



Addis Ababa University

Addis Ababa Institute of Technology

**School of Civil and Environmental
Engineering**

Msc. Thesis

On

**Performance Assessment of Drainage
Systems on ALEMGENA-BUTAJIRA Road**

September 2015

Addis Ababa, Ethiopia

Addis Ababa University
Addis Ababa Institute of Technology
School of Civil and Environmental
Engineering
Msc. Thesis

On

Performance Assessment of Drainage
Systems on ALEMGENA-BUTAJIRA Road

A thesis Submitted to the School of Graduate Studies in partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Hydraulic Engineering) at Addis Ababa Institute of Technology, Addis Ababa University.

By

Alemu Tamiru

Advisor: Dr. Agizew Nigussie

Certification

I , the undersigned certify that I have read the thesis entitled “Performance Assessment of Drainage Systems on Alemgena- Butajira Road ”and here by recommend for acceptance by Addis Ababa University in partial fulfillment of Master of Science in Hydraulic Engineering.

X

Agizew Nigussie (Ph . D)
Supervisor

Declaration and copyright

Alemu Tamiru 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.

X

Signature

This thesis is copyright material protected under the Berne conventions, the copyright act 1999 and other international and national enhancements, in that behalf, all intellectual property. It may not be reproduced by any means, in full or in part, except for short extracts in fair dealing for research or private study, critical scholarly review or discourse with an acknowledgement, without written permission of the school of postgraduate studies, on the behalf of both the author and Addis Ababa University.

Acknowledgement

First of all I would like to thank a lot the Almighty God for his gift that cannot be described in words, protection and help. And I am extremely thankful to St. Virgin Mary and all Saints supporting me during my work.

I would like to take this opportunity to express my sincere appreciation to Dr. Agizew Nigussie, for his supervision, guidance and expert advice during the period of our study to the preparation of the master's thesis proposal.

I am thankful for his unreserved and continuous advice and great support being as instructor, and Brother from the initial to final successful accomplishment of this thesis.

I am also very grateful to the Ethiopian Road Authority (ERA) for covering all my financial expenses and full time salary payment during my study period.

I would like to express my deepest gratitude and appreciation to Ato Yisihak Gebre for providing me his assistance in facilitating my data collection work. And also to Ato Tesfaye Abebe who has been giving technical input and comment at the right time.

My acknowledgement is also due to the staff of ENMA, MOWIE for their support in providing required documents.

My special appreciation goes to my wife Altayech Yitbark for her moral and material support throughout the study period.

Abstract

This Master of Science thesis focuses on performance assessment of drainage structures on Alemgena-Butajira road segment. The main target of this research is to indicate a quantity of work that should have been done already, but has not yet been done in keeping the bridge and culverts in their performance to good condition.

Bridges and culverts on the road segment that require intervention have been assessed, selected and illustrated in this thesis work.

In preparation of this research work, 26 bridges and 169 culverts were registered. Among them 19 bridges and 45 culverts were found defective which need detail inspection for maintenance. The remaining requires only visual regular inspection. The existing status of the structures along the route is presented with support of pictures.

Even though Ethiopian Roads Authority has launched regular bridge inspection program that detail inspection in 3 years interval and visual inspection every year for each bridge and culvert structure, the attention given for bridges and culverts maintenance along the route has been very little. A number of the bridges and culverts along the route were found defective and they should be maintained urgently to keep them safe and operational.

Table of Contents

Certification	II
Declaration and Copyright.....	III
Acknowledgement	IV
Abstract.....	V
Table of Contents.....	VI
List of Tables	VIII
List of Figures.....	VIII
List of Acronyms	X
Chapter1: Introduction	1
1.2 General	1
1.2.1 Brief information about the study area.....	2
1.2.2 Statement of the problem.....	2
1.2.3 Research Questions	3
1.2.4 Objective of the research study	3
1.2.5 Significance of the Research	4
1.2.6 Scope and limitations of the study	4
2.1. Introduction	5
2.6.1 Rainfall.....	10
2.6.2 Flood History	10
2.7 Flood History of Existing Structures	12
2.8 Methods of Determining Flood Magnitudes	13

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.....	21
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).....	21
Chapter 3: Description of the study area	27
3.1 General	27
3.2 Assessment of Performance and condition of the structure	35
Chapter 4: Materials and methods	41
4.1.1 Design rainfall	41
4.2 Hydrological equations for peak flood computation	42
4.3. Hydraulic equations (hydraulic calculation).....	53
4.4. Types of data needed	56
CHAPTER 5- Results and Discussion	59
5.1 Hydrologic Analysis	59
5.2 Delineation of catchment Area.....	59
5.3 Computation of Peak Discharge by Rational Method	60
CHAPTER 6- Hydraulic Analysis Results and Discussion	77
6.1. Hydraulic Analysis	77
Chapter 7: conclusion and recommendation	85
7.1. Conclusion.....	85
7.2. Recommendation	86
8. References	87

Appendix 1:Time of Flow, Unit Peak Discharge, Velocity of flow and SCS CN Charts	73
Appendix 2:Roughness and Runoff Coefficient Values	81
Appendix 3:Parametes for the Design of Drainage Structures	84
Appendix 4: Typical hydrologic soil groups of Ethiopia.....	90

List of Tables

Table 5.1: Delineation of previous and proposed drainage structures	50
Table 5.2: Comparison of computation results of previous and proposed culverts	56
Table 5.3: Computation results of surface runoff	61
Table 5.4 : catchment area segment results	63
Table 5.5: Comparison of computation results of previous and designed culvert	64
Table 6.1: Hydraulic comparison of culvert	66
Table 6.2:Hydraulic parameters for proposed drainage structure at station 47+100	67

List of Figures

Figure 3.1a and 3.1b: Location Map of the Study Area.....	19
Figure 3.2: Topographic Map of Alemgena- Butajira Road	21
Figure 3.3: Rainfall Isohytes.....	23
Figure 3.4: Geological Map of the study Area	24
Figure 3.5: Soil Map of the study Area	25

Figure 3.6: Previous damaged RC slab culvert type bridge at station 47+100.....	28
Figure 3.7a and 3.7b: Previousdamaged slab culvert at station 81+430.....	32
Figure 5.1 Delineation of previous and proposed Drainage Structures.....	49
Figure 5.2: Previous Culvert at Chainage 81+430	51
Figure 5.3: Land use cover map of the culvert	52
Figure 5.4: Average Monthly rainfall of the Study Area (Butajira)	53
Figure 5.5: Average Monthly rainfall of the Study Area (Bui)	53
Figure 5.6: Arithmeatic mean Rainfall of Butajira and Bui Stations.....	54
Figure 5.7: Measured Monthly flow Hydrograph near the Study Area in Rift Valleys Basin	54
Figure 5.8: Previous Bridge at Chainage 47+100	57
Figure 5.9: Soil map of Bridge catchement area	59
Figure 5.10: Average Monthly Temperature of the Study Area (Lemen)	59
Figure 5.11: Average Monthly Rainfall of the Study Area (Lemen)	60
Figure 5.12: Measured Monthly Flow data Hydrograph near the Study Area in Awash Basin	60
Figure 5.13: Catchment Area Segments	62

List of Acronyms

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

ERA - Ethiopian Roads Authority

FDRE- Federal Democratic Republic of Ethiopia

FHWA- Federal Highway Administration

GDM- Geometric Design Manual

HDS- Hydraulic Design Series

HSG- Hydrological Soil Group

HVR-High Volume Roads

IDF- Intensity-Duration-Frequency

NBIS- National Bridge Inspection Standards

NMA- National Meteorological Agency

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

Chapter1: Introduction

1.2 General

In order to protect Drainage structures from any major problems of damage, identification of deteriorations and damages is very essential. To reduce any trouble and difficulty of travel due to drainage structures, correct measures through proper maintenance and repair must be used.

The aim in highway drainage is to prevent on-site water standing on the surface and convey the off-site storm runoff from one side of the roadway to the other. To accomplish the offsite drainage either a culvert or a bridge can be used.

Culverts are cross-drainage structures that convey water from streams and side channels below the road. A culvert is usually, although not always, differentiated from a bridge by virtue of the fact that the top of the culvert does not form part of traveled roadway. More frequently, culverts are differentiated from bridges on the basis of span length. On an arbitrary basis, structures having a span of 6m or less are called culverts, whereas those having spans of more than 6m are called bridges (ERA, 2002). Culverts also differ from bridges in that they are usually designed to flow full under certain conditions, while bridges are designed to pass floating debris or vessels.

Provision of adequate drainage is an important factor in the location and geometric design of highways. It is desirable that they be designed economically and provides an adequate level of service. Factors such as initial cost, design life, and the risk of loss of use of the roadway for a time due to runoff exceeding the capacity of the drainage structure, need to be considered in the design.

1.2. Brief information about the study area

Alemgena- Butajira road is found in Shewa Province in a southerly direction, passing through the towns of Lemen, Suten, Bui, and Butajira. The road starts in Alemgena, at the junction with the Addis Ababa-Jimma Road, some 20 Km from Addis Ababa.

The area through which the road passes is characterized by extensive small- scale agricultural activities, which result in the continuous movement of produce along the road, especially during the harvest seasons.

It is one of the corridor routes that link the central part of the country to the southern regions specially the Oromiya and Southern National and Nationalities Administrative Regional States. The route was constructed by double surface treatment pavement structure in recent years. Nevertheless severe damage is noticed in the most parts of the road segment. The road is 120 Kilometers long. Throughout the road length, there are many bridges and culverts even if some of them are not functioning properly during the rainy season.

1.3 Statement of the problem

In Alemgena – Butajira road, there are drainage structures which are not properly functioning. The main causes are: - deposition of material in and around structure buried or severely completely clogged culverts, blockage of bridge deck drains, low attention and priority given to drainage structures and lack of regular maintenance and timely repair. Drainage structures are not kept free of debris and obstructions.

The main problems are the inadequacy of drainage structures during the rainy season to pass the peak flood, poor quality construction and workmanship, inappropriate site selection, Cracks on the pier and abutment of the bridges, Aggradations or increasing of river bed elevation 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. Especially at Tinsua lemen Bridge station 47+100 and culvert station 81+430 on Alemgena-Butajira road, such problem is seen on the existing bridge. 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.

1.4 Research Questions

The very important questions that are to think about a problem or a situation and to find out facts about problem by the research are:-

- What are the major defects of the Alemgena- Butajira road drainage structures?
- What are the causes of the defects of the drainage structures on Alemgena- Butajira road?
- How is the severity and extent of the damage?
- How has preserving of the Alemgena- Butajira drainage structures been carried out?
- What reduction measures can improve the drainage problems?
- Are the hydraulic capacities of the drainage structures adequate?

1.5 Objective of the research study

1.5.1 General objective

- ❖ To evaluate the performance of drainage structures i.e. side drains, Bridges and culverts and to suggest mitigation measures that can minimize the untimely deterioration of the road and frequent maintenance of drainage structures.

1.5.2 Specific objectives

- To investigate the severity and extent of the drainage structures damage.
- To explore the causes of the drainage structures problems.
- To evaluate the drainage structures hydraulic capacity.
- To recommend appropriate remedial measures.

1.6 Significance of the Research

In the design of highway, drainage structures are extremely important components. Provision of adequate drainage is an important factor in the location and geometric design of highways. Adequate level of service can be acquired by properly designing them. Initial cost, design life, and the risk of loss of use of the road way for a time due to runoff exceeding the capacity of the drainage structure, need to be considered in the design.

This study aims at improving the situation of the Alemgena- Butajira road drainage structures by assessing the performances and suggesting reduction measures. It is beneficial for researchers who conduct similar researches on road drainage structures.

1.7 Scope and limitations of the study

This thesis is limited to the performance assessment of drainage systems on Alemgena-Butajira road. The intended purpose is to suggest reduction measures for problems that are found on the existing drainage structures. The research included both hydraulic and hydrologic analyses for drainage structures that are susceptible to failure.

CHAPTER 2: Literature Review

2.1. Introduction

Hydrologic analysis is an important step prior to the hydraulic design of a highway drainage structure. Such an analysis is necessary for determining the rate of flow, runoff or discharge that the drainage facility will be required to accommodate. The design discharge is a hydraulic “load” on the highway facility and the determination of its magnitude and possibly its duration is as important as the determination of the proper structural load.

Highway drainage facilities range from very small culverts and channels to multi-million dollar storm drains and bridges. Although some hydrologic analysis is necessary for all highway drainage facilities, the extent of such studies should be commensurate with the importance of the structure.(AASHTO, 2007).

2.2 General Description of Road Drainage Structures

Highway hydraulic structures perform the vital function of conveying, diverting, or removing surface water from the highway right-of-way. They should be designed to be commensurate with risk, construction cost, importance of the road, economy of maintenance, and legal requirements. One type of drainage facility will rarely provide the most satisfactory drainage for all sections of a highway.

Drainage design covers many disciplines, of which two are hydrology and hydraulics. The determination of the quantity and frequency of runoff, surface and groundwater is a hydrologic problem. The design of structures with the proper capacity to divert water from the roadway, remove water from the roadway, and pass collected water under the roadway is a hydraulic problem. (FHWA-NHI-08-090 June 2008)

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 commonly used for cross drainage and can range in size from a single small culvert draining an isolated depression to multiple barrel designs and/or very large culverts for passing major stream channels under a roadway. Small culverts are also used for down drains to protect fill slopes or to divert roadway water from a bridge deck.

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).

2.2.1 Types of Culverts

A culvert typically represents a significant contraction of flow over conditions in the upstream and downstream channels and often is a hydraulic control point in the channel. Provision of a more gradual flow transition at the inlet of a culvert can improve the discharge capacity of the culvert by reducing the energy losses associated with flow contraction. Culvert inlets are available in a variety of configurations and may be prefabricated or constructed in place. Commonly used inlet configurations include projecting culvert barrels, cast-in-place concrete headwalls, precast or prefabricated end sections, and culvert ends mitered to conform to the fill slope. Structural stability, aesthetics, erosion control, fill retention, economics, safety, and hydraulic performance are considerations in the selection of an inlet (FHWA-NHI-08-090 June2008)

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, 150 - 450 if conditions do not permit to install normal to the stream channel. Installing culverts normal to the stream channel decreases construction cost. Where large skew angles are required, consideration of the most appropriate road alignment is significant (Austroads, 1994).

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.

2.2.2 Road Surface Drainage

As one of its most basic characteristics, a roadway must be carefully shaped and graded to remain “high and dry”. The combination of: (1) centerline grade (with respect to existing ground), (2) typical section (the shape of the roadway structure including ditches), and (3) vertical alignment (the succession of centerline grades) must be established to prevent inundation of the roadway and its supporting structure. Various means of roadway surface drainage collection and conveyance are described in the following sections.

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, 2002) 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.

2.3 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.

2.4 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).

2.5 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)

2.6 Design Flood for Road Drainage Structures

2.6.1 Rainfall

Although the relationship between rainfall and runoff is not well defined, runoff usually increases in proportion to the rainfall on a drainage basin. Basin characteristics and antecedent conditions, particularly precipitation, have a major effect on the proportion of rainfall which becomes runoff. For example, most of the rain falling on frozen or saturated ground runs off quickly, while most of the rain falling on dry, porous soil infiltrates. There is little correlation between the recurrence interval of rainfall and the recurrence interval of the corresponding peak runoff (Hiemstra L. A. V etal). However, studies (Horner, V. W et, al) , Relation Between Rainfall and Runoff from Small Urban Areas, have shown that when peak runoff and rainfall were considered separately, the ratio of peak runoff rate of a given frequency to rainfall intensity for the same frequency remained reasonably constant for the various frequencies. This indicates that rainfall can be used to estimate design floods, although a rainfall on a given frequency will seldom produce a peak runoff of the same frequency for any one storm.(Matthai etal, 1968).

2.6.2 Flood History

Good highway design practice recognizes that flood hazards must be evaluated whenever highway locations cross or encroach upon flood plains. The history of past floods and their effect on existing structures are of exceptional value in making flood hazard evaluation studies, including needed information for sizing structures. Flood-control works and land-use planning

data relative to restricting flood heights and reducing flood discharges are also a necessary part of a flood hazard evaluation.(AASHTO, 2007).

2.6.2.1 Historical Floods

Major floods that have been experienced before the start of records are often called historical floods. In describing these events, it is necessary to determine the period of years during which they have occurred, as well as their magnitude, in order that the information may be fully utilized. Some information on past floods usually can be obtained from old newspaper accounts, long-term residents, and other sources. Often, experienced personnel of the Geological Survey and other agencies who make flood determinations can find flood marks or other positive evidence of the height of historical floods. Changes in channel and watershed conditions should be evaluated in relating historical floods to the present. Historical floods of unusual magnitude are valuable data in flood-frequency analysis, particularly when the gaging-station record is short.(AASHTO, 2007).

2.6.2.2 Flood Data

Much stream flow and flood-related data are available to the highway engineer. Most stream flow data are obtained by the Geological Survey at numerous gaging stations over the United States. These data and that collected by other agencies are published periodically in the surface water records of the Geological Survey and available at their local offices.

Railroad maintenance files often contain accurate information regarding flood stages that have been experienced at railway structures or along tracks bordering a stream. Newspaper accounts and magazine articles should not be overlooked as sources of documentation of unusual floods.

All of these sources may provide valuable assistance and supplementary information that can be used advantageously; however, discrepancies sometimes are revealed when these data are compared. This indicates the need for verification and evaluation of flood data, regardless of the source. Development within the watershed should be evaluated before using old flood data.

(AASHTO, 2007).

2.7 Flood History of Existing Structures

Structures that have existed for many years may have experienced unusual floods. If an existing structure is located in the vicinity of the proposed highway structure, a field inspection may indicate flood heights and damage that has occurred. Local witnesses and examination of maintenance records may be helpful in evaluating past floods at a structure.

High water elevations, indicated by deposits of debris, by seed or mud lines on tree trunks and bridge abutments, by wash-lines or fine-debris lines on banks, by whisps of grass or hay lodged in tree limbs or fences, and by other flood evidence, such as erosion and scour, can provide information for arriving at flood discharges and reliable flood stages for use in designing a proposed structure. More obvious items of flood evidence such as large deposits of debris or prominent wash-lines do not necessarily indicate the true peak stage. Usually the actual peak is somewhat higher than would be indicated by the rather obvious marks, unless such marks were affected by pile up or the rebound of trees or shrubs after the flood. Interviews with highway maintenance foremen and the long-time residents in the area can be very helpful.

A record of the performance of drainage structures during floods, including photographs is valuable for use in designing future structures and for determining modifications to structures which might reduce maintenance or increase safety. Such records may also be helpful in defending the state against damage claims.

These records might include: Maximum flood height, upstream and downstream from a structure. Observed differences in water surface elevations on the upstream and downstream side of the embankments at several points from each abutment.

Distribution of flow and approximate velocities in different sections of the stream, relative quantity of overbank flow and how it returns to the channel direction of flow with respect to the piers and the low-water channel observe drift-size and concentration. Remarks on clearance or freeboard duration of flooding magnitude of flood and its relation to other notable floods headwater at culverts scour erosion and sediment or gravel deposits damage to structure and adjacent property.

All of these observations may not be necessary for every structure. The size of the structure, magnitude of the flood, extent of damage, or probability of legal action might determine the extent of the observations.(AASHTO, 2007).

2.8 Methods of Determining Flood Magnitudes

The accurate determination of flood magnitudes requires a background in open-channel hydraulics and knowledge of floodwater behavioral patterns; however, knowledge must be coupled with experience if the results are to be correctly interpreted. The basic methods of measuring flood flow are discussed in the following

2.8.1 Direct Measurements

The direct measurement of flood flow consists of measurements that are made during a flood (Buchanan, et, al, 1969). Discharge is determined by simultaneously measuring the flow depth and velocity at a sufficient number of points in a cross section to define significant changes in either depth or velocity. From these measurements, the area and average velocity can be determined and the discharge calculated. Discharge measurements at various stages at a site or gaging station provide data for developing a rating curve (Carter, et, al, 1965) or a plot of stage versus discharge. Continuous records of stage gaging stations provide discharge data for studying the recurrence interval or frequency of floods (Buchanan, et, al, 1968). And (Carter, et al 1968).(AASHTO, 2007).

2.8.2 Indirect Measurements

Indirect measurements are made when it is impossible or impractical to measure flood flows directly. Generally, these measurements are made after the flood subsides (Benson, et, al, 1968). Such measurements include high-water marks, channel geometry, and an estimate of roughness coefficients (Barnes, et, al, 1967). From these data, the flood magnitude is calculated using basic hydraulic equations. Indirect methods for determining the magnitudes of actual floods include the slope-area (Dalrymple, et, al, 1967), flow-through culverts (Bodhaine, G. L., 1968), contracted opening (Matthai, et, al, 1968), and flow over dams. This tool in measuring flood flows is most valuable to the highway engineer and a thorough understanding of the methods used in the listed publications is necessary. (AASHTO, 2007).

2.9 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).

2.9.1 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.

2.9.2 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).

2.10 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.

2.10.1 Scour caused by the river

If the river scours the foundation or the bank in front of the abutment, the abutment will move and the bridge may even fall. Bank seat abutments can easily be damaged if the bank under them is scoured by the river or eroded by rain water, so you must look very carefully for scour or erosion near bank seat abutments.

- ❖ Check for erosion and scour near the base abutment or scour of the bank in front of the abutment. If the water is now low enough or clear enough to see, then use a long pole to fill if the river has cause scour.
- ❖ Check for the movement of the abutment. This can be a serious problem. Look for disturbance of the ground around the abutment. If the abutment has moved, there are often cracks in the soil. There may even be cracks in the roads behind the abutment.
- ❖ Check for vegetation growing on or in the abutment. Look in cracks or drains in the abutments, or in cracks between the abutment and wing walls and retaining walls.
- ❖ **Natural scour**: Scour that along a channel reach due to an unstable stream, no exterior causes.

- ❖ **General scour:** Scour involving the removal of material from the bed and banks across or most of the width of a channel and is not localized at an element such as a pier, abutment, or other obstruction to flow. Bed material surrounding the foundation is washed out, and concrete surface is deteriorated.

2.10.1.1 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 Pier

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](#)).

2.10.1.2 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.

2.10.2.1 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

2.10.3 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](#)).

2.11 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.

2.11.1 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.

2.12 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° as measured from a line perpendicular to the roadway centerline. Culvert skews should be constructible with standard designs of 15° , 30° and 45° 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).

2.12.1 Use of culverts as alternatives to bridges

Culverts may be considered as alternatives to bridge wherever their use appears economical and feasible, keeping in mind both their advantage & disadvantages. The hydraulic design requirements should be consistent for both alternatives, to ensure acceptable hydraulic performance of either structure. (Project Committee on Bridge Hydraulics, 1972)

2.12.2 Basic criteria and procedures for hydraulic design

2.12.2.1 Hydraulic requirements for bridges

The following basic hydraulic requirements should be met to a bridge crossing a river or other body of water.

- Location

The site selected should enable construction of a safe, economical, and easily maintained crossing, having regard to routing and its environment, and to the use of such training works as may be appropriate to deal with adverse natural features.

- Height

The height of the deck should be such that the superstructure is not endangered by the action of flowing water, ice, floating debris, or waves, and the roadway is not rendered impassable except under clearly understood and permitted conditions.

- Length

The length of bridgeworks should be such that the waterway opening is able to pass the maximum flows that may reasonably be expected without endangering the bridge or adjacent structures by scour, without creating major maintenance problems without causing unacceptable backwater effects upstream, and without causing currents, waves, or turbulence unacceptable to navigation or other legitimate interests. It should be possible to pass expected quantities of ice, logs, and other debris without endangering the structure or adjacent property as a result of jams and accumulations.

- Arrangement and details

The arrangement and details of piers, abutments, approaches, training works, and temporary construction facilities, so far as is compatible with requirements of structural adequacy, safety,

economy, and aesthetics, should be designed to minimize local scour, obstruction of flow, and inconvenience to legitimate interests. (Project Committee on Bridge Hydraulics, 1972)

Chapter 3: Description of the study area

3.1 General

The study area starts in Alemgena, at the junction with the Addis Ababa-Jima road, some 20 Km from Addis Ababa. The road links the central part of the country to the southern regions specially the Oromya and southern National and Nationalities Administrative regional states. The road is 120 Kilometers long and runs in a southerly direction, passing through the towns of Boneya, Melka kuntre, Lemen, Bui and Butajira. Throughout the road length, there are many bridges and culverts. The area through which the road passes is characterized by extensive small- scale agricultural activity, which results in the continuous movement of produce along the road. The road was constructed by double surface treatment pavement structure in recent years; nevertheless, severe damage is noticed in most parts of the road segment.

3.1.1 Location of the study area

The study area is located south west of Addis Ababa in central Ethiopia.

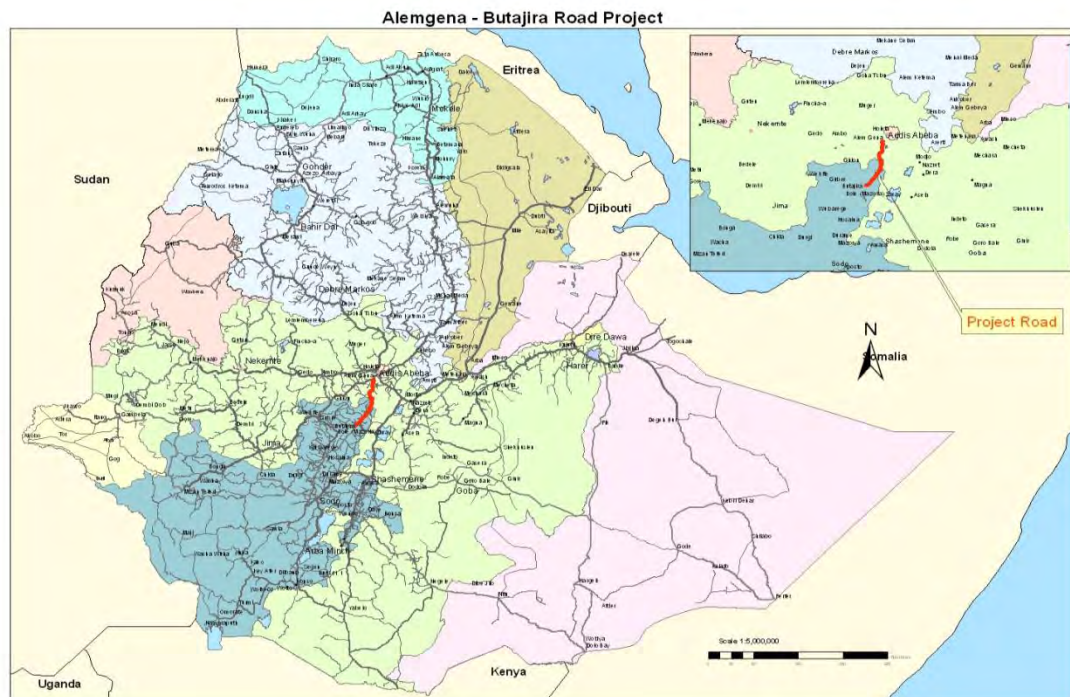


Figure: 3.1 a

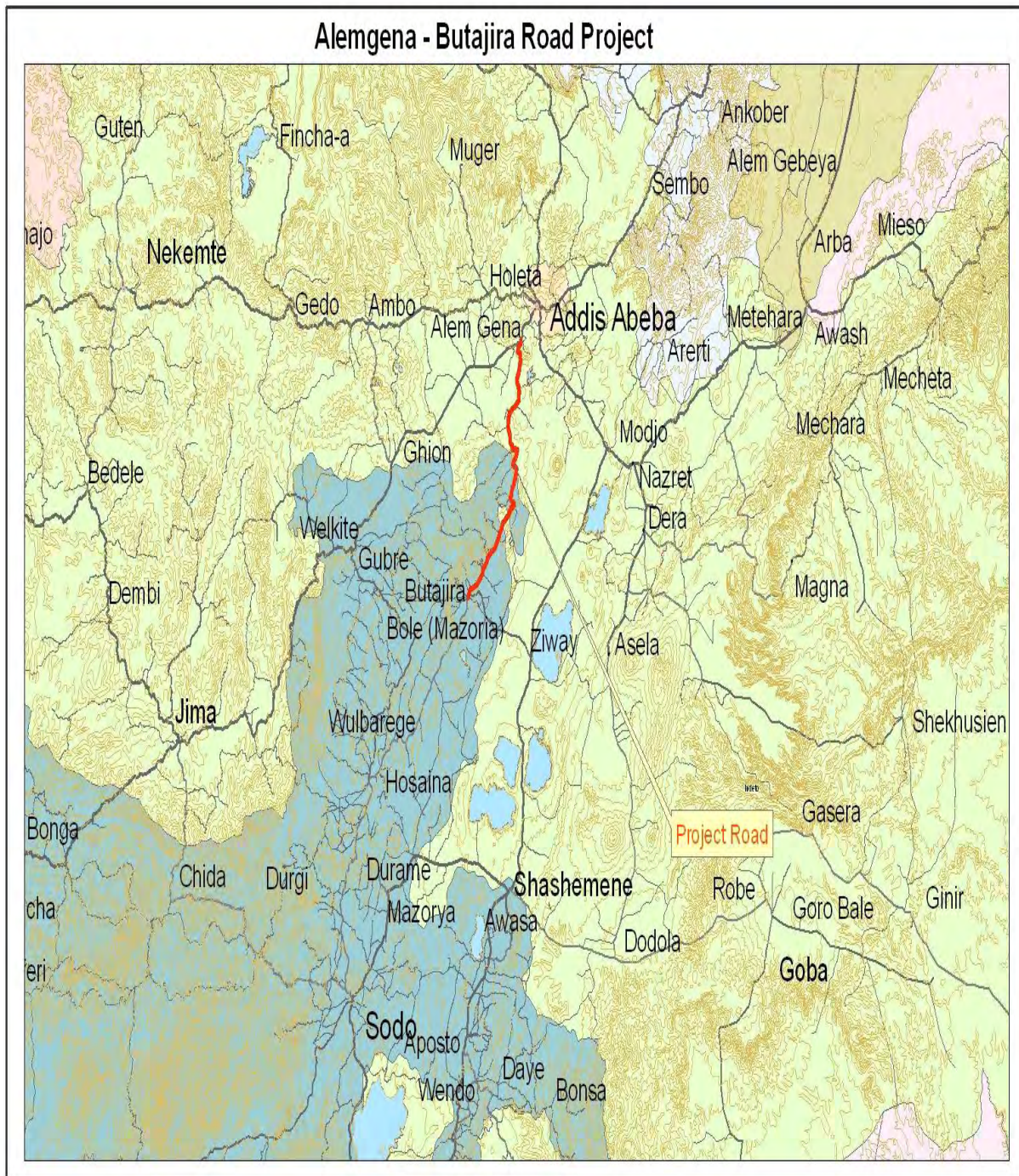


Figure: 3.1 b

Figures 3.1 a & b: Location maps of the study area

while the remaining 12% includes settlements, eroded areas, gorges, steep slopes and rocky areas unsuitable for farming.

Individual farmers manage almost all the cultivated land and grazing is exploited both communally and individually. The area is densely populated and in most cases the plots are very small, varying between 0.5 ha (towards the South) and 2.0 ha in the drier areas.

Land Cover: -The visual impact of the depletion of the natural vegetation seems greater along the northern sections. This depletion may be due to relatively lower rainfall.

Patches of natural vegetation mainly open bush and shrub lands and scattered trees remain on road boundaries and in area unsuitable for cultivation and grazing such as hillsides, gorges, eroded areas.

The grass lands which remain between the large tracts of ploughed areas are short and of a poor quality due to overgrazing.

In terms of bio-mass, eucalyptus is dominant. It is treated as a crop and widely planted around homesteads, along plot boundaries, grazing areas, roads and footpaths. It is the major source of timber, fuel and construction material.

(c) Climate

Rainfall: -There are two wet seasons. The so-called „little rains“ (belg) generally fall between March and May while the main rainy seasons, the „heavy rains“ (keramt), occur between June and September. The rains, whether „heavy“ or „little“ are characterized by falling in a large amounts that often starts suddenly



Figures 3.3 Rainfall Isohyets' of study area(Engineering Report of Alemgena-Hossana-Sodo)

Temperature: - The area lies between 6.75°N (Sodo) and 9°N (Alemgena) the elevation of the area results in a definitely cooler climate than its being near in distance to the equator may suggest another typical of a place is not existing extreme temperatures.

3.1.3. GEOLOGY

The Alemgena-Butajira road follows the north-western edge of the Ethiopian Rift Valley, next to what is known as the Lakes Region. The geology along the route is therefore dominated by the geological formation of the Rift Valley. This section describes the geological formations that occur along the route. Topography of the area is primarily determined by the rift system of faulting. Rift margin faults have undergone a long period of erosion while those of the rift floor are mostly recent. The rift floor consists mainly of extensive flat lands while the escarpment areas have mostly steep slopes and cliffs.

The Ethiopian Rift valley is part of the East African Rift valley. Following the regression of the Mesozoic Sea to the southeast, a major uplift which is known as the Arabo- Ethiopian swell occurred, this subsequently resulted in the formation of the East African Rift as well as the Red Sea and Gulf of Aden. The tertiary uplift and formation of the rift was associated and followed

by the extrusion of large masses of basaltic magma through fissures. Although the magnitude of the tertiary uplift was affected by later tectonic events, it can be observed that tremendous uplift occurred where the present rift valley lies. Here the basement surface was probably uplifted over 2800 M (Chernet, 1982)

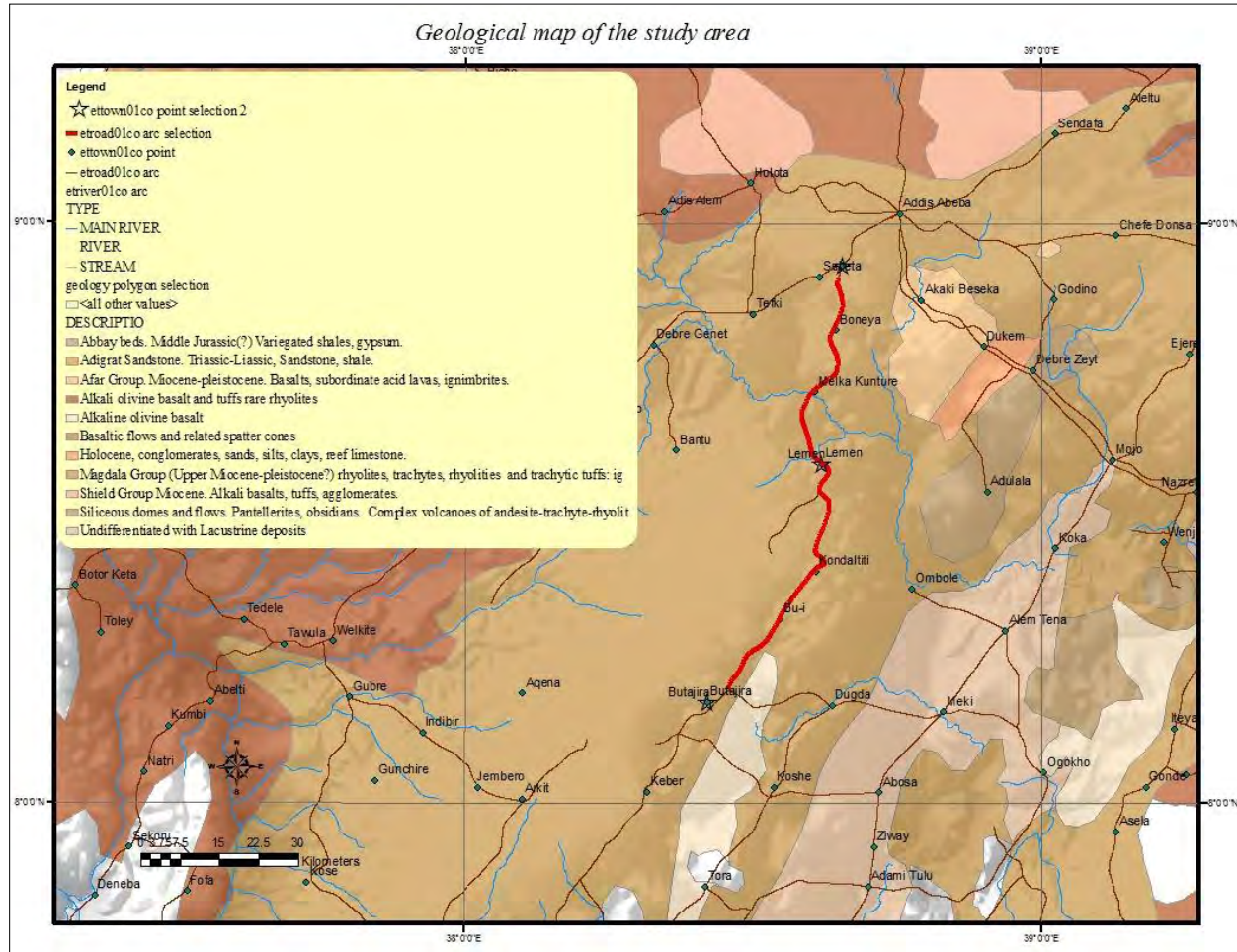


Figure 3.4 Geological Map of the Study area (Awash River Basin Master Plan Study)

3.1.3.1 Soils

Three main types of soils are observed in the area: Sandy/Silty soils, lateritic clay soils and brown to dark brown clayey soils. The sandy soils are derived from ignimbrites, unwelded pumiceous pyroclastics and some beds of lacustrine sediments. Lateritic clay soils occur along the Butajira-Hossana road. Similar lateritic paleosols are observed in the same areas between ignimbrite flows. The thickness of the lateritic soil is variable, but mostly less than 2m. Deep gullies cut in the soil are common. The region is classified as a region of high soil erosion with more than 2000 tons per square kilometer per year (Atlas of Ethiopia, 1970). Dark clay soils occur where there are basalt flows of fine-grained quaternary sediments. The soils on lacustrine

sediments are generally thin in layers of thickness less than 1 meter. In the Butajira basalt area, dark clayey and silty soils occur with thickness greater than 1 meter.

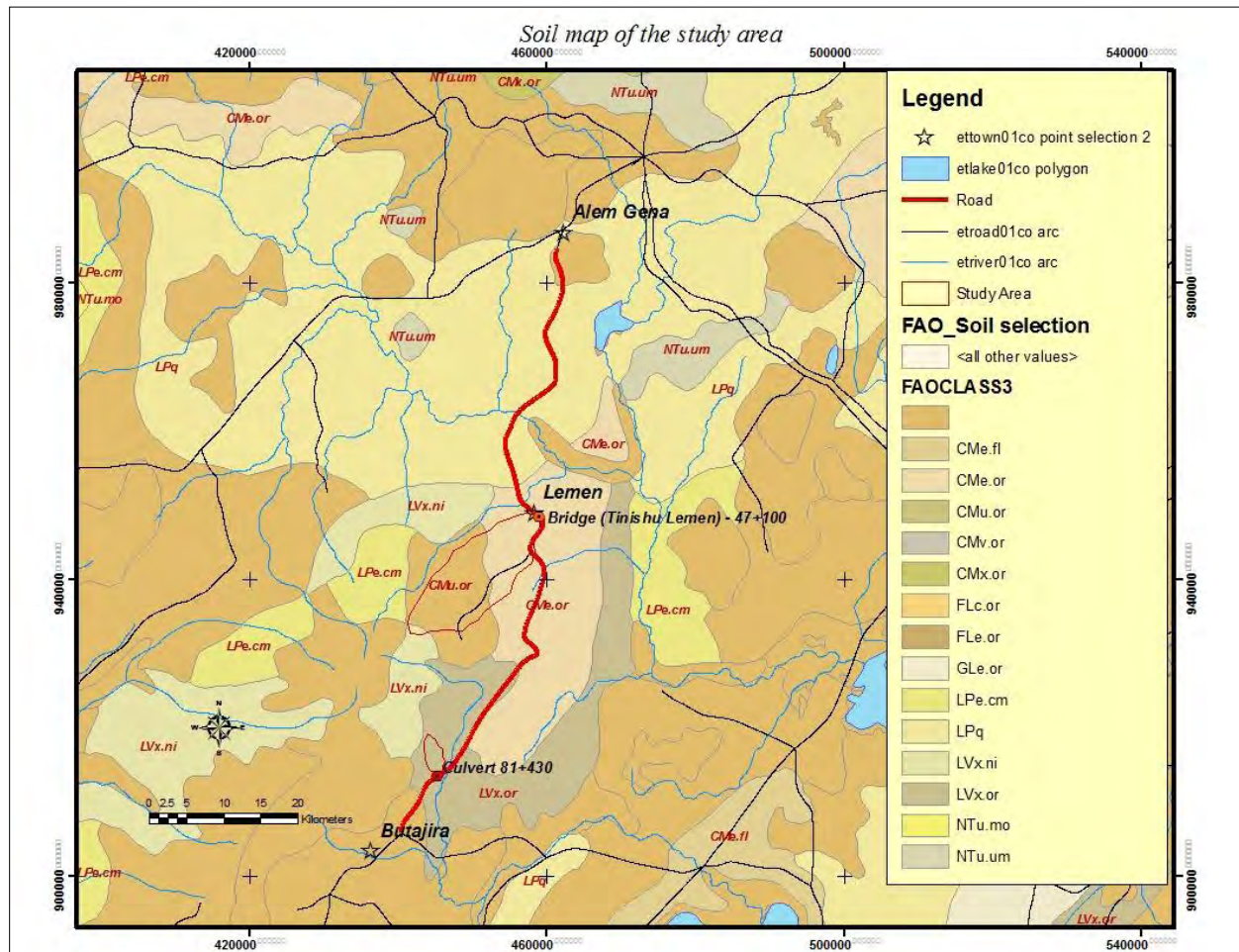


Figure 3.5 soil Map of the Study area(FAO)

3.1.3.2. Stratigraphy

a) In general

The short descriptions of the main formation in the Stratigraphy of the, area adopted from the „Geological Map of the Ethiopian Rift, 1980“ is presented below as described by Chernet. (Chernet, 1982)

Nazareth Group

These are alkaline and pearalkaline stratoid Silicics, ignimbrites, unwelded triffs, ash flows, rhyolites, domes and trachytes. They range in age between 2 and 9 million years and are mostly located on the escarpments. Flow thickness varies widely, from 1 to 30 meters on the plateau and up to 250 meters in the rift.

i. Chilalo formation

This formation consists mainly of trachytic lava flows with frachy-basalt, peralkaline rhyolite, overlain by subordinate alkaline basalts.

ii. Dino formation

The dino formation includes peralkaline Silicics. It consists of ignimbrites tuffs, waterlain pyroclastick, occasional lacustrine beds which are overlain by coarse unwelded pumiceous pyroclastics. Typical ignimbrite is a hard, well-welded rock with nicely developed „fomme“ containing small inclusions of foreign rocks. The typical ignimbrite is the most common of the three rock types in the rift pyroclastic formation.

The layered pumiccous pyroclastics consist of unwelded pumice of small size. The rift ignimbrites are highly faulted and they outcrop in most places of the rift. Most of these pyroclastics were probably erupted through fissures.

iii. Basalts

There are mildly alkaline (transitional) basalts and trachyte. A main field of recent basalt flow having an area of about 800 square kilometers is in the Butajira-Silte area. This flow consists of a lot of scoria and its texture varies from aphyric to porphyritic. Basalts of the Bishoftu formation occur in the area near Alemgena.

Some rhyolitic lava flows and domes are found associated with the rift ignimbrites. Ignimbrites and pumic are the result of gas-rich Silicic magma, most probably, the same magma, having lost its gases during the explosive activity, was erupted later on as viscous land flows and domes.

iv. Alluvial and Lacustrine Deposits

The lacustrine Sediments include silts, clays, diatomites, volcanoclastic sediments and tuffs. Silts and clays are however dominant.

v. Rhyolitic Volcanic Centers

These include obsidian pitchstone, pumice, ignimbrite, tuff and subordinate trachytic flows.

b) Along the route

Alemgena to Butajira

This section crosses large portions of the Nazareth Group. Near Alemgena the route passes over or near to trachytes and basalts of the Bishoftu and chilalo formation. Just north of Butajira the road crosses onto the dino formation volcanic cinders occur at a few locations in

characteristically straight-sided cone-shaped hills which frequently have large concave depressions in their topped or sides where mixtures of solids and gases were released during the formation of the cone. Cinders vary in colour often within the same cone and may be red, brown, gray or black.

3.2 Assessment of Performance and condition of the structure

3.2.1. Identification of Selected Bridge and culvert condition

On Alemgena- Butagira Road Segment there are different types of damages of bridge and culverts Therefore, based on material used and structural nature of the structures and components is shown below,

Components of Bridge based on material and structural type.

- Bridge pavement
- Curb & Railing
- Expansion Joint
- Deck Slab
- Rc Girder & Rc Arch
- Bearing
- Deck Drain
- Pier & Foundation
- Abutment
- Embankment around Wing Wall & Approach Roads
- Rip Rap
- Masonry Arch , Wing Wall and Retaining Wall

Selected Bridge 47+100

- Location (Km, from AlemGena):- 47+ 100
- Bridge type: - RC Slab Culvert
- Bridge No: - BS1- 1- 012
- Bridge Length (Mt):- 22.00
- Type of surface: - Asphalt
- Type of guard railing: - RC
- Detour possibility: - Yes
- Road segment: - Lemen- Butagira



Figure 3.6:- Previous damage RC Slab culvert type Bridge



1.1. Damage to Wing Wall

❖ Scour

Foundation is washed out



1.2. Damage to Deck Slab

- ❖ Water leakage

Leaking of water and leaching of free lime through cracks or voids.



- ❖ Void

Cavity or voids due to poor vibration in placement of concrete.



❖ Rebar exposure

Exposure of rebar due to peeling or spalling off surface concrete and rebar corrosion caused by oxidation or other rusting condition.



Selected culvert 81+430

Location (Km, from Alem Gena):- 81+430

Culvert type: - Slab (SC)

Culvert No: - B51-1-C-130

Culvert /Barrel length (Mt):- 11.0

Type of surface: - Asphalt

Type of Guard Rail/Parapet Material:-Masonry

Detour possibility: - Yes

Road Segment: - Lemen-Butajira

1. Damage to paved water way

- ❖ Cracking

Cracking of paved water way

- ❖ Scouring of paved water way



Figure 3.6a Cracking and scouring of paved water way



Figure 3.7b scouring of waterway outlet side

Figure 3.6a and figure 3.7b:- previous damaged slab at station 81+430

Chapter 4: Materials and methods

During field survey in the study area the items used were measuring tapes, digital camera, GPS, small knife, caliper, ball pin hammer, pen& pencil, cap, safety jacket, hand torch. The problem of each drainage system inlet and outlet area, waterway condition, sedimentation and size were assessed for their performance based on criteria. In addition flood marks were observed and information was gathered.

For this thesis I used descriptive and explanatory types of research. The descriptive type of research was used to describe the existing system whereas explanatory type was used to examine carefully the existing performance condition of drainage structures.

The design flows for these drainage structures have been calculated on the basis of Rational and SCS equations. It should be noted that an official suggestion in ERA 2002 and 2011 for HVRs drainage design manuals were used to determine peak discharges. I used these manuals as the reference while doing my thesis work. Because these manuals are intended for use in the design of all roads in Ethiopia and also it gives guidance and recommendations to those engaged in the design of road drainage in Ethiopia.

4.1. Hydrological Model

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

4.1.1 Design rainfall

The hydrological study deals with estimation of design discharge by analyzing rainfall data, stream flow record, topographic map, Arial photograph and surveyed data. There is no continuous gauge record data available for such as an assessment in the study area. The

momentary peak flow is the basis on which the drainage structures for a road of sized and designed, therefore the lack of such records is a significant handicap.

The design discharge is the most useful parameter when the waterway area of the drainage structures has to be designed. When design flood is known, the water stage and the velocity of flow can be determined by means of hydraulic calculation for each drainage structure. There are many methods developed for calculation of the design flood but their applicability depends mainly on the availability of hydrological data. For the current hydrological study, the information required for estimating the design discharge was based on previous studies, ERA drainage design manual IDF curves, and topographic map of the study area.

The IDF curves of ERA were produced by classifying rainfall regions into four major rainfall and eight sub-rainfall regions in the country. For comparing the produced IDF curve with created IDF curve of the study area local rainfall data are required. However, near the study area representative ample rainfall data are not available. To determine rainfall intensity the already produced IDF curve by ERA was used.

For rainfall regions in the country ERA produced four IDF curves. The produced curves are for A1&A4, A2&A3, B, and C&D.(See Appedix I, Figure f). The study area lies in sub-regionA2and the IDF curve corresponding to this zone was used to determine design rainfalls of different return periods.

4.2 Hydrological equations for peak flood computation

Assessment of the design flow can be grouped into two major categories, namely a. Statistical and b. deterministic.

Statistical methods do not require much objective judgments and experience to apply. This method applies the techniques and procedures of modern statistical analysis to actual or synthetic data and fit the required design parameters directly.

Deterministic methods often require a large amount of judgments and experience to be used. In this methods the physical aspects of the rainfall- runoff process either conceptually or empirically, where the relationship between rainfall and runoff is quantified based on measured data and experience.

There are many methods developed for calculation of the design flood but their applicability depends mainly on the availability of hydrological data. Due to unavailability of hydrological data, in most cases Rational method and soil conservation services (SCS) methods of design flood estimation are applied for minor drainage structures. According to the area of the

catchment, and based on concepts mentioned before, rational and SCS mathematical equations are adopted for this thesis.

4.2.1 Rational Method

The rational method is a method for determining peak runoff in terms of m^3/s at the point of interest. It is based on a simplified representation of the process and variables involved in flood run-off. Rainfall intensity is an important input to the calculations. Because uniform area and time distribution of rainfall have to be assumed, the method is normally only recommended for catchments smaller than about 0.5 Km^2 . Judgment and experience on the part of the user play an important part in this method, but owing to improved methods for determining run-off coefficients, the part played by subjective judgment is becoming increasingly less important. However, for this thesis uniform area and time distribution of rainfall are considered. Only a portion of the rain that falls on a watershed appears as surface run-off in a stream. Rainfall becomes run-off when all loss processes are satisfied. Infiltration losses have the greatest effect on surface runoff. The rate of infiltration is a function of the soil and is highly heterogeneous over space and time.

Any rational formula to determine the amount of runoff must consider the following factors:-

Drainage area tributary to a point under design, the shape of the area and the topography of the area, all of which will indicate the time factor on the rate at which the runoff will reach the drainage structure

- ❖ Probable future condition of the drainage area, i.e., percentage of impervious surface or character of land use when developed to the extent assumed.
- ❖ Selection of an appropriate runoff coefficient for the entire area or for component inlet areas
- ❖ Rainfall and intensity-duration curve for the locality.
- ❖ Time of concentration including both inlet and conduit flow time to point of design.

The rational method applies to small watersheds where storage routing is not necessary. The method is useful for determining peak flows from small subdivisions and development projects or to determine flows to catch basins. The error in the run-off estimate increases as the size of the drainage area increases. Owing to these facts, the rational method is not used to determine the rate of run-off for large drainage areas. The use of the rational method for the design of highway drainage structures is restricted up to 50 hectares in Ethiopia.

The rational method is expressed by the formula:-

$$\text{➤ } Q_{10} = 0.00278C_iA \quad (4.1)$$

$$\text{➤ } Q_{25} = 0.00278C_fC_iA \quad (4.2)$$

In which Q = the runoff in m^3/s from a given area

I = the intensity of rainfall in mm per hour for a duration equal to the time of concentration.

C = a coefficient representing the ratio of runoff to rainfall

A = the drainage area in hectares

C_f = frequency factor

The value of C to be used must be estimated from a study of the soil, the slope, and condition of the surface, the imperviousness of the surface and a consideration of probable future changes in the surfaces within the area.

The value of I to be selected depends upon the curves for the intensity of rainfall plotted for the local vicinity and the assumed period of reoccurrence as well as the period of concentration required for surface runoff to flow from the most distant point in the area under study, to the culvert.

The value of A is measurable and can be accurately determined.

To design drainage structures a thorough study must be made of the drainage area, of the available rainfall and runoff data. Careful consideration must be given to each variable that enters the runoff problem. Experience and good judgment must be used in good measure in order to insure a culvert of adequate capacity yet not wasteful oversized.

For this thesis the maximum value of the catchment area, 28 hectares is considered.

4.2.1.1. Run-off Coefficient

The run-off coefficient, C , in the rational formula is the ratio of the rate of run-off to the rate of rainfall at an average intensity, I , when all the drainage area is contributing. It is, more simply, the percentage of rainfall on a given area that flows off as free water. The run-off factor seldom will reach 100 percent because even impervious surfaces on steep slope will absorb some moisture, and small depressions and irregularities will hold back additional amounts. As impervious areas become thoroughly wetted, soil becomes saturated, and depressions become filled, the amount of run-off will gradually increase; the coefficient thereafter will remain nearly constant, varying directly with the intensity of rainfall. The composite effect of all these factors must be considered. When the drainage area is composed of several types of cover the coefficient used in the formula should be a weighted average of the C values for each types of cover present.

(The weighted average, C_w , may be computed from the following

$$C_w = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \dots}{A_1 + A_2 + A_3 + \dots} \quad (4.3)$$

Where C_w = weighted average coefficient of run-off for the whole area,

$C_1, C_2, C_3, \text{ etc}$ = run-off coefficient for the type of cover present,

$A_1, A_2, A_3, \text{ etc}$ = areas representing each type of cover present.

4.2.1.2 Rainfall Intensity

The intensity of a design storm increases as the return period becomes longer and as the duration of the storm decreases. To obtain the largest possible peak flow for a given return period using the rational method, the storm rainfall must have duration equal to the time of concentration. If the storm has a shorter duration, it will not be possible for all the parts of the catchment to contribute simultaneously to run-off at the point of measurement.

Consequently, the effective catchment area will be smaller than the actual catchment. If the storm lasts longer than the time of concentration, it will have a lower intensity. Apart from the duration and return period, the intensity of rainfall is also related to the mean annual rainfall and to the rainfall region.

To quantify rainfall, I used ERA regionalized IDF curves. The location of the study area is in the rainfall region of Ethiopia, in rainfall sub-region A2 as shown on Appendix 1 on figure f.

4.2.1.3. Time of Concentration

The theory underlying the rational method, maximum discharge at any point in a drainage system occurs when:-

The entire area tributary to that point is contributing to the flow

The rainfall intensity is at the maximum which can be expected for rainfall duration equal to the time of concentration. (Concise Hand book of civil engineering, 2002). By the time of concentration is meant the time required for run-off from most remote point of the catchment to contribute to the peak discharge.

In most cases the longest water path includes both overland and channel flow. In large catchments the channel flow is usually the dominant part, but in small catchments it may be necessary to determine T_c as the sum of the flow times for both overland and channel flow. To obtain a broad indication, it can usually be accepted that a defined watercourse exists when the

average slope of the catchment is greater than 5 percent and the catchment itself is larger than 5 Km².

Waterfalls and high rapids must not be taken into account in the height differences.

In urban areas the time of concentration must be determined, where applicable, by means of the flow velocity according to the Chezy or Manning equations.

With road drainage the volume of water that runs off as a result of storms of less than 15 minutes duration is usually not large and much of this run-off is absorbed in filling of the watercourses. Time of concentration of less than 15 minutes is therefore generally not important. It is sound practice to calculate the average flow velocity ($v = \frac{L}{T_c}$) after T_c has been determined in order to ensure that it falls within realistic limits.

Typical values of the flow velocity range from 0.1 to 4 m/s, depending on the natural conditions.

Overland flow: - This usually occurs in small, flat catchments or in the upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface.

The Kerby formula is recommended for the calculation of T_c in this case. It is only applicable to parts where the slope is fairly even.

$$T_c = 0.604(rL/S^{0.5})^{0.467} \text{ (hours)} \quad (4.4)$$

Where: - r = Roughness Coefficient, L = Hydraulic length of catchment (Km),

$$S = H/1000L \quad (4.4.1)$$

H = Height of most remote point above outlet of catchment (m)

Recommended values of r are as follows:-

$r = 0.1$ for clean compacted soil, no stones,

$r = 0.02$ for paved areas,

$r = 0.3$ for sparse grass over fairly rough surface,

$r = 0.4$ for medium grass cover,

$r = 0.8$ for thick grass cover.

Defined watercourse: - in a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels is the one developed by the US soil conservation service.

$$T_c = (0.87L^2 / 1000 S_{av})^{0.385} \quad (\text{hours}) \quad (4.5)$$

Where: - L = Hydraulic length of catchment (Km),

S_{av} = average slope

4.2.2. The SCS Curve Number Method

The SCS Curve Number Method (United States Soil Conservation Service Hydrograph Generating Technique) is particularly suitable for computing flood peaks and run-off volumes for catchments smaller than 10 Km² and with slopes of less than 30 percent. It is mainly applicable to rural catchments but may also be used for urban areas. The basic method requires a considerable amount of calculation, but this can be greatly reduced by using nomograms.

The SCS Method takes into account most of the factors that affect run-off, such as quantity, time distribution and duration of rainfall, land use, soil type, prevailing soil moisture conditions, and size and characteristics of the catchment. An advantage of the SCS Method is that it enables empirical hydrographs to be fully calculated.

4.2.2.1. Catchment Area

Topographical maps (1: 50,000) are usually used to determine the area of a catchment. However, the accuracy and contour intervals on these maps are not always as required and it is often very useful to obtain unchecked pencil compilations to a scale 1: 10,000.

Ortho-photographs should also be used, if available. However, it is considered essential for the designer to visit the site so that he can confirm the boundaries of the catchment and the location of the watercourses.

4.2.2.2. Rainfall-runoff equation

In contrast to the rational and some other methods, the SCS Curve Number method does not contain rainfall intensity as a basic variable, but 24-hour rainfall data for different return period are used to calculate the storm volume for design purposes. The typical time distribution of 24-hour storms for the climatic region are defined for three levels of maximum intensity, type I being the least intense and type III being the most intense. According to ERA drainage design manual (ERA, 2002) a type II distribution is used. It is applicable to areas where convection activity (summer thunderstorms) is the main cause of flood rainfall over small catchments. Therefore, for Ethiopia type II distribution is suitable. So I used type II distribution for the study area.

A relationship between accumulated rainfall and accumulated runoff was derived by SCS from experimental plots for numerous soils and vegetative cover conditions. The equation was developed mainly for small watersheds for which only daily rainfall and watershed data are

ordinarily available. It was developed from recorded storm data that included total amounts of rainfall in a calendar day but not its distribution with respect to time. The SCS runoff equation is therefore a method of estimating direct runoff from 24-hour or 1-day storm rainfall.

$$\text{The equation is: } -Q = (P - I_a)^2 / (P + I_a) + S \quad (4.6)$$

Where: - Q = accumulated direct runoff, mm

P = accumulated rainfall (potential maximum runoff, mm

I_a = initial obstruction including surface storage interception, and infiltration, prior to runoff, mm,

S = potential maximum retention, mm

The relationship between I_a and S was developed from experimental watershed data. It removes the necessity for estimating I_a for common usage.

The empirical relationship used in the SCS runoff equation is:-

$$I_a = 0.2S \quad (4.6.1.)$$

Substituting 0.2S for I_a in equation (4.6) the SCS rainfall-runoff equation becomes: -

$$Q = (P - 0.2S)^2 / (P + 0.8S) \quad (4.6.2.)$$

4.2.2.3. Travel time estimation

Travel time (T_t) is the time it takes water to travel from one location to another in watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the water-shed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

Following is a discussion of procedures and equations for calculating travel time and time of concentration.

Travel time: - water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection. Travel time is the ratio of flow length to flow velocity:-

$$T_t = L / (3600V) \quad (4.7.)$$

Where: - T_t = travel time, hr,

L = flow length, m,

V = average velocity, m/s,

3600 = conversion factor from second to hours.

4.2.2.4. Time of concentration

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

$$T_c = T_{t1} + T_{t2} + \dots + T_{tm} \quad (4.8.)$$

Where: - Tc = time of concentration, hr,

m = number of flow segments.

4.2.2.5. Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the head water of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.03m or so. (Appendix 2 Table C) gives Manning's n values for sheet flow for various surface conditions length.

For sheet flow of less than 90m, use Manning's kinematic solution (Overton and Meadows 1976) to compute Tt:-

$$T_t = [0.007(nL)^{0.8} / (P_2)^{0.5} S^{0.4}] \quad (4.9.)$$

Where: - Tt = travel time, hr,

n = Manning's roughness coefficient (Appendix 2 Table C),

L = flow length, m,

P₂ = 2-year, 24-hour rainfall, m,

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

This simplified form of the Manning's kinematic solution is based on the following:-

- ❖ Shallow steady uniform flow,
- ❖ Constant intensity of rainfall excess (rain available for runoff),

- ❖ Rainfall duration of 24-hours,
- ❖ And minor effect of infiltration on travel time.

4.2.2.6. Shallow concentrated flow

After a maximum of 90m, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure D-1 on the Appendix -----, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.0015 m/m, use equations given below figure D-1 average velocities for estimating travel time for shallow concentrated flow using figure D-1

$$\text{Unpaved} \quad V = 16.1345(S)^{0.5} \quad (4.10.)$$

$$\text{Paved} \quad V = 20.3282(S)^{0.5} \quad (4.11.)$$

Where: - V = average velocity, m/s

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

These two equations are based on the solution of Manning's equation with different assumptions for n (Manning's roughness coefficient) and r (hydraulic radius, m).

For unpaved areas, n is 0.05 and r is 0.4;

For paved areas, n is 0.025 and r is 0.2.

After determining average velocity using figure D-1, use equation 4.7 to estimate travel time for the shallow concentrated flow segment.

4.2.2.7. Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological survey (USGS). Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

$$\text{Manning's equation is: - } V = 1/nS^{1/2}(A/P)^{2/3} \quad (4.12)$$

Where: - V = average velocity, m/s,

r = hydraulic radius, m (equal to a/P_w),

a = cross sectional flow area, m^2 ,

P_w = wetted perimeter, m,

S = slope of the hydraulic grade line, m/m,

n = Manning's roughness coefficient.

After average velocity is computed using equation 4.12, T_t for the channel segment can be estimated using equation 4.7.

4.2.2.8. Runoff and curve numbers

The curve number (CN) is SCS designated measure that indicates the runoff potential of an area. Its determination is based on soil condition and land use (ground cover) and is estimated from a classification in one of the four hydrologic soil groups (A, B, C or D) together with the hydrologic soil condition (poor, fair, and good).

Care shall be taken in the selection of curve numbers (CN'S). Use a representative average curve number, CN, for the catchment area. Selection of overly conservative CN'S will result in the estimation of excessively high runoff and consequently excessively costly drainage structures. Selection of conservatively high values for all runoff variables results in compounding the runoff estimation. It is better to use average values and design for a longer storm frequency. The hydrologic designer could consider doing both in making the most appropriate selection of design discharge.

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 obstruction.

$$\text{This empirical relationship is: } - F/S = Q/P-I_a \quad (4.13.)$$

Where, F = actual retention (mm)

S = potential maximum retention (mm)

Q = accumulated runoff depth (mm)

I_a = initial obstruction and runoff (mm)

The actual retention is the difference between rainfall minus initial obstruction and runoff

$$F = P- I_a- Q \quad (4.13.1.)$$

Maximum retention S has been converted to the curve number CN to make the operations more nearly linear.

A parameter CN (hydrological soil cover complex number) is introduced as:-

$$CN = 25400/S+254 \quad (4.14.)$$

The curve number method was developed with daily rainfall data measured with non-recording gauges. Therefore rainfall intensity is not included in the estimate of runoff depth

4.2.2.9. Hydrological soil groups

Soil properties influence the relationship between runoff and rainfall since soils have differing rates of infiltration. Infiltration is the movement of water through the soil surface into the soil. Based on infiltration rates, the soil conservation services (SCS) has divided soils into four hydrologic soil groups as follows:-

- Group A soils having a low runoff potential due to high infiltration rates. These soils consist primarily of deep, well drained sands and gravels.
- Group B soils having a moderately low runoff potential due to moderate infiltration rates. These soils consist primarily of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.
- Group C soils having a moderately high runoff potential due to slow infiltration rates. These soils consist primarily of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.
- Group D soils having a high runoff potential due to very slow infiltration rates. These soils consist primarily of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

4.3. Hydraulic equations (hydraulic calculation)

4.3.1 General

Only three fundamental laws are applied in hydraulic calculations. These laws related to: -

- Conservation of mass (continuity principle);
- Conservation of energy;
- Conservation of momentum.

Depending on what information is available and what is needed, every design calculation basically involves the application of one or more of these laws.

Where complete mathematical description of flow conditions becomes too difficult or cumbersome, empirical coefficients are used to compensate for the simplifying assumptions that are made.

4.3.2. Manning's equation

Manning's equation for a given depth of flow in a channel with a steady, uniform flow, the mean velocity, V , can be computed with Manning's equation:

$$V = (1/n) R^{2/3} S^{1/2} \quad (4.15.)$$

Where: V = velocity, m/s,

n = Manning's roughness coefficient

R = hydraulic radius = A/P , m, P = wetted perimeter, m

S = slope of the energy gradeline, m/m (for steady uniform flow, S = channel slope, m/m)

The selection of Manning's „ n “ is generally based on observation; however, the range of „ n “ values for various types of channels and flood plains is given in Appendix 2 table c.

For a given channel geometry, slope, and roughness, and a specified value of discharge Q , a unique value of depth occurs in steady uniform flow. It is called the normal depth. The normal depth is used to design artificial channels in steady, uniform flow and is computed from Manning's Equation:

$$Q = (1/n) A R^{2/3} S^{1/2} \quad (4.16.)$$

Where: Q = discharge, m³/s,

n = Manning's roughness coefficient,

A = cross-sectional area of flow, m²,

R = hydraulic radius = A/P , m,

P = Wetted perimeter, m,

S = channel slope, m/m

4.3.3. Equations of Heads in Culvert Barrels

$$\text{Losses} \quad H_l = H_e + H_f + H_v + H_b + H_j + H_g \quad (4.17)$$

Where: - H_l = total energy loss, m,

H_e = entrance loss, m,

H_f = friction losses, m,

H_v = exit loss (velocity head), m,

H_b = bend losses, m,

H_j = losses at junctions, m,

H_g = losses at grates, m,

$$\text{Velocity: - } V = Q/A \quad (4.18.)$$

Where: - V = average barrel velocity, m/s,

Q = flow rate, m³/s,

A = cross sectional area of flow with the barrel full, m²

$$\text{Velocity Head: - } H_v = V^2/2g \quad (4.19)$$

Where: - g = acceleration due to gravity, 9.8 m/s²

$$\text{Entrance loss: - } H_e = K_e (v^2/2g) \quad (4.20.)$$

Where: - K_e = entrance loss coefficient,

$$\text{Friction Loss: - } H_f = [(19.63n^2 L) / R^{1.33}] [V^2/2g] \quad (4.21.)$$

Where: - n = manning's roughness coefficient

L = length of the culvert barrel, m

R = hydraulic radius of the full culvert barrel = A/P , m

P = wetted perimeter of the barrel, m

$$\text{Exit loss: - } H_o = 1.0[(V^2/2g) - (V_d^2/2g)] \quad (4.22.)$$

Where: - V_d = channel velocity downstream of the culvert, m/s (usually neglected, resulting in equation 4.22.)

$$H_o = H_v = V^2/2g \quad (4.22.)$$

$$\text{Barrel Losses: - } H = H_e + H_o + H_f \quad (4.23.)$$

$$H = [1 + K_e + (19.63 n^2 L/R^{1.33})] [V^2/2g] \quad (4.23.1.)$$

4.4. Types of data needed

4.4.1. General

The designer must compile the data that are specific to the subject site.

Following are the major types of data that may be required: -

- Watershed characteristics;
- Stream reach data (especially in the vicinity of the facility);
- Other physical data in the general vicinity of the facility such as utilities, easements, etc. Hydrologic and meteorological data (stream flow and rainfall data related to maximum or historical peak as well as low flow discharges and hydrographs applicable to the site);

Existing and proposed land use data in the subject drainage area and in the general vicinity of the facility, anticipated changes in land use and/or watershed characteristics; and floodplain and environmental regulations. Watershed, stream reach and site characteristic data, as well as data on other physical characteristics can be obtained from a field reconnaissance of the site. Examination of available maps and aerial photographs of the watershed is also an excellent means of defining physical characteristics of the watershed.

4.4.2. Drainage surveys

A complete field or aerial drainage surveys of the site and its contributing watershed should always be undertaken as part of the hydraulic analysis and design. Survey requirements for small drainage facilities such as 36 inch culverts are less extensive than those for major facilities such as bridges. However, the purpose of each survey is to provide an accurate picture of the conditions within the zone of hydraulic influence of the facility.

Following are the data that can be obtained or verified: -

- Contributing drainage area characteristics;
- Stream reaches data-cross sections and thalweg profile;
- Existing structures;

Location and survey for development, existing structures, etc. , that may affect the determination of allowable flood levels, capacity of proposed drainage facilities, or acceptable outlet velocities; drift/debris characteristics; general ecological information about the drainage area and adjacent lands; and high water elevations including the date of occurrence. Much of these data must be obtained from an on-site inspection.

4.5 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 collected but around the study area there is no sufficient meteorological station that recorded the rainfall, temperature, and relative humidity. The main choice is using the IDF curve developed by ERA for roads drainage design manual in 2002 for Ethiopian rainfall regions.

According to ERA drainage design manual ERA drainage design manual 2002, the study area is found on the hydrological sub-region of B2. Hence, the IDF curve developed for B, C and D are used.

There are no ample 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.

4.6 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.

CHAPTER 5- Results and Discussion

5.1 Hydrologic Analysis

On Alemgena-Butajira road, drainage structures performance assessment, the maximum peak flood was 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. Two structures are selected on Alemgena-Butajira road based on the prioritized problem. Therefore, the Hydrologic analysis for one culvert and one bridge has been done by using Rational and SCS methods respectively.

5.2 Delineation of catchment Area

Delineation of the catchment areas for existing and proposed drainage structures is shown in Figure 5.1.

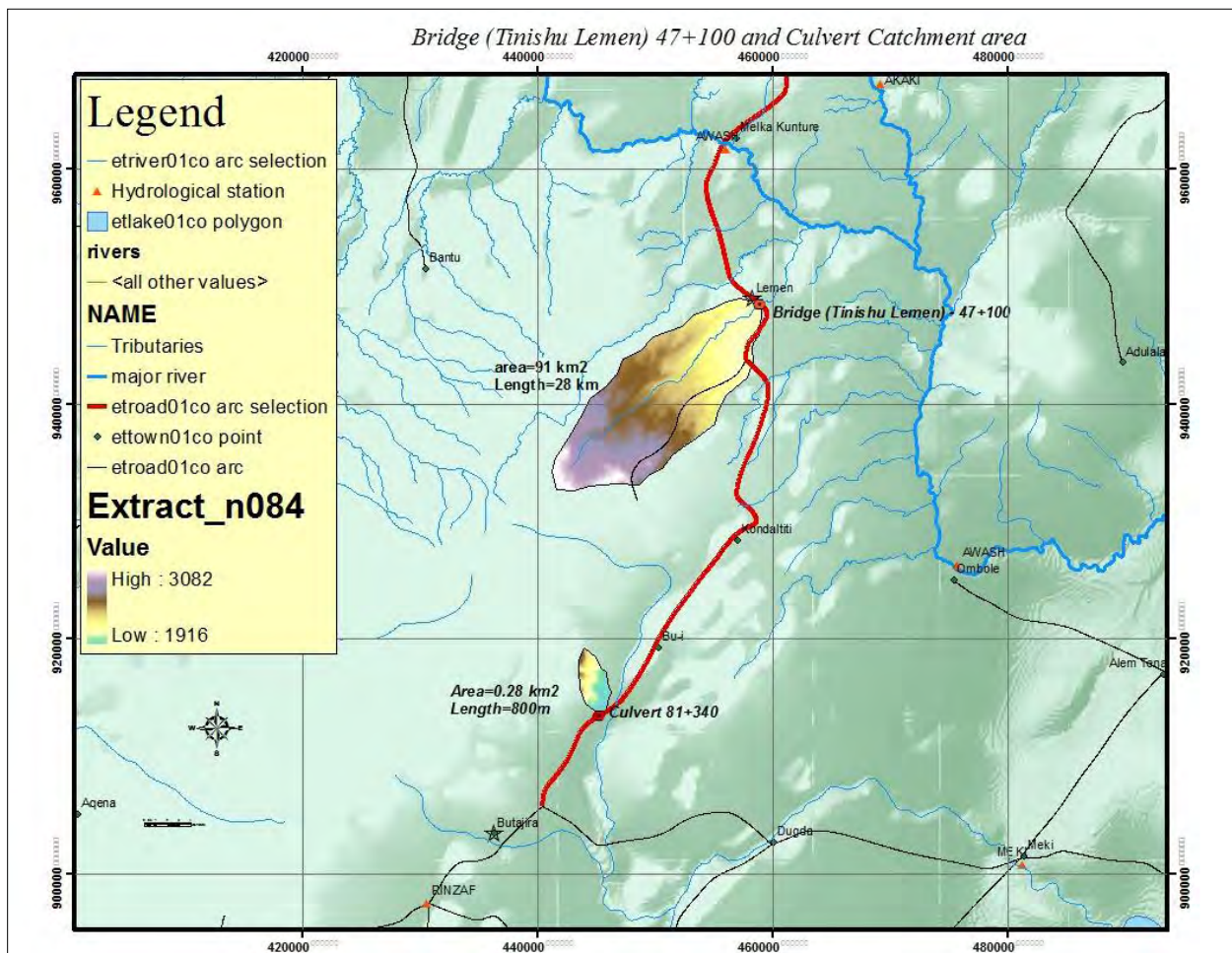


Figure 5.1: Delineation of previous and Proposed Drainage Structures

Table 5.1: Delineation of Previous and Proposed Drainage Structures

S.No.	Proposed Cross Drainage Structures catchment characteristics				
	Station	Location from AlemGena (km)	Catchment Area	River length (km)	Catchment slope (%)
1	81+430 (culvert)	81.430	28 ha	0.800	(15m fall from high to low) (15/800)=1.8%
2	47+100(Bridge)	47.100	91.4 Km ²	28.10	4.51%

N.Bthe Existing Cross-Drainage Structures watershed Data and the proposed one is almost similar

5.3 Computation of Peak Discharge by Rational Method

5.3.1 Culvert (BS1-1-C-130) -81+430

From a topographic map and DEM, the area of the drainage basin upstream from the selected culvert (81+430) was found to be 28 hectares. The Rational Method is selected as per the standard; the catchment area is less than 50 hectares.



Figure 5.2 Existing culvert at chainage 81+430

The existing culvert is small. The road has a functional classification of a Link Road, with a design standard of DC4 (*From Geometric Design Manual*), requiring, as per Table 2-1, a design storm frequency of 10 years. Design runoff corresponding to a 10-year return period rainfall was used and then checked for a 25-year return period runoff.

The following data were measured:

No.	Station	Catchment Area [km ²]	Catchment Area [ha]	Length of Main Channel [m]	ELEVATIONS [m]			River Channel Slope (%)
					At Crossing	Mean Altitude	At high point	
C1	1+548	0.28	27.78	988.58	2059.70	2082.00	2104.30	4.51

Terrain Classification

Terrain Classification	
Terrain Classification	Cs
Soft to Moderate	0.10

LandUse and Soil Data

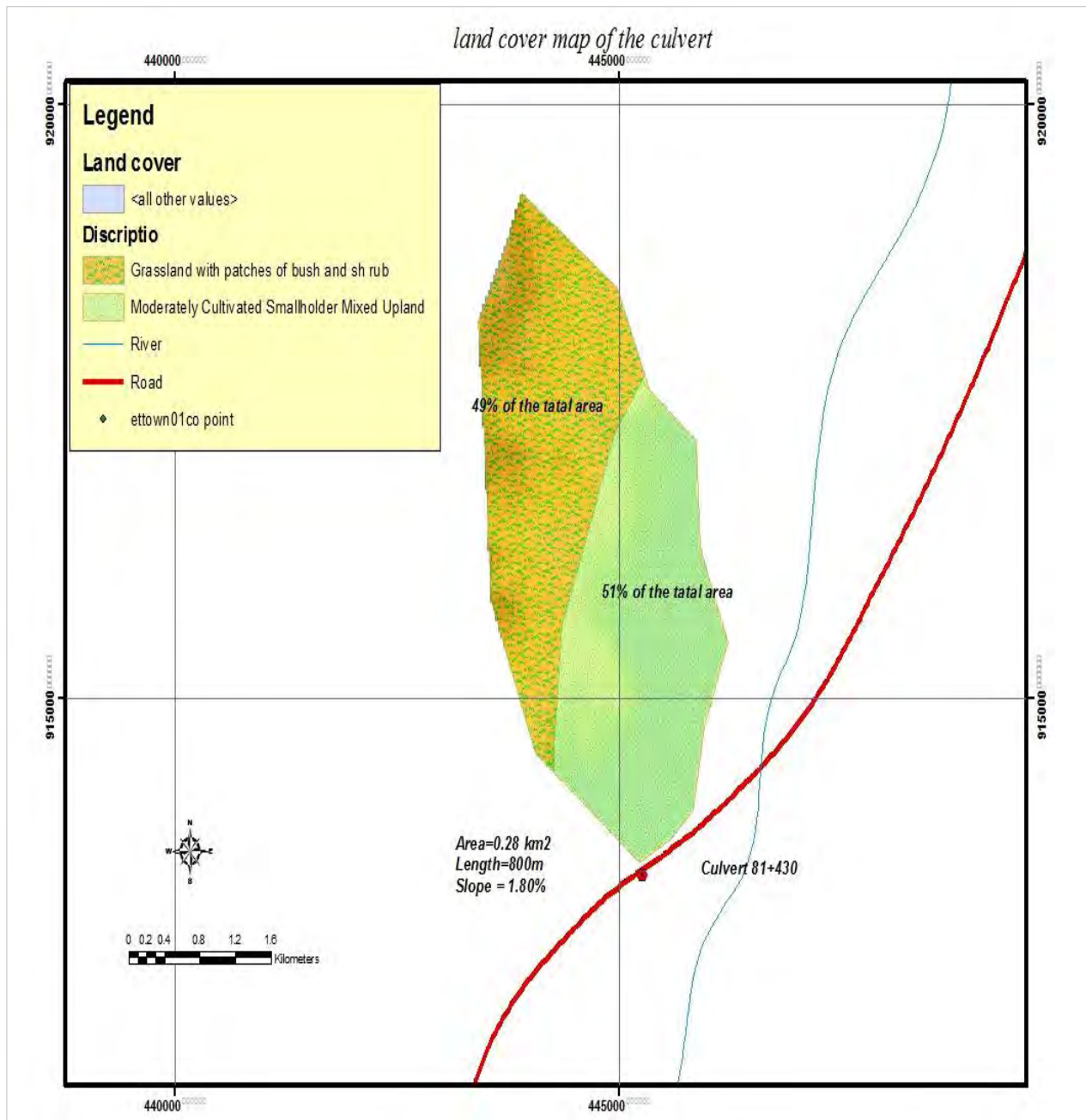


Figure5.3 Land use cover map of the culvert

- From existing land use maps, the land use for the drainage basin was estimated to be:

Vegetation cover in %age						
Dense Forest	Sparse Forest	Grassland	Cultivation	Sparse Grassland	Barren	C_v
0%	15%	30%	45%	10%	0%	0.18

- From existing soil maps, the soil for the drainage basin was estimated to be:

Soil type & coverage in %age					
Well Drained Soil	Fair Drained Soil	Poorly Drained Soil	Impervious Soil	Black Cotton Soil	C_p
20%	30%	40%	10%	0%	0.13

Hydro-Metrological data near. Bui and Butajira area

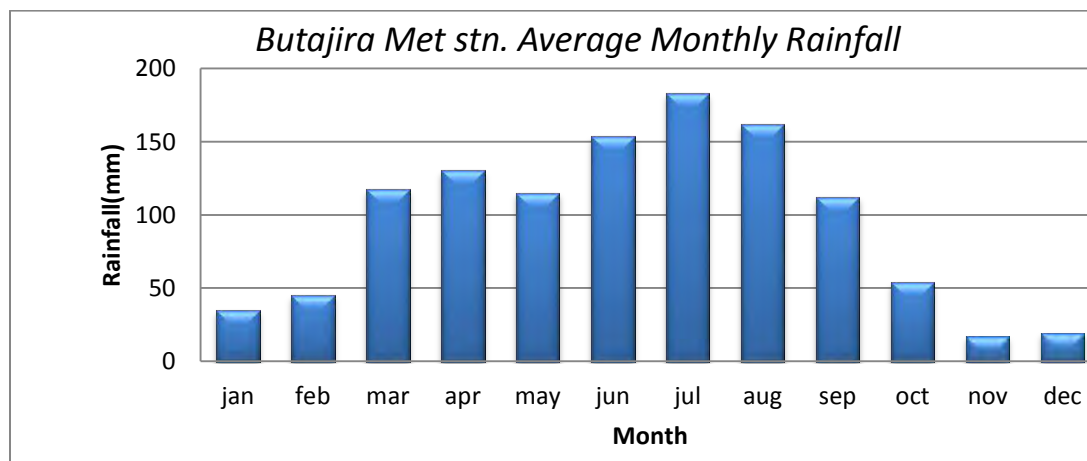


Figure 5.4 Average monthly rainfall of the study area (Butajira)

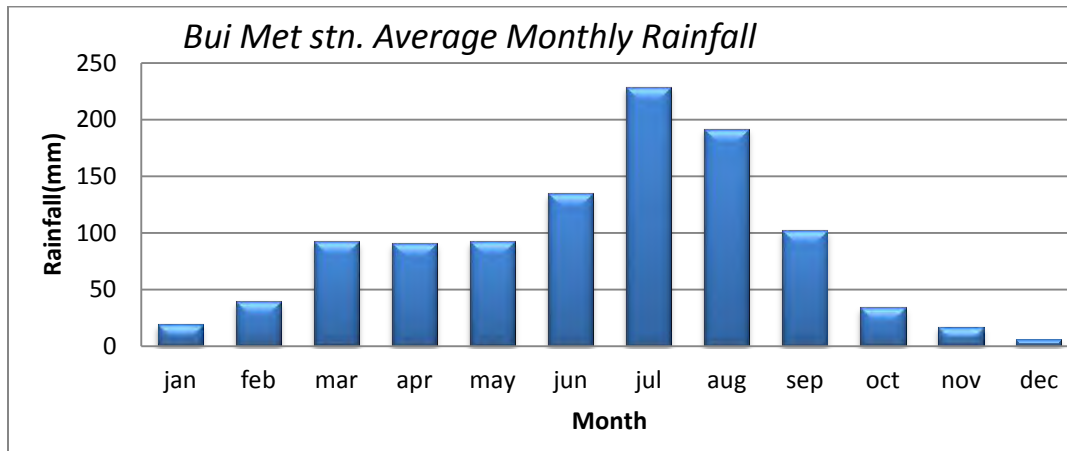


Figure 5.5 Average monthly rainfall of the study area (Bui)

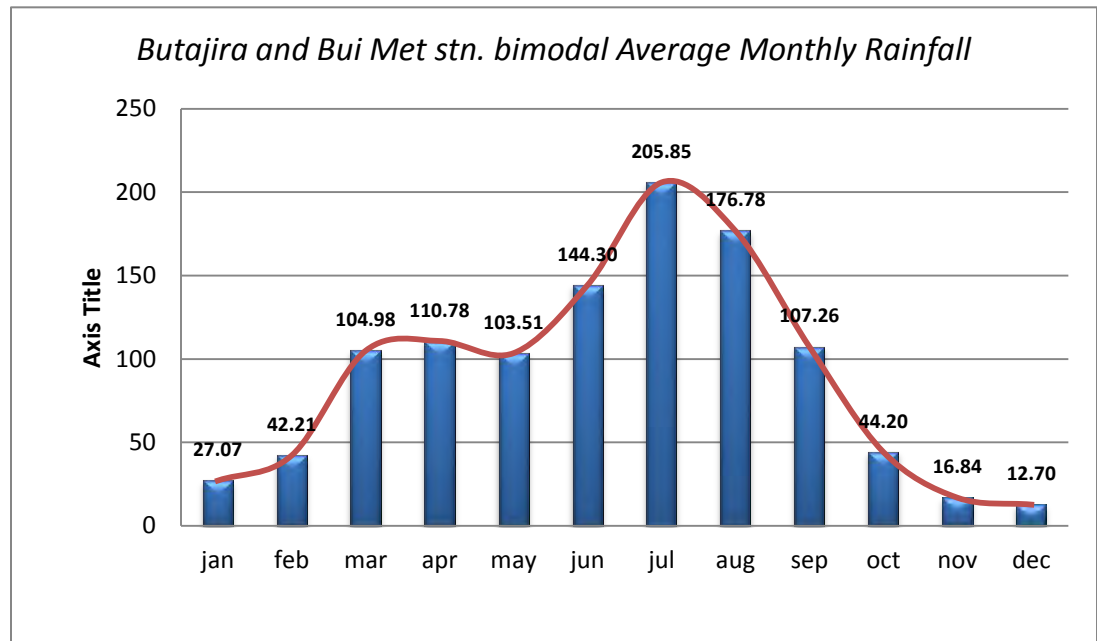


Fig.5.6 Arithmetic mean value of Butajira and Bui Station

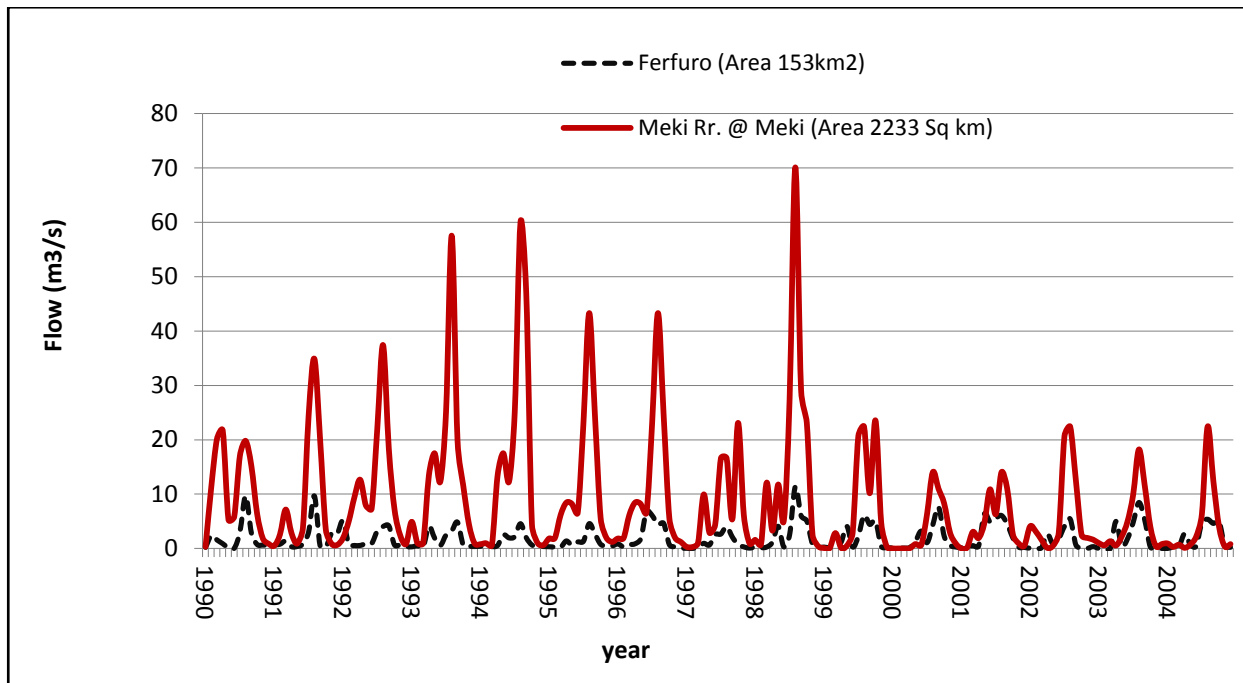


Fig.5.7 Measured Monthly flow hydrograph near the study area in Rift valley basin

Computation of Time of Concentration

1. Overland Flow coefficient

Total Runoff Coefficient (C)

$$C = C_s + C_p + C_v$$

$$C = 0.1 + 0.13 + 0.18$$

$$C = 0.4$$

2. Overl and flow Tc:-

By using Eqn. 4.4 with an overland flow length of 0.98858km, slope of 4.51% and $r = 0.29$, the inlet time is

$$T_c = 0.604 (rL / S^{0.5})^{0.467} \text{ (hours)} \quad (4.4)$$

$$T_c = 0.604(0.29 * 0.98858 / 0.0451^{0.5})^{0.467} = 0.91 \text{ hr} = \mathbf{16 \text{ min}}$$

3. Defined watercourse or channel flow Tc:-

By using Eqn. 4.5 With a channel flow length of 0.800km and slope of 2.00%, the time of concentration is

$$T_c = (0.87L^2 / 1000 S_{av})^{0.385}$$

$$T_c = (0.87 * 0.800^2 / 1000 * 0.02)^{0.385} = 0.05 \text{ hr} = \mathbf{3 \text{ min}}$$

$$\text{Total Time of Concentration} = 16.25 + 15 = \mathbf{31 \text{ min}}$$

Computation of Rainfall Intensity

From Appendix 1 Figure h Alemgena Butajira in Region B2. From Appendix 1 Figure g (IDF curve) for Region B2 with duration equal to 13.01 minutes and with a given return period, the rainfall intensity is:

$$\begin{array}{ll} I_{10} & \text{(10-yr return period)} = 134 \text{ mm/hr} \\ I_{25} & \text{(25-yr return period)} = 161 \text{ mm/hr} \end{array}$$

Runoff Coefficient

A weighted runoff coefficient (C) for the total catchment area is determined in the following table by using the values from Appendix 2 Table b

(1)	(2)	(3)	Weighted Run off Coefficient (2) x(3)
Percent of Total Land Use	Weighted Runoff Land Area	Runoff Coefficient	
Moderately Cultivated Smallholder (Soil Group B)	0.51	0.16	0.08
Grassland with patches of bush and shrub(Soil Group B)	0.49	0.23	0.11
Total Weighted Runoff Coefficient			0.19

Peak Run off Computation

From the rational equation (Eqn. 4.1& 4.2):

$$Q_{10} = 0.00278CIA = 0.00278 \times 0.4 \times 134 \text{ mm/hx28ha} = \mathbf{4.17m^3/s}$$

$$Q_{25} = C_f CIA = 1.1 \times 0.00278 \times 0.4 \times 161 \text{ mm/hx28ha} = \mathbf{5.514m^3/s}$$

These are the estimates of peak runoff for a 10-year and 25-year design storm for the culvert catchment (basin).The culvert, channel, and erosion protection design would proceed with these values.

Table5.2: Comparison of computation results of Existing and proposed Culvert

Parameters considered	Existing	Proposed Culvert	
Return Period (years)	10	10	25
Rainfall Depth(MAP)-(mm)	900	900	900
Overall Rational C value	0.23	0.4	0.4
Rainfall Intensity (mm/hr)	54.6	134	161
Time of Concentration (min)	78	31	31
Catchment Area (Ha)	28	13.01	13.01
Catchment Length (km)	0.80	0.98858	0.98858
Peak Discharge (m ³ /s)	0.99	4.17	5.514

*Rational Method is used to compute the Peak discharge for both the existng and the proposed culverts

The data for the existing culvert design parameters is from (ENGINEERING REPORT OF ALEMGENA-HOSSAENA-SODO road upgrading project Annexure 7B hydraulics analysis 10km^2).

As we can see the comparison result from the above table especially the 10 years return period, the methods for both studies are the same and also the results are different. The difference of the result may be arising from the current attention given by the government and the public in water harvesting activities thoroughly. Considering future sustainability of the culvert and for better hydraulic design, so it is better to use the 10 years return period estimated design discharge $4.17\text{m}^3/\text{s}$ for the proposed culvert.

5.3.2 Bridge (Lemen Tinshua) -47+100

5.3.2.1 SCSMETHOD

Estimate the maximum rate of run off at the inlet to a proposed drainage structure located Alemgena Butajira road (47+100)


From a topographic map and DEM file, the area of the drainage basin upstream from the point in question is found to be 91.4km^2 . The SCS curve number Method is selected as per the area selected is greater than 50 hectares.



Figure 5.8 Previous bridge at chainage 47+100

The road has a functional classification of a Trunk Road, with a design standard of DS4 (see *Geometric Design Manual*, Table 2-1, on Chapter 2). The type of structure is Bridge. ERA standard for Bridges indicates a range of design storm frequencies between 25 and 100 years. Thus, determine the maximum rate of runoff for a 50 year return period. The following data were measured:

River Channel Data

RIVER CHANNEL DATA	Length of Main Channel (L), [m]		31519.31 m
	Length of Channel at 0.15 L, [m]		4727.90 m
	Length of Channel at 0.3 L, [m]		9455.79 m
	Catchment Area, [km ²]		91.438 km ²
	Crossing Elevation (from DEM)		2098.400 m
	Crossing Elevation (From Surveying Data)		2098.400 m
	Elevation at 0.15 (L)		2787.300 m
	Elevation at 0.3 (L)		2475.870
	High Point Elevation		3232.200 m
	Direction of flow		R - L
	Average River Channel Slope		5.90 %
	Terrain Classification		Soft to Moderate

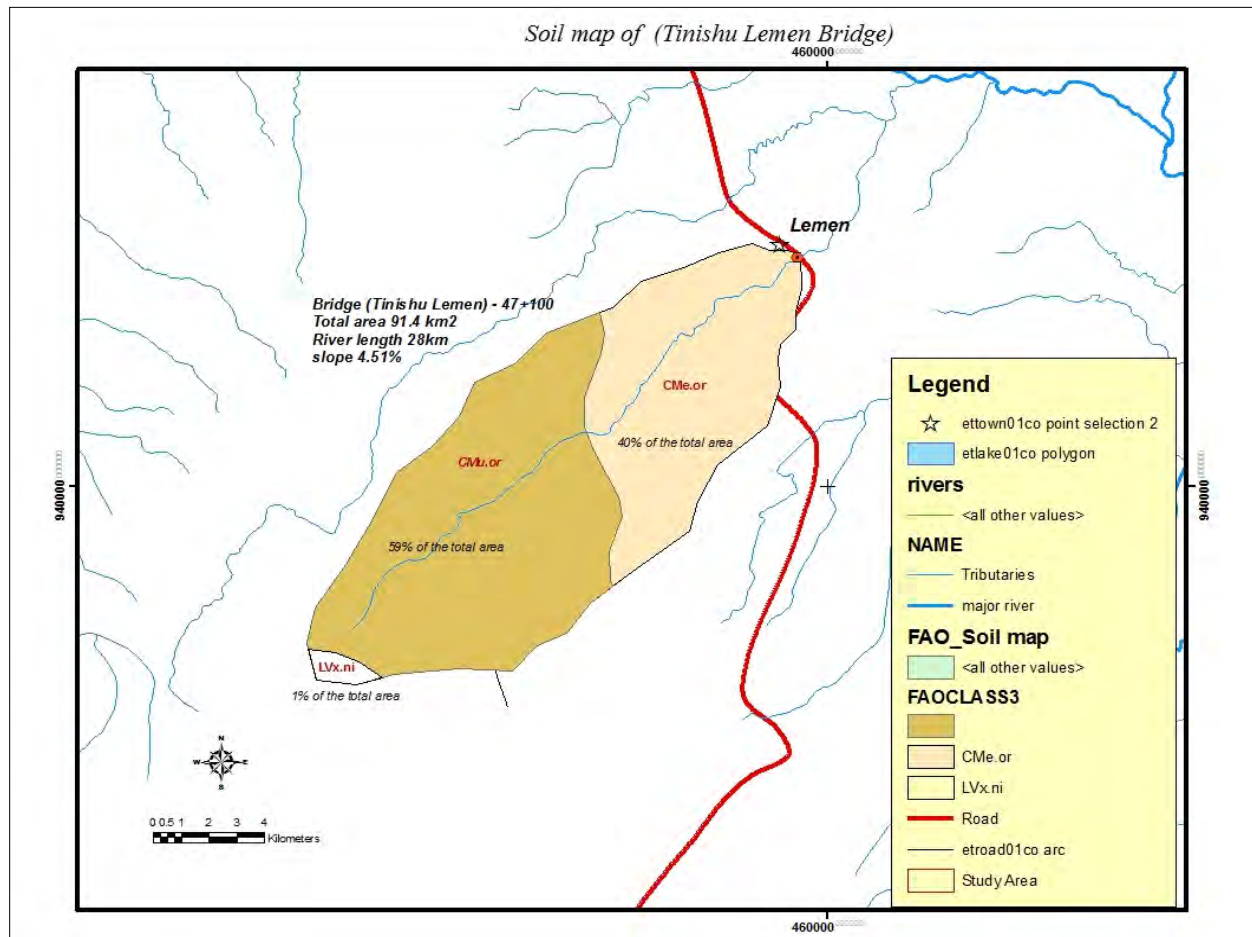


Figure 5.9 Soil map of Bridge catchment area

Hydro-Metrological data near Lemen village

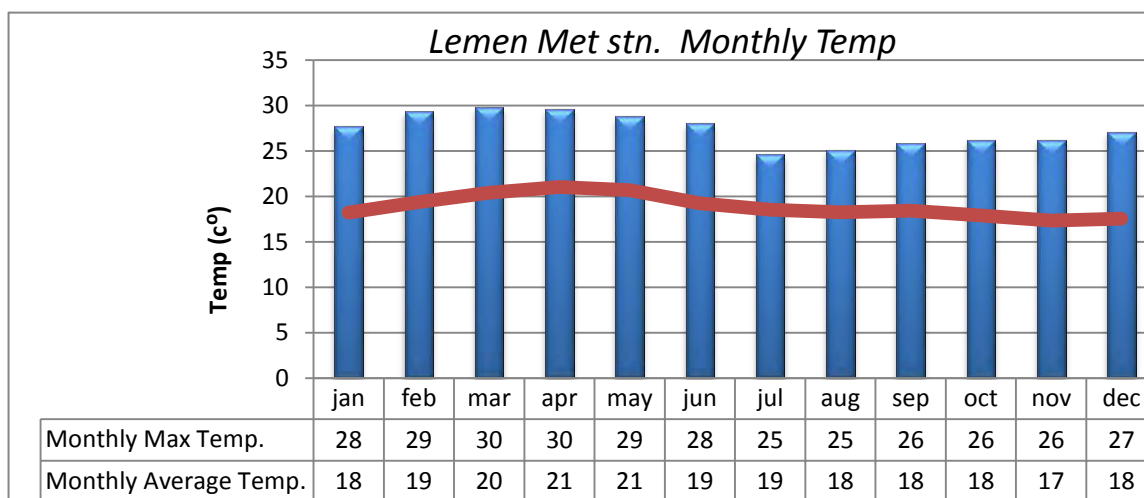


Figure 5.10 Average monthly Temperature of the study area (Lemen)

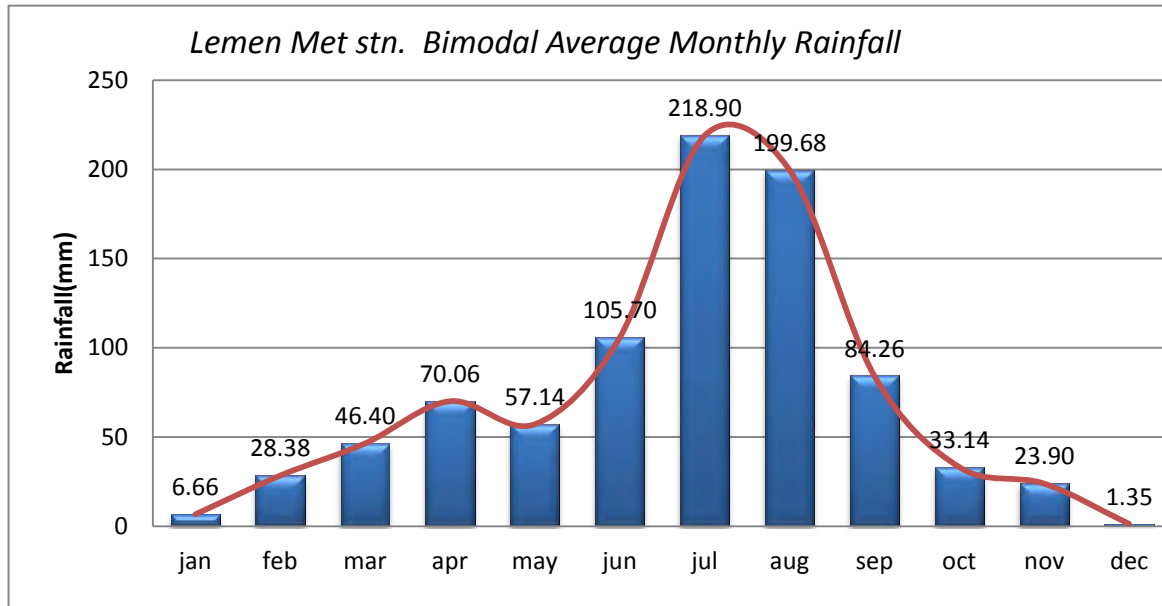


Figure 5.11 Average monthly rainfall of the study area (Lemen)

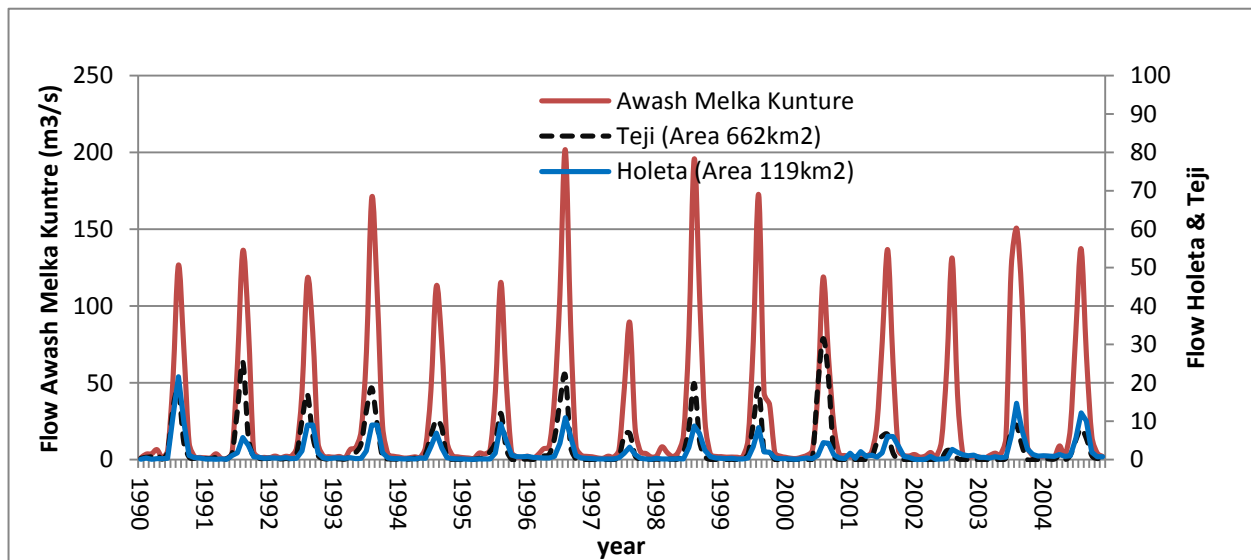


Fig.5.12 Measured Monthly flow data hydrograph near the study area in Awash basin

LandUse and Soil Data

- From the Soils Map of Ethiopia, and from the field survey, it appears that the soils at „Lemen Tinishu“ Bridge catchment are:

Table-Soil Type and Hydrologic Soil Group Data

Item No.	Soil Type	Area (Km2)	Coverage in %
1	Chromic Luvisols	1.053	1.151
2	Eutric Nitosols	77.302	84.540
3	Vertic Cambisols	13.084	14.309
TOTAL		91.438	100.000

Table-Hydrologic Soil Group Data

Item No.	Hydrologic Soil Group	Area (Km ²)	Coverage in %
1	B	91.44	100.00
TOTAL		91.44	100.00

- From existing land use maps, the land use for the drainage basin was estimated to be:

Table-Land Use Type

Item No.	Land Use Type	Area (Km2)	Coverage in %
1	MODERATELY CULTIVATED	19.96	21.83
2	INTENSIVELY CULTIVATED	71.47	78.17
TOTAL		91.44	100.00

SCS Curve Number (CN values)

Table-Runoff curve numbers (CN values)

Item No.	CN Number	Area (Km2)	Coverage in %
1	71	71.47	78.17
2	76	19.96	21.83
TOTAL		91.44	100.00
Average CN Number			72.09

Peak Runoff computation

After sizing a drainage facility using a flood discharge corresponding to the design frequency, it is necessary to review or check this proposed facility with a base discharge. This is done to insure that there are no unexpected flood hazards inherent in the proposed facilities. The review (check) flood shall be at least the 100-year event. In some cases, a flood event larger than the specified review flood might be used for analysis to ensure the safety of the drainage structure and downstream development.

The peak runoff computation is done for 50 years Return Period for design and 100 years return period for reviews (check).

Design Flood Discharge Computation (50 Years Return Period)

Design Flood SCS Parameters	Return Period	50.00 Years
	Time of Concentration T_c	3.96 Hours
	Rainfall Region (ERA DRAINAGE MANUAL 2013)	Region B2
	SCS CN Number for the Whole Catchment	72.09
	Antecedent Moisture Condition (AMC)	Normal
	SCS CN Number for the Whole Catchment for AMC = normal	72.09
	Potential Infiltration S	98.34 mm
	Initial Abstraction I_a	19.67 mm
	Design Storm P (24 Hours Maximum Rainfall)	132.00 mm
	$I_a/P =$	0.149
	Direct Runoff q	59.90 mm
	Unit Peak Discharge Q_U	0.0553 m³/s
	SCS Q for 50 Year Return Period	302.71 m³/s

Check (Review) Flood Discharge Computation (100 Years Return Period)

Check (Review) Flood SCS Parameters	Return Period	100.00 Years
	Time of Concentration T_c	3.96 Hours
	Rainfall Region (ERA DRAINAGE MANUAL 2013)	Region B2
	SCS CN Number for the Whole Catchment	72.09
	Antecedent Moisture Condition (AMC)	Normal
	SCS CN Number for the Whole Catchment for AMC =	Normal 72.09
	Potential Infiltration S	98.34 mm
	Initial Abstraction I_a	19.67 mm
	Design Storm P (24 Hours Maximum Rainfall)	147.00 mm
	$I_a/P =$	0.134
	Direct Runoff q	71.85 mm
	Unit Peak Discharge Q_u	0.0558 m³/s
	SCS Q for 100 Year Return Period	366.26 m³/s

Table 5.5: Comparison of computation results of previous and Designed Bridge

Parameters considered	Previous study (Modified Rational Method)	Proposed Designed (SCS curve number Method)	Proposed Review(SCS curve number Method)
Return Periods(years)	50	50	100
Catchment Area (km ²)	91.4	91.4	91.4
Peak Discharge (m ³ /s)	101.60	302.71	366.26

All the existing Bridge design parameters are extracted from (ENGINEERING REPORT OF ALEMGENA-HOSSAENA-SODO road upgrading project Annexure 7A hydraulics analysis >10km²).

As we can see the comparison result from the above table especially the 50 years return period, the methods are different (the previous study used modified Rational Method) but this study use the SCS curve number method, this might be the reason for the two results has a big difference. The existing study may be under estimate the peak discharge it means undersized the cross drainage structure (the Bridge). This study proposes the flood discharge of **302.71 m³/sto** design the „Tinishu Lemen“ Bridge.

CHAPTER 6- Hydraulic Analysis Results and Discussion

6.1. Hydraulic Analysis

6.1.1 Adequacy of Existing Drainage Structures

Hydraulic calculations are carried out for drainage structures at stations 47+100, 81+430, using the peak discharges that are tabulated in Tables 5.5 and 5.2 respectively.

6.1.2 Hydraulic Calculation for Drainage Structure at Station 81+430(Culvert)

The drainage structure at station 81+430 is culvert as shown on Figure 5.2. The hydraulic calculation is carried out using equation (4.19). In Table 5.2, the design and existing discharges are $0.99\text{m}^3/\text{sec}$ and $4.15\text{m}^3/\text{sec}$ respectively. The existing culvert was installed at a slope of 1.09%.

No.	Station	Structure Type	Drainage Area [km ²]	Stream Channel Slope (%)	Length of culvert	Design Discharge	
						for 10 Year Return Period	for 25 Year Return Period
1	81+430	Slab/Box Culvert	0.28	5.00%	12.00	4.15	5.48

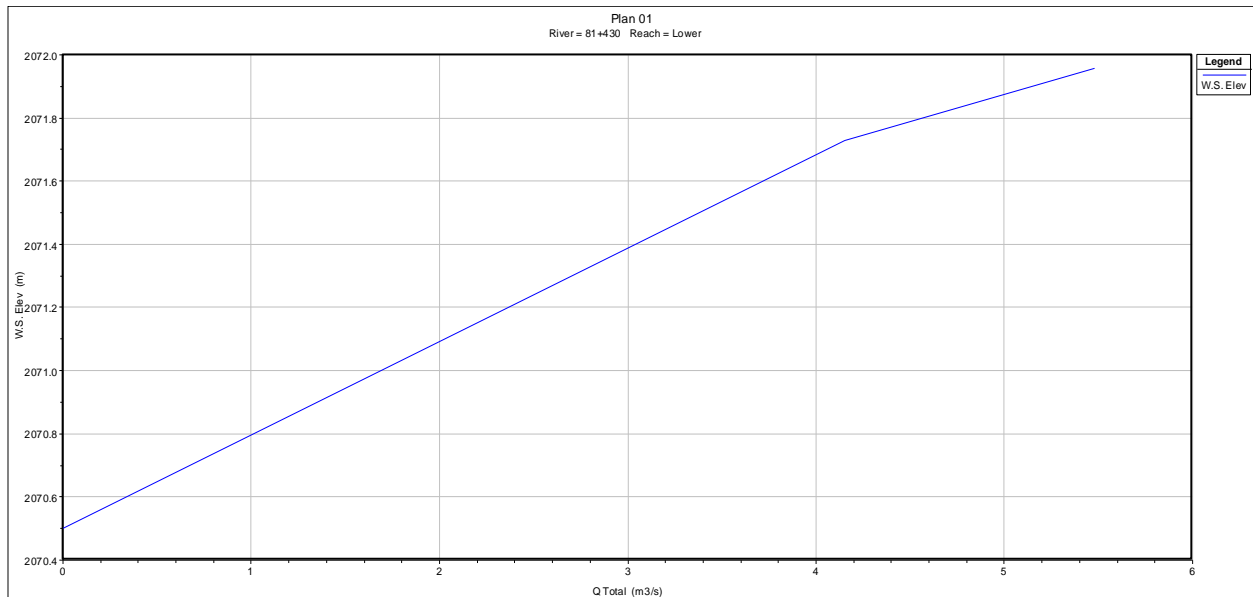
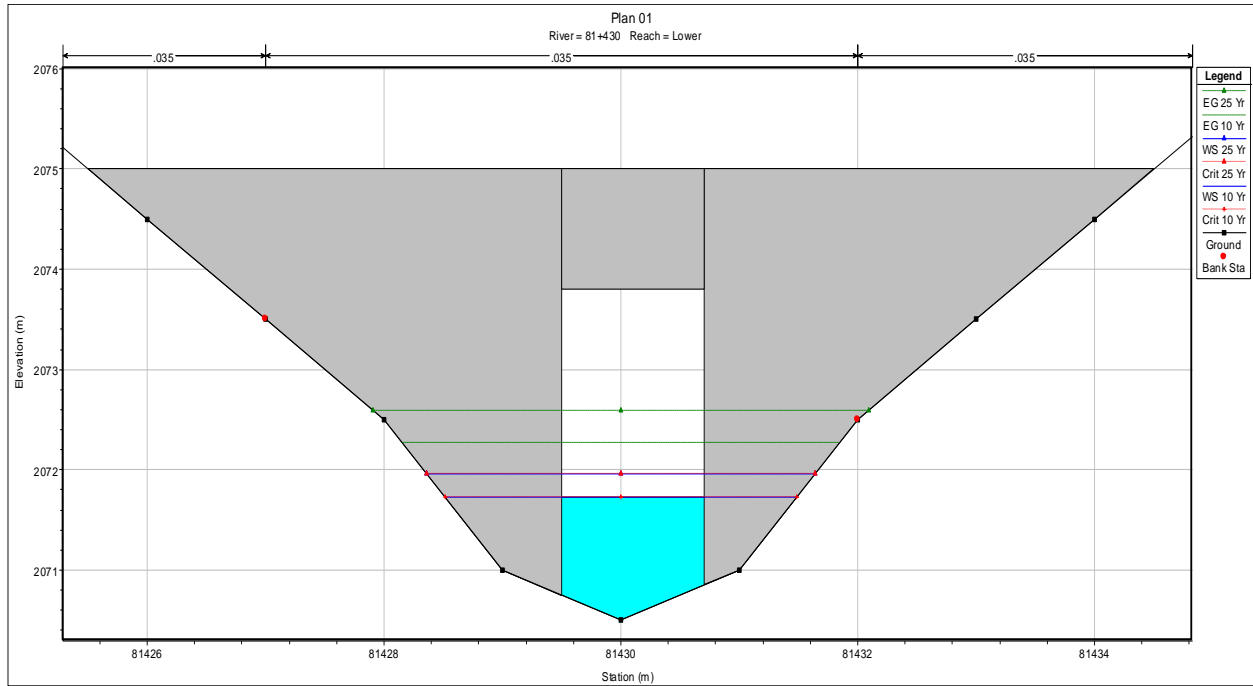
Manning's n	Normal Depth Calculations								
	Depth of Water hw (m)	Span	No. of Barrels	Wetted Perimeter P (m)	Area A [m ²]	Hydra. Radius R (m)	Check Q	Free Board	Clear Depth of Structure
0.030	0.91	1.20	1	3.03	1.10	0.36	4.15	0.60	1.51

OPENING SIZE DETERMINATION

Existing Opening Size			Design Opening Size		
Span	No. of Barrels	Clear Depth	Design Span	No. of Barrels	Design Clear Depth
1.20	1.00	3.80	1.20	1	1.51

From computation results, the existing length of the drainage structure that the flood should pass without disturbing the structure is 12.8 m and the design length is 12 m. The Design culvert has a clear depth of 1.51. Therefore, the appropriate drainage structure that is recommended slab/box culvert, i.e. 12- meter clear span length.

Hydraulic Adequacy of Designed Culvert.



Hec Ras Hydraulic Summary

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m ³ /s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m ²)	(m)	
Lower	7	10 Yr	4.15	2071.5	2072.5	2072.5	2072.81	0.019882	2.49	1.67	2.67	1
Lower	7	25 Yr	5.48	2071.5	2072.64	2072.64	2073	0.019706	2.67	2.05	2.86	1.01
Lower	6	10 Yr	4.15	2071	2072.2		2072.37	0.009147	1.87	2.22	2.93	0.69
Lower	6	25 Yr	5.48	2071	2072.56		2072.69	0.005161	1.62	3.37	3.42	0.52
Lower	5	10 Yr	4.15	2070.5	2072.27	2071.5	2072.32	0.001742	1.01	4.11	3.69	0.31
Lower	5	25 Yr	5.48	2070.5	2072.6	2071.64	2072.66	0.001422	1.01	5.44	4.21	0.28
Lower	4.5		Culvert									
Lower	4	10 Yr	4.15	2070	2071	2071	2071.31	0.019857	2.49	1.67	2.67	1
Lower	4	25 Yr	5.48	2070	2071.14	2071.14	2071.5	0.019857	2.68	2.05	2.86	1.01
Lower	3	10 Yr	4.15	2069.5	2070.5	2070.5	2070.81	0.020046	2.5	1.66	2.67	1.01
Lower	3	25 Yr	5.48	2069.5	2070.64	2070.64	2071	0.019823	2.68	2.05	2.86	1.01
Lower	2	10 Yr	4.15	2069	2070	2070	2070.31	0.019914	2.49	1.67	2.67	1.01
Lower	2	25 Yr	5.48	2069	2070.14	2070.14	2070.5	0.01989	2.68	2.05	2.86	1.01

Lower	1	10 Yr	4.15	2068.5	2069.57	2069.5	2069.82	0.015002	2.24	1.85	2.76	0.87
Lower	1	25 Yr	5.48	2068.5	2069.71	2069.64	2070.01	0.015002	2.42	2.27	2.95	0.88

Table 6-1 Hydraulic Comparison of culvert

Parameters of Design and Review	Existing	Designed
Return Periods(years)	10	10
Slope of culvert (%)	1.09	5
Manning's Roughness Coefficient for Natural Stream	0.016	0.03
Opening Span of the Structure(length) (m)	12.8	12
Clear depth of Culvert (m)	3.80	1.51
Maximum Freeboard (m)	0.67	0.6

6.1.2.1 Proposing Drainage Structure at Station 81+430

From the above result, the computed existing and designed peak discharges are 0.99 m³/sec and 4.15m³/sec respectively. Therefore, the proposed drainage structure is a clear span reinforced concrete culvert (RC slab culvert) that has length of 12m and span of culvert 1.2 meter.

**6.1.3 Hydraulic Calculation for Drainage Structure
At Station 47+100(Bridge)**

6.1.3.1 Opening Size Determination

Design Discharge		Check (Review Discharge)	
Design Discharge=	302.71 m ³ /s	Check (Review) Discharge =	366.26 m ³ /s
Try a Bridge of Span =	10.00 m	Try a Bridge of Span =	10.00 m
No. of Spans=	2	No. of Spans=	2
Depth of Water =	2.57 m	Depth of Water =	2.94 m
Area =	25.73 m ²	Area =	29.40 m ²
Wetted Perimeter =	15.15 m	Wetted Perimeter =	15.88 m
Hydraulic Radius =	1.70 m	Hydraulic Radius =	1.85 m
Channel Slope =	1.54%	Channel Slope =	1.54%
Manning's n =	0.030	Manning's n =	0.030
Free Board =	1.20		
Q =	302.71 m ³ /s	Q =	366.26 m ³ /s
clear height H=	3.80 m	clear height H=	3.00 m

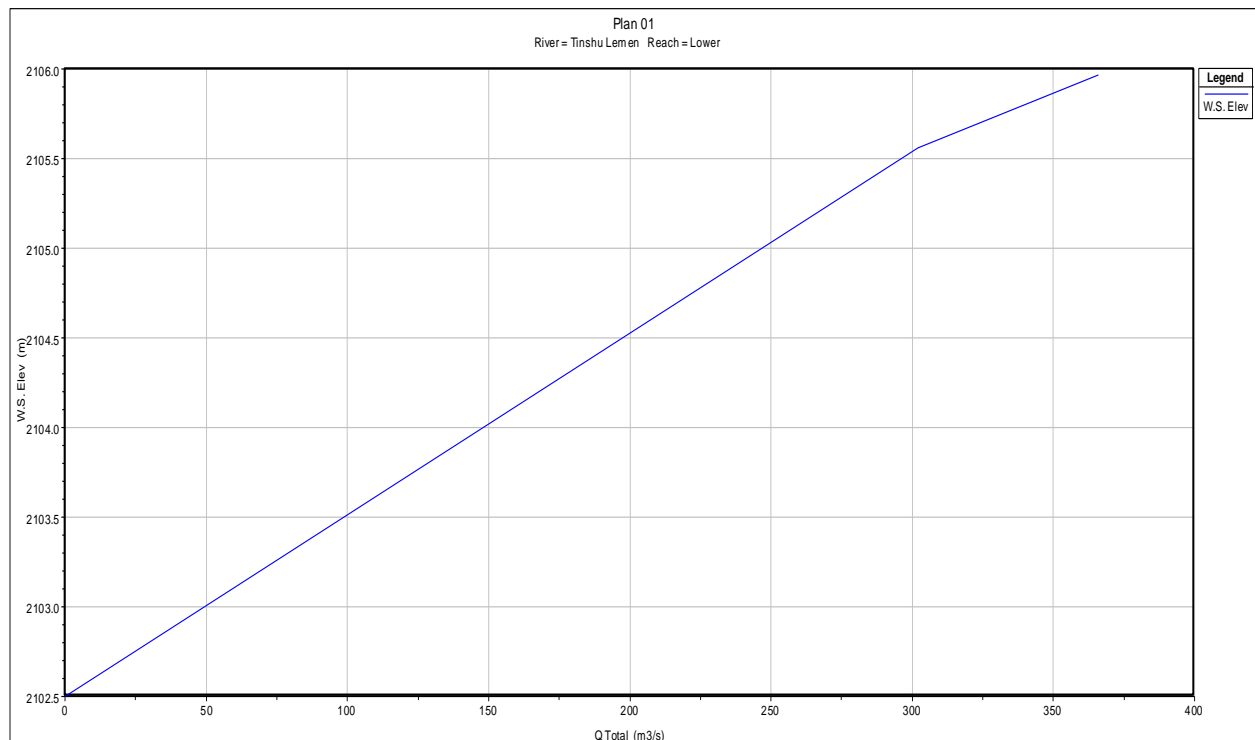
From the above result, the previous and design discharges for 50 years return periods are 101.6m³/sec and **302.71 m³/s** respectively. The stream normal Manning's roughness coefficient is 0.03 and the catchment slope is 1.54%. The Bridge has two spans of the Bridge length is 30-meter and the width of the Bridge is 10 meters.

The depth of flow is about 2.57 meters. Therefore, the structure is safe from overtopping flood. The previous Bridge design discharge is 101.6 m³/sec; the design (calculated) discharge **302.71m³/sec** and 366.26 m³/sec for review. This velocity is erosive velocity because of the silty clay soil formation where the drainage structure is constructed. Therefore, the drainage structure is not safe from scouring and requires redesign and rehabilitation and additional protection work for the scouring of the abutment.

Table 6.2: Hydraulic Parameters for Proposed Drainage Structure at station 47+100

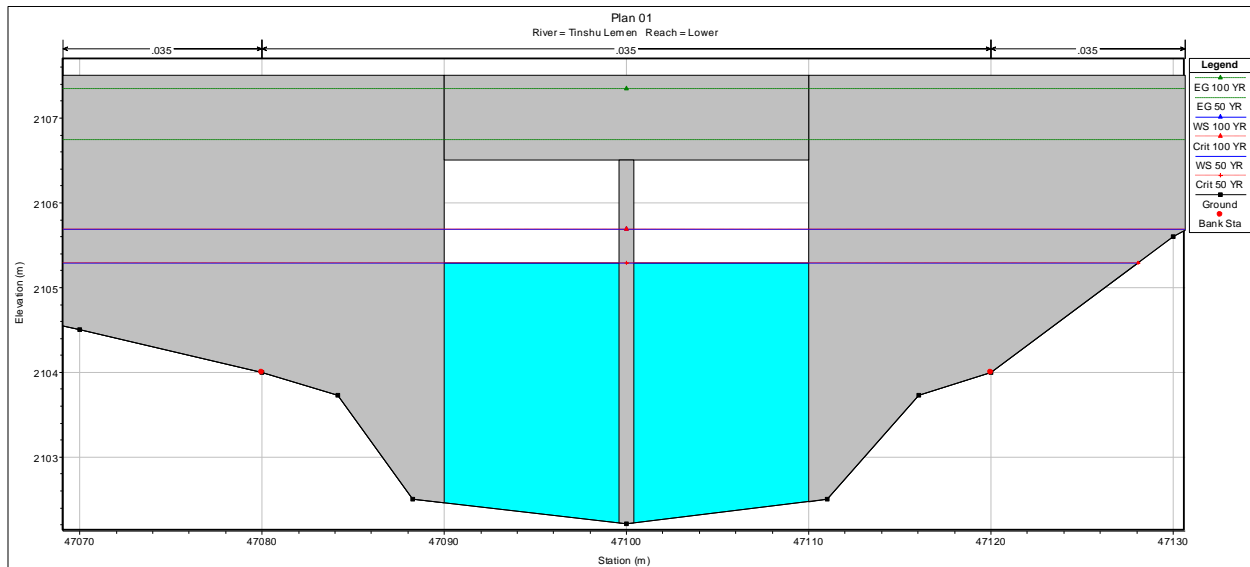
Parameters of Design and Review	previous	Design
Return Periods(years)	50	50
Channel Slope (%)	1.54	1.54
Span length of the Structure (m)	11+11	10+10
Clear Height m	3.3	3.8
Area of flow cross-section (m ²)	30.8	25.73
Wetted Perimeter (m)	24.8	15.15
Maximum Freeboard (m)	1.2	1.2
Maximum Water Depth (m)	1.4	2.57

Stage Discharge Curve



Hydraulic Adequacy of Existing Bridge

Upstream View



6.1.3.1 Proposing Drainage Structures at 47+100

Designing highway drainage structures involves many factors including estimating flood peaks, hydraulic performance, structural adequacy, and overall construction and maintenance costs.

Therefore, the Bridge can be constructed at the location as per the revised design.

Chapter 7: conclusion and recommendation

From the investigations of the site condition assessment, remedial action recommendations are provided to maintain the desired level of service.

7.1. Conclusion

Condition assessment of the drainage structures along the route (Alemgena Butajira) carried out to ensure assets deliver the standard of service required.

The drainage structures capacity and fitness for the intended purpose were identified through hydrologic and hydraulic analysis.

Asset condition reflects the physical state of the asset, which may or may not affect its performance. The performance of the asset is the ability to provide the required level of service to customers. Generally this can be measured in terms of reliability, availability, capacity, and meeting customer demands and needs. All this critical information for determining the remaining useful life of an asset and more importantly the timing for possible intervention steps to bring level of service, provided by the asset, back to a desired standard.

Negligence regarding Design, maintenance, rehabilitation and renewal may lead to the premature failure, which leaves the organization with only one option- to replace the asset (generally the most expensive options).

Based on the research finding on the structure 81+430(culvert) should be revised as per the analysis results .The computed previous and designed peak discharges are 0.99m³/sec and 1.183m³/sec respectively. Therefore, the proposed drainage structure is a clear span reinforced concrete culvert (RC slab culvert) that has length of 22.63 and opening width of culvert one meter.

The previous length of the drainage structure that the flood should pass without disturbing the structure is 12.8 m and the reviewed new length is 22.63 m. The previous culvert has 0.5m total opening, therefore, it is not adequate. Due to the bank-full width, the appropriate drainage structure that is recommended is culvert that has 23-meter clear span.

The outlet velocity of flow for the previous and reviewed drainage structures is erosive due to clayey and silty streambed. Therefore, it requires erosion protection treatment.

The structure 47+100 (Bridge) should be revised as per the analysis results. Designing highway drainage structures involves many factors including estimating flood peaks, hydraulic performance, structural adequacy, and overall construction and maintenance costs.

Therefore, the Bridge can be constructed at the location based on the revised design.

The previous and designed discharges are $101.6\text{m}^3/\text{sec}$ and $225.23\text{m}^3/\text{sec}$ respectively. The stream normal Manning's roughness coefficient is 0.13 and the catchment slope is 5%. The Bridge has a two span of length 30-meter and a width of 8.5 meters.

The depth of flow is about 1.4 meters. Therefore, the structure is safe from overtopping flood. This velocity is erosive velocity because of the silty clay soil formation where the drainage structure is constructed. Therefore, the drainage structure is not safe from scouring and requires redesign and rehabilitation and additional protection work for the scouring of the abutment.

The unforeseen failure of an asset can have major consequences that constitute a business risk or potential loss to the organization.

7.2. Recommendation

On Alemgena- Butajira road segment, even though there is regular evaluation of drainage systems no corrective action is applied (no maintenance, rehabilitations, renewal). Effective drainage maintenance should therefore be prioritized ahead of any other measures.

- ❖ Maintenance is neglected in the study area, this resulted flooding, washouts. To avoid these problem pipe culverts clogged with debris or sediment needs cleaning.
- ❖ Vegetation and brush that obstruct water flow need to be mowed or cut.
- ❖ It is necessary to re-seed, mulch, or use other erosion protection methods on steep slopes or in areas sensitive to severe erosion.
- ❖ A comprehensive program for maintaining drainage structures in good repair and operating condition will reduce the probability of failure and prove to be cost effective.
- ❖ Inlet and outlet areas must be adequately protected by placing rip-rap, or fitted stone.
- ❖ Head walls and end wall must be properly constructed

8. References

1. Ethiopian Roads Authority Drainage Design Manual-2002
2. Highway drainage guideline, America association of Highway and Transport officials, Washington.
3. Meinzer, Oscar E. ed., Hydrology, McGraw-Hill, Book Co., Inc., New York, N.Y., 1942.
4. Hydrology Handbook ASCE Manual of Engineering Practice- No. 28, New York, N.Y., 1949, 184 p.
5. Introduction to Highway Hydraulics -Publication No. FHWA-NHI-08-090 June 2008
6. Federal High way Administration. 1990. HYDRAIN Documentation
7. Hiemstra, L. A. V., and Reich, B. M., (1967) Engineering Judgment.
8. Horner, V. W. and Flynt, F. L., Relation Between Rainfall and Runoff from Small Urban Areas, Trans. ASCE, Vol. 101, 1936, p. 140-206.
9. Schaake, J. C., Jr., and Knapp, J. W., (1967), Experimental Examination of the Rational Method, New York.
10. Buchanan, Thomas J., and Somers, William P.,(1969), Discharge Measurements at Gaging Stations, U.S. Geological Survey Techniques Water-Resources Investigations book 3, chapter A8, Washington, D.C.
11. Carter, R. W., and Davidian, Jacob, (1965), Discharge Ratings at Gaging Stations, U.S. Geological Survey Surface Water Techniques, book 1, chapter 12.
12. Buchanan, T. J., and Somers, W. P., (1968), Stage Measurements of Gaging Stations, U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A7, Washington, D.C.
13. Carter, R. W. and Davidian, Jacob,(1968), General Procedure for Gaging Streams, U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A6, Washington, D.C.
14. Benson, M.A., and Dalrymple, Tate,(1968), General Field and Office Procedures for Indirect Discharge Measurements, U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A1, Washington, D.C.
15. Barnes, Harry H., Jr., (1967), Roughness Characteristics of Natural Channels, U.S. Geological Survey Water-Supply Paper 1849, Washington, D.C.

16. Dalrymple, Tate, and Benson, M.A.,(1967), Measurement of Peak Discharge by the Slope-area method U.S.Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A2,Washington, D.C.
17. Bodhaine, G. L., (1968), Measurement of Peak Discharge at Culverts by Indirect Methods, U.S. Geological Survey Techniques of Water-Resources Investigations, book 3 chapter A3, Washington, D.C.
18. Matthai, Howard, F.,(1968), Measurement of Peak Discharge at Width Contractions by Indirect Methods, U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A5, Washington, D.C.
19. Benson, M. A., and Dalrymple, Tate,(1968), General Field and Office procedures for Indirect Discharge Measurements, U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A1,Washington, D.C.
20. Dalrymple, Tate, (1960), Flood-Frequency Analyses, U.S. Geological Survey Water-Supply Paper 1543-A, Washington, D.C.
21. Benson, M. A., (1962), Evolution of Methods For Evaluating the Occurrence of Floods, U.S. Geological Survey Water-Supply Paper 1580-A, Washington, D.C.
22. Riggs, H.C., (1968), Frequency Curves, U.S. Geological Survey Techniques of Water-Resources Investigations, book 4, chapter A2, Washington, D.C.
23. Subcommittee on Hydrology, Inter-Agency Committee on Water Resources, (1966), Methods of Flow Frequency Analysis, Notes on Hydrologic Activities, Bulletin No. 13, Washington, D.C.
24. Dalrymple, Tate, (1960), Flood-Frequency Analyses, U.S. Geological Survey Water-Supply Paper 1543-A, Washington, D.C.
25. Tennent, Richard C., and Harrison, Lawrence j.,(1968), Flood Record Compilation and Frequency Plot, BPR Program No. HY-5, Federal Highway Administration, Washington, D.C.
26. Hydrology Committee, (1967), A Uniform Technique for Determining Flood Flow Frequencies, Water Resources Council Bulletin No. 15, Washington, D.C.
27. Schaake, J. C., Jr., Geyer, J. c., and Knapp, J. W., (1967), Experimental Examination of the Rational Method, Proc. ASCE, Hydraulics Division Journal HY 6 Paper 5607, New York.

28. Searcy, James K.,(1965), Design of Roadside Drainage Channels, Bureau of Public Roads, now Federal Highway Administration Hydraulic Design Series No. 4, Washington, D.C.
29. Hershfield, David M., (1961), Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years, Weather Bureau, Washington, D.C.
30. Jens, S. W. and McPherson, M. B., (1965), Hydrology of Urban Areas in Handbook of Applied Hydrology, section 20, McGraw-Hill Book Co., Inc., New York.
31. Guide to Bridge Hydraulics, published for roads and Transportation association of Canada by university of Toronto press.
32. Cedergen, Herry R. Drainage of highway and airfield pavements. Printed in the United States of America.
33. Concise Hand book of civil engineering 2002 S.CEHAND and Company L T D RAMNAGAR, NEWDELHI- 110055.

Appendix 1: Time of Flow, Unit Peak Discharge, Velocity of flow and SCS CN Charts

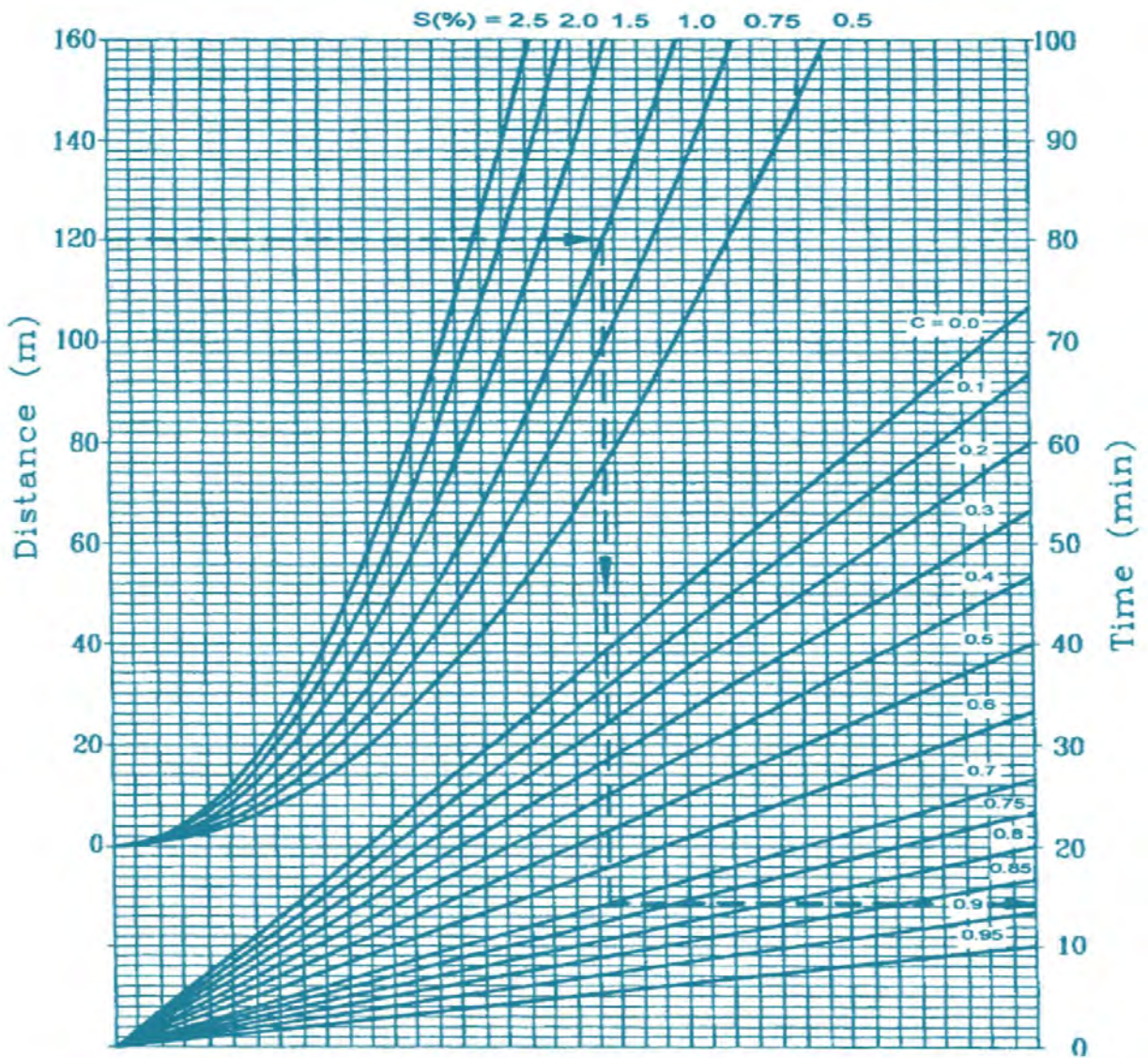


Figure a: Overland Time of Flow

Source: Air Port Drainage, Federal Aviation Administration, 1965

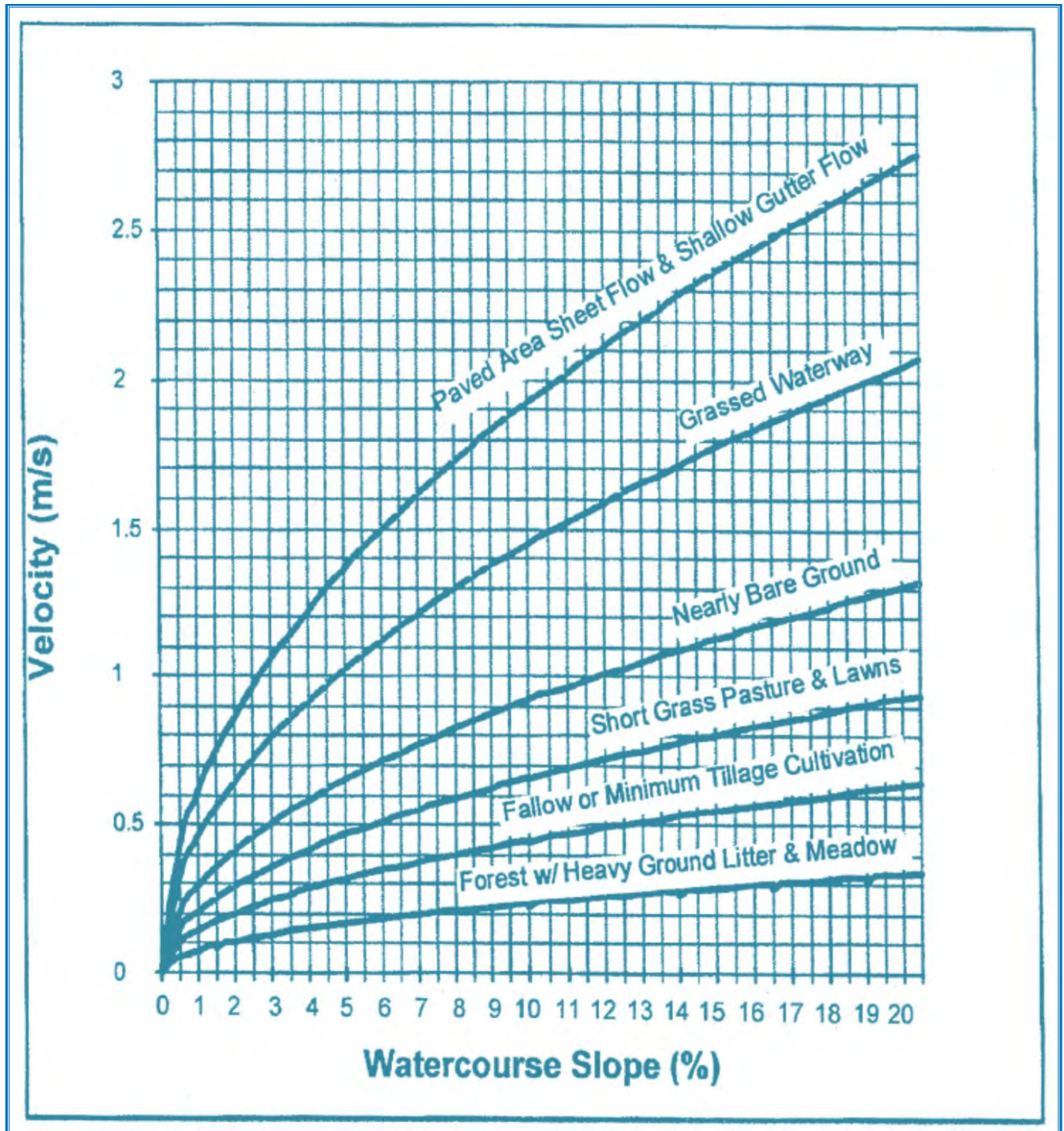


Figure b: Velocities for Upland Method of Estimating Time of Concentration

Source: HEC No. 19, FHWA

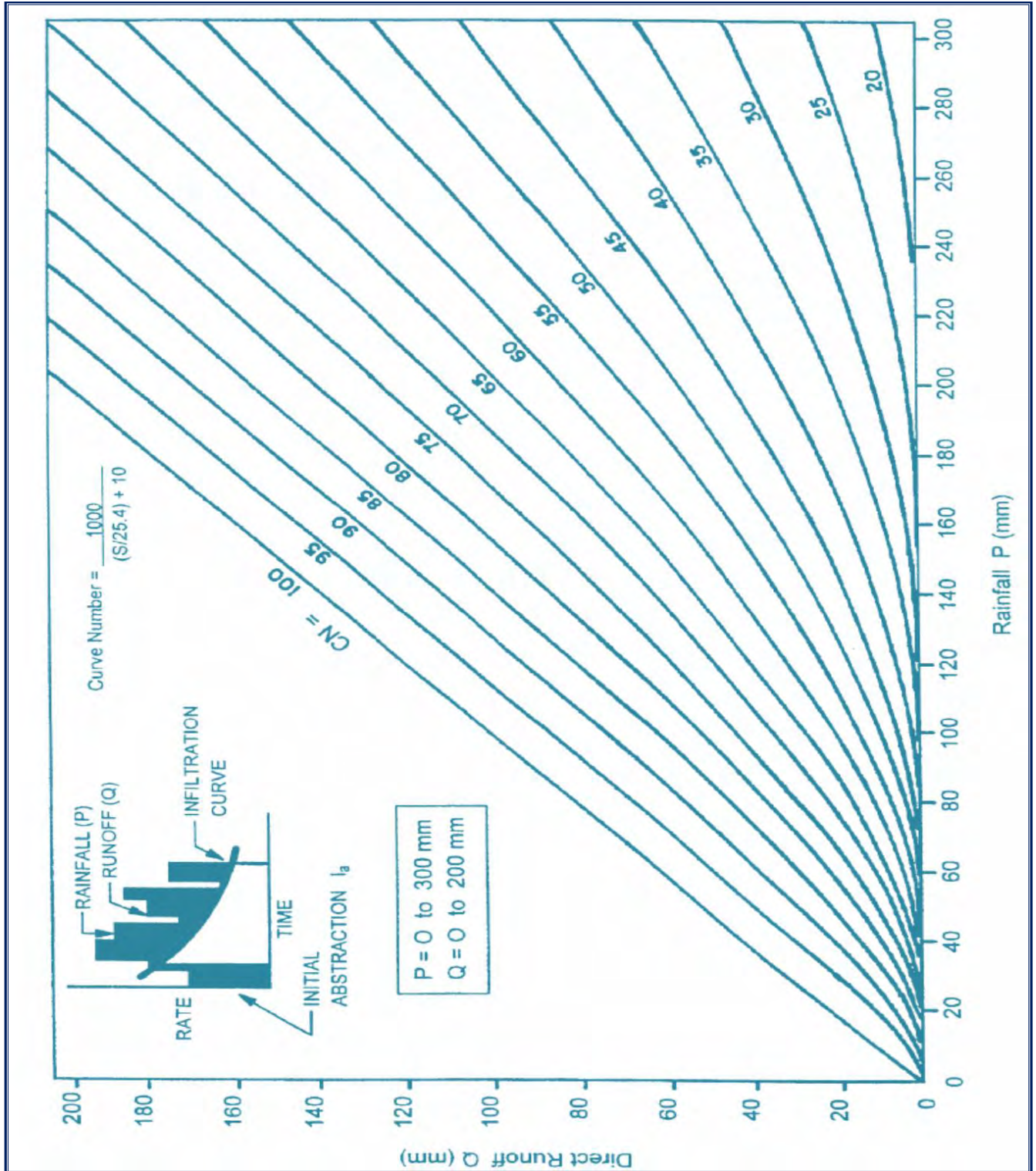


Figure c: SCS Relation between Direct Runoff, Curve Number and Precipitation

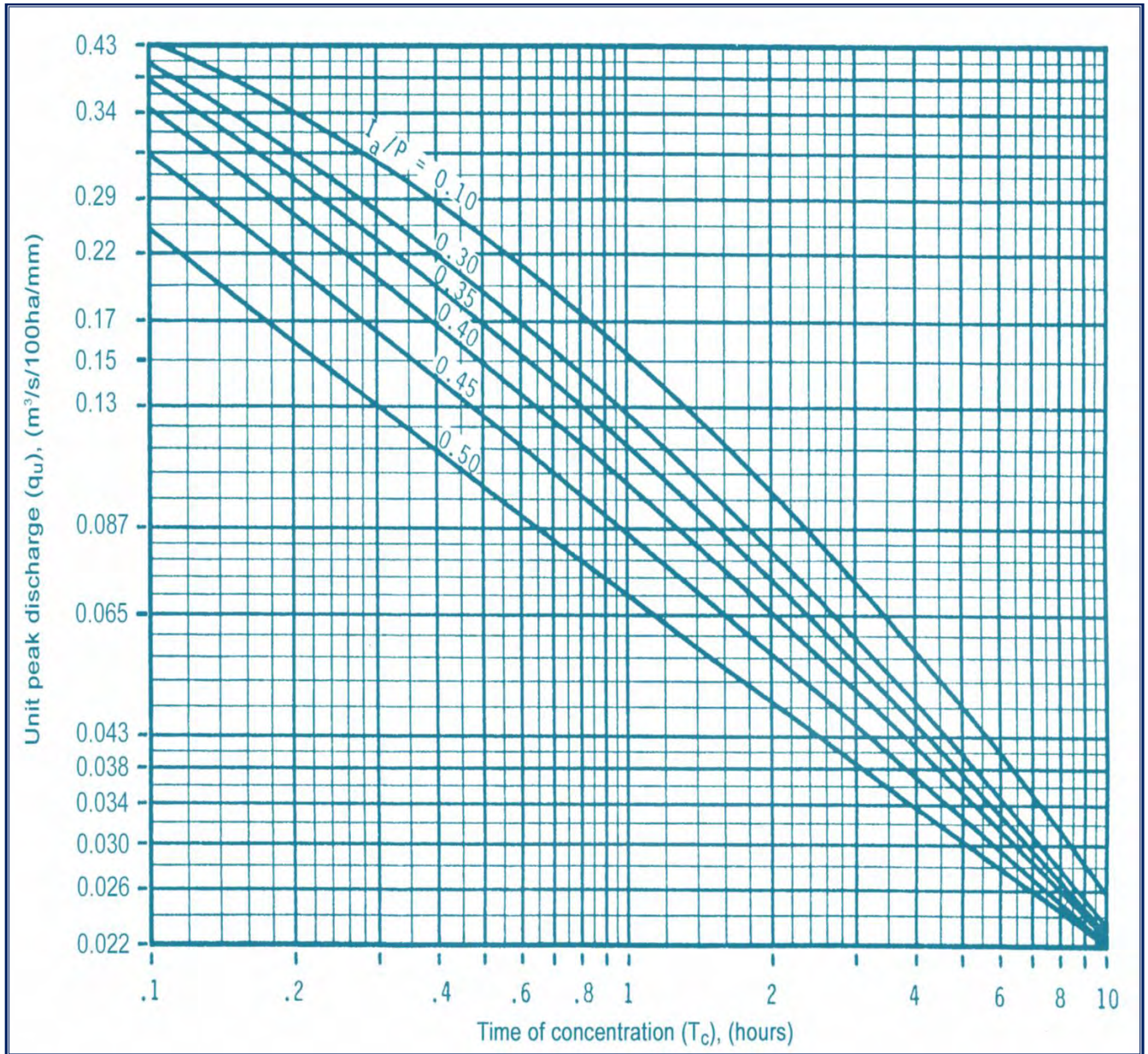


Figure d: Unit Peak Discharge, Type II rainfall

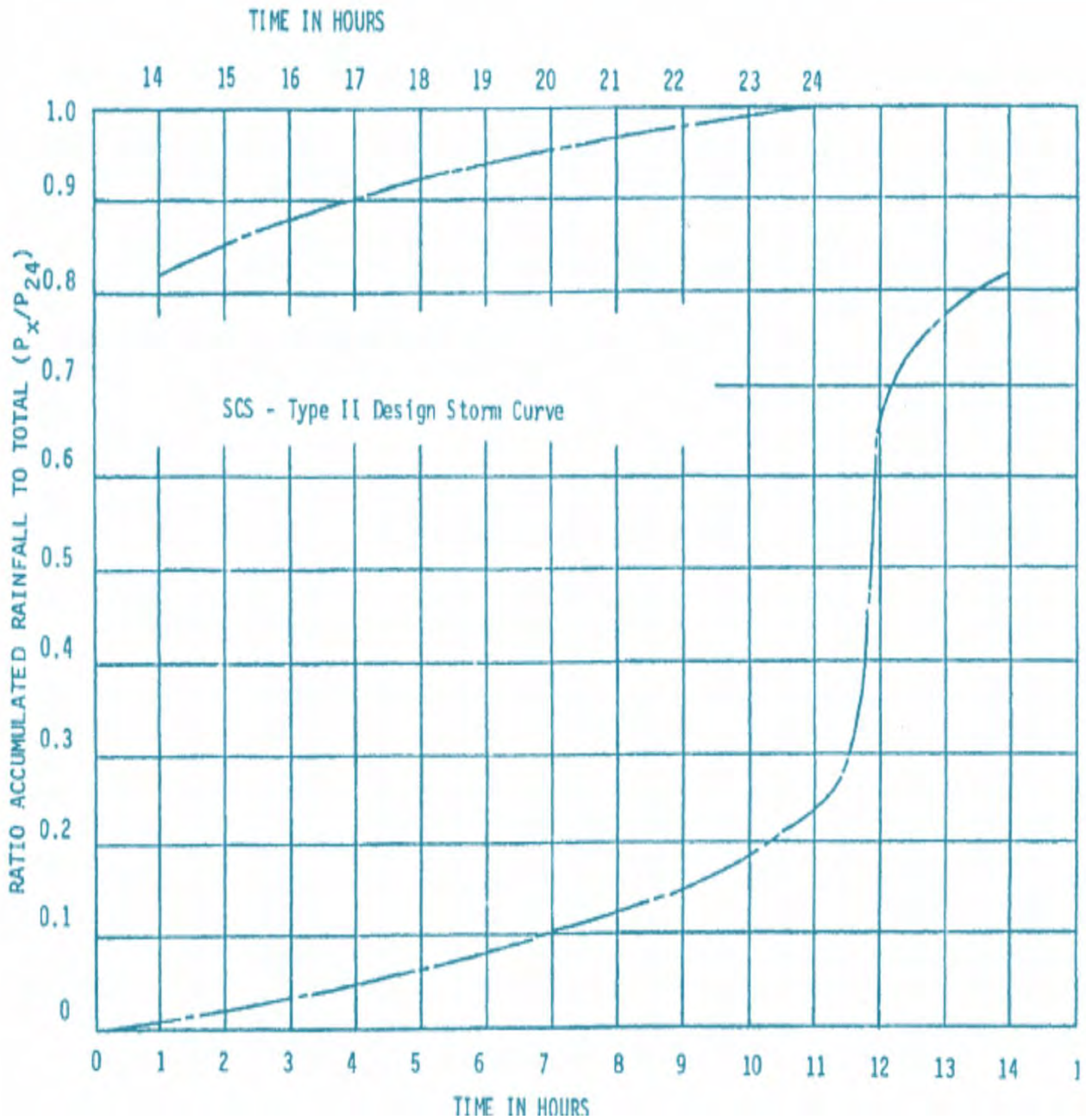
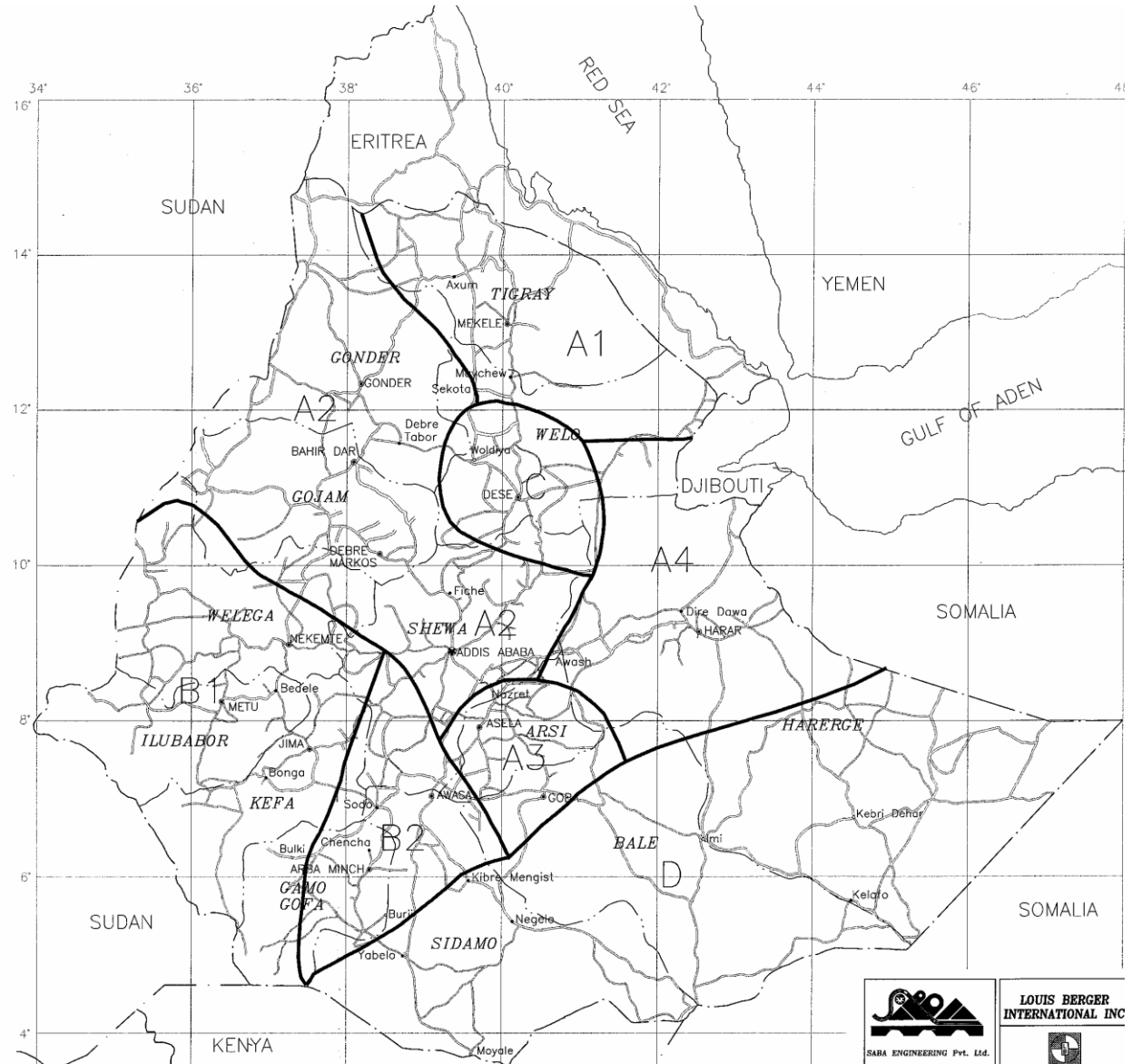


Figure e: Type II SCS Storm Curve

Figure f Rainfall Regions of Ethiopia

Intensity-Duration-Frequency Curves for Regions B, C & D (ERA DDM, 2002)



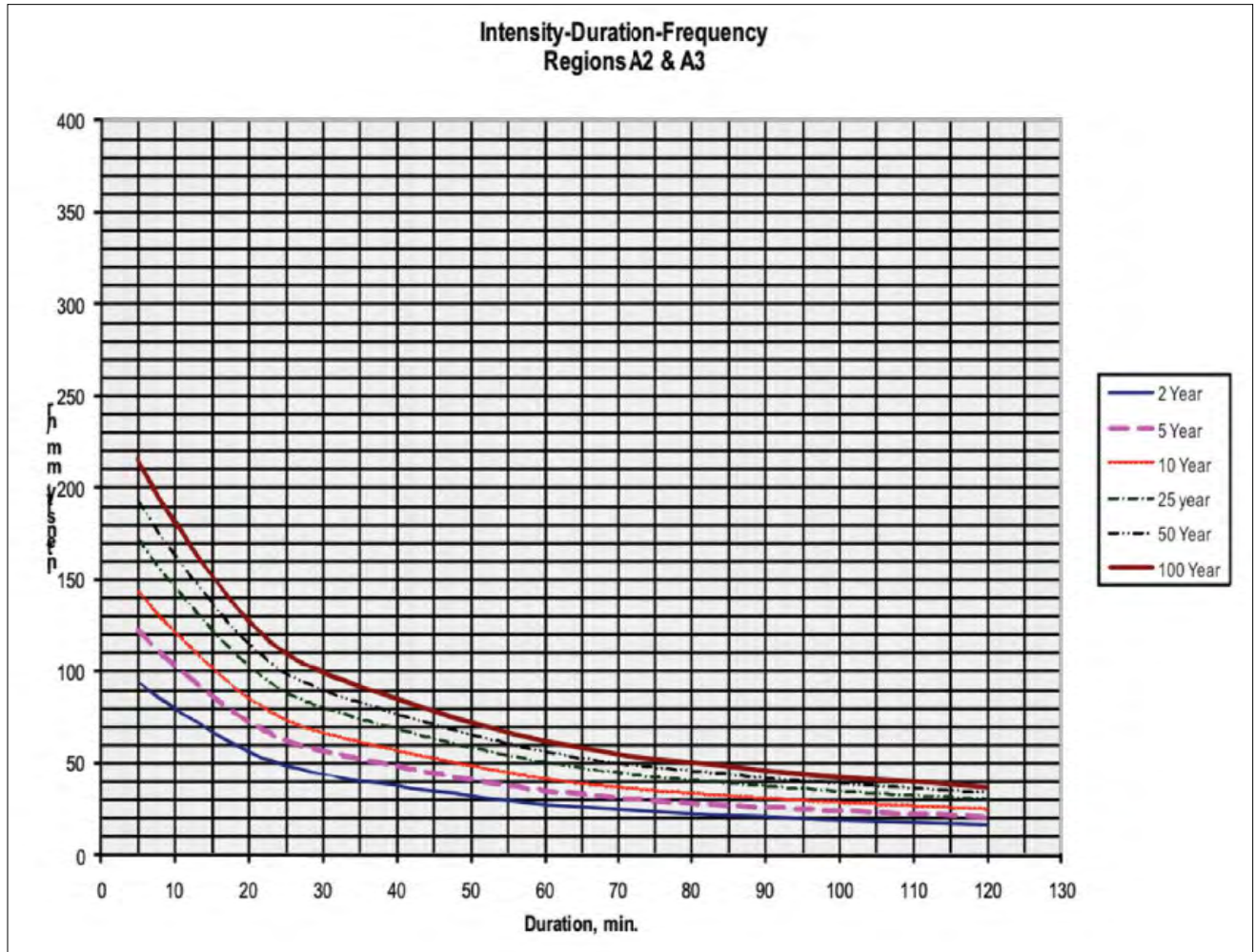


Figure g: IDF Curve for Rainfall Regions of A2 and A3 in Ethiopia (ERA DDM, 2002 & 2011)

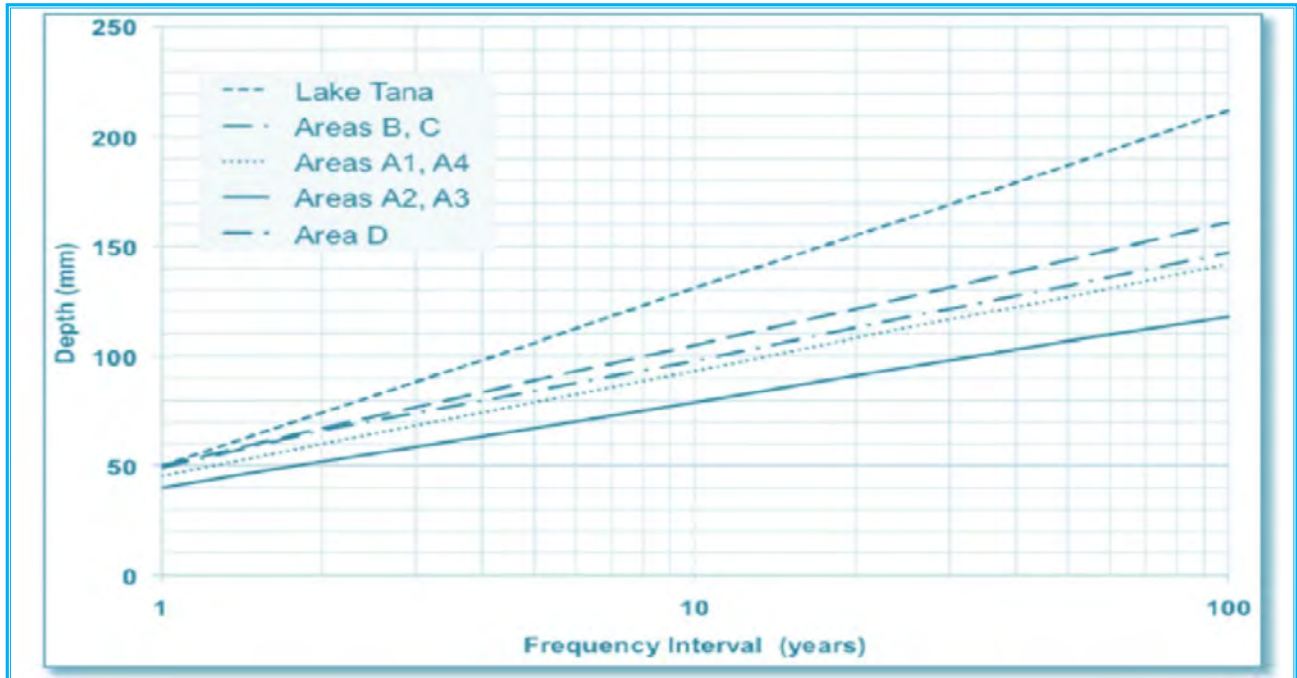


Figure h: 24 hour Depth-Frequency Curve (ERA DDM, 2011)

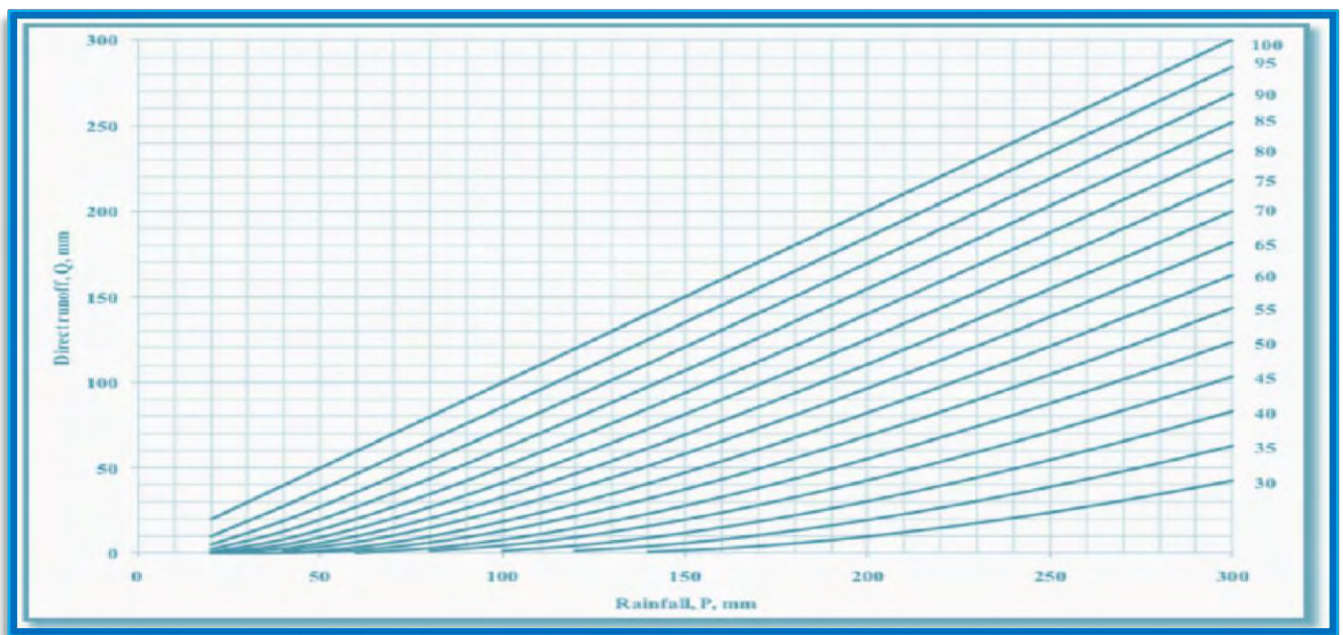


Figure i: Relationships between Precipitation, Direct Runoff and Curve Number (ERA, 2011)

Appendix 2: Roughness and Runoff Coefficient Values

Table a: Values of Roughness Coefficient (n) for Uniform Flow

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

Type of Channel and Description	Minimum	Normal	Maximum
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Dense willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	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 b: Recommended Runoff Coefficient (C) for Various Selected Land Uses

Slope :	Runoff Coefficient, C					
	Soil Group A			Soil Group B		
	< 2%	2-6%	> 6%	< 2%	2-6%	> 6%
Forest	0.08	0.11	0.14	0.10	0.14	0.18
Meadow	0.14	0.22	0.30	0.20	0.28	0.37
Pasture	0.15	0.25	0.37	0.23	0.34	0.45
Farmland	0.14	0.18	0.22	0.16	0.21	0.28
Res. 1 acre	0.22	0.26	0.29	0.24	0.28	0.34
Res. 1/2 acre	0.25	0.29	0.32	0.28	0.32	0.36
Res. 1/3 acre	0.28	0.32	0.35	0.30	0.35	0.39
Res. 1/4 acre	0.30	0.34	0.37	0.33	0.37	0.42
Res. 1/8 acre	0.33	0.37	0.40	0.35	0.39	0.44
Industrial	0.85	0.85	0.86	0.85	0.86	0.86
Commercial	0.88	0.88	0.89	0.89	0.89	0.89
Streets: ROW	0.76	0.77	0.79	0.80	0.82	0.84
Parking	0.95	0.96	0.97	0.95	0.96	0.97
Disturbed Area	0.65	0.67	0.69	0.66	0.68	0.70

Rational Method Runoff Coefficients - Part I

Slope :	Runoff Coefficient, C					
	Soil Group C			Soil Group D		
	< 2%	2-6%	> 6%	< 2%	2-6%	> 6%
Forest	0.12	0.16	0.20	0.15	0.20	0.25
Meadow	0.26	0.35	0.44	0.30	0.40	0.50
Pasture	0.30	0.42	0.52	0.37	0.50	0.62
Farmland	0.20	0.25	0.34	0.24	0.29	0.41
Res. 1 acre	0.28	0.32	0.40	0.31	0.35	0.46
Res. 1/2 acre	0.31	0.35	0.42	0.34	0.38	0.46
Res. 1/3 acre	0.33	0.38	0.45	0.36	0.40	0.50
Res. 1/4 acre	0.36	0.40	0.47	0.38	0.42	0.52
Res. 1/8 acre	0.38	0.42	0.49	0.41	0.45	0.54
Industrial	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.89	0.89	0.90	0.89	0.89	0.90
Streets: ROW	0.84	0.85	0.89	0.89	0.91	0.95
Parking	0.95	0.96	0.97	0.95	0.96	0.97
Disturbed Area	0.68	0.70	0.72	0.69	0.72	0.75

Rational Method Runoff Coefficients - Part II

Table c: Roughness Coefficient Values (Manning’s n) for Sheet Flow

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

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

Appendix 3: Parameters for the Design of Drainage Structures

Table a: 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 b: 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 c: 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 d: 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 e: 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 f: 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 g: 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 h: 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 i: 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 j: 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

Appendix 4

Table: a Typical Hydrologic Soil Groups of Ethiopia

	Soil Types	Hydrologic Soil Group
Ao	Orthic Acrisols	B
Bc	Chromic Cambisols	B
Bd	Dystric Cambisols	B
Be	Eutric Cambisols	B
Bh	Humic Cambisols	C
Bk	Calcic Cambisols	B
Bv	Vertic Cambisols	B
Ck	Calcic Chernozems	B
E	Rendzinas	D
Hh	Haplic Phaeozems	C
Hi	Luvic Phaeozems	C
I	Lithosols	D
Jc	Calcaric Fluvisols	B
Je	Eutric Fluvisols	B
Lc	Chromic Luvisols	B
Lo	Orthic Luvisols	B
Lv	Vertic Luvisols	C
Nd	Dystric Nitosols	B
Ne	Eutric Nitosols	B
Od	Dystric Histosols	D
Oe	Eutric Histosols	D
Qc	Cambric Arenosols	A
Rc	Calcaric Regosols	A
Re	Eutric Regosols	A
Th	Humic Andosols	B
Tm	Mollic Andosols	B
Tv	Vitric Andosols	B
Vc	Chromic Vertisols	D
Vp	Pellic Vertisols	D
Xh	Haplic Xerosols	B
Xk	Calcic Xerosols	B
Xl	Luvic Xerosols	C
Yy	Gypsic Yermosols	B
Zg	Gleyic Solonchaks	D
Zo	Orthic Solonchaks	B