Addis Ababa University

Addis Ababa Institute of Technology

School of Civil and Environmental Engineering

Postgraduate Program in Hydraulic Engineering

Performance Assessment of Road Drainage Structures and Proposed Mitigation Measures: The case of Daleti-Odagodere Gravel Road in Benishangul-Gumuz Region.

By

Yifred Kassa

A Thesis Submitted to the School of Postgraduate Studies in Partial Fulfillment of the Requirements for the Degree of Master of Science in Hydraulic Engineering at Addis Ababa Institute of Technology, Addis Ababa University.

November 2013

Addis Ababa, Ethiopia

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Advisor: Dr. Agizew Nigussie

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Certification

I, the undersigned certify that I have read the thesis entitled “Performance Assessment of Drainage Structures and Proposed Mitigation Measures: The Case of Daleti-Odagodere Gravel Road in Benishangul-Gumuz Region” and here by recommend for acceptance by Addis Ababa University in Partial fulfillment of Master of Science in Hydraulic Engineering.

____

Agizew Nigussie (Ph.D)

Supervisor

Declaration and Copyright

Yifred Kassa, declares that this thesis is my own original work that has not been presented and will not be presented by me to any other University for similar or any other degree award.

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Signature

Acknowledgement

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Abstract

This thesis presents results of the assessment of drainage structures; performance on Daleti-Odagodere gravel road in Benishangul-Gumuz region and proposed mitigation measures. Mitigation measures were proposed based on ERA drainage design manuals 2002 and 2011 for Low Volume Roads.

Descriptive and exploratory methods of research were used for this thesis work. Field visits of the catchment area that contributes runoff to the drainage structures were made and the existing problems were described.

The necessary secondary data for this research are land cover map, topographical map, geological map, and feasibility study of the road before construction. The primary data are photographs that show the existing drainage structures conditions, flood level marks and information that is gathered from the residences and road desk office about the performance of the drainage structures during the rainy season. Hydrological analysis was carried out by using Rational and SCS equations. Hydraulic parameters are determined by using Manning's equation.

Structural and hydraulically failures of drainage structures and roadways were investigated. Moreover, stations of the road were investigated that require construction of minor drainage structures but not constructed.

Suitable mitigation measures were proposed in order to make the road and drainage structures serve for the intended purposes sustainably. New drainage structures were proposed where they are lacking in the existing system.

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List of Acronyms

AACRA-Addis Ababa City Roads Authority

AAM- Average Antecedent Moisture

AASHTO- American Association of State Highway and Transportation Officials

ACPA- American Concrete Pipe Association

ADOT - Arizona Department of Transportation

ASCE- American Society of Civil Engineers

BDM- Bridge Design Manual

BMS- Bridge Management System

CN- Curve Number

DC- Design Class

DDDM- Draft Drainage Design Manual

DDM- Drainage Design Manual

EMA- Ethiopian Mapping Agency (Current nomenclature)

EMA- Ethiopian Mapping Authority (Previous nomenclature)

ERA - Ethiopian Roads Authority

FDRE- Federal Democratic Republic of Ethiopia

FHWA- Federal Highway Administration

GDM- Geometric Design Manual

HDS- Hydraulic Design Series

HEC- Hydraulic Engineering Circular (FHWA)

HEC- Hydrologic Engineering Center (USACE)

HEC-RAS- Hydrologic Engineering Center for River Analysis System

HEC-HMS- Hydrologic Engineering Center for Hydrology Modeling System

HSG- Hydrological Soil Group

IDF- Intensity-Duration-Frequency

LVRs- Low Volume Roads

NBIS- National Bridge Inspection Standards

NMA- National Meteorological Agency

OCHA-Office for the Coordination of Humanitarian Affairs

USACE- United States Army Corps of Engineers

USAID- United States Agency for International Development

US NRCS- United States Natural Resources Conservation Service

USGS- United States Geological Survey

US SCS- United States Soil Conservation Service

CHAPTER 1: Introduction

1.1 General Background of Drainage Structures

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. To carry out the offsite drainage either a culvert or a bridge should be used. Culverts are closed conduits in which the top of the structure is covered by embankment at a minimum thickness of 30cm (AACRA, 2004). Bridges are mainly provided for large streams and rivers. United States National Bridge Inspection Standards (USNBIS, 1990) and ERA drainage design manuals (ERA DDM, 2002 and 2011) define bridges as those structures that have at least 6 meters of span along the roadway centerline.

The main operational differences between culverts and bridges are described in terms of economics, hydraulics, structural aspects and maintenance attention requirements (Kaan and Larry, 2004). The common properties of culverts and bridges are both increase stream velocities, turbulence of flow, aggradations, scour, and bank erosion downstream of the crossing structure (Richardson and Richardson, 1999). All culverts should be designed with headwalls and wing walls or flared-end sections at the inlet and outlet. Erosion protection should be provided at the outlet.

Adequate drainage is essential during the design of roadways, since it affects the serviceability and usable life of the roadway, including the structural strength of the pavement. Satisfactory cross-drainage facilities will limit the buildup of pond against the upstream side of roadway embankments and avoid overtopping of the roadway. If formation of pond on the carriageway occurs, hydroplaning (sliding of vehicles) becomes an important safety concern. Rapid removal of storm water from the pavement minimizes the phenomenon, which can result in the hazardous of hydroplaning. Adequate cross-slope and longitudinal grade enhance such rapid removal of storm water.

Drainage design involves providing facilities that collect, transport and remove storm water from the roadway. The design must also consider the storm water reaching the roadway embankment through natural stream flow or manmade ditches.

1.2 Background of Drainage Structures of the Study Area

Daleti-Odagodere road is a gravel road that is found in Benishangul-Gumuz region in the Northwestern part of Ethiopia in Odabuldigilu district, Dabus sub-basin. The region and the study area are characterized by extended and large volumes of rainfall. Nonetheless, environmental factors that adversely affect drainage structures performance are serious issues in Odabuldigilu district.

Daleti-Odagodere road is a low volume road and it is categorized under design standard six (DS6) or design class two (DC2). According to ERA geometric design manual 2011 for low volume roads, DC2 low volume roads carry 25-75 vehicles per day (ERA, 2011) and the road is classified under feeder road.

According to Highway Engineers and Consultants PLC, feasibility study report (2005), the road is very important for economic development of the region in general and Odabuldigilu district in particular. The road is 42 kilometers long and it has a width of 6 meters and on average 3 meters wide earthen side ditches on both sides of the road. Throughout the road length, there are many bridges and culverts even if some of them were not functioning properly during the rainy season. The road was designed to carry 50 vehicles per day on average in both directions, which makes it lie under the category of design class two (DC2).

At some stations, the drainage structures are lacking even if they are required for the road to function properly. Alignment of existing drainage structures at some stations is improper. Due to this reason, stream crosscurrents are negatively affecting the proper functioning of the drainage structures and carriageways. The overtopping runoff erodes the embankments as well as the road-wearing course. The water that infiltrates in to the carriageway over saturates the wearing course as well as sub-grade. Due to this, the bearing capacity of the carriageway is weakened and traffic interruption was common in Daleti-Odagodere road in the previous years during the rainy season.

Therefore, for uninterrupted functioning of the road, the drainage structures performance should be assessed and mitigation measures should be proposed. The proper functioning of the road drainage system will avoid the interruption of traffic movement and the economic activities will not be endangered in Odabuldigilu district.

1.3 Statement of the Problem

In Benishangul-Gumuz region on Daleti-Odagodere road, drainage structures are not properly functioning. The main causes are the inadequacy of drainage structures during the rainy season to pass the flood, poor quality construction, inappropriate site selection and improper alignment of some drainage structures with respect to the road alignment. These shortcomings cause damage to superstructures of drainage structures and stream crosscurrents are significant factors. Improper skew i.e. improper alignment of drainage structures with respect to the natural channel and the roadway can greatly aggravate the magnitude of scour. At station 32+300 on Daleti-Odagodere road, such problem is seen on the existing bridge.

Deforestation of land occurred on both sides of the road due to the agricultural activities of investors and indigenous people. This has resulted in accelerated soil erosion and its accumulation in the drainage structures. This causes storm water to overflow on the carriageway and clogging of culverts by silts. In addition to silts, the logs and tree branches are transported to the drainage structures on the upstream side of the culverts. These are the main causes for the clogging of these drainage structures, which causes overtopping of embankment by flood. As a result, spending of a large amount of money during the rainy season is common every year for the removal of the logs, branches of trees and the silt accumulated in the drainage structures.

Runoff, which is in excess of the drainage structures capacity, overtops the road embankment and makes the road to function improperly due to erosion and ponding. The wearing-course and sub-grade of the road become weak due to high moisture content and the road could not carry traffic as the intended design requirement. Moreover, at some stations even if construction of bridge was required, culverts with inadequate rows of pipe were constructed. This created the road to be malfunction during the rainy season every year due to overtopping. To alleviate this problem, culvert and bridge drainage structures performance should be evaluated and mitigation measures should be proposed for sustainable and proper functioning based on ERA drainage design manuals 2002 and 2011 for low volume roads.

1.4 Questions of the Research Study

The fundamental questions that are addressed and investigated are:

- What are the types of drainage structures failures on Daleti-Odagodere road?
- What are the major causes of drainage structures failures on Daleti-Odagodere road?
- Are the hydraulic capacities of the different drainage elements of the road adequate?
- What mitigation measures can improve the drainage problems?

1.5 Objective of the Research Study

1.5.1 General Objective of the Study

The general objective of the study is to evaluate the performances of the existing road drainage structures and to propose mitigation measures that minimize frequent maintenance of drainage structures and roadways on Daleti-Odagodere gravel road.

1.5.2 Specific Objectives of the study

- To assess the conditions of the existing drainage infrastructure inlets, lines and outlets
- To evaluate the hydraulic capacity of the different drainage elements
- To recommend appropriate mitigation measures

1.6 Significance of the Study

The study, design and construction of road drainage structures require skilled work force and intensive financial resources. If the drainage structures fail, high investment is required to maintain them in order to avoid traffic interruption. To minimize maintenance expenses proper protection and management of these road assets is important.

Therefore, this study is beneficial to the region for future road drainage structures construction to avoid problems by assessing the performances of the existing drainage structures and proposing mitigation measures to avoid improper functioning.

The study is expected to propose appropriate solutions to the drainage systems whose implementation will contribute to the sustainability of the case study road.

The study is beneficial for academicians and researchers who conduct similar researches on other road drainage structures, soil conservation strategies, erosion and scouring prevention mechanisms and aggradations/degradations of the stream channel. It may also support policy makers in their effort to address similar problems

1.7 Scope and Limit of the Study

The thesis is limited to the performance assessment of existing drainage structures and proposing mitigation measures that are found only on Daleti-Odagodere road. The research does not include structural design of all types of drainage structures except proposing the type and size of the required drainage structures. However, hydrologic analysis and hydraulic parameters determination for drainage structures that are susceptible to failure are included in the research.

CHAPTER 2: Literature Review

2.1 General 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. 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.

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). 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).

2.1.1 Types of Culverts

Culverts can be classified into two based on their functional types, stream crossing and runoff management.

Stream crossing culvert is a drainage structure installed on the stream with recommended skewed angle, 15^0 - 45^0 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).

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.

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.

Generally, drainage structures 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.

2.1.2 Road Surface Drainage

If surface water penetrates into the road body, it reduces the load bearing capacity of the pavement, which may cause further damage of the road. To avoid these problems, it is important to secure adequate drainage of the road surface. According to ERA geometric design manual (ERA, 2011) the normal cross-slope is not less than 3% in order to dispose water from the roadway quickly that avoids infiltration of water into the roadway. If the cross-slope is less, water will get time to infiltrate into the roadway and weakens the pavement that cannot withstand traffic load.

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 & 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 natural channel slope may have problems related to both sediment deposition and bed scour, and this affects hydraulic performance.

A culvert invert slope should match the streambed slope. Placing the culvert on a flatter or steeper gradient from the natural streambed can cause sediment deposition in the barrel. It can also cause scour that removes sediment from the barrel.

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.

2.3 Backwater Effect on Road Drainage Structures

When a roadway crosses a natural drainage way, the resistance to flow of the structure may increase the water depth upstream of the drainage structure. This backwater effect may cause areas close to the drainage way to become flooded where previously they remained above the floodwaters. When dwellings or other manmade structures are close to the drainage way, a limitation placed on the maximum backwater effect tolerated for drainage structure design.

Aggradations increase the backwater effect; affect the pressure on the structure, and passes ability of the bridge (Johnson et al., 2002). Bridges seem to more readily allow sediment transport than culverts and therefore have less accumulation up stream of the crossing (Wellman et al., 2000).

2.4 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 exceeds the erosive 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 works shall not exceed the erosive velocity of the channel or the natural velocity of the channel, whichever is greater.

Table 2.1: Target Outlet Velocities

Material Downstream of Culvert Outlet	Target Outlet Velocity (m/sec.)
Rock	4.5
Stones 150mm. diameter or larger	3.5
Gravel 100mm. or grass cover	2.5
Firm loam or stiff clay	1.2-2.0
Sandy or Silty clay	1.0-1.5

Source: derived from Austroads GRD PART 5(2008)

2.5 Design Flood for Road Drainage Structures

Drainage works designed for storms having a recurrence interval of at least that are presented in Appendix C of Table 1. 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 C of Table 1. All other drainage structures checked for the storm having the next lower frequency 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.

2.5.1 The Criteria for Roadside Channels

In Ethiopia, the design discharge frequency for permanent roadside ditch linings should be according to the values that are presented in Appendix C of Table 2 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

2.6 Description and Function of Road Drainage Structures

Storm drainage facilities consist of curbs, gutters, inlets, storm drains, ditches, 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 systems,

- 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).

2.6.1 Description of Road Drainage Structures

Two different types of drainage systems 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 systems.

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 ERA Low volume Roads drainage design manual the fall of 3-5% allowed on culverts to ensure that water flows without depositing silt and other debris. 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.

2.6.2 Functions of Road Drainage Structures

Drainage structures 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 in side drains.

In steep terrain, culvert capacity is usually governed by inlet control. The water depth at the entrance conditions governs the capacity of culverts subject to inlet control. The entrance conditions include the geometry of the opening, the wing walls, head walls, the angle of wing walls & head walls and the protection of the culvert in to the headwater pond.

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).

2.7 Failures of Road Drainage Structures

The roadway shall not obstruct the general flow of surface water or stream water in any unreasonable manner to cause an unnecessary accumulation either of water flooding or water soaking uplands, or an unreasonable accumulation and discharge of surface water flooding or water soaking lowlands.

The failure of culvert occurred on Daleti-Odagodere gravel road due to inadequate capacity of the culvert. If the failure is sudden and catastrophic, it can result in injury or loss of life and property.

Water passing through undersized culverts will scour away the surrounding soil over time. This can cause a sudden failure during rain events. Degradation in streams can cause the loss of bridge piers in stream channels, as well as piers and abutments in caving banks.

2.7.1 Bridge Scour

Scour is the erosion or removal of streambed or bank material from bridge foundations due to flowing water (Kattell and Eriksson, 1998). It is the most common cause of roadway bridge failures. Every bridge over water assessed as to its vulnerability to scour in order to determine the prudent measures for that bridge and the entire inventory (Richardson and Davis, 1995). Scour can have a long-term impact on bed degradation and affect entire channel reaches (Simon and Johnson, 1999).

Hydraulic conditions and rates of erosion are vastly different at abutments and piers at any bridge site. Extent of erosion at abutments minimized, by placing them away from the riverbanks. Piers are located in the middle of peak flood zones, where flood velocity is the highest. The direction of flow is at right angles to the pier, which acts as an obstruction, with the water flowing on both of its sides. Hence, foundation all around a pier scoured. On the other hand, the foundation only on side exposed to the flow in case of an abutment may be scoured.

Total scour at bridge footings is primarily sum of degradations and aggradations, local scour and contraction scour. Degradation is a general and progressive (long-term) lowering of the channel bed due to erosion over a relatively long channel length. Local scour is due to increase in local flow velocities and turbulence levels because of obstruction caused by bridge piers and abutments to the water flow. Contraction scour is because of increased water velocity in the bridge opening due to decrease in cross-sectional area of waterway at the bridge crossing.

Figure 2.1 Typical Scour of Bridge at Abutment

Scour at a bridge crossing a river classified as general scour, contraction scour, or local scour. General scour occurs irrespective of the existence of the bridge and can occur as either long-term or short-term scour. Short-term general scour develops during a single or several closely spaced floods.

Long-term general scour has a considerably longer timescale, normally of the order of several years or longer and includes progressive degradation and (lateral) bank erosion. Degradation is the general lowering of the riverbed. Bank erosion may result from channel widening, meander migration, a change in river controls, or a sudden change in the river course.

1. General scour is a process of streambed erosion or degradation. It is associated with the natural variations in the flow and occurs irrespective of the presence of the bridge.

2. Contraction scour results from general increases of the velocities where the flow is constricted during the velocity approaches the bridge opening and is characterized by a general lowering in the bed elevation due to the contracted section. Contraction scour can be further split into two types of scour viz., live bed scour, occurs when sediment transported into the bridge area scours the streambed. The other is clear water scour occurs during clear water stages and the increased flow velocities create higher shear stresses and thus scour the streambed (Richardson and Richardson, 1999).

3. By contrast, local scour is due to changes in the local flow pattern at the bridge, which is usually associated with three-dimensional flows and vortex systems. It is also characterized by the formation of scour holes at the base of the bridge foundation. In general, local scour is a continuous process of streambed degradation that results from turbulence of water at the floodplains and underneath the bridge.

Localized scour is the combination of local and contraction scour. The types of localized scour include clear-water scour and live-bed scour. When the bed resistance upstream of the scoured area is equal to or less than the critical or threshold shear stress for the commencement of the particle motion, clear water scour occurs. The maximum scour depth in clear-water scour attained when the flow is not able to get rid of the particles from the scour hole anymore.

Live-bed scour is also known as scour with sediment transport. It occurs when general bed load is transported by the stream. Similar scour depths are achieved when the materials removed from the scour hole is equal to materials supplied to the scour hole from upstream after some time. Differentiation of the two types of scour is needed because it is the main key point of the increment of the scour hole with time and approach flow velocity (Raudkivi and Ettema, 1983).

2.7.2 Causes of Culvert Scour

1. If a culvert is blocked with debris or the stream changes course, the culvert will be inadequate to handle design flows.
2. Poor culvert location
3. Changes in upstream land use such as real estate development, deforestation, clearing
due to settlement.
4. Inadequate design or poor construction activities of culvert
5. Changes of slope, flow velocity, width and depth of channel and invert elevation

These entire scour causes may further result in excessive pond formation, washing out of roadway embankment and flooding of nearby properties.

2.7.3 Factors Affecting Scour at Culverts

The following factors must be considered for evaluating long-term scour at culverts:

1. Area of opening of the culvert
2. Flood velocity
3. Angle of flow
4. Longitudinal slope
5. Head water and tail water elevations
6. Invert elevation

2.7.4 Protection Measures of Failure on Drainage Structures

According to ERA drainage design manual 2002 a check dam, which is a low dam or weir constructed across a channel, is one of the most successful techniques for halting degradation on small to medium streams in Ethiopia. Providing erosion protection measures at structures is significant to protect against the erosive force of turbulent flow. Gabions are used to protect bridge piers, abutments, and culvert wing walls.

Longitudinal stone dikes placed at the toe of channel banks can be effective countermeasures for bank caving in degradation streams. Precautions to prevent outflanking, such as tie backs to the banks, may be necessary where installations are limited to the vicinity of highway stream crossing. In general, channel lining alone is not a successful countermeasure against degradation problems (ERA, 2002).

Current measures in use to alleviate aggradations problems at roadways include channelization, bridge modification, continued maintenance, or any combination of these. Channelization may include excavating and cleaning channels, constructing cutoffs to increase the local slope, constructing flow control structures to reduce and control the local channel width, and constructing relief channels to improve the capacity at the crossing. Except for relief channels, these measures are intended to increase the sediment transport capacity of the channel, thus reducing or eliminating problems with aggradations (ERA, 2002).

Culvert drainage structures shall be adequate to avoid hazardous flooding and failures of road or embankment structures. Poorly designed culverts are also more appropriate to become jammed with sediment and debris during medium to large-scale rain events. This can cause the road to fail, often introducing a large amount of fine sediment that can clog other structures downstream and also damage crops and property. Hard bank armoring and a proper sized structure can help to alleviate this pressure.

Providing scour protections are important at both inlet and outlet for all culverts to protect the structure from damage. Providing rock armor is significant protection measure of scour for inlets and outlets of culverts. Moreover, headwalls and end walls utilized to control erosion and scour, to anchor the culvert against lateral pressures, and to ensure bank stability. Constructing all headwalls from reinforced concrete material is significant and may be straight and parallel to the channel, however, flared or warped, with or without aprons is possible when the site and hydraulic conditions permit.

To prevent the possible piping failure, cement stabilized fill can be used to form the culvert invert bedding for a suitable length. These measures found to perform well in clayey/silty/sandy soils (Sherard et al., 1963).

2.8 Erosion Hazards at Culvert Inlets and Outlets

Erosion hazard may exist if a defined approach channel aligned with the culvert axis. Aligning the culvert with the approach channel axis will minimize erosion at the culvert inlet. When aligning the culvert with the channel neglected and modification of channel carried out to bend into the culvert, erosion can occur at the bend in the channel. Riprap or other revetment needed to protect the hazard of erosion.

At design discharge, water will normally pond at the culvert inlet and flow from this pool will accelerate over a relatively short distance. Significant increases in velocity only extend upstream from the culvert inlet at a distance equal to the height of the culvert. Velocity near the inlet is approximated by dividing the flow rate by the area of the culvert opening. The risk of channel erosion should be judged based on this average approach velocity. The protection provided should be adequate for flow rates that are less than the maximum design rate. Since depth of pondage at the inlet is less for smaller discharges, greater velocities may occur. This is especially true in channels with steep slopes where high velocity flow prevails.

Culvert inverts are sometimes placed below existing channel grades to increase culvert capacity or to meet minimum cover requirements. Hydraulic Design Series No.5 (HDS 5) (Normann, et al., 2001) discusses the advantages of providing a depression or fall at the culvert entrance to increase culvert capacity. However, the depression may result in progressive degradation of the upstream channel unless resistant natural materials or channel protection is provided.

Caution must be exercised in attempting to gain the advantages of a lowered inlet where placement of the outlet flow line below the channel would also be required. Locating the entire culvert flow line below channel grade may result in deposition problems.

Recessing the culvert into the fill slope and retaining the fill by either a headwall parallel to the roadway or by a short headwall and wing walls does not produce significant erosion problems. This type of design decreases the culvert length and enhances the appearance of the roadway by providing culvert ends that conform to the embankment slopes. A vertical headwall parallel to the embankment shoulder line and without wing walls should have sufficient length so that the embankment at the headwall ends remain clear of the culvert opening. Normally riprap protection of this location is not necessary if the slopes are sufficiently flat to remain stable when wet.

Wing walls flared with respect to the culvert axis are commonly used and are more efficient than parallel wing walls. The effects of various wing wall placements upon culvert capacity are discussed in HDS 5 (Normann, et al., 2001). Use of a minimum practical wing wall flare has the advantage of reducing the inlet area requiring protection against erosion. The flare angle for the given type of culvert should be consistent with recommendations of HDS 5.

Most inlet failures reported have occurred on large, flexible-type pipe culverts with projected or mitered entrances without headwalls or other entrance protection. When soils adjacent to the inlet are eroded or become saturated, pipe inlets can be subjected to buoyant forces. Lodged drift and constricted flow conditions at culvert entrances cause buoyant and hydrostatic pressures on the culvert inlet edges that, while difficult to predict, have significant effect on the stability of culvert entrances.

2.8.1 Erosion Hazards at Culvert Outlets

Erosion at culvert outlets is a common condition. Determination of the local scour potential and channel erosion should be standard procedure in the design of all highway culverts. Culvert outlet velocity is the primary indicator of erosion potential.

Local scour is the result of high-velocity flow at the culvert outlet, but its effect extends only a limited distance downstream as the velocity transitions to outlet channel conditions. Natural channel velocities are usually less than culvert outlet velocities because the channel cross-section, including its flood plain, is generally larger than the culvert flow area. Thus, the flow rapidly adjusts to a pattern controlled by the channel characteristics.

Long, smooth-barrel culverts on steep slopes will produce the highest velocities. These cases will require protection of the outlet channel at most sites without any doubt. However, protection is also often required for culverts on mild slopes. For these culverts flowing full, the outlet velocity will be critical velocity with low tail-water and the full barrel velocity for high tail-water. Where the discharge leaves the barrel at critical depth, the velocity will usually be in the range of 3 to 6 m/s (FHWA, 2006).

A common mitigation measure for small culverts is to provide at least minimum protection and then inspect the outlet channel after major storms to determine if the protection must be increased or extended. Under this procedure, the initial protection against channel erosion should be sufficient to provide some assurance that extensive damage could not result from one runoff event.

Culverts are generally constructed at crossings of small streams, many of which are eroding to reduce their slopes. This channel erosion or degradation proceeds in a uniform manner over a long length of stream or it may occur abruptly with drops progressing upstream with every runoff event. Information regarding the degree of instability of the outlet channel is an essential part of the culvert site investigation. If substantial doubt exists as to the long-term stability of the channel, measures for protection should be included in the initial construction (FHWA, 2006).

Standard practice is to use the same end treatment at the culvert entrance and exit. However, the inlet is designed to improve culvert capacity or reduce head loss while the outlet structure should provide a smooth flow transition back to the natural channel or into an energy dissipater (FHWA, 2006). Outlet transitions should provide uniform redistribution or spreading of the flow without excessive separation and turbulence. Therefore, it may not be possible to satisfy both inlet and outlet requirements with the same end treatment or design.

2.9 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).

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. Constructing drainage structures on hard basement avoids scouring problem.

The culvert skew shall not exceed 45^0 as measured from a line perpendicular to the roadway centerline. Culvert skews should be constructible with standard designs of 15^0, 30^0 and 45^0 skew (ADOT, 2007). 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). 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. 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).

CHAPTER 3: Description of the Study Area

3.1 Location

The study area is located in the Northwestern part of Ethiopia in Benishangul-Gumuz region in Asossa zone in Odabuldigilu district. The geographical location of the region is 10°38′20″N35°43′59″E. The study area is located in geographic coordinates between 9^{0}58'N to 10^{0}31'N latitude and 34^{0}11' to 35^{0}43'E longitude. It is bordered with Menesibu district of Oromia region to the south and east, Menge district to the west and Kamash zone to the north. The region covers with an estimated area of 50,380 square kilometers and the area of Odabuldigilu district is estimated 1,387.19 square kilometers.

Benishangul-Gumuz region is divided in to three administrative zones and one special district. The administrative zones have nineteen districts. Odabuldigilu district is one of the nineteen districts that is found in Asossa zone, which is economically and in any infrastructure the backward district when it is compared with other districts in the region. The only engineered road in the district is Daleti-Odagodere gravel road.

Figure 3.1: Location Map of the Study Area

3.2 Topography

Altitude of the study area ranges from 580 meters to 2,731 meters above mean sea level. Odabuldigilu district is located on the eastern slopes of the Dabus river, with elevations ranging from approximately 2000 meters above mean sea level in the east to just under 1000 meters at the bottom of the Dabus valley.

3.3 Climate

3.3.1 Temperature

Benishangul-Gumuz region has three major climatic zones viz., 75% lowland, 24% temperate and 1% highland. The mean annual temperature of Odabuldigilu district ranges from lowest of 25^0c to the highest of 38^0c during the months of January to May. The highest wind speed is observed during the hot seasons of the year. According to FDRE Ministry of Agriculture land administration and use directorate agro-climatic classification, the study area is in the warm sub-humid region of the country.

3.3.2 Rainfall

Benishangul-Gumuz region is characterized by high amount of annual rainfall. The rainfall ranges from 800mm to 2000mm and long duration of rainy months, May to October, whereas the average annual rainfall in the study area is 1000mm with maximum and minimum rainfall being 1300mm and 906mm respectively.

3.4 Soil Characteristic

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 thesis, since the FAO classification system is recent, the FAO classification (FAO, 1998) is selected.

The study area is found on the Northwestern part of Ethiopia. The type of soil on the study area is Nitisols (FAO, 1998) that covers almost 100% of the total soil coverage. In Nitisols about 70% of the soil is silt loam of hydrologic soil group B and the remaining 30% is clay (FAO, 1998) of hydrologic soil group C.

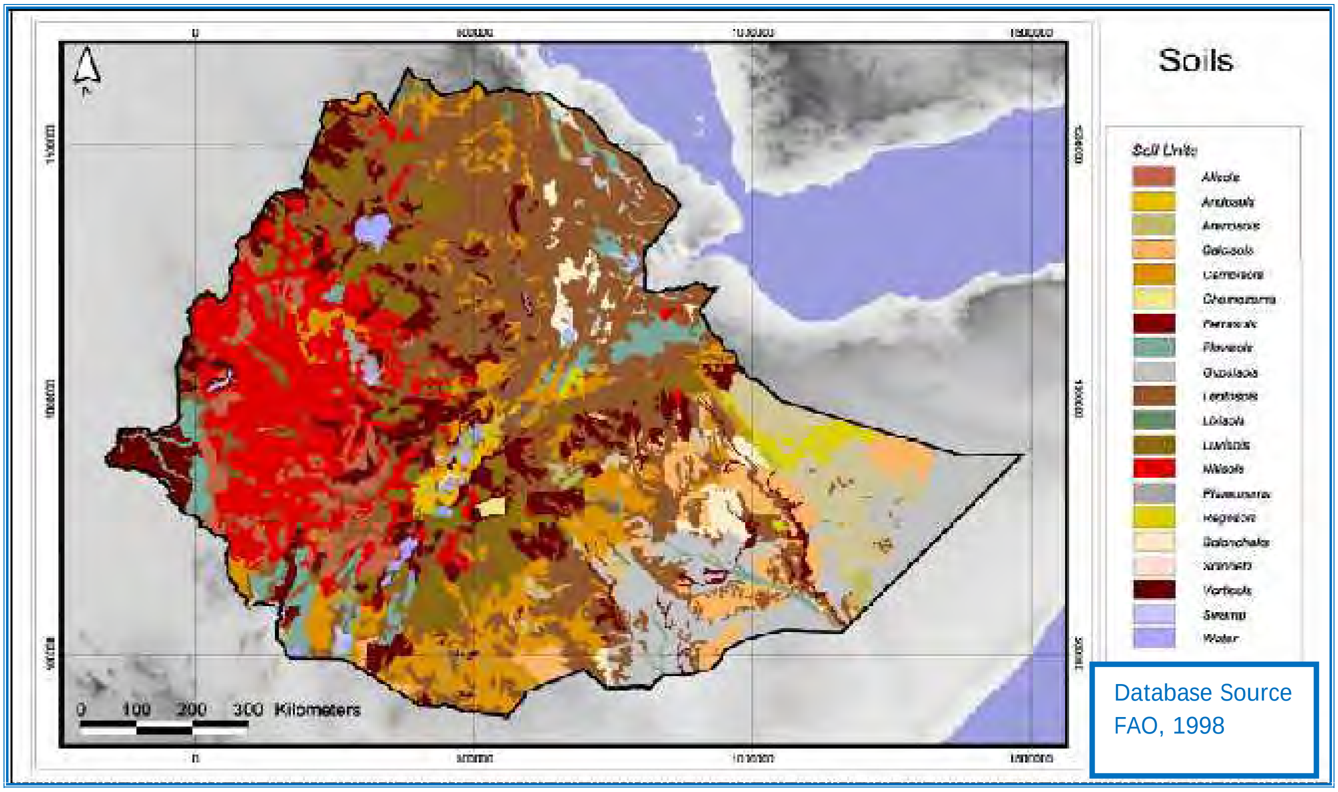

Figure 3.2: Soils in Ethiopia (FAO, 1998)

3.5 Demography

The 2007 national census reported a total population of 54,584, for Odabuldigilu district. From the total population 28,885 were men and 25,699 were women; 3,165 or 5.8% of its population were urban dwellers. The population density was 39.35 people per square kilometer. The majority of the inhabitants were Muslims, with 67.53% of the population reporting they observed this belief, while 27.37% of the population was Protestant and 4.14% practiced Ethiopian Orthodox Christianity.

Based on figures from the Central Statistical Agency in 2005, this district has an estimated total population of 29,604, of whom 15,282 were men and 14,322 were women. With an estimated area of 1,387.19 square kilometers, Odabuldigilu District had a population density of 21.3 people per square kilometer, which is greater than the Zone average of 19.95

The 1994 national census reported a total population of 22,320 in 4,743 households, of whom 11,573 were men and 10,747 were women; no urban dwellers were recorded in this district. The population density was 16.09 people per square kilometer. The population density increased rapidly from 16.09 to 39.35 people per square kilometer from 1994-2007.

The three largest ethnic groups reported in Odabuldigilu district were the Berta (77.7%), the Oromo (18.6%), and the Gumuz (3.4%); all other ethnic groups made up 0.3% of the population. The language of Berta (Wutawutigna) is spoken as a first language by 77.4%, 20% speak Oromiffa, and 2.4% speak Gumuz; the remaining 0.2% spoke all other primary languages reported.

Concerning education, 2.73% of the population were considered literate, which is less than the Zone average of 18.49%; only 0.55% of children aged 7-12 were in primary school, whether the children aged 13-14 were in junior secondary school, nor were any of the inhabitants aged 15-18 in senior secondary school. Concerning sanitary conditions, 3.7% of all houses had access to safe drinking water, and 2.7% had toilet facilities at the time of the census.

3.6 Hydrology

The hydrological feature of the study area is the seasonal rivers and streams water. The rivers and streams are not gauged and so the actual discharges are not known.

According to Odabuldigilu District Agriculture Office, land use is dominated by moderately cultivated land covering approximately 68% of the total area of the district. On both sides of the study road, the land is cultivated by investors and indigenous farmers that contribute siltation on side ditches and minor drainage structures. During rainy season, the intensity of runoff is high due to moderately and low infiltration soil conditions. The high intensity runoff carries tree branches, logs and debris that clog minor drainage structures and causes the erosion of embankments by overtopping.

3.7 Road Infrastructure

Odabuldigilu District has limited road network, small transport fleet and a low coverage of road transport services. The majority of the rural population is dependent on traditional form of transport. The limited access to transport has significantly limited agricultural productivity and the participation of rural community in the market. The only engineered road in Odabuldigilu District is Daleti-Odagodere gravel road that was constructed by Ethiopian Rural Travel and Transport Program (ERTTP).

3.8 Socio-Economy

According to a number of socio-economic indicators and parameters, services and infrastructural facilities in Odabuldigilu district are below the actual requirements. Due to shortage of educated personnels in development management, inadequate provision of roads, educational and health facilities the district lagged from other districts in the region in economic development. The main economic activities are small-scale agriculture and traditional gold mining.

CHAPTER 4: Research methods, materials and procedures

Descriptive and exploratory types of research are used for this thesis. The descriptive type of research is used to describe the existing performance condition whereas exploratory type of research is used to explore the existing performance condition of drainage structures.

Topography field visiting of the study area is carried out to determine existing performance condition of drainage structures. Observing flood marks, measuring the size of the existing drainage structures, measuring the elevation difference between river/stream bed and flood mark as well as gathering information is carried out about the overall performance of drainage structures during the rainy season.

The mathematical equations that are used to determine peak discharges are Rational and SCS equations. Recommendations in ERA 2002 and 2011 for LVRs drainage design manuals are used to determine peak discharges. These manuals are the lead information documents and main reference tools for my thesis work. The main reason that I used these manuals as the lead documents is, in our country these manuals are guidelines and best of all materials regarding drainage system design and performance evaluation.

The materials that are used for the study of the research are digital camera, GPS device, and measuring tape. All these materials are used during field visit of the study area.

4.1 Hydrological Information

The hydrological information required for estimating the design floods are obtained from previous studies, 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. To compare the developed IDF curve with generated IDF curve of the study area local rainfall data are required. However, local rainfall data are not available near the study area. The already developed regionalized IDF curve by ERA is used to determine rainfall intensity.

ERA developed four IDF curves for rainfall regions in the country. The developed curves are for A1&A4, A2&A3, B, C & D and Bahir Dar & Lake Tana rainfall regions. The study area lies on sub-region B1 and the IDF curve was constructed for B, C and D rainfall regions together. Therefore, I used the rainfall intensity from the IDF curve for the corresponding return period.

4.2 Hydrological Equations for Determining Peak Flood

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

- 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 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.

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. Based on the aforementioned concepts, rational and SCS mathematical equations are used for this thesis according to the area of the catchment.

4.2.1 Rational Method

Rational formula is particularly useful if local stream flow data do not exist (Keller and Sherar, 2003). 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 ($0.5km^2$).

Actual runoff is far more complicated than the values that are calculated by rational formula. Rainfall intensity is seldom the same over an area of appreciable size or for any substantial length of time during the same storm. Even if a uniform intensity of rainfall of duration equal to the time of concentration that occurs on all parts of the drainage area, the rate of runoff would vary in different parts of the area because of differences in the characteristics of the land surface and the non-uniformity of antecedent conditions. However, for this thesis, the same characteristics of the land surface and uniform antecedent conditions are considered.

Under some conditions, maximum rate of runoff occurs before all of the drainage areas are contributing. Temporary storage of storm water routing toward defined channels and within the channels themselves accounts considerable reduction in the peak rate of flow except on very small areas. The error in the runoff estimate increases as the size of the drainage area increases.

Due to these facts, for this thesis the rational method is not used to determine the rate of runoff for large drainage areas. For the design of highway drainage structures, the use of the rational method should be restricted to drainage areas up to 50 hectares in Ethiopia. Hence, for this thesis the maximum value of the catchment area, 50 hectares, is considered.

Peak discharge is expressed as:

$$Q = 0.00278CIA \tag{4.1}$$

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)

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

1. 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.
2. The recurrence interval of the peak discharge is the same as the recurrence interval of the average rainfall intensity.
3. 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 may be 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.

The procedures in rational method to determine peak flood are:

1. Obtain the necessary information for each sub area:

   i. Drainage area
   ii. Land use
   iii. Soil types (highly permeable or impermeable)
   iv. Distance from the farthest point of the drainage area to the point of discharge
   v. Difference in elevation from the farthest point of the drainage area to the point of discharge

2. Determine the time of concentration for the selected recurrence interval with duration equal to the time of concentration
3. Determine the rainfall intensity for the selected recurrence intervals
4. Select the appropriate runoff coefficient
5. Compute the design flow (Q= 0.00278CIA)

4.2.1.1 Runoff Coefficient

The most common definition of a runoff coefficient is the ratio of the peak rate of direct runoff to the average intensity of rainfall in a storm (Chow et al., 1988). The runoff coefficient is a dimensionless ratio intended to indicate the amount of runoff generated by a watershed given an average intensity of precipitation for a storm. While it is implied by the rational method, intensity of runoff is proportional to intensity of rainfall; calibration of the runoff coefficient has usually depended on comparing the total depth of runoff with the total depth of precipitation.

The runoff coefficient accounts for the effects of infiltration, detention storage, surface retention, evapotranspiration, surface retention, flow routing and interception. The product of runoff coefficient and rainfall intensity is the rainfall excess of runoff per hectare. The runoff coefficient should be weighted to reflect the different conditions that exist within a watershed.

$$C_w = (A_1C_1 + A_2C_2 + \text{---} + A_nC_n) / (A_1 + A_2 + \text{---} + A_n) \qquad (4.2)$$

C_w=Weighted Runoff Coefficient

C_1, C_2, --------C_n= coefficient of runoff for parts of the drainage area.

A_1, A_2, ------, A_n= parts of drainage areas with different runoff coefficients.

4.2.1.2 Rainfall Intensity

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. Furthermore, as storm duration increases average precipitation intensity decreases.

The relation between storm duration, storm intensity, and storm return interval, is represented by a family of curves called the intensity-duration-frequency curves, or IDF curves. Quantification of rainfall is generally carried out using isopluvial (Return Period) maps and intensity-duration-frequency (IDF) curves (Chow et al., 1988). Various rainfall contour maps developed to provide the design rain depths for various return periods and durations (Hershfield, 1961). 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 are used to quantify rainfall. The study area is found in the rainfall region of Ethiopia, in rainfall sub-region B1 as shown on Appendix A on Figure 6.

4.2.1.3 Time of Concentration

The time of concentration of a watershed is often defined to be the time required for a parcel of runoff to travel from the most hydraulically distant part of a watershed to the outlet. It is not possible to point to a particular point on a watershed and say, "the time of concentration is measured from this point". Neither it is possible to measure the time of concentration. Instead, the concept of time concentration is useful for describing the time response of a watershed to a driving impulse, namely that of watershed runoff.

The velocity of flow depends on the catchment characteristics and slope of the watercourse. It is estimated from appendix A on Figure 2, according to ERA drainage design manual 2011 for LVRs. The design return periods are taken from Appendix C in Table 1.

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.

$$T_c = 0.604 \left(RL/S^{0.5} \right)^{0.467}$$

(4.3) Kerby Formula

Where: T_c – Time of concentration in hours

L – Length of overland flow in kilometers

S – Slope in m/m

R – Roughness coefficient

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

(4.4) 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).

4.2.1.4 Catchment Area

Catchment area can be determined from topographic maps and field surveys. For this thesis, the catchment area is determined from topographic map of the study area. For large catchment areas, it is 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 drainage structures and assess their effects on the flood flows. For this thesis, a field inspection of existing or proposed drainage systems has been made to determine if the natural drainage divides have been altered. These alterations could make significant changes of the size and slope of the sub-catchment areas. However, it is obtained that the alterations do not occur.

4.2.2 The SCS method

The SCS runoff equation was developed to estimate total storm runoff from total storm rainfall (NRCS, 2004) that is, the relationship excludes time as a variable. Rainfall intensity is ignored. The SCS method for calculating rates of runoff requires much of the same basic data as the rational method namely catchment area, a runoff factor (curve number), time of concentration, and rainfall. However, the SCS method also considers the time distribution of the rainfall, the initial rainfall losses to interception and storage, and an infiltration rate that decreases during the course of a storm. It is therefore, potentially more accurate than the rational method and is applicable when the catchment area is larger than 50 hectares (ERA, 2011)

4.2.2.1 Catchment Area

In general, the catchment area can be determined from topographic maps 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.

4.2.2.2 Rainfall-Runoff Equation

The SCS method is based on a 24-hour storm event. The characteristics of storms defined in terms of the relationship between the percentages of the total storm rainfall that has fallen as a function of time. Three basic types of storm are defined for three levels of maximum intensity, Type I being the least intense and Type III being the most intense. In Ethiopia, according to ERA drainage design manual a Type II distribution is used (ERA, 2002). It is applicable for interior rather than the coastal regions. Hence, type II distribution is appropriate for Ethiopia (ERA, 2002) and so for the study area.

The SCS 24-hour storm distributions are based on the generalized rainfall depth-duration-frequency relationships collected for rainfall events lasting from 30 minutes up to 24 hours. Working in 30-minute increments, the rainfall depths are arranged with the maximum rainfall depth assumed to occur in the middle of the 24-hour period. The next largest 30-minute incremental depth occurs just after the maximum depth; the third largest rainfall depth occurs just prior to the maximum depth, etc. This continues with each decreasing 30-minute incremental depth until the smaller increments fall at the beginning and end of the 24-hour rainfall.

A relationship between accumulated rainfall and accumulated runoff derived by SCS for numerous hydrologic and vegetative cover conditions are important for peak discharge determination. The storm data included 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 storm rainfall.

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$ For P>0.2S

Q =0 for P≤0.2S (4.5)

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 $P \Rightarrow \infty$, P-Ia 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).

$$S = 25.4\left(\frac{1000}{CN} - 10\right)$$ (S is in millimeter) (4.6a)

$$CN = \frac{1000}{10 + S}$$ (S is in inches) (4.6b)

The relationship between I_a and S was found to be;
I_a= 0.2S (4.7a)

I_a = $$50.8\left(\frac{100}{CN} - 1\right)$$ (4.7b)

Substituting into (3.7)

$Q = [P - 50.8(100/CN - 1)]^2 / [P + 203.2(100/CN - 1)]$ (4.8)

4.2.2.3 Time of Concentration

Time of concentration is the time it takes water to flow from the edge of the catchment area to the point of interest. 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 is the ratio of flow length to flow velocity:

$$T = L/(3600V) \quad (4.9)$$

Where: T = travel time, hr

L = flow length, m

V = average velocity, m/s

The U.S. SCS formula to estimate time of concentration is:

$$T_c = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385} \quad (4.10)$$

Where, T_c – Time of concentration in hours

S_{av} – Average slope in m/m

L – Hydraulic length of catchment along the flow path from the catchment boundary

to the place where the flood needs to be determined (km)

Travel time is the time it takes water to travel from one location to another in a catchment area. T_t is a component of time of concentration.

$$T_c = T_{t1} + T_{t2} + \text{----} + T_{tm} \qquad (4.11)$$

1. Sheet Flow

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

$$T_t = [0.091(nL)^{0.8} / (P_2)^{0.5} S^{0.4}] \qquad (4.12)$$

Where, T_t=travel time, hr

n= Manning's roughness coefficient

L=flow length, m

P_2 = 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 can be determined by the following formulae according to the type of surface which water flows i.e. paved and unpaved. In these formulae, average velocity is a function of watercourse slope and type of channel.

Unpaved Surface: $V = 4.9178(S)^{0.5}$ (4.12a)

Paved Surface: $V=6.1961(S)^{0.5}$ (4.12b)

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 (3.9) is used to estimate travel time for the shallow concentrated flow segment.

3. Open Channel Flow

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 (including streams) appear on Ethiopian Mapping Agency (EMA) topographic maps (1:50,000). Average velocity is usually determined for bank-full elevation. Manning's equation or water profile information used to estimate average flow velocity. When the channel section and roughness coefficient are available, then the average velocity can be calculated by using Manning's equation. For this thesis, topographic map of the study area was used that was produced in 1:50,000 scale.

$$V= (R^{2/3}S^{1/2})/n \tag{4.13}$$

After average velocity is calculated, T_t is calculated by using equation (3.9)

$$T_c=T_{t1}+T_{t2}+T_{t3} \tag{4.14}$$

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

T_{t2}=travel time for shallow concentrated flow

T_{t3} =travel time for open channel flow

Using the calculated time of concentration, unit peak discharge is obtained from Appendix A on Figure 4. After unit peak discharge is obtained, design peak discharge is determined using the formula:

$$\text{Design Peak Discharge, } Q_p =Q_u*Q*A \tag{4.15}$$

Where, Qp= Design Peak Discharge, m^3/sec

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

Q= Direct Runoff, mm

A= Area of the catchment, ha

4.2.2.4 Runoff and Curve Numbers

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.

To describe these curves mathematically, SCS assumed that the ratio of actual retention to potential maximum retention is equal to the ratio of actual runoff to potential maximum runoff, the latter being rainfall minus initial abstraction. In mathematical form, this empirical relationship is

$$\frac{F}{S} = \frac{Q}{P - I_a} \qquad (4.16)$$

Where, F = actual retention (mm)

S = potential maximum retention (mm)

Q = accumulated runoff depth (mm)

P = accumulated rainfall depth (mm)

Ia = initial abstraction (mm)

After runoff has started, all additional rainfall becomes either runoff or actual retention (i.e. the actual retention is the difference between rainfall minus initial abstraction and runoff).

$$\mathbf{F} = \mathbf{P} - \mathbf{I}_a - \mathbf{Q} \tag{4.17}$$

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

$$CN = \frac{25400}{254 + S} \tag{4.18}$$

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 (S = ∞), which is an infinitely abstracting watershed. As the potential maximum retention (S) can theoretically vary between zero and infinity (3.18) 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. In drainage basins, the reality will be somewhere in between. Therefore, equation (3.7b) and (3.8) will be defined.

The curve number method was developed with daily rainfall data measured with non-recording gauges. The relationship therefore excludes time as an explicit variable (i.e. rainfall intensity is not included in the estimate of runoff depth).

4.2.2.5 Hydrological Soil Groups

Soils are classified into hydrologic soil groups (HSGs) to indicate the minimum rate of infiltration obtained for bare soil after prolonged wetting. The HSGs are A, B, C and D

Group A Soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels and have a high rate of water transmission (greater than 7.62mm/hr). Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam or silt loam textures can be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments (NRCS, 2007).

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (3.81mm/hr-7.62mm/hr). Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures can be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments (NRCS, 2007).

Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine-to-fine texture. These soils have a low rate of water transmission (1.27mm/hr to 3.81mm/hr). Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments (NRCS, 2007).

Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-1.27mm/hr). Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential (NRCS, 2007).

4.3 Hydraulic Equations

4.3.1 Manning's Equation

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

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

$$Q = \frac{AR^{2/3}S^{1/2}}{n} \tag{4.19}$$

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

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

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

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

R is the hydraulic radius = A/P.

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

Roughness coefficients represent the resistance to flood flows in channels and flood plains (USGS, 2009). Roughness values for flood plains can be quite different from values for channels; therefore, roughness values for flood plains should be determined independently from channel values. For this research, the Manning's roughness coefficients were used for different materials that are presented in ERA drainage design manuals 2002 and 2011 for LVRs.

4.3.2 Equations of Heads in Culvert Barrels

The head or energy to pass the quantity of water through a culvert flowing in outlet control with the barrel flowing full throughout its length is made up of three major parts. These three parts are velocity head, an entrance loss, and a friction loss. This energy is obtained from pondage of water at the entrance (FHWA, 1965).

The friction loss is the energy required to overcome the roughness of the culvert barrel (FHWA, 1965). Friction loss can be expressed in several ways. Since in most highways engineering Manning's roughness coefficient is familiar, the following expression is used:

$$H_f = \frac{n^2 V^2 L}{R^{4/3}(2g)} \tag{4.20}$$

Where, H_f = Friction loss, meter

n= Manning's roughness coefficient

V= Mean Velocity, m/sec

L= Length of Pipe, meter

R= Hydraulic Radius, meter

g= Acceleration due to gravity, 9.81m/sec^2

Energy line is the total energy at any point along the culvert barrel and pressure line is the hydraulic grade line. The energy line and the pressure line are parallel over the length of the barrel except in the immediate vicinity of the inlet where the flow contracts and re-expands. The velocity head is the difference between the energy line and the pressure line.

$$H_v = V^2/2g \tag{4.21}$$

Exit loss is one of the components of total head loss that occurs at the entrance of the culvert barrel and it is expressed as:

$$H_e = k_e V^2/2g \tag{4.22}$$

Total head loss is the sum of velocity head, friction loss and exit loss. It is expressed as:

$$H = (1+k_e+n^2L/R^{1/3})V^2/2g \tag{4.23}$$

Where, H= Total head, meter

H_v = Velocity head, meter

H_e = Exit loss, meter

k_e = Entrance loss coefficient

In computing headwater depths for outlet control, when the above bevel is used, ke equals 0.25 for corrugated metal barrels and 0.2 for concrete barrels (FHWA, 1965).

The headwater depth is the water elevation at the entrance of the culvert barrel and it is computed using the following expression.

$$H_w = H + h_o - LS \tag{4.24}$$

Where: H = total head loss, meter

ho = ½(critical depth + D) or tail water depth, whichever is greater (maximum = D)

L = culvert length

S = culvert slope

4.3.3 Equation for ford crossing structure

When the water depth is deeper than 15.25cm, it is not appropriate to recommend constructing stream-crossing ford due to the difficulty of traffic and pedestrians (McDonald and Anderson, 2003). The hydraulic equation for unvented ford for flow depth calculation is equation (2.25)

$$H = \left[\frac{nQ}{BS^{1/2}}\right]^{3/5} \tag{4.25}$$

Where, Q is design discharge

n is the streambed roughness

B is stream width

H is flow depth

S is stream slope

$$Q_e = \frac{(wH)^{5/3}\ S^{1/2}}{n(w+2H)^{2/3}} \tag{4.26}$$

Where, Q_e is the design discharge in m^3/second

w is the channel width in meter

H is the depth of flow in meter

S is the channel slope in meter/meter

n is Manning's roughness coefficient

Knowing "Q_e, w, n, and S" from data collection, the depth of flow can be determined through trial and error.

4.4 Data Types and Sources

To conduct the research both quantitative and qualitative data types are used. Study reports, topographical maps of 1:50,000 for catchment characteristics (area, slope, etc.) determination, soil, and land use/land cover map of 1:2,000,000 for determination of soil and land cover of the catchment for flood estimation, geological maps of 1:2,000,000 to determine geological formation that influence flood and channel characteristics are secondary data. The previous and the existing land cover are considered significantly. This is because during the construction period and at the existing condition the runoff entering in to the drainage structures is quite different.

Meteorological data are not collected because around the study area there is no meteorological station that recorded the rainfall, temperature, and relative humidity. The Asossa and other meteorological stations in the country are very far from the study area to obtain hydrological data for the study area accurately. The main choice is using the IDF curve developed by ERA for low volume roads drainage design manual in 2011 for Ethiopian rainfall regions.

According to ERA drainage design manual for low volume roads 2011 and ERA drainage design manual 2002, the study area is found on the hydrological sub-region of B1. Hence, the IDF curve developed for B, C and D are used.

The rivers and streams in the study area are not perennial. There are no recorded data of flow to use peak discharges like many rivers and streams in the country. The rainfall intensity is taken from the already developed IDF curve for the corresponding return periods.

4.5 Analysis method

Analysis of the collected data is 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 and flood plain characteristics including geometry and configuration, natural and artificial controls, channel modification, aggradations/degradations, and debris;
- Floodplain characteristics including vegetation cover and channel storage;
- Meteorological characteristics including precipitation amounts and type, storm cell size and distribution characteristics. It also includes storm direction, and time rate of precipitation (hyetograph).

These parameters can be 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.

Road drainage structures should be designed by analyzing hydrologic parameters of the catchment area and determining hydraulic parameters before construction. Adequate hydrologic analysis and hydraulic design is significant for road drainage structures to pass peak discharge without disturbance, damage the drainage structure and property adjoining the road crossing. Moreover, after adequate hydrologic analysis and hydraulic design proper construction is very important for the road drainage structure to function properly for the intended purpose.

5.1 Design and Construction Practice at Station 16+400

On Daleti-Odagodere road at station 16+400, the embankment on the downstream side was eroded by overtopping flood. Initially, three rows of pipe culvert that each pipe has one-meter internal diameter was constructed. The three rows of pipe culvert could not accommodate the peak discharge during the rainy season after constructed because the active channel width is greater than the span of the culvert. In order to mitigate the overtopping problem one row pipe that has one-meter internal diameter on the left side was constructed additionally. However, the constructed additional one row pipe did not mitigate the overtopping problem of the peak flood during the rainy season as a result the embankment was eroded catastrophically.

The main cause for the embankment erosion was the lack of detail flood information during rainy season and inadequate hydraulic design. The construction of the culvert was carried out without some rational or statistical assessment of the expected flow i.e. the construction was carried out by trial and error rather than considering hydrological analysis and calculating hydraulic parameters during the design stage. The hydrologic analysis is required to estimate peak discharge that is a major component of the overall design effort. In general, drainage crossings must be designed to pass the appropriate storm flows and debris or to survive overtopping.

In order to increase the hydraulic capacity, construction of wing walls and head walls are required. However, the four rows of pipe culvert that each pipe has one-meter internal diameter, wing walls were not constructed as shown on Figures 4.1 and 4.2, as a result the hydraulic efficiency decreased due to flow constriction and the peak flood overtops the embankment.

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 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+400 on Daleti-Odagodere road is under design that costs much every year for maintenance as shown on Figure 5.2. Therefore, appropriate design, and construction should be carried out in order to the road to function properly as intended for the road users. The size and type of drainage structure at this station is recommended in chapter five.

Figure 5.1: Initially constructed three Rows of Pipe Culvert at Station 16+400

Figure 5.2: Four Rows of Pipe Culvert after one Row Pipe on the left added.

5.2 Design and Construction Practice at Station 32+300

The primary function of bridges is to carry vehicles, bicycles and pedestrians. Bridge abutments and piers shall generally be aligned to match the alignment of the existing watercourse. Relocation of existing stream channels shall be avoided. Bridge structures should be on a tangent alignment if such can be accomplished without sacrificing the overall geometric design of the roadway. Tangent alignment affords easier bridge construction thereby resulting in lower structure cost.

Road drainage structures in general should be aligned properly with respect to the roadway cross-section in order to avoid traffic hazard and damage of structures by flowing water crosscurrents. On Daleti-Odagodere road at station 32+300, the alignment of bridge with respect to the roadway alignment is improper as shown on Figure 5.3. When the alignment of road structure is improper, the superstructure will be damaged by traffic hazard as a result frequent maintenance will occur on the structure and finally its serviceability life will decrease.

As it is shown on Figure 5.3, the embankment of the bridge is eroded due to the absence of wing walls construction that protects the embankment from being eroded by retaining the embankment material. The flows from the side ditches, the roadway and the adjoining land are the main causes for the erosion of embankment. Wing walls stabilize the embankment by protecting from flood erosion that flows from roadway and adjoining land that finally enters to the side ditches. Therefore, including the design and construction of wing walls are significant for long serviceability of road drainage structures as well as the road network. As I observed during the field visit, the construction practice on this bridge is very poor; as a result, it may cause catastrophic damage on life and property.

The approach roads should be straight with respect to the bridge alignment to a minimum distance of 50 meters on both sides to avoid hazard of traffic and damage of the structure by traffic. However, the bridge at station 32+300 is curve at the approaches of the bridge.

On the right side, the bridge abutment was constructed improperly as shown on Figure 5.3. The abutment is expected endangered within a short period without providing the expected benefit for the road users as intended.

Figure 5.3: Improper Alignment of Bridge with Respect of Roadway

5.3 Design and Construction Practice at Station 36+000

Fords are drainage structures that are used to pass floodwater over and under them. When the fords are constructed to flow flood over and under them, they are called vented fords, but if they are constructed to flow flood only over them, they are called unvented fords. These structures are constructed for low flow rates, at a relatively narrow, shallow stream location and should be in an area of bedrock or coarse soil for good foundation conditions. A ford can be narrow or broad, but should not be used in deeply incised drainages that require a high fill or excessively steep road approaches.

The ford cross-sectional area should be equal to or greater than the natural channel cross-sectional area. If the flow rate is high, it is not a feasible structure to be constructed because for high flow rates the depth of flood cannot be known accurately to pass traffic without hazard. This causes waiting of vehicles and pedestrians until the flood will decrease. This is waste of time for the road users.

The depth, width, and velocity of flow, as well as sediment and floating debris content, erosion on the waterway, especially of the ford surfaces and downstream of the ford, the probability of flooding over the ford and the associated risks and consequences should be considered seriously during design before deciding to construct ford on a specific site.

On Daleti-Odagodere road at station 36+000 reinforced concrete ford was constructed, however, as shown on Figure 5.4, due to inappropriate drainage structure construction for stream crossing on the aforementioned station, the ford was washed away by flood without serving for the intended purpose. The active channel width of the seasonal stream where ford was constructed is very large which accommodates high flood during the rainy season.

The active channel width is 20 meter. The width of the crossing structure should be equal or greater than the average active channel width. Based on the bank-full/active channel width, it is clear to understand the type of the crossing structure to be bridge. This stream is dry in winter season and experiences high amount of flow rate during summer season. In general, not all the streams that are found on Daleti-Odagodere road are perennial but during wet season they experience high amount of flood flow which is infeasible to construct ford as crossing structure.

As it is shown on Figure 5.4, approach gradients, including the vertical and horizontal curves are not to the standard design. Approach gradients will depend on vehicle configuration, sight distances, traction requirements, and site conditions and layout. Therefore, it is very dangerous for traffic movement during rainy season. It is also dangerous even without flood flows because traffic accident will occur due to its steepness on both sides of the ford.

Practices that should be avoided during ford construction are:

- Sharp and vertical curves should be avoided during construction of fords.
- Placing, approach fills material in the drainage channel.
- Crossing fords during high water flows.

- Placing low-water crossings on scour susceptible, fine grained soil deposits, or using designs without scour protection.
- Construction of fords should be avoided that hinders the movement of fish to upstream or downstream of streams/rivers.

Therefore, it is essential to design and construct bridge with girder/beam appropriately in order to mitigate the problems of the road users. The design should consider both hydrology of the catchment area and hydraulic parameters to avoid over or inadequate design; both are uneconomical, as well as for the appropriate function of the structure as intended.

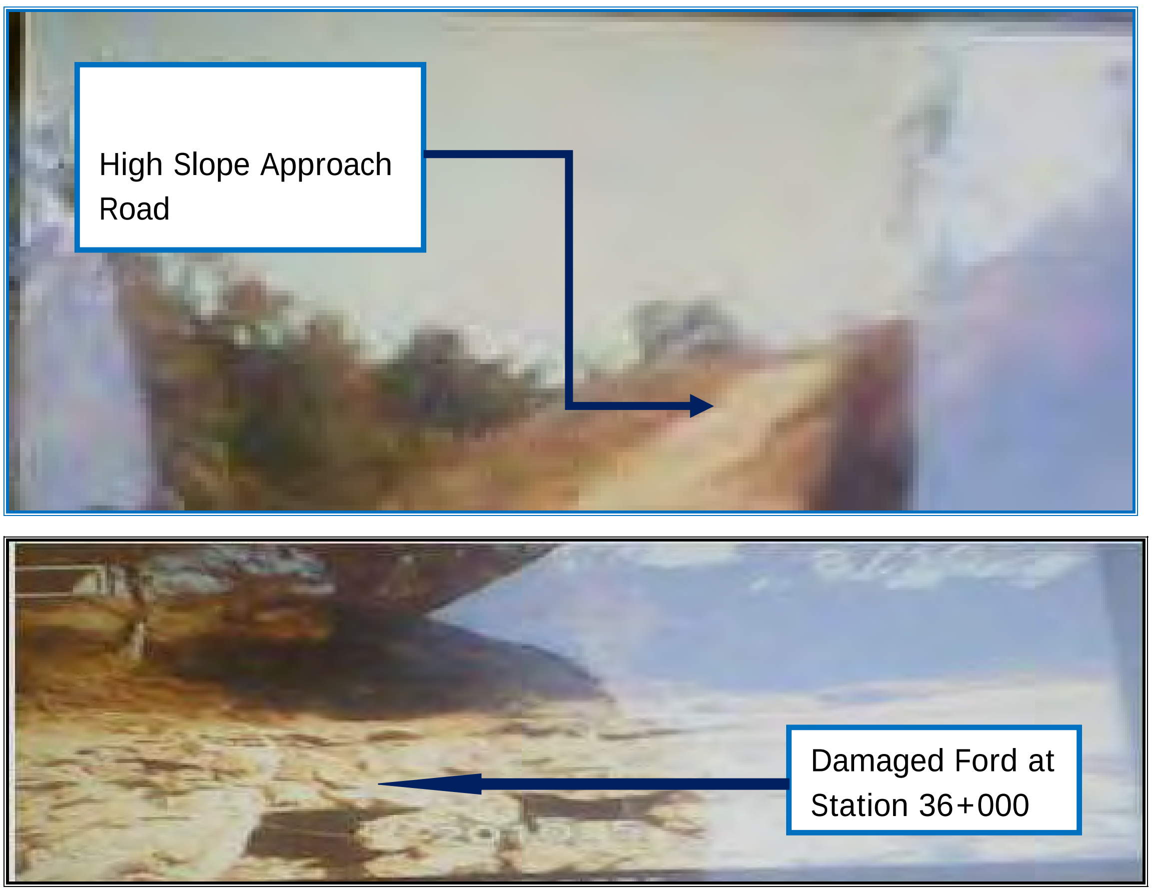

Figure 5.4: Ford Constructed at deep stream Channel at Station 36+000

5.4 Design and Construction Practice at Station 41+700

5.4.1 Design Practice

Selecting the proper location for a bridge is as important as the characteristics of the bridge itself. Problems associated with slab culvert location and construction as shown on Figure 5.5 on Daleti-Odagodere road can be alleviated by conducting a proper site investigation, paying attention to geomorphic indicators, knowing road template design needs, and understanding how streams and watersheds function.

A poor location of the slab culvert on Daleti-Odagodere road at station 41+700 as shown on Figure 5.5 makes to be susceptible to failure. Selection of good drainage structure site must involve many disciplines that include preliminary engineering, hydrology & hydraulics, geomorphologic concerns, roadway alignment, and environmental & geological concerns in order to make the structure sustainable after construction.

5.4.1.1 Preliminary Engineering

For site investigations collecting topographical maps, infrared photography, remote sensing images, GIS coverage, and aerial photographs are required. Topographic maps can help when we are locating a bridge/culvert site. Infrared maps may show areas that are prone to wet and other problem areas (springs or wetlands).

Reviewing multiple years of aerial photography is helpful when determining the stability of streams. Stable streams will show up in the same location year after year, while unstable streams may change locations or widths in photographs taken during different years. Not all these had been carried out for a slab culvert constructed at station 41+700 during site selection stage.

5.4.1.2 Hydrology and Hydraulics

Hydrological parameters calculation should be completed familiar with the local conditions and stream flows. The calculation of hydrology are carried out using equations such as rational, and SCS. Hydraulic parameters calculations are carried out using Manning's equation.

The flood damage potential of bridges and major culverts (greater than 1.22m diameter) should be reviewed for the 50-year and 100-year frequency according to ERA drainage design manuals 2002 and 2011 for LVRs. The scour potential for bridge substructures should be reviewed for the 500-year frequency or overtopping event.

5.4.1.3 Geomorphologic Concerns

The location of a stream reach in its watershed determines its channel morphology and responsiveness to natural or manmade disturbances (Gubernick et al., 2003). Slope, discharge, sediment, and vegetation are the main controlling factors, which also vary with topography and position in the watershed. The way a channel is configured provides information that can help us to decide whether a crossing is a good, safe location or an expensive, complex location that will require extensive analysis and design. Channel classification has been an excellent tool for describing stream configurations and for interdisciplinary communication.

The geomorphology of the watershed and channel play key roles during the selection of sites of drainage structures. Basic geomorphic principles allow designers to understand the geomorphic processes and difficulties presented when drainage structures cross various positions in the watershed. These processes change with location in the watershed and along the reach where the crossing will be located. Channels are extremely dynamic, responding to changes in the watershed by propagating changes downstream to upstream and vice-versa depending on the channel position in the watershed, the type of disturbance, and the channel types along the stream.

On Daleti-Odagodere road, as it was observed during the field visit and as shown on Figure 5.5 geomorphology study was not carried out because the location of the slab culvert is on the area of weak clay soil that is highly susceptible to scouring on both upstream and downstream.

5.4.1.4 Road Alignment

A good horizontal road alignment provides adequate sight distance with required horizontal curves and/or straight approaches for the design road speed. Vertical road alignments are also important. Bridges or slab culverts with a slight grade will shed water.

5.4.2 Construction Practice

Unless conditions do not permit, bridge and culvert should be constructed at a narrow channel location and should be in an area of bedrock or coarse soil and rock for a bridge site with good foundation conditions. Many bridge failures occur due to foundations placed upon fine materials that are susceptible to scour (Keller and Sherar, 2003).

The proper construction practice is important after proper design for drainage structures to function properly for the road users as intended. Only proper design by itself does not make the drainage structure to serve properly up to its design life but also proper construction practice must be carried out by appropriate personnel according to the design.

On Daleti-Odagodere road as shown on Figure 5.5, the slab culvert was damaged within a short period of time after construction due to poor construction practice as well as poor design of abutment and slab. The slab culvert was constructed with a thickness of 30cm abutment as shown on the figure from bottom to top, which cannot carry vehicles and also cannot resist the water pressure during the rainy season. Moreover, it cannot withstand the lateral pressure of soil at the bottom. On the left side of the slab culvert, the abutment is completely damaged.

The slab reinforcement arrangement and size as I observed from the damaged slab of the slab culvert during the field visit was not according to the standard specification of any bridge span and culvert construction in ERA drainage and bridge design manuals of 2002.

The site selection was not carried out based on the geomorphology investigation because the site is highly scoured both upstream and downstream of the site, this makes reconstruction problem. Therefore, to reconstruct the slab culvert, change of route corridor is required in order to find hard stratum at the channel bottom.

5.4.2.1 Factors that Affect Site Selection

The factors that should be considered in selecting site of bridges and culverts are:

- Permanency of the channel
- Presence of high and stable banks
- Narrowness of the channel
- Straight reach of the river upstream and downstream of the proposed site
- Freedom of any obstruction upstream and downstream
- Possibility of normal (right angled) crossings
- Possibility of good approach alignment

However, on Daleti-Odagodere road as shown on Figure 5.5 in addition to other constraints permanency of the channel and presence of high and stable banks are not satisfied, as a result failure of the structure occurred and became serious problem in traffic mobility.

Figure 5.5: Damaged Slab Culvert at Station 41+700

To reconstruct the slab culvert on the same site is difficult due to the silty clay loam soil condition of the existing site of the slab culvert. Hence, to reconstruct the slab culvert, relocation of the slab culvert site is important. In general, the site is not suitable for drainage structure construction. The fact is due to unavailability of hard basement which protects the drainage structure abutment from scouring without the need of scouring countermeasures.

5.5 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. On Daleti-Odagodere gravel road at stations 15+500, 22+500 and 23+100 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.

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 minor culverts, culverts that have internal diameter less than or equal to 1.22m was required, at the aforementioned three stations before pondage can be created

Figure 5.6: Weakened Carriageways at Station 15+500

CHAPTER 6- Results and Discussion

6.1 Hydrologic Analysis

On Daleti-Odagodere road, drainage structures performance assessment, the maximum peak flood is computed taking into consideration the road standard and the design life span of the structure. The existing bridges that are found throughout the road length are short span bridges. Therefore, according to Appendix C in Table 1, the design and check floods are determined (See Table 6.1 and 6.2).

Case I- During the diameter of culvert is less than 2 meters.

Table 6.1: Design and Checking of 24-hour Rainfall Depths

Culverts		Bridges	
Design Rainfall Depth	Checking Rainfall Depth	Design Rainfall Depth	Checking Rainfall depth
112mm	118mm	118mm	132mm

Case II- During the diameter of culvert is greater than 2 meters.

Table 6.2: Design and checking of 24-hour Rainfall Depths

Culverts		Bridges	
Design Rainfall Depth	Checking Rainfall Depth	Design Rainfall Depth	Checking Rainfall depth
112mm	132mm	118mm	132mm

6.2 Delineation of Watershed Area

Delineation of the catchment area for existing drainage structures and proposed drainage structures are tabulated in Table 6.3 that are determined by digital elevation model.

Table 6.3: Delineation of Existing and Proposed Drainage Structures

S.No.	Existing Drainage Structures		Proposed Drainage Structures	
	Station	Area (hectare)	Station	Area (hectare)
1	16+400	900	15+500	18.81
2	36+000	600	22+500	15.99
3	41+700	480	23+100	15.12

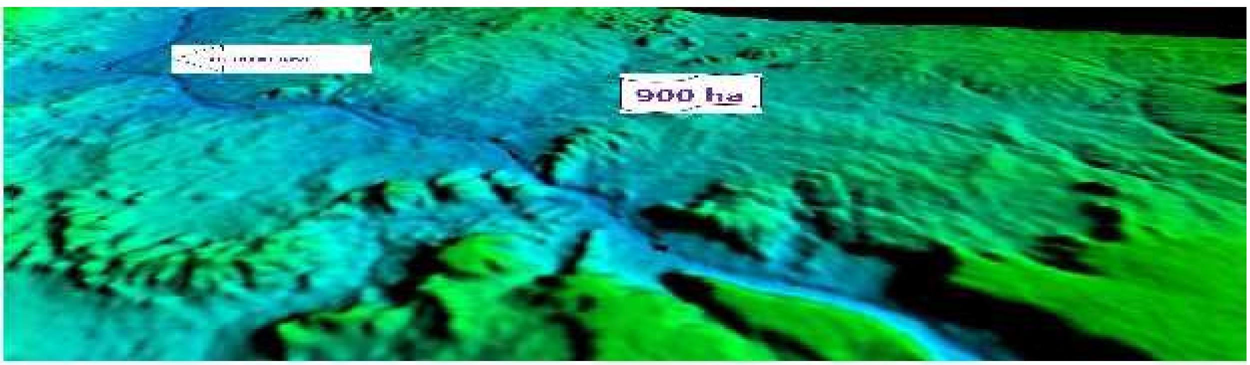

Figure 6.1: Catchment Area for Drainage Structure at Station 16+400

Figure 6.2: Catchment Area for Drainage Structure at Station 36+000

Similarly, the other catchment areas are delineated using the same procedure.

6.3 Computation of Catchment Parameters

6.3.1 Catchment Parameters at Station 16+400 Drainage Structure

After the watershed areas delineation, watershed properties like the land use coverage, soil type and curve number are computed. About 70% of the catchment is cultivated, and the remaining 30% is covered by small trees, shrubs, and scarcely distributed trees.

From Appendix C on Table 8, the runoff curve number for pasture, poor condition and cultivated land without treatment average hydrologic soil group B is 80 and hydrologic soil group C is 87. Therefore, average runoff curve number is (0.70x80) + (0.30x87) =84.9 but the nearest CN value is 85.

i. Runoff Curve Number

As per Appendix C in Table 6, hydrological characteristics of soil groups, the region is a wet antecedent moisture condition (AMC) region. From Appendix C in Table 9, $CN85_{avg}$= $CN94_{wet}$.

ii. 24-hour rainfall depth

Since the drainage structure at station 16+400 is culvert that has diameter greater than 2-meter as shown on Figure 5.2, the 24-hour rainfall depth is 112mm for design and 132mm for checking (See Appendix C in Table 5).

iii. Direct runoff depth

Direct runoff (Q) is determined from Appendix A on Figure 3, by using rainfall depth of 112mm for design, 132mm for checking and CN of 94. Therefore, Q_{20}=98mm and Q_{50}=110mm.

iv. Slope of the watershed

The average slope of the overland flow is approximated 2% (0.02) by referring from topography and field reconnaissance. Since the topographical map was produced 24-years ago, there is radical change between the existing topography and the topography from the topographical map. Hence, I used the slope that I obtained from field reconnaissance.

v. Time of concentration

a. sheet flow

The sheet flow occurs up to 100 meters. Sheet flow, natural range, slope of 0.02 m/m, and length of 100m and from Appendix B, Table 3, for range (natural) Manning's roughness coefficient is 0.13. The 2-year, 24-hour rainfall depth is determined from Appendix A in Figure 8 or Appendix C in Table 5 to be 65mm. Hence, from Equation (4.12), travel time for sheet flow is determined as:

$T_t = [0.091(nL)^{0.8} / (P_2)^{0.5} S^{0.4}]$

$= 0.42hr$

b. Shallow Concentrated Flow

For shallow concentrated flow, unpaved watershed slope is approximated 0.02m/m and length from topography map is 800m. From equation (4.12a), $V=4.9178(S)^{0.5}$ for unpaved watershed. $V=4.9178(0.02)^{0.5}$ =0.70m/sec. From equation (4.9), travel-time is determined as:

$T_t = L/ (3600V)$

$= 800/ (3600X0.70)$

$= 0.32hr$

c. Channel Flow

For channel flow, natural stream channel, winding with weeds and pools, slope is 0.01m/m, and length is 886m. By direct measuring the average bottom width of the stream channel is 2.5m, side slopes are 1V:1.5H, 20-year storm depth is observed from flood mark and measured to be 1.5m. From Appendix B in Table 3, Manning's roughness coefficient for fallow (no residue) channels is 0.050.

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

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

$= 5.875m^2$

Wetted perimeter (P_w) $= b+2y\,(1+z^2)^{0.5}$

$= 2.5+2x1.5\,(1+1.5^2)^{0.5}$

$= 7.91m$

Hydraulic radius (R) $= A/P$

$= 5.875/7.91$

$= 0.743m$

From Equation (3.16), $V = (R^{2/3}S^{1/2})/n$

$= 1.64m/sec.$

From equation (3.11), $T_t = L/\ (3600V)$

$= 0.15hr$

Total Time of Concentration (T_c) is (0.42 + 0.32+ 0.15) = 0.89 hr

By the same procedures, catchment parameters at stations 36+000 and 41+700 are determined.

6.4 Peak Discharge Computation

6.4.1 Peak Discharge at Station 16+400 Drainage Structure

Peak discharge is calculated using equation (4.15). The peak discharges on other stations are computed with the same procedure. The values for all the three drainage structures that are computed are tabulated in Tables 6.4, 6.5 and 6.6

Table 6.4: Catchment Parameters for Design and Check (Station 16+400)

Parameters of Design and Review	Design	Review
Return Periods(years)	20	50
Time of Concentration (hours)	0.89	0.89
Curve Number(CN)	94	94
Potential Maximum Retention(mm)	16.21	16.21
Initial Abstraction Ia (mm)	3.24	3.24
Design Storm (24-hr maximum rainfall)	112	132
Ia/P	0.029	0.025

Direct Runoff (mm)	98	110
Unit Peak Discharge ($m^3/s/km^2/mm$)	0.25	0.27
Peak Discharge (m^3/sec)	220.5	267.3

Table 6.5: Catchment Parameters for Design and Check (Station 36+000)

Parameters of Design and Review	Design	Review
Return Periods(years)	10	15
Time of Concentration (hours)	0.82	0.82
Curve Number(CN)	94	94
Potential Maximum Retention(mm)	16.21	16.21
Initial Abstraction Ia (mm)	3.24	3.24
Design Storm (24-hr maximum rainfall)	98	105
Ia/P	0.033	0.031
Direct Runoff (mm)	81	90
Unit Peak Discharge ($m^3/s/km^2/mm$)	0.224	0.226
Peak Discharge (m^3/sec)	108.87	122.04

Table 6.6: Catchment Parameters for Design and Check at Station 41+700

Parameters of Design and Review	Design	Review
Return Periods(years)	20	50
Time of Concentration (hours)	0.82	0.82
Curve Number(CN)	94	94
Potential Maximum Retention(mm)	16.12	16.12
Initial Abstraction Ia (mm)	3.24	3.24
Design Storm (24-hr maximum rainfall)	112	132
Ia/P	0.029	0.025
Direct Runoff (mm)	105	125
Unit Peak Discharge ($m^3/s/km^2/mm$)	0.223	0.224
Peak Discharge (m^3/sec)	112.4	134.4

6.5 Hydraulic Analysis

6.5.1 Adequacy of Existing Drainage Structures

Hydraulic calculations are carried out for drainage structures at stations 16+400, 36+000 and 41+700, using the peak discharges that are tabulated in Tables 6.4, 6.5 and 6.6 respectively.

6.5.1.1 Hydraulic Calculation for Drainage Structure at Station 16+400

The drainage structure at station 16+400 is four rows pipe culvert as shown on Figure 5.2. The hydraulic calculation is carried out using equation (4.19). In Table 6.4, the design and check discharges are 220.5m^3/sec and 267.3m^3/sec respectively. The existing culvert was installed at a slope of 0.5%.

From equation (4.19), the design diameter of the drainage structure that the flood should pass without disturbing the structure is 7.55m and the review diameter is 8.11m. The existing four rows pipe culvert has 4-meters total opening, therefore, it is not adequate. The bank-full width as shown on Figure 5.2 is wide. Due to the bank-full width, the appropriate drainage structure that is recommended is bridge that has 12-meter clear span.

The outlet velocity of flow for the existing drainage structure for design and review are erosive due to clayey and silty streambed. Therefore, it requires erosion protection treatment.

Table 6.7: Hydraulic Parameters for Proposed Drainage Structure at Station 16+400

Parameters of Design and Review	Design	Review
Return Periods(years)	25	50
Slope of natural stream (%)	0.5	0.5

Manning's Roughness Coefficient for Natural Stream	0.022	0.022
Clear Span of the Structure (m)	12	12
Outlet Velocity (m/sec)	4.59	5.56
Hydraulic Radius (m)	2.4	2.4
Area of flow cross-section (m^2)	48	48
Wetted Perimeter (m)	20	20
Maximum Freeboard (m)	1.5	1.5
Maximum Water Depth (m)	4.0	4.0

6.5.1.2. Hydraulic Calculation for Drainage Structure at Station 36+000

The drainage structure at station 36+000 on Daleti-Odagodere road is reinforced concrete ford; however, it does not have cut walls at the downstream and upstream. Unavailability of cut walls caused scouring and finally it was susceptible for washing away by flood without serving for the intended purpose. The existing ford at station 36+000 on Daleti-Odagodere road is unvented ford.

According to Table 6.5, the design and check discharges are 108.87m^3/sec and 122.04m^3/sec respectively. The stream roughness coefficient is 0.022 and average slope of the stream is 0.005. The average bottom width of the stream is 8-meter. From equation (4.25), the water depth (H) is 2.38m. Since the water depth is greater than 15.25cm, construction of ford on the channel bottom is not acceptable. The bank-full width is 20m even if the average bottom width of the streambed is 8m and the bottom elevation of stream is very deep with respect to the roadway cross-section.

Table 6.8: Hydraulic Parameters for Proposed Drainage Structure at Station 36+000

Parameters of Design and Review	Design	Review
Return Periods(years)	50	100

Slope of natural stream (%)	0.5	0.5
Manning's Roughness Coefficient for Natural Stream	0.022	0.022
Clear Span of the Structure (m)	20	20
Outlet Velocity (m/sec)	2.56	2.29
Hydraulic Radius (m)	1.92	1.92
Area of flow Cross-section (m^2)	47.6	47.6
Wetted Perimeter (m)	24.76	24.76
Maximum Freeboard (m)	1.5	1.5
Maximum Water Depth (m)	2.38	2.38

6.5.1.3 Hydraulic Calculation for Drainage Structure at Station 41+700

From Table 6.6, the design and check discharges are 112.4m^3/sec and 134.4m^3/sec respectively. The stream normal Manning's roughness coefficient is 0.022 and the stream slope is 0.008. The span of the slab culvert is 6-meter and the opening height is 4.60 meters.

Using equation (4.19) to obtain flow depth by trial and error, the depth of flow is about 3.1 meters. Therefore, the structure is safe from overtopping flood. The calculated outlet velocity is 6.04m/sec for design and 7.22m/sec for review. This velocity is erosive velocity because of the silty clay soil formation where the drainage structure is constructed (See Table 2.1). Therefore, the drainage structure is not safe from scouring and requires changing the site to reconstruct the slab culvert.

Table 6.9: Hydraulic Parameters for Proposed Drainage Structure at station 41+700

Parameters of Design and Review	Design	Review
Return Periods(years)	20	50

Slope of natural stream (%)	0.8	0.8
Manning's Roughness Coefficient for Natural Stream	0.022	0.022
Opening Span of the Structure (m)	6.0	6.0
Outlet Velocity (m/sec)	6.04	7.22
Hydraulic Radius (m)	1.52	1.52
Area of flow cross-section (m^2)	18.6	18.6
Wetted Perimeter (m)	12.2	12.2
Maximum Freeboard (m)	1.5	1.5
Maximum Water Depth (m)	3.1	3.1

6.5.2 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.

6.5.2.1 Proposing Drainage Structure at Station 15+500

From equation (4.19), the computed peak discharges are 2.49m^3/sec and 2.67m^3/sec respectively. Therefore, the proposed drainage structure is a one-row reinforced concrete pipe culvert that has length of nine-meters and internal diameter of one-meter.

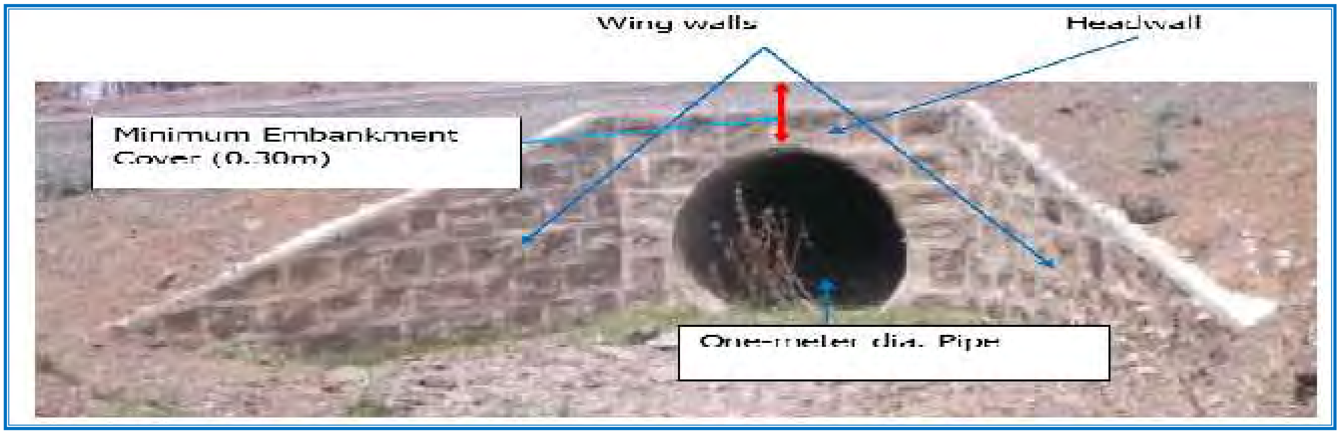

Figure 6.3: Proposed Pipe Culvert at Station 15+500

Table 6.10: Design Parameters for Proposed Drainage Structure at Station 15+500

Parameters of Design and Review	Design	Review
Return Periods (years)	20	25
Peak Discharges (m^3/sec)	2.49	2.67
Rainfall Intensity (mm/hr)	112	120
Runoff Coefficient	0.2	0.2
Slope of Pipe Culvert (%)	2.5	2.5
Manning's Roughness Coefficient for Concrete Pipe	0.017	0.017
Diameter of Concrete Pipe (m)	1.0	1.0
Outlet Velocity (m/sec)	3.69	3.69
Hydraulic Radius (m)	0.25	0.25
Area of the Opening (m^2)	0.79	0.79
Wetted Perimeter (m)	3.14	3.14
Length of Pipe Culvert (m)	9.0	9.0
Minimum Embankment Cover (m)	0.30	0.30
Maximum Inclination of Wing Walls	45^0	45^0
Head Water Depth (m)	0.84	0.84

The same procedures are followed for stations 22+500 and 23+100, therefore, the design parameters are tabulated on Tables 6.11 and 6.12

6.5.2.2 Proposing Drainage Structure at Station 22+500

Figure 6.4: Proposed Pipe Culvert at Station 22+500

Table 6.11: Design Parameters for Proposed Drainage Structure (Station 22+500)

Parameters of Design and Review	Design	Review
Return Periods (years)	20	25
Peak Discharges (m^3/sec)	2.12	2.50
Rainfall Intensity (mm/hr)	112	120
Runoff Coefficient	0.2	0.2
Slope of Pipe Culvert (%)	2.5	2.5
Manning's Roughness Coefficient for Concrete Pipe	0.017	0.017
Diameter of Concrete Pipe (m)	0.90	0.90
Outlet Velocity (m/sec)	3.45	3.45
Hydraulic Radius (m)	0.23	0.23
Area of the Opening (m^2)	0.64	0.64
Wetted Perimeter (m)	2.83	2.83
Length of Pipe Culvert (m)	9.0	9.0
Minimum Embankment Cover (m)	0.30	0.30
Maximum Inclination of Wing Walls	45^0	45^0
Head Water Depth (m)	0.74	0.74

6.5.2.3 Proposing Drainage Structure at Station 23+100

Figure 6.5: Proposed Pipe Culvert at Station 23+100

Table 6.12: Design Parameters for Proposed Drainage Structure (Station 23+100)

Parameters of Design and Review	Design	Review
Return Periods (years)	20	25
Peak Discharges (m^3/sec)	2.0	2.35
Rainfall Intensity (mm/hr)	112	120
Runoff Coefficient	0.2	0.2
Slope of Pipe Culvert (%)	2.5	2.5
Manning's Roughness Coefficient for Concrete Pipe	0.017	0.017
Diameter of Concrete Pipe (m)	0.90	0.90
Outlet Velocity (m/sec)	3.45	3.45
Hydraulic Radius (m)	0.23	0.23
Area of the Opening (m^2)	0.64	0.64
Wetted Perimeter (m)	2.83	2.83
Length of Pipe Culvert (m)	8.0	8.0
Minimum Embankment Cover (m)	0.30	0.30
Maximum Inclination of Wing Walls	45^0	45^0
Head Water Depth (m)	0.74	0.74

CHAPTER 7: Conclusion and Recommendation

Conclusions are drawn from the investigations of the results of the research. Recommendations are provided based on the findings of the results of the research.

7.1 Conclusion

The conditions of the existing drainage structures and roads were assessed through critical site observations. The capacity and adequacy of drainage structures were assessed through hydrologic and hydraulic analysis. Under hydrologic analysis, return periods, IDF curves, 24-hour rainfall analysis, delineation of watershed area, computation of catchment parameters, and peak discharge computation were carried out. The hydraulic analysis was used to assess the adequacy of existing drainage structures, and propose new drainage structures where required.

The failure of drainage structures due to inadequate size of drainage structures is caused by negligence of hydrological analysis and hydraulic design. Moreover, improper site selection for drainage structures, drainage structures at critical points where failure of carriageway due to over saturation and overtopping is expected, improper alignment of drainage structures at sections where scouring due to stream crosscurrent occurs and poor construction workmanship contribute to the problem.

7.2 Recommendation

On Daleti-Odagodere road, drainage structures failures have had serious negative impact on road users. In order to minimize these negative impacts, the following appropriate mitigation measures are recommended.

- At station 15+500, weakening of carriageway occurred due to lack of drainage structure. Therefore, to avoid this problem a pipe culvert of one-meter internal diameter is recommended.

- At station 16+400, the four rows of pipe culvert was inadequate to pass flood through it during wet season. This resulted in erosion of embankment at the downstream. Therefore, to avoid this problem, construction of 12-meter clear span bridge is important.
- At stations 22+500 and 23+100, drainage structures were not constructed; as a result, weakening of carriageways occurred. Therefore, to avoid this problem, construction of pipe culverts of 0.90-meter internal diameter is important at both stations.
- At station 32+300, the improper alignment of the short span bridge has made it is susceptible to scour, and damage by vehicles. Therefore, replacement of the bridge with proper alignment and same size is required.
- At station 36+000, stream-crossing ford was constructed; however, due to high flood; it was washed away by flood without serving for road users as intended. The stream is very wide and deep that accommodates high flood. Therefore, construction of Girder Bridge is important in order to pass flood during wet season without disturbing the drainage facilities and stream embankments.
- At station 41+700, slab culvert was constructed; however, due to poor quality construction; the slab culvert was damaged without serving the intended purpose. Therefore, relocation of slab culvert is important in order to reconstruct based on ERA bridge design manual.

In general, it is important to use appropriate hydrological analysis, hydraulic design, appropriate site selection, stream morphological study and workmanship for road drainage structures.

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

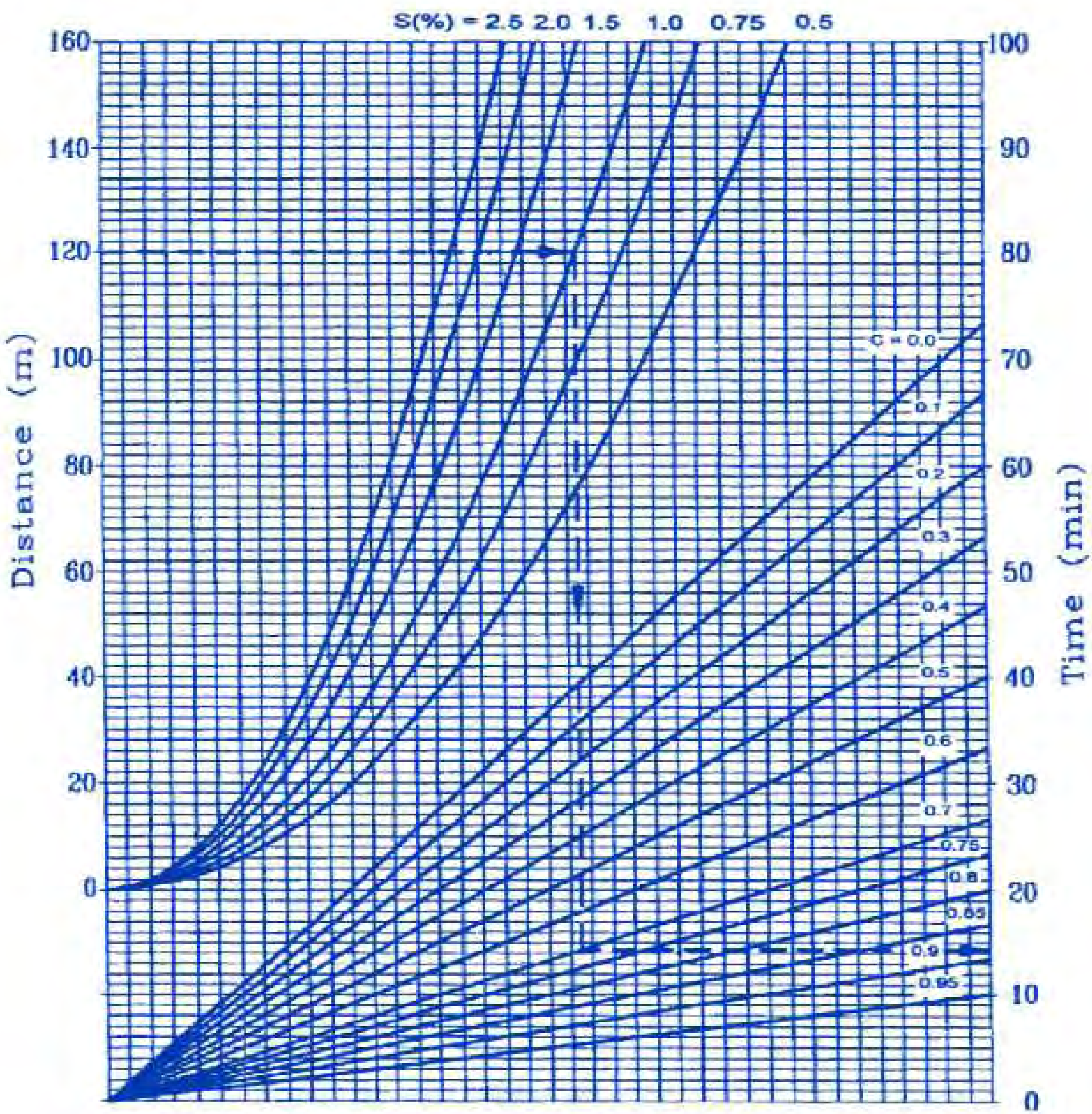

Figure 1: Overland Time of Flow

Source: Air Port Drainage, Federal Aviation Administration, 1965

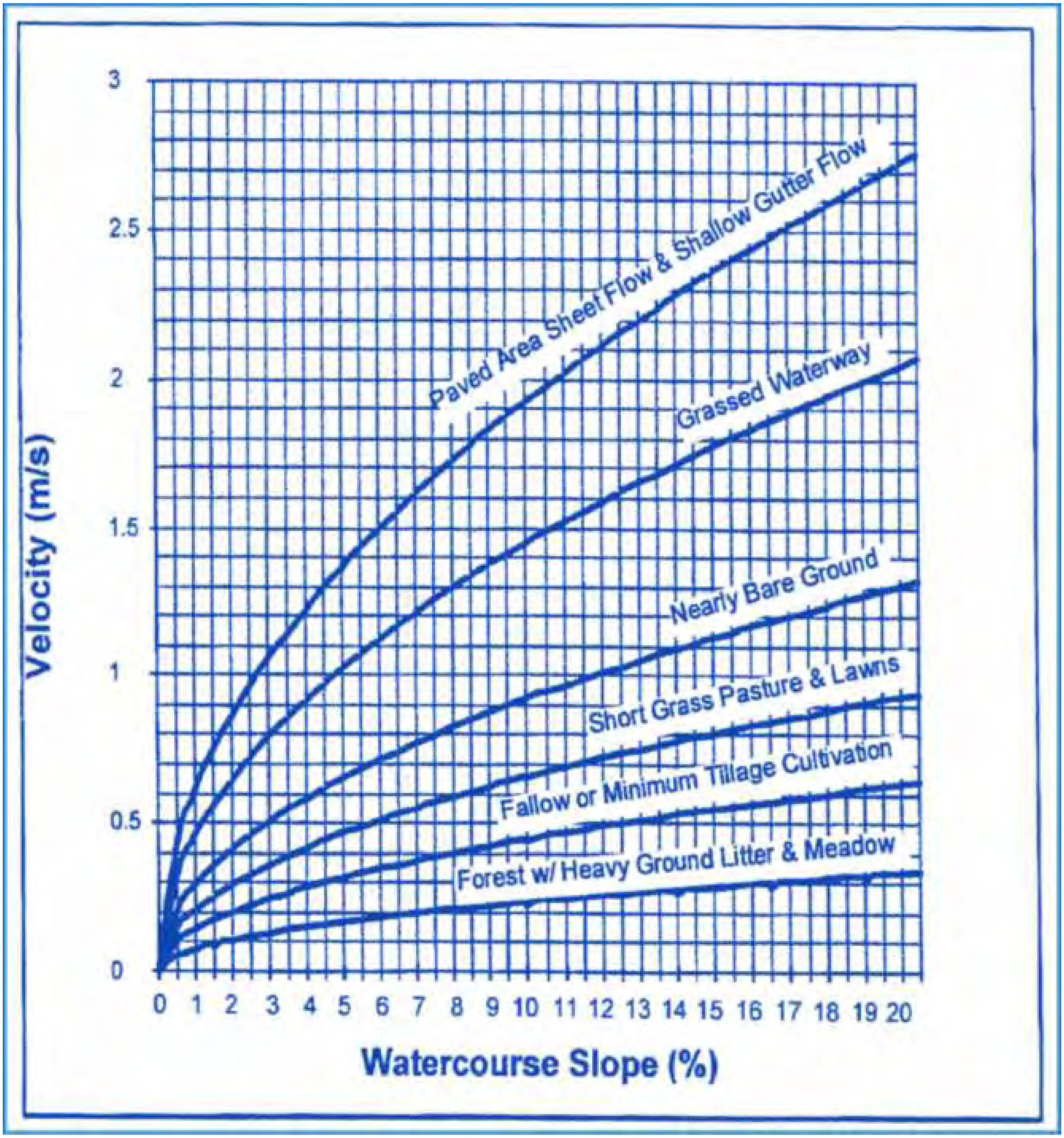

Figure 2: Velocities for Upland Method of Estimating Time of Concentration

Source: HEC No. 19, FHWA

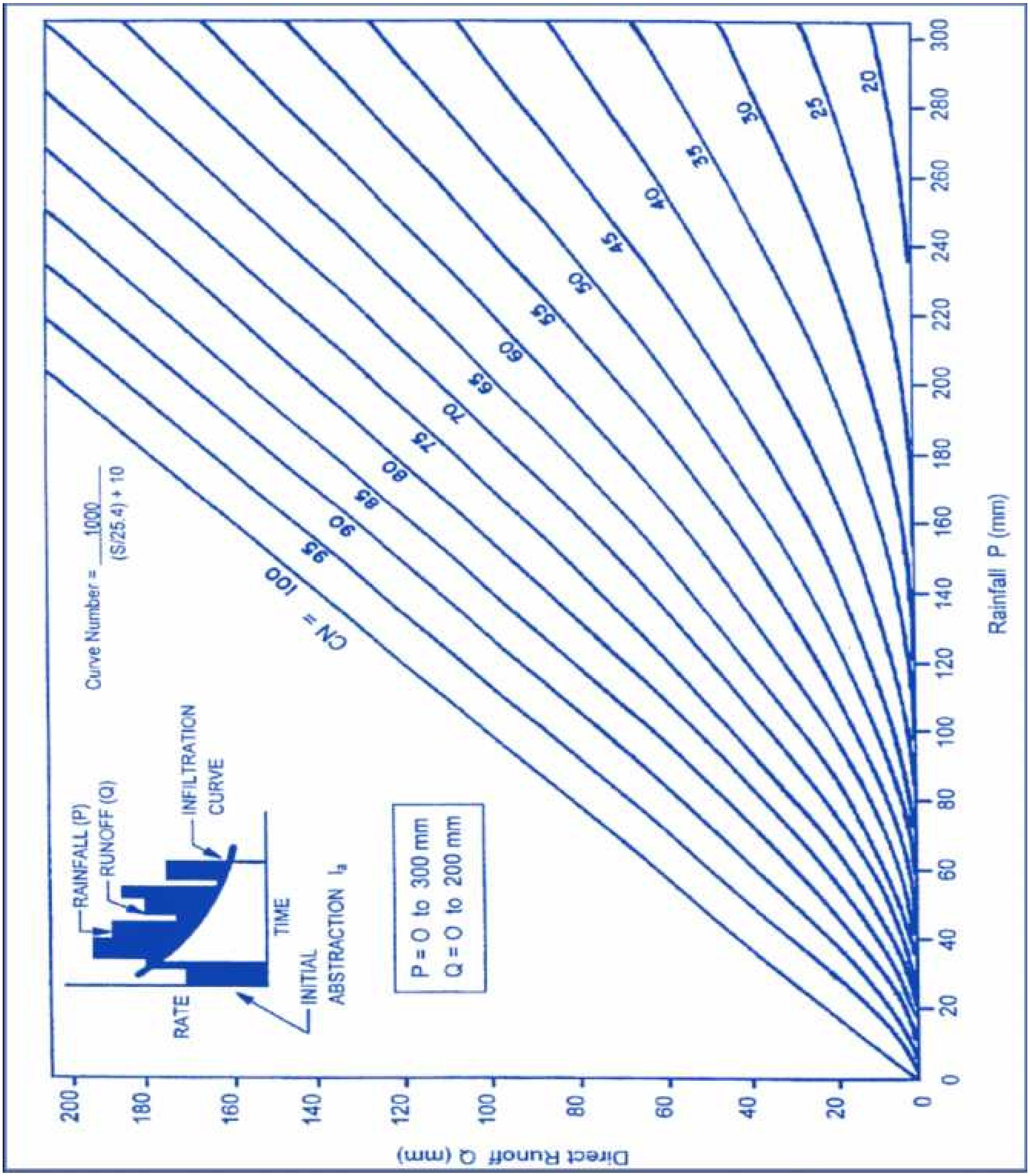

Figure 3: SCS Relation between Direct Runoff, Curve Number and Precipitation

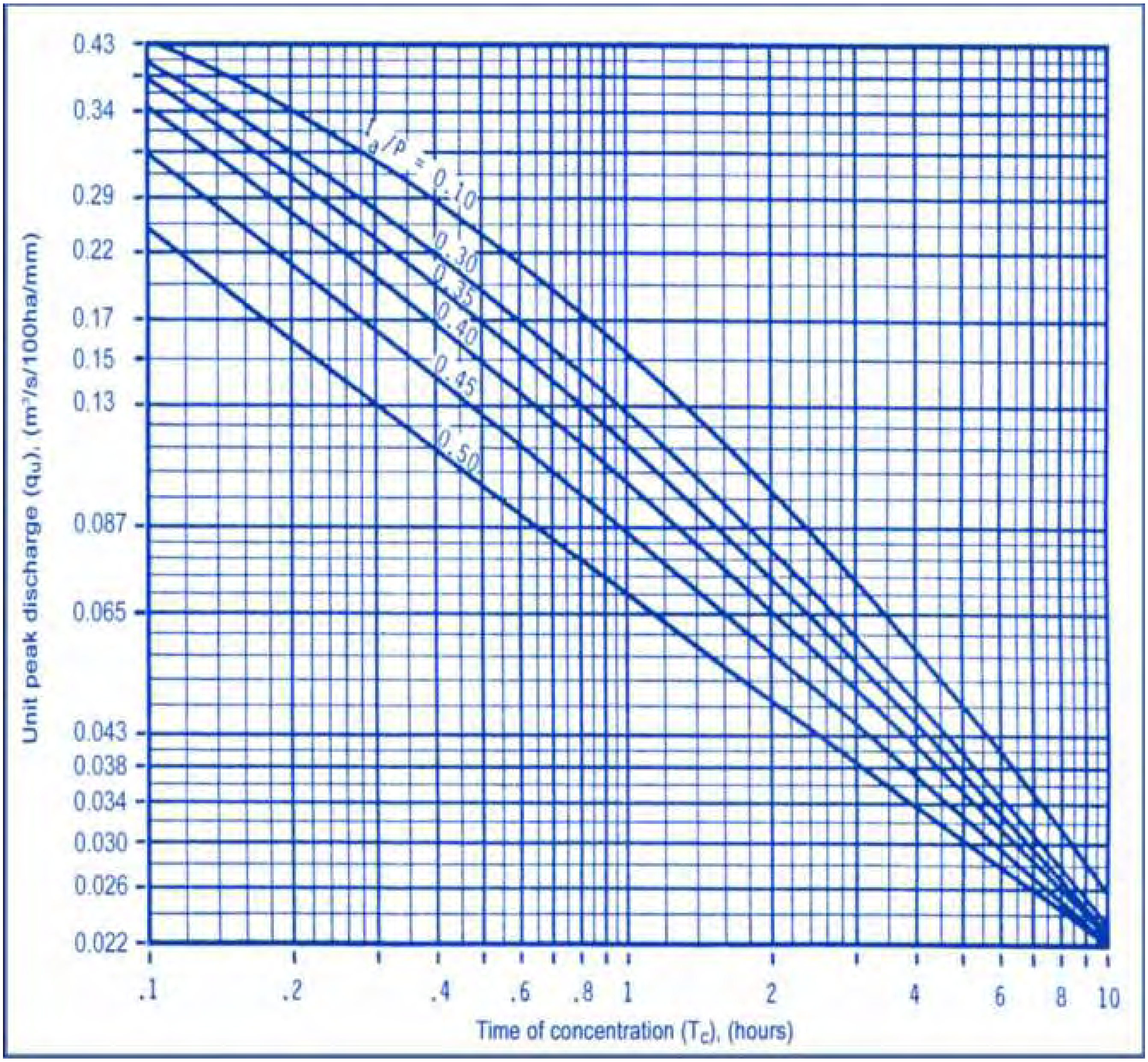

Figure 4: Unit Peak Discharge, Type II rainfall

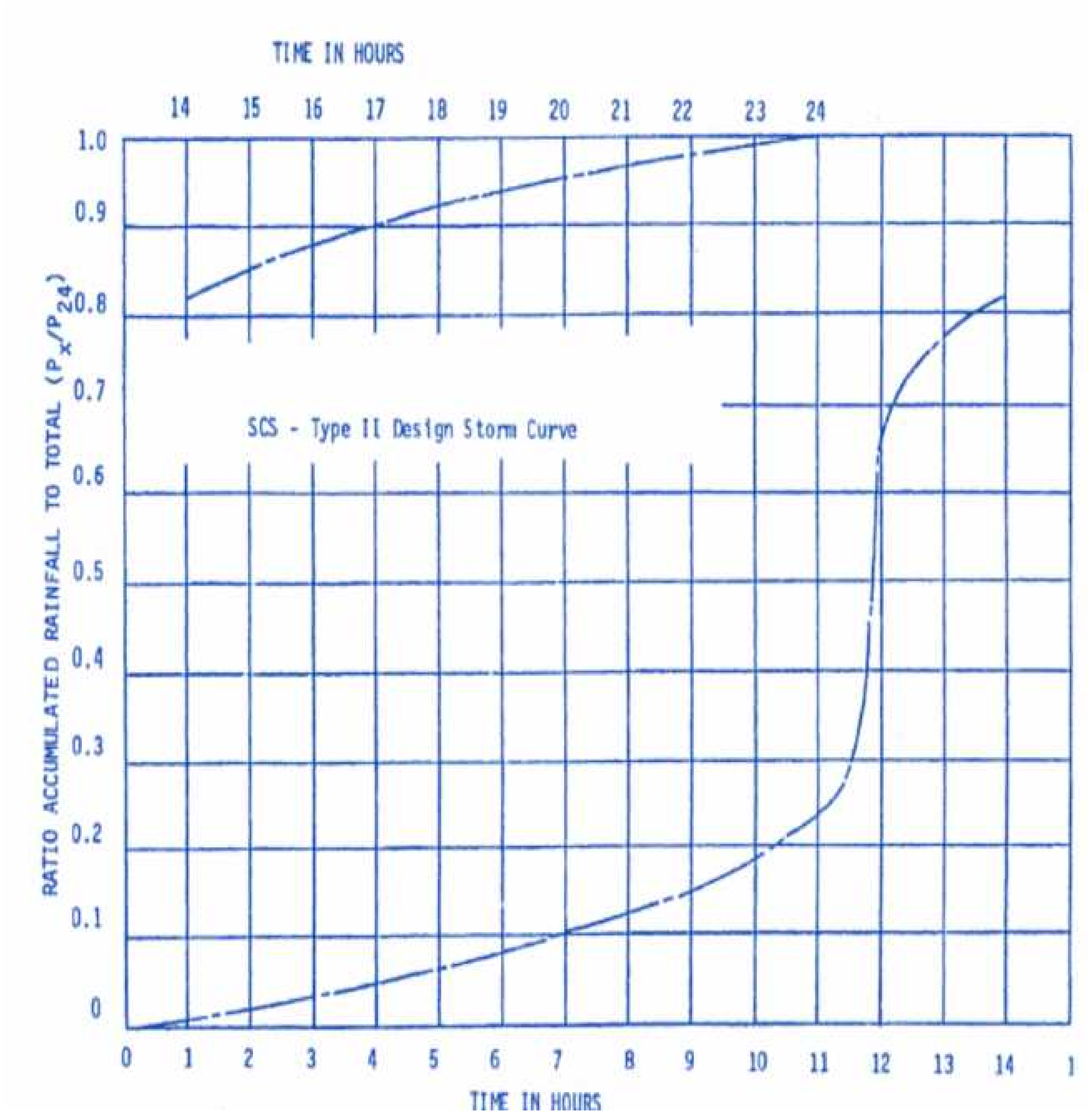

Figure 5: Type II SCS Storm Curve

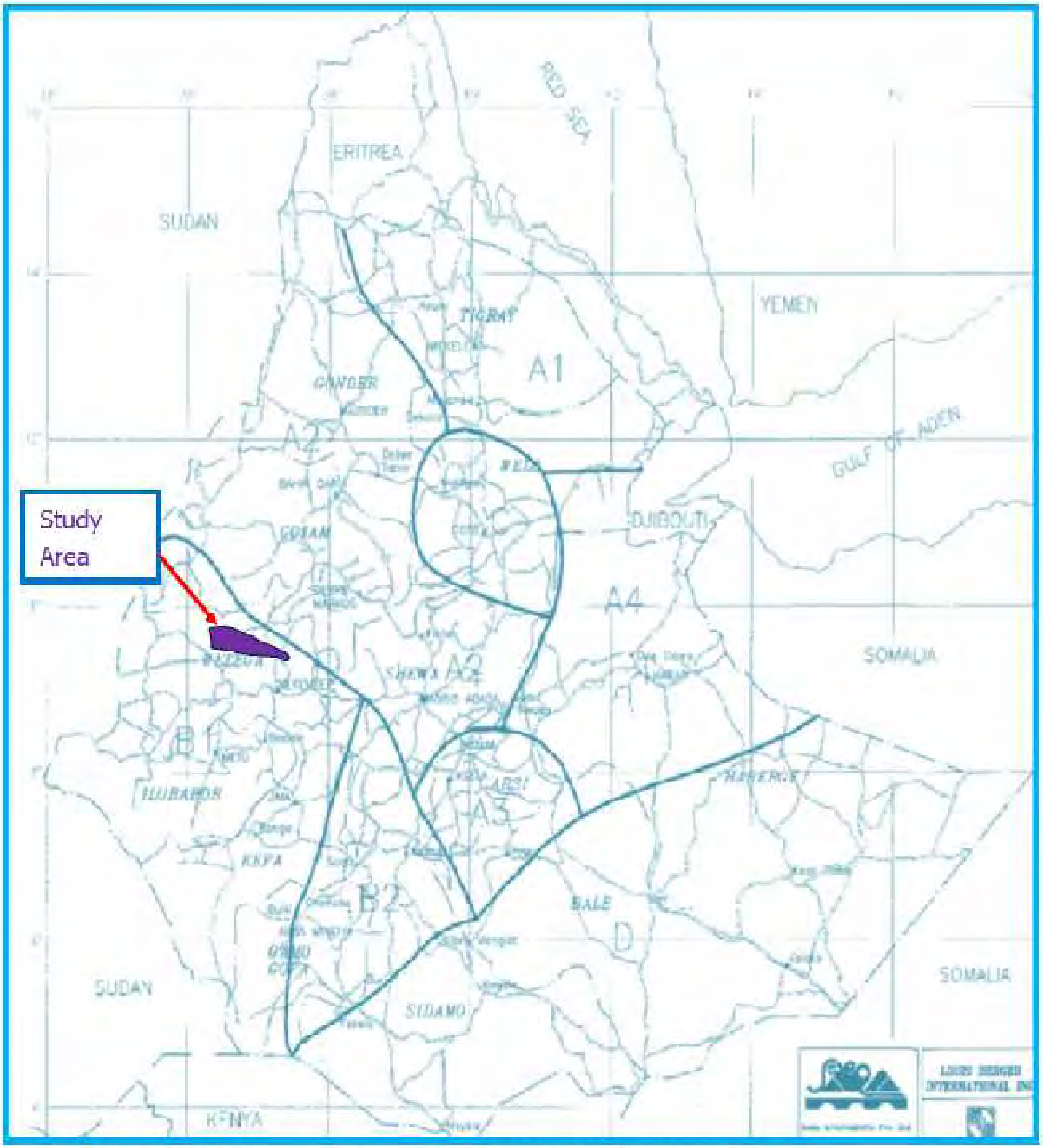

Figure 6 Rainfall Regions of Ethiopia

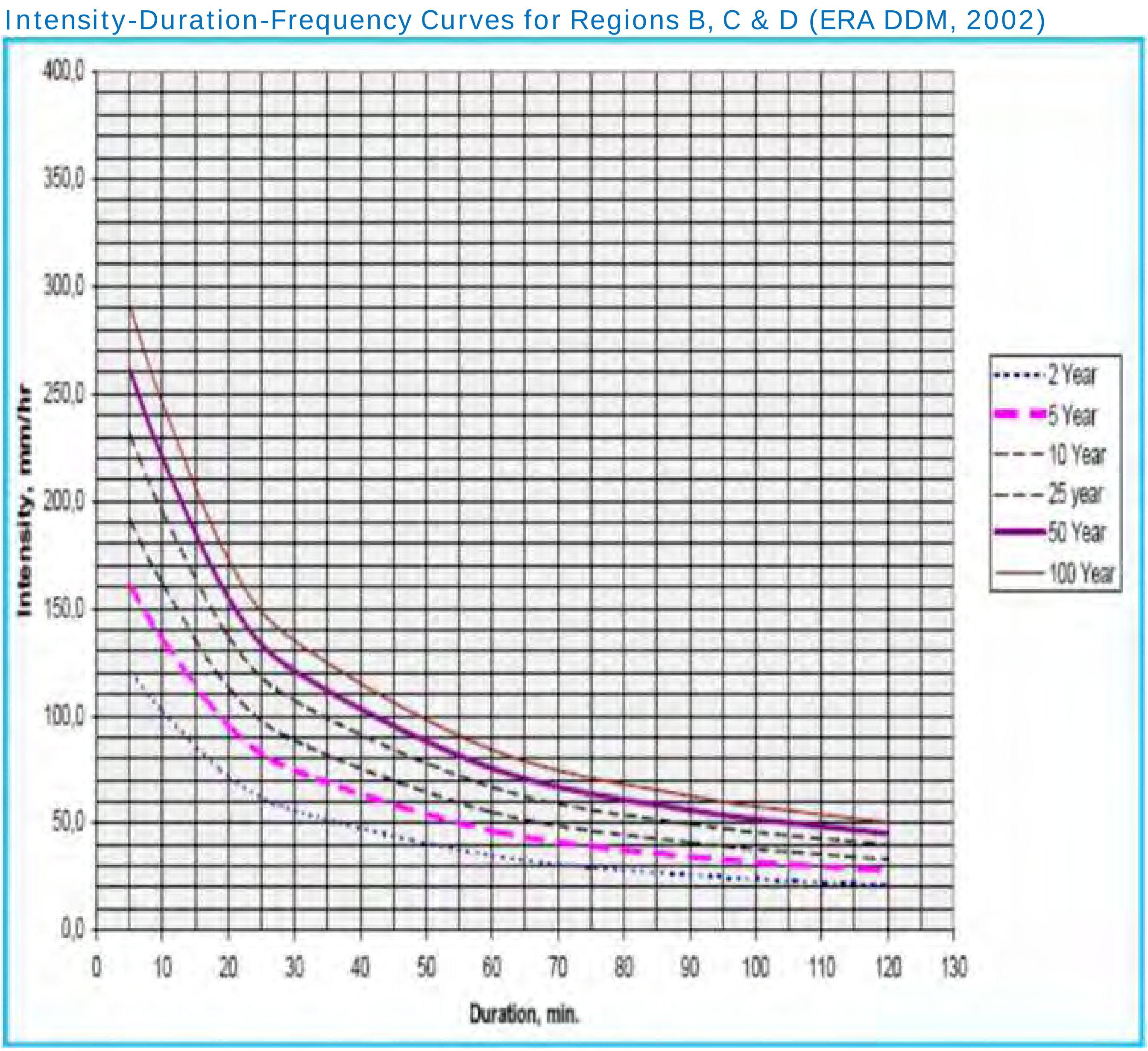

Figure 7: IDF Curve for Rainfall Regions of B, C and D in Ethiopia (ERA DDM, 2002 & 2011)

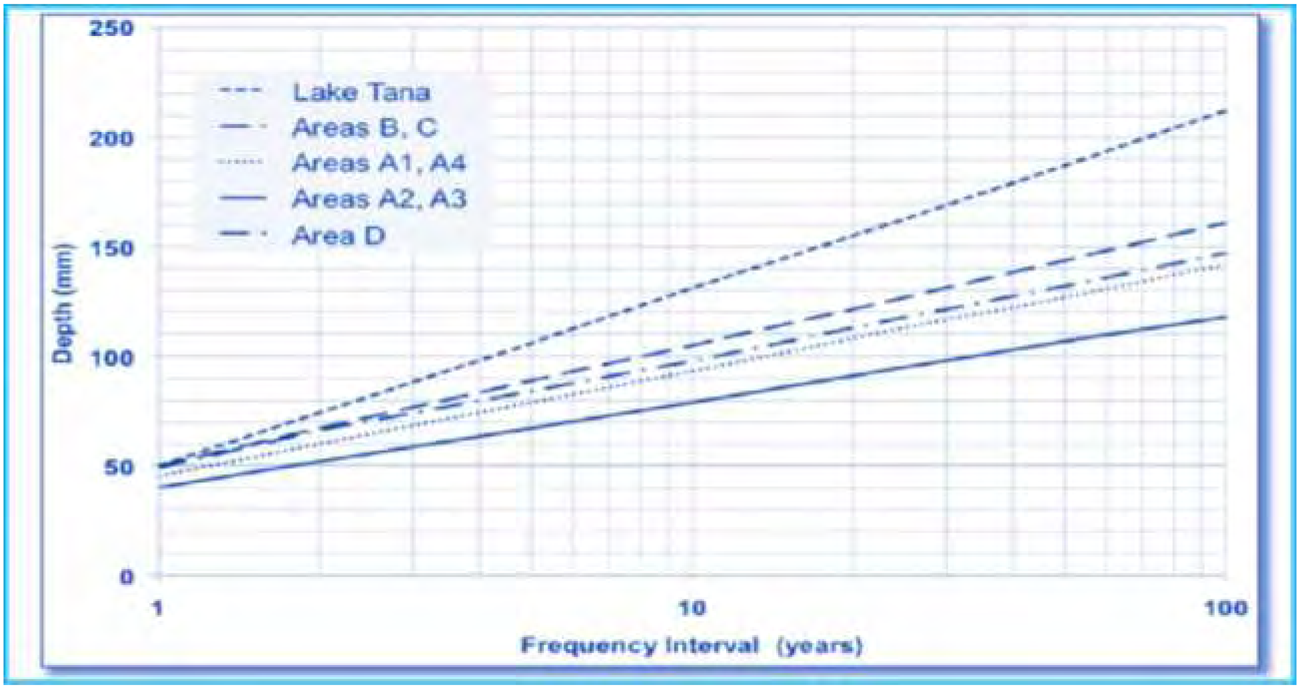

Figure 8: 24 hour Depth-Frequency Curve (ERA DDM, 2011)

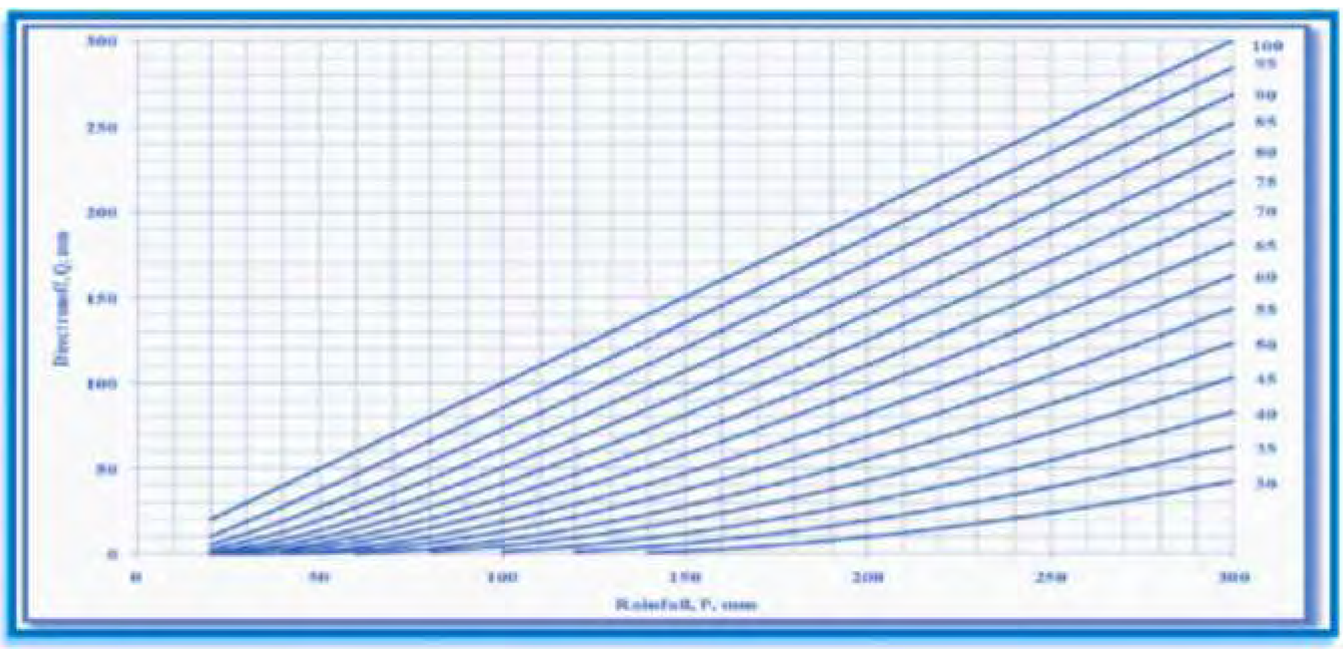

Figure 9: Relationships between Precipitation, Direct Runoff and Curve Number (ERA, 2011)

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 rifts 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: Recommended Runoff Coefficient (C) for Various Selected Land Uses

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

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

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 complied 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

Appendix C: Parameters for the Design of Drainage Structures

Table 1: Storm Design Return Period –years (ERA DDM, 2011)

Structure Type	Geometric Design Standard			
	DC4	DC3	DC2	DC1
Gutters and Inlets	2	2	2	1
Side ditches	10	5	5	2
Ford	10	5	5	2
Drift	10	5	5	2
Culvert diameter <2meter	15	10	10	5
Large culvert diameter >2meter	25	15	10	5
Gabion abutment bridge	25	20	15	-
Short span bridge(<15meter)	25	25	15	-
Masonry arch bridge	50	25	25	-
Medium span bridge (15-50 meter)	50	50	25	-
Long span bridge(>50meter)	100	100	50	-

Table 2: Runoff Coefficient: Humid Catchment (ERA Drainage Design Manual, 2011)

Average ground slope	Soil Permeability			
	Very low(Rock and hard clay)	Low(clay loam)	Medium (sandy loam)	High (sand and gravel)
Flat (0-1%)	0.55	0.40	0.20	0.05
Gentle (1-4%)	0.75	0.55	0.35	0.20
Rolling (4-10%)	0.85	0.65	0.45	0.30
Steep (>10%)	0.95	0.75	0.55	0.40

Table 3: Runoff Coefficient: Semi-arid Catchment (ERA DDM, 2011)

Average ground slope	Soil Permeability			
	Very low(Rock and hard clay)	Low(clay loam)	Medium (sandy loam)	High (sand and gravel)
Flat (0-1%)	0.75	0.40	0.05	0.05
Gentle (1-4%)	0.85	0.55	0.20	0.05
Rolling (4-10%)	0.95	0.70	0.30	0.05
Steep (>10%)	1.00	0.80	0.50	0.10

Table 4: 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 5: 24-hour Rainfall Depth

Region	Frequency Interval (years)					
	2	5	10	25	50	100
A1, A4	60	79	93	113	127	142
A2, A3	52	67	79	95	107	118
B, C	65	84	98	118	132	147
D	67	89	105	127	144	161
Lake Tana	74	106	131	163	187	211

Table 6: Hydrological Characteristics of Soil Groups (ERA DDM, 2011)

Soil Group	General Description	
A	Well drained, sandy	High infiltration, low runoff
B	Sandy loam, low plasticity	
C	Clayey loam, medium plasticity	
D	High plastic clay	Low infiltration, high runoff

Table 7: Antecedent Moisture Conditions (ERA DDM, 2011 for LVRs)

Regions(*)	Antecedent Moisture Conditions
D	Dry
B	Wet
All other regions	Average
Bahir Dar area	Although in region A, use wet

* The rainfall regions of Ethiopia

Table 8: 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 9: Conversion of CN from AAM conditions to dry and wet conditions

CN for average conditions	Corresponding CN's	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

Table 10: I_a 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