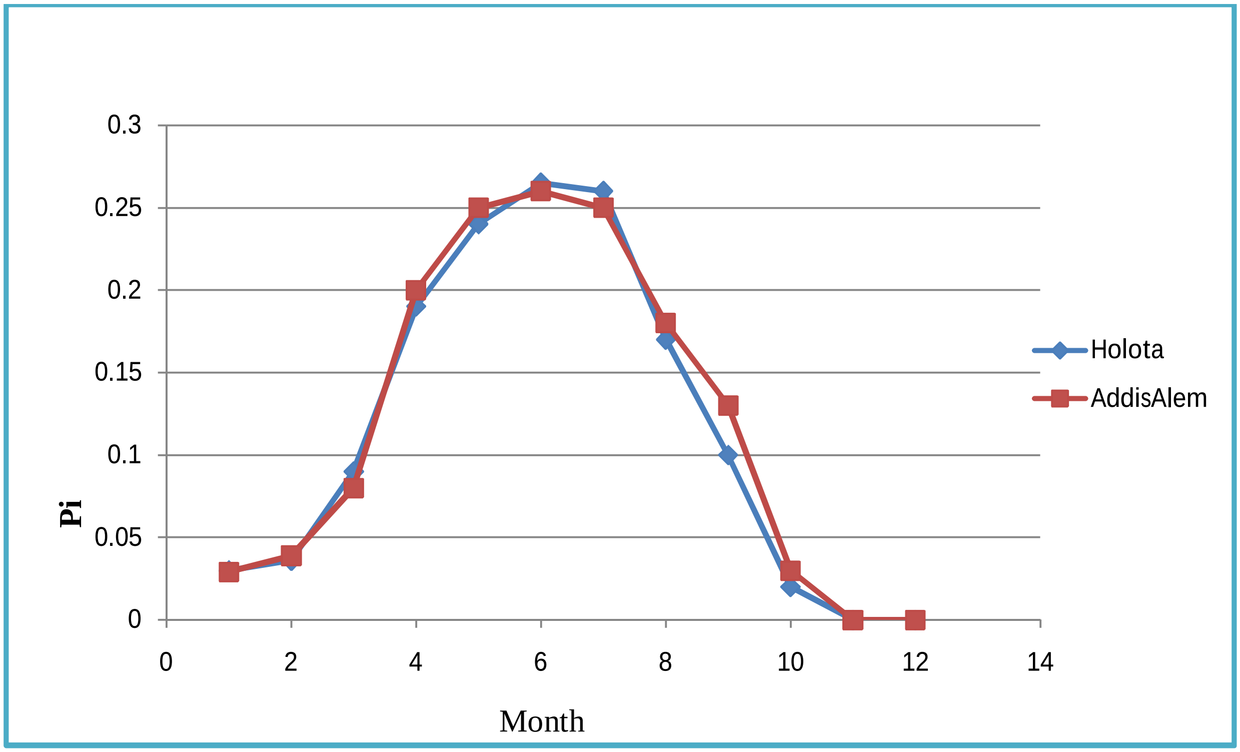


Figure C- 1: Homogeneity test for Holota and AddisAlem meteorological stations

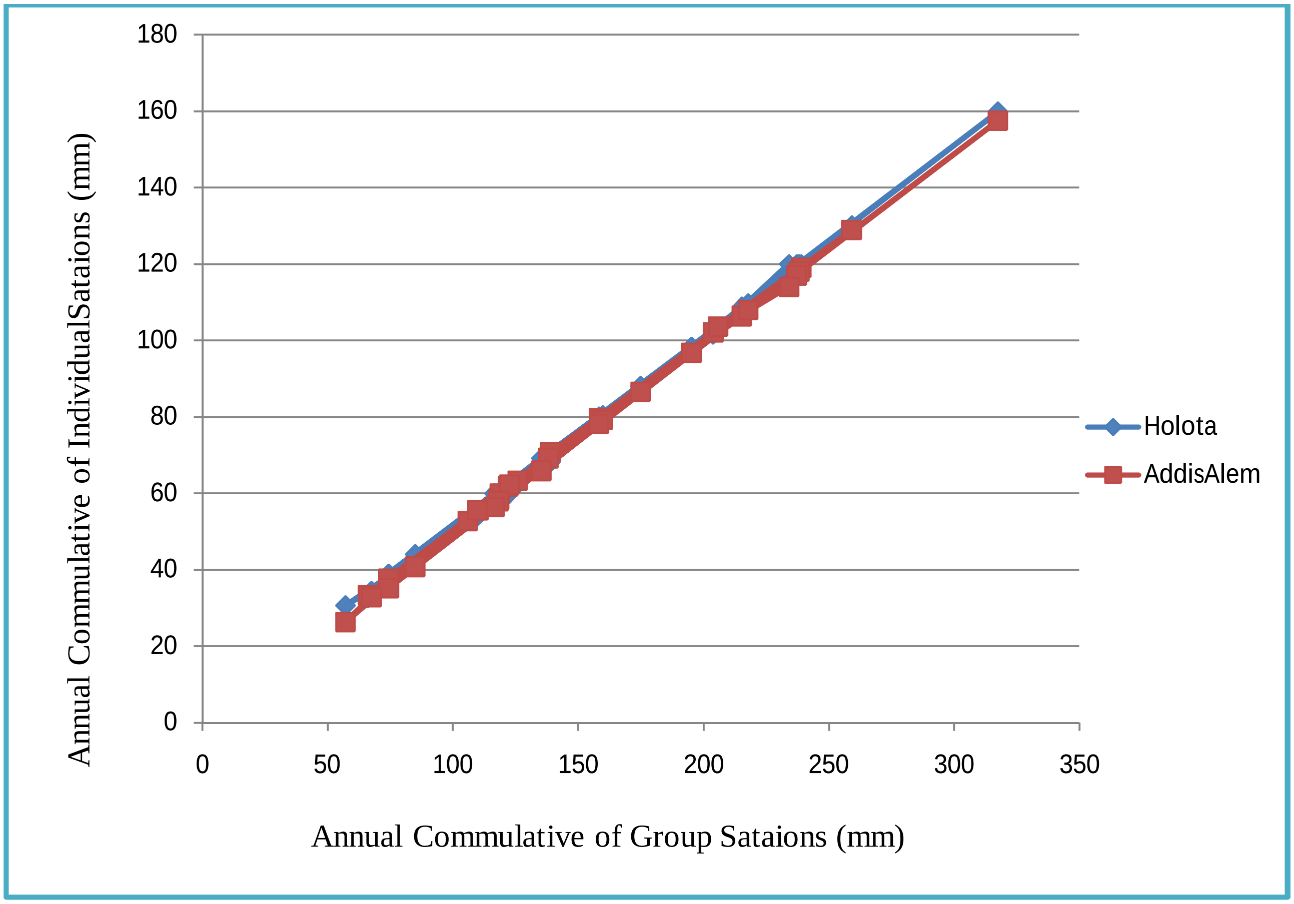


Figure C-2: Consistency test for Holota and AddisAlem meteorological stations