ADDIS ABABA INSTITUTE OF TECHNOLOGY

DEPARTEMENT OF CIVIL ENGINEERING

MSc. PROGRAM IN WATER SUPPLY AND ENVIRONMENTAL ENGINEERING

THESIS PAPER

ON

# INVESTIGATION ON STORM DRAINAGE PROBLEM OF ADDIS ABABA

**(Case Study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road)**

**Master Thesis by Desalegn Getachew Tessema**

**Advisor Dr. Yilma Seleshi**

**July 2011**

# INVESTIGATION ON STORM DRAINAGE PROBLEM OF ADDIS ABABA

(Case Study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road)

A Thesis

Presented to the

School of Graduate Studies

Addis Ababa University

Institutes of Technology

In partial fulfillment of the requirements for the degree master of science in Water Supply and Environmental Engineering

By Desalegn Getachew Tessema

July 2011

INVESTIGATION ON STORM DRAINAGE PROBLEM OF ADDIS ABABA

(Case study at Gotera – Wollo Sefer, Saris - Gotera and Ring Road)

A Thesis Presented to the School of Graduate Studies Addis Ababa University Institutes of Technology in Partial Fulfillment of the Requirements for the Degree of Master of Science in Water Supply and Environmental Engineering.

Date of Defense ----------------------------

Approval by board of Examiners

| | | |
|---|---|---|
| 1. ---------------------------- | ---------------- | ------------------- |
| Chairman | Signature | Date |
| 2. ---------------------------- | ---------------- | ------------------- |
| Adviser | Signature | Date |
| 3. ---------------------------- | ---------------- | ------------------- |
| Internal examiner | Signature | Date |
| 4. ---------------------------- | ---------------- | ------------------- |
| External examiner | Signature | Date |

## Declaration and copy right

I, Desalegn Getachew, declares that this thesis is my original work and that it has not been presented and will not be presented by me to any other university for similar or any other degree award.

Signature ……



Addis Ababa University
July 2011

## Acknowledgement

First of all I would like to thank the almighty GOD for his unspeakable gift, help and protection during my work.

I would like to express my genuine gratitude and appreciation to Dr. Yilma Seleshi, whose encouragement, guidance and support from the initial to the final level enabled me to develop an understanding of the subject I was working and day to day follow up for the completion of this thesis.

Also, I am deeply grateful to Ato Wolday Berhe, Defense Construction Enterprise General Manager, who supports me throughout the master program.

Lastly, I offer my regards and thanks to my Father and W/t Kidist for being with me to complete my study.

Desalegn G.

May 2011, Addis Ababa, Ethiopia

## Abstract

Proper drainage system is required in developing urban areas. In Addis Ababa, drainage problem become an issue during rainy season. This study deals with investigation of storm drainage problem of Addis Ababa and a possible mitigation measure to overcome the problem. Despite there are many places in the city facing storm drainage problem; Ethio china road (Gotera – Wollo sefer), Saris Gotera road (Debrezeit road) and Ring roads are areas selected for this study.

Based on primary and secondary data collected, the problems in the areas are categorized as construction, management and design problem. The method used to investigate management problem is direct field data collection and site visit but the construction problem is analyzed using field survey as well as comparison of design with what is implemented in the ground. Design of the study area is evaluated by redesigning of the system using the computation sheet used in American Federal Highway Administration (FHWA) urban drainage design manual and Addis Ababa City Road Authority (AACRA) urban drainage design manual with some modification. Design of storm drainage system evaluated in this research includes inlet spacing, pipe sizing and inlet type selection. The Values of inlet spacing and pipe sizing obtained by redesigning is compared with the original design.

The result of this paper shows that the problem in Gotera - Wollo Sefer and Ring road is caused by insufficient drainage operation, over spaced inlet spacing and minimum pipe size is used. The investigation in Saris – Gotera road also shows; the curb inlets are over spaced and constructed with very small opening, the operation system of the drainage is in sufficient and curbs are not constructed according to the design.

Finally, based on the result obtained a possible mitigation measures is recommended.

## Table of Content page

***List of Tables*** ***page***

***List of Figures*** ***page***

# 1. Introduction

## 1.1. General

Drainage systems are required in developing urban areas for road safety and proper environmental condition. The two types of water that requires drainage are waste water and storm water. Urban drainage systems handle these two types of water with separate or combined drainage system. In Addis Ababa, capital city of Ethiopia, most of the drainage systems are separate and this study deals with the investigation of storm drainage problem on the Ring road, Gotera – Wollo sefer road and Saris - Gotera road.

Storm water is generated by rain fall and consists of that portion of rainfall that runs off from urban surface. Hence, the properties of storm water, in terms of quality and quantity, are intrinsically linked to the natural and characteristic of both the rainfall and the catchment (Butlers and Davies, 2004)

Storm drainage design is an integral component in the design of highway. Drainage design for highway facilities must strive to maintain compatibility and minimize interference with existing drainage patterns, control flooding of the roadway surface for design flood events, and minimize potential environmental impacts from highway related storm water runoff.

Storm water collection systems must be designed to provide adequate surface drainage. Traffic safety is intimately related to surface drainage. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity (FHWA, 2001)

The sites which are selected for investigation are among the major roads which are constructed in Addis Ababa recently and along these roads areas which are highly affected by the storm drainage problem are identified for detail study.

Using the available data the drainage system of the roads is evaluated and compared with the original design as well as what is implemented in the ground. Depending on the problem investigated a possible mitigation is also recommended.

## 1.2. Study area

The research is conducted at varies location in Addis Ababa where significant storm drainage problem exist. Addis Ababa lies at an altitude of 2300mt above sea level, located at 9$^o$ 1’ 48’’ N 38$^o$ 44’ 24’’ E. The city lies at the foot of Mount Entoto. Its lowest point is around Akaki and its highest point is at Entoto. The city posses a complex mix of highland and climate zones, with temperature difference up to 10$^o$c, depending on elevation and prevailing wind pattern. The proposed areas for this study are

- Gotera – Wollo sefer road
- Saris – Gotera road and
- Along Ring road where considerable drainage problem exist,

The sites are indicated on the maps below

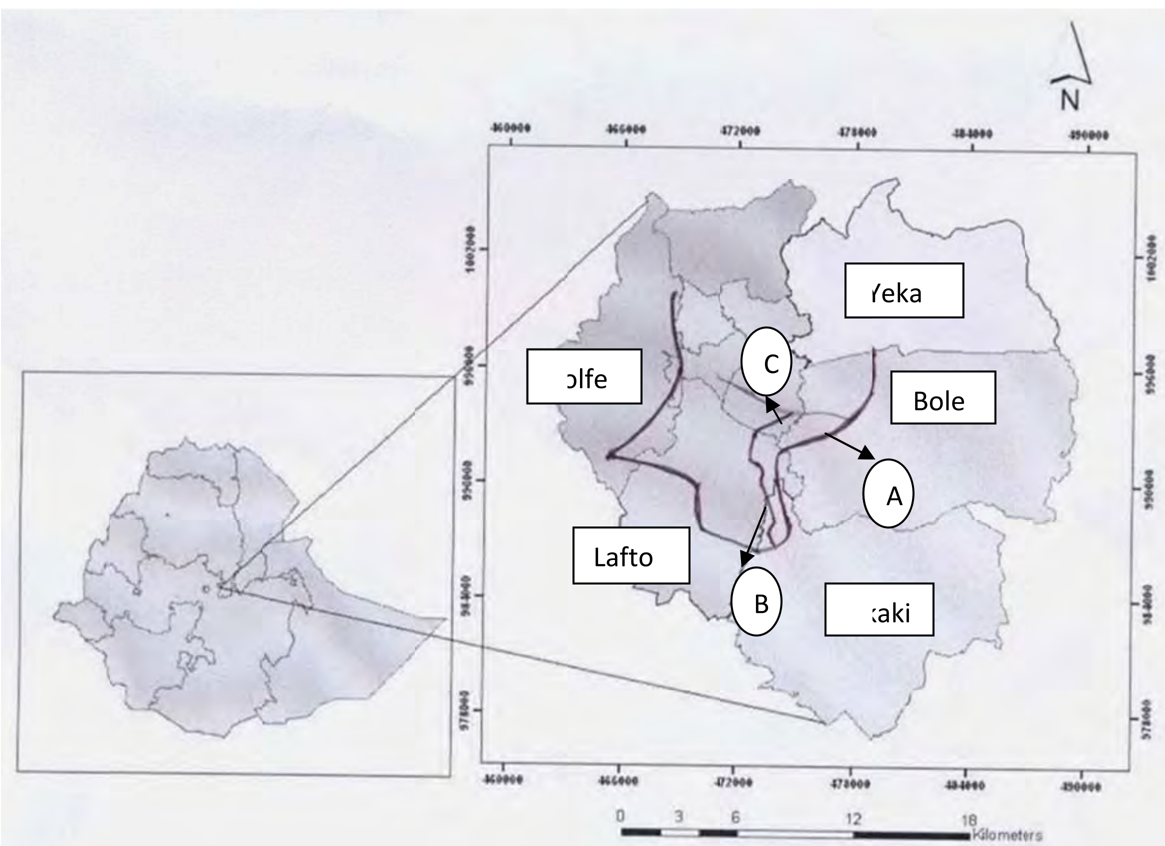



*Fig1.1 The location of the study area*

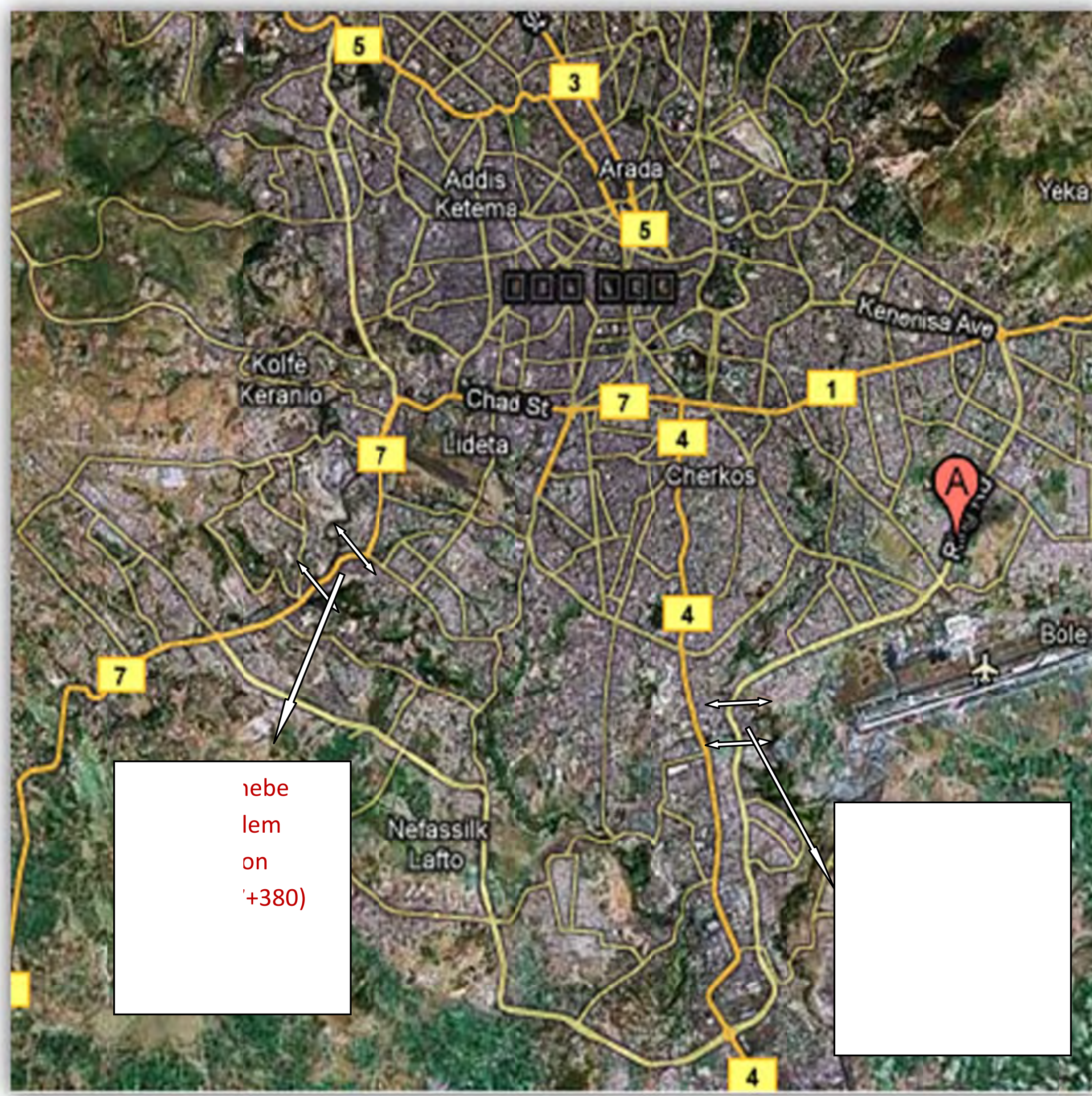



*Fig 1.2 map of ring road and identified problem area (detail A in fig 1.1)*


*Fig 1.3 Map of Gotera – Wollo sefer (detail C in fig 1.1) and Saris - Gotera (detail B in gig 1.1) and identified problem areas.*

### *1.3.* Statement of the Problem

Due to various reasons which are going to be investigated in this research, the storm drainage system in the mentioned roads is not working properly. This leads to

- Flooding during and after the rainfall which is the main cause of congested traffic flow,
- Erosion of pavement and
- Negative impact on health and difficulty on day today activity of people who are using the road and living at the side of the road.

*Fig1.4 a) Flooding due to poor drainage problem at Addis sefer (August 2010)*

*Fig1.4b) Flooding due to poor drainage problem at Zenebework (August 2010)*

*Fig 1.4) Drainage problem at Ring road*

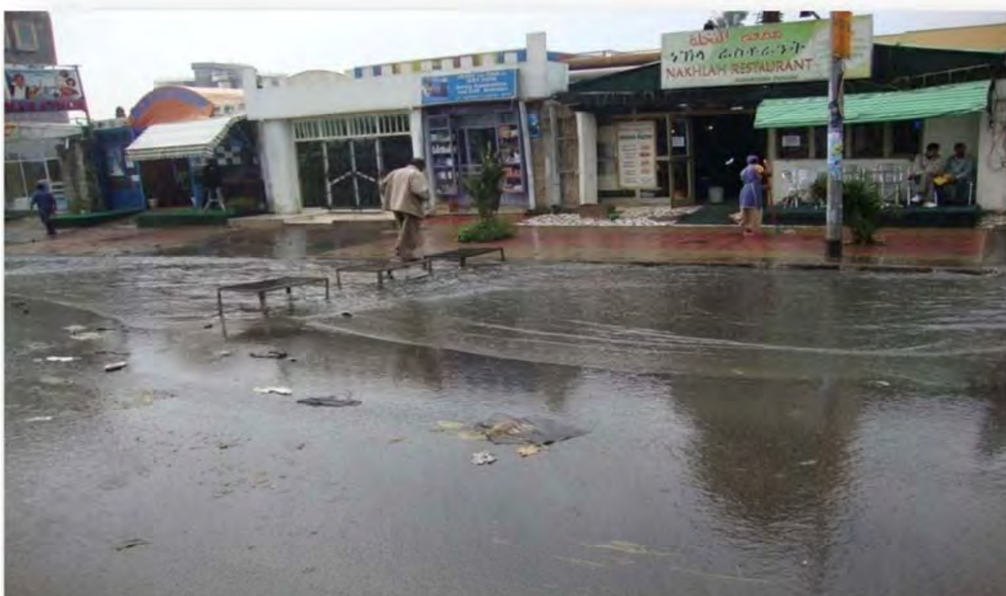



*Fig1. 5 a) Difficulty for movement of people due to poor drainage at Gotera - Wollo sefer (August 2010)*

*Fig1. 5 b) Congested traffic flow due to poor drainage at Gotera – Wollo sefer (August 2010)*

*Fig1. 5) Drainage problem at Gotera – Wollo sefer road*

## 1.4. Objective of the study

### 1.4.1. Genera objective

The general objective of the study is to investigate the storm drainage problem of Addis Ababa and to give a possible solution for the problem.

### 1.4.2. Specific Objectives

The specific objectives of the study are:

- To investigate the storm drainage problem of Gotera – Wollo sefer, Saris – Gotera and Ring road.
- To identify success and short coming of the design and construction of
  - Inlet spacing
  - Drainage pipes
  - Drainage at sag point

- To evaluate drainage operational management at the study area
- To give an appropriate solution for the storm drainage problem of the study areas

## 1.5. Scope of the study

This study investigates the storm drainage problem which occurs in the above mentioned roads. Based on the available data; the design, the construction and the operational management of the drainage system is evaluated and a possible mitigation measure is recommended.

# 2. Literature Review

## 2.1. History of Urban Drainage Engineering

The use of drainage systems by humans has a long history dating back to the early third millennium B. C. during the Indus civilization. Not far behind were the Mesopotamians (Adams, 1981). The Minoan civilization on Crete, in the second millennium B.C. also had extensive drainage systems. Knossos, approximately 5 kilometers from Herakleion, the modern capital of Crete, was one of the most ancient and most unique cities of the Aegean and of Europe. The drainage systems at Knossos were most interesting, consisting of two separate systems, one to collect the sewage and the other to collect rain water (see Figures below). After the collapse of the Minoan civilization and before the Greek influence, which was roughly from 1100 to 700 B.C., there was disarray in the Aegean society. The uses of drains were fairly extensive in Minoan palaces and later their use was rediscovered by the Greeks, as they started living in settlements.

Community drainage systems were a relatively late development of the Greeks (Crouch, 1993). Drainage in Greek cities included sewers under the streets in residential areas and drainage channels in public areas. Components of the drainage systems included eavetroughs for individual buildings, drain pipes through walls or foundation of individual houses, collector channels in neighborhoods, and drains in public areas.

After the Greeks, many of the cities and towns were eventually taken over by the Romans. Many Roman cities did not have any type of drainage system, especially those in the outer parts of the Roman Empire. In the more developed communities, stone drains were provided. In the old established cities that were originally built without storm drains, it was difficult to install them during later times. This is why cities such as Pompeii did not have a full network of storm drains. The older parts of the cities had a somewhat random layout because of no urban planning, whereas the newer parts of the cities were built on a square grid street pattern. The downtown core of Pompeii, around the forum, does have the random layout; whereas rectangular city blocks were used in the later expansion of the city. Ironically, the older part of Pompeii was the only part that did have storm drain (Mays, 2004)

*Fig 2.1 Drainage system at Knossos* (Greek) *(Mays, 2004)*

*Fig 2.2 Drainage system at Pompeii (Italy) (Mays, 2004)*

## 2.2. Components of a drainage system

A complete storm 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 have been historically considered as part of the "storm drainage system". These components include

- Curbs
- Gutters
- Ditches
- Inlets
- Access holes
- Pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc.

The minor system is normally designed to carry runoff from 10 year frequency storm events.

The major system provides overland relief for storm water flows exceeding the capacity of the minor system. This usually occurs during more infrequent storm events, such as the 25-, 50-, and 100-year storms. The major system is composed of pathways that are provided – knowingly or unknowingly -for the runoff to flow to natural or manmade receiving channels such as streams, creeks, or rivers (AACRA, 2003)

### 2.2.1. Storm water Collection

Storm water collection is a function of the minor storm drainage system which is accommodated through the use of roadside and median ditches, gutters, and drainage inlets. Roadside and median ditches are used to intercept runoff and carry it to an adequate storm drain. These ditches should have adequate capacity for the design runoff and should be located and shaped in a manner that does not present a traffic hazard. If necessary, channel linings should be provided to control erosion in ditches. Where design velocities will permit, vegetative linings should be used.

Gutters are used to intercept pavement runoff and carry it along the roadway shoulder to an adequate storm drain inlet. Curbs are typically installed in combination with gutters where runoff from the pavement surface would erode fill slopes and/or where right-of-way requirements or topographic conditions will not permit the development of roadside ditches. Pavement sections are typically curbed in urban settings. Parabolic gutters without curbs are used in some areas.

Inlets are the receptors for surface water collected in ditches and gutters, and serve as the mechanism whereby surface water enters storm drains. When located along the shoulder of the roadway, storm drain inlets are sized and located to limit the spread of surface water on to travel lanes. The term "inlets," as used here, refers to all types of inlets such as grate inlets, curb inlets, slotted inlets, etc. Drainage inlet locations are often established by the roadway geometries as well as by the intent to reduce the spread of water onto the roadway surface. Generally, inlets are placed at low points in the gutter grade, intersections, crosswalks, cross-slope reversals, and on side streets to prevent the water from flowing onto the main road. Additionally, inlets are placed upgrade of bridges to prevent drainage onto bridge decks and downgrade of bridges to prevent the Flow of water from the bridge onto the roadway surface (Mays, 2004)

### 2.2.2. Storm water Conveyance

Upon reaching the main storm drainage system, storm water is conveyed along and through the right-of-way to its discharge point via storm drains connected by access holes or other drainage access structures. In some cases, storm water pump stations may also be required as a part of the conveyance system.

Storm drains are defined as that portion of the storm drainage system that receive runoff from inlets and conveys the runoff to some point where it is discharged into a channel, water body, or other piped system. Storm drains can be closed conduit or open channel; they consist of one or more pipes or conveyance channels connecting two or more inlets.

Access holes, junction boxes and inlets serve as access structures and alignment control points in storm drainage systems. Critical design parameters related to these structures include access structure spacing and storm drain deflection. Spacing limits are often dictated by maintenance activities. In addition, these structures should be located at the intersections of two or more storm drains, when there is a change in the pipe size, and at changes in alignment (horizontal or vertical) (AACRA, 2003)

In areas where gravity drainage is impossible or not economically justifiable, storm water pump stations are often required to drain depressed sections of roadways.

Detention/ retention facilities are used to control the quantity of runoff discharged to receiving waters. A reduction in runoff quantity can be achieved by the storage of runoff in detention/retention basins, storm drainage pipes, swales and channels, or other storage facilities. Outlet controls on these facilities are used to reduce the rate of storm water discharge. This concept should be considered for use in highway drainage design where existing downstream receiving channels are inadequate to handle peak flow rates from the highway project, where highway development would contribute to increased peak flow rates and aggravate downstream flooding problems, or as a technique to reduce the size and associated cost of outfalls from highway storm drainage facilities (FHWA, 2001)

### 2.3. Storm water Generation

Storm water is generated by rainfall, and consists of that proportion of rainfall that runs off from urban surfaces. The transformation of a rainfall hyetograph into a surface runoff hydrograph involves two principal parts. Firstly, losses due to interception, depression storage, infiltration and evapo transpiration are deducted from the rainfall. Secondly, the resulting effective rainfall is transformed by surface routing into an overland flow hydrograph (Butlers and Davies, 2004)

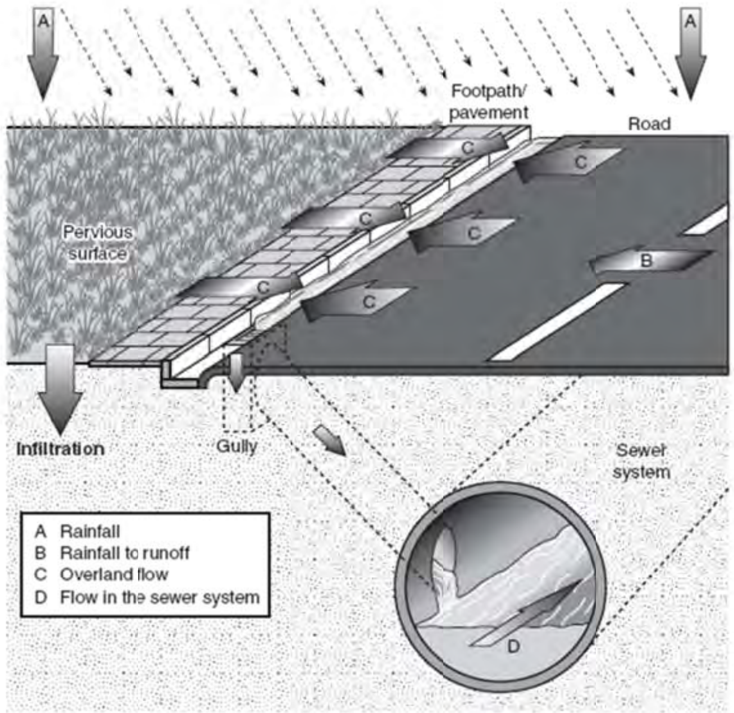



*Fig2.3 Storm water runoff generation processes (Butlers and Davies, 2004)*

### 2.3.1. Storm Water Computation

One of the most commonly used equations for the calculation of peak flow from small areas is the rational formula, given as:

$$Q = \frac{CIA}{K_u} \tag{2.1}$$

Where:

Q = Flow, $m^3/s$

C = dimensionless runoff coefficient

I = rainfall intensity, mm/hr

A = drainage area, hectares, ha

$K_u$ = units conversion factor equal to 360

Assumptions inherent in the rational formula are as follows

- Peak flow occurs when the entire watershed is contributing to the flow
- Rainfall intensity is the same over the entire drainage area.
- Rainfall intensity is uniform over a time duration equal to the time of concentration, tc. The time of concentration is the time required for water to travel from the hydraulically most remote point of the basin to the point of interest.
- Frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 10-year rainfall intensity is assumed to produce the 10-year peak flow.
- Coefficient of runoff is the same for all storms of all recurrence probabilities.

Because of these inherent assumptions, the Rational formula should only be applied to drainage areas smaller than 80 ha (200 ac) (Mays, 2004)

#### 2.3.1.1. Runoff Coefficient

The runoff coefficient, C, in equation above is a function of the ground cover. It relates the estimated peak discharge to a theoretical maximum of 100 percent runoff. Typical values for C are given in the table (see table A-21 in the appendix III). If the basin contains varying amounts of different land cover or other abstractions, a composite coefficient can be calculated through areal weighing as follows

$$C = \frac{\sum CxAx}{A\ Total} \tag{2.2}$$

Where

x = subscript designating values for incremental areas with consistent land cover

$C_x$ = runoff coefficient for area $A_x$

$A_x$ = area having runoff coefficient $C_x$

#### 2.3.1.2. Rainfall Intensity

Intensity duration frequency curves (IDF curve) are necessary to calculate the intensity of rainfall used for the design. Regional IDF curve are available in AACRA design manual which are shown in fig 2.4 and 2.5 are used for computation. Depending on the location of the study area with respect to the station where the IDF is produced, we choose the IDF curve for a given computation (AACRA, 2003)

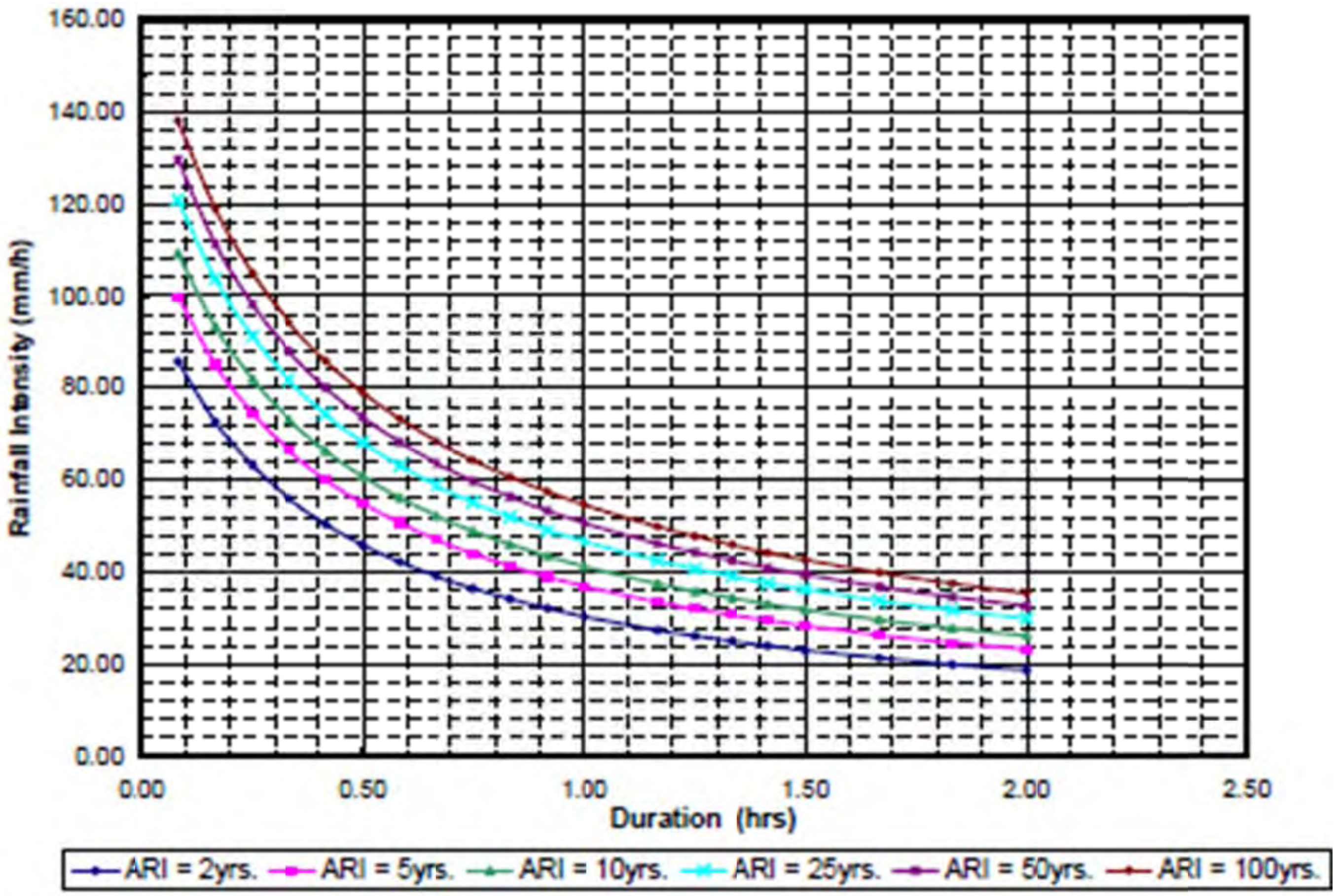



*Fig2.4 Intensity Duration Frequency Curve (Addis Ababa Bole)* (AACRA, 2003)

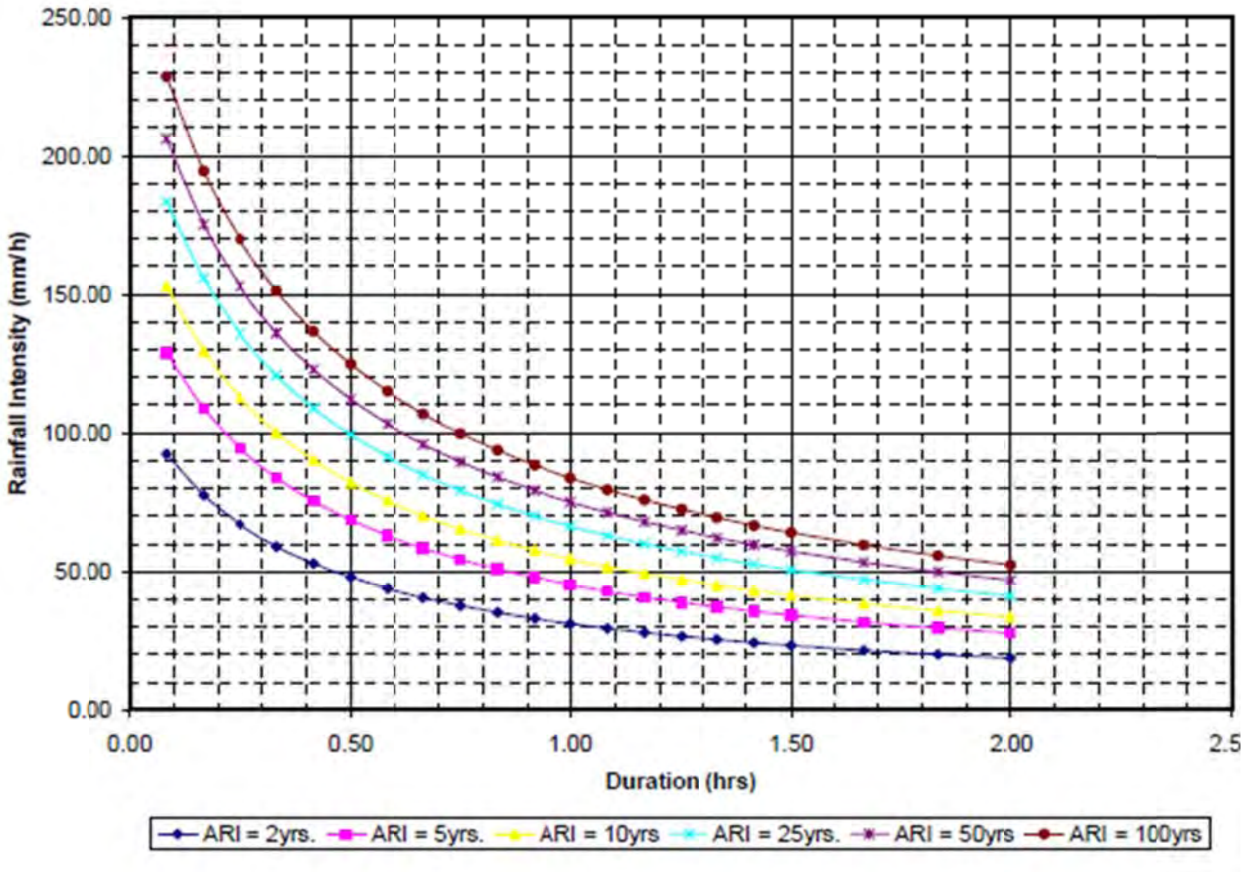



*Fig 2.5 Intensity Duration Frequency Curve (Addis Ababa Observatory)* (AACRA, 2003)

The IDF curve at Observatory is used for Total – Zenebework study area but for Gotera – Wollo Sefer, Saris – Gotera and Addis Sefer – Abo study areas, we use the IDF curve of Bole station. The criterion for choosing a given station is the nearness of the station to the site.

#### 2.3.1.3. Time of Concentration

It is defined as the time required for surface runoff to flow from the remotest part of the catchment area to the point under consideration. Each point in the catchment has its own time of concentration (ERA, 2002)

There are a number of methods that can be used to estimate time of concentration (tc), some of which are intended to calculate the flow velocity within individual segments of the flow path (e.g., shallow concentrated flow, open channel flow, etc.). The time of concentration can be calculated as the sum of the travel times within the various consecutive flow segments.

The time of concentration can be calculated as the sum of the travel times within the various consecutive flow segments (FHWA, 2001)

**Sheet Flow Travel Time**

Sheet flow is the shallow mass of runoff on a planar surface with a uniform depth across the sloping surface. This usually occurs at the headwater of streams over relatively short distances, rarely more than about 130 m, and possibly less than 25 m. Sheet flow is commonly estimated with a version of the kinematic wave equation, a derivative of Manning's equation, as follows (FHWA, 2001)

$$Tti = \frac{Ku}{I^{0.4}}\left(\frac{nL}{\sqrt[2]{S}}\right) \qquad (2.3)$$

Where:

$T_{ti}$ = sheet flow travel time, min

n = roughness coefficient. (See table A-22 in the appendix III)

L = flow length, m

I = rainfall intensity, mm/hr

S = surface slope, m/m

$K_u$ = empirical coefficient equal to 6.92

Since I depend on tc and tc is not initially known, the computation of tc is an iterative process. An initial estimate of tc is assumed and used to obtain I from the IDF curve for the locality. The tc is then computed from equation 2.3 and used to check the initial value of tc. If they are not the same, the process is repeated until two successive tc estimates are the same.

**Shallow Concentrated Flow Velocity**

After short distances of at most 130 m, sheet flow tends to concentrate in rills and then gullies of increasing proportions. Such flow is usually referred to as shallow concentrated flow. The velocity of such flow can be estimated using a relationship between velocity and slope as follows (FHWA, 2001)

$$V = K_u\ k\ S_p^{0.5} \tag{2.4}$$

Where:

$K_u = 1.0$

V = velocity, m/s

k = intercept coefficient (table 2.1)

Sp = slope, percent

*Table 2.1 Intercept Coefficients for Velocity vs. Slope Relationship (FHWA, 2001)*

| Land Cover/ Flow Regime | K |
|---|---|
| Forest with heavy ground litter, hay meadow (Overland flow) | 0.076 |
| Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow) | 0.152 |
| Short grass pasture (overland flow) | 0.213 |
| Cultivated straight row (overland flow) | 0.274 |
| Nearly bare and untilled (overland flow); alluvial fans in western mountain region | 0.305 |
| Grassed waterway (shallow concentrated flow) | 0.457 |
| Unpaved area (shallow concentrated flow); small upland gullies | 0.491 |
| Paved area (shallow concentrated flow); small upland gullies | 0.619 |

**Open Channel and Pipe Flow Velocity**

Flow in gullies empties into channels or pipes. Cross-section geometry and roughness should be obtained for all channel reaches in the watershed. Manning's equation can be used to estimate average flow velocities in pipes and open channels as follows: (FHWA, 2001)

$$V = \frac{Ku}{n} R^{2/3} S^{1/2} \qquad (2.5)$$

Where:

n = roughness coefficient (see table A-22 in appendix III)

V = velocity, m/s

R = hydraulic radius (defined as the flow area divided by the wetted perimeter) m

S = slope, m/m

$K_u$ = units conversion factor equal to 1

For a circular pipe flowing full, the hydraulic radius is one-fourth of the diameter. For a wide rectangular channel (W > 10 d), the hydraulic radius is approximately equal to the depth. The travel time is then calculated as follows:

$$T_{ti} = \frac{L}{60V} \qquad (2.6)$$

Where:

$T_{ti}$ = travel time for segment i, min

L = flow length for segment i, m

V = velocity for segment i, m/s

## 2.4. Pavement drainage

### 2.4.1. Design Frequency and Spread

The objective of highway storm drainage design is to provide for safe passage of vehicles during the design storm event. The design of a drainage system for a curbed highway pavement section is to collect runoff in the gutter and convey it to pavement inlets in a manner that provides reasonable safety for traffic and pedestrians at a reasonable cost. As spread from the curb increases, the risks of traffic accidents and delays, and the nuisance and possible hazard to pedestrian traffic increase.

The process of selecting the recurrence interval and spread for design involves; decisions regarding acceptable risks of accidents, traffic delays and acceptable costs for the drainage system. Risks associated with water on traffic lanes are greater with high traffic volumes, high speeds, and higher highway classifications than with lower volumes, speeds, and highway classifications (Debo and Reese, 2003)

Table 2.2 provides suggested minimum design frequencies and spread based on the type of highway and traffic speed.

*Table 2.2 Suggested Minimum Design Frequencies and Spread (FHWA, 2001)*

| Road Classification | Design Frequency | Design Spread |
|---|---|---|
| High volume or divided or bi - directional | | |
| < 70 km/hr (45 mph) | 10 Years | shoulder + 1m (3 ft) |
| > 70 km/hr (45 mph) | 10 Years | Shoulder |
| Sag Point | 50 Years | Shoulder + 1m (3 ft) |
| | | |
| Collector | | |
| < 70 km/hr (45 mph) | 10 | 1/2 Driving Lane |
| >70 km/hr (45 mph) | 10 | Shoulder |
| Sag point | 10 | 1/2 Driving Lane |
| Low Street | | |
| Low ADT | 5 | 1/2 Driving Lane |
| High ADT | 10 | 1/2 Driving Lane |
| Sag point | 10 | 1/2 Driving Lane |

### 2.4.2. Flow in Gutters

Gutter Flow calculations are necessary to establish the spread of water on the shoulder, parking lane, or pavement section. A modification of the Manning's equation can be used for computing flow in triangular channels. The modification is necessary because the hydraulic radius in the equation does not adequately describe the gutter cross section, particularly where the top width of the water surface may be more than 40 times the depth at the curb. To compute gutter flow, the Manning's equation is integrated for an increment of width across the section. The resulting equation is: (FHWA, 2001)

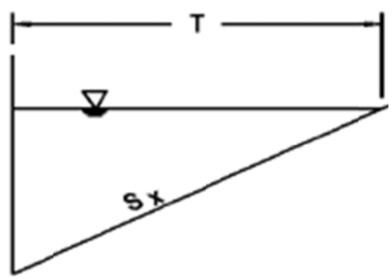



*Fig. 2.6 Triangular gutter cross section*

$$Q = \frac{\mathrm{KuSx}^{1.67}\mathrm{SL}^{0.5}\mathrm{T}^{2.67}}{\mathrm{n}} \qquad (2.7)$$

Or in terms of T

$$\mathrm{T} = \left(\frac{\mathrm{Qn}}{\mathrm{KuSx}^{1.67}\mathrm{SL}^{0.5}}\right)^{0.375} \qquad (2.8)$$

Where:

$K_u$ = 0.376

n = Manning's coefficient (refer table 2.3)

Q = flow rate, $m^3/s$

T = width of flow (spread), m

$S_x$ = cross slope, m/m

$S_L$ = longitudinal slope, m/m

Spreads on the pavement and flow depth at the curb are often used as criteria for spacing pavement drainage inlets.

$$d = TSx \qquad (2.9)$$

Where:

d = depth of flow, m

*Table 2.3 Manning's n for Street and Pavement Gutters (FHWA, 2001)*

| Types of Gutter or Pavement | manning's |
|---|---|
| concrete gutter | 0.012 |
| Asphalt Pavement | |
| Smooth texture | 0.013 |
| Rough texture | 0.016 |
| Concrete gutter asphalt pavement | |
| Smooth texture | 0.013 |
| Rough | 0.015 |
| Concrete Pavement | |
| Float Finish | 0.014 |
| Broom Finish | 0.016 |
| For gutter with small slope, where sediment may accumulate , increase above values of "n" by | 0.02 |

### 2.4.3. Flow in Sag Vertical Curves

As gutter flow approaches the low point in a sag vertical curve the flow can exceed the allowable design spread values as a result of the continually decreasing gutter slope. The spread in these areas should be checked to insure it remains within allowable limits. If the computed spread exceeds design values, additional inlets should be provided to reduce the flow as it approaches the low point (FHWA, 2001)

### 2.4.4. Gutter Flow Time

The Flow time in gutters is an important component of the time of concentration for the contributing drainage area to an inlet. To find the gutter flow component of the time of concentration, a method for estimating the average velocity in a reach of gutter is needed. The velocity in a gutter varies with the flow rate and the flow rate varies with the distance along the gutter, i.e., both the velocity and flow rate in a gutter are spatially varied. The time of flow can be estimated by dividing the length of the gutter with an average velocity obtained in equation 2.10 (Debo and Reese, 2003)

$$V = \frac{K_u(S_L^{0.5} S_x^{0.67} T^{0.67})}{n} \qquad (2.10)$$

$K_u$ = 0.752

T = width of flow (spread), m

$S_x$ = cross slope, m/m

$S_L$ = longitudinal slope, m/m

n = manning coefficient (table 2.3)

V = velocity in the triangular channel, m/s

### 2.4.5. Drainage Inlet Design

The hydraulic capacity of a storm drain inlet depends upon its geometry as well as the characteristics of the gutter flow. Inlet capacity governs both the rate of water removal from the gutter and the amount of water that can enter the storm drainage system. Inadequate inlet capacity or poor inlet location may cause flooding on the roadway resulting in a hazard to the traveling public (Butlers and Davies, 2004)

#### 2.4.5.1. Inlet Types

Storm drain inlets are used to collect runoff and discharge it to an underground storm drainage system Inlets used for the drainage of highway surfaces can be divided into the following four classes:

- Grate inlets
- Curb-opening inlets
- Slotted inlets
- Combination inlets

Grate inlets consist of an opening in the gutter or ditch covered by a grate. Curb opening inlets are vertical openings in the curb covered by a top slab. The principal advantage of grate inlets is that they are installed along the roadway where the water is flowing. Their principal disadvantage is that they may be clogged by floating trash or debris. Curb inlets are most effective on flatter slopes, in sags, and with flows which typically carry significant amounts of floating debris. The interception capacity of curb-opening inlets decreases as the gutter grade steepens. Consequently, the use of curb-opening inlets is recommended in sags and on grades less than 3%. Slotted inlets consist of a pipe cut along the longitudinal axis with bars perpendicular to the opening to maintain the slotted opening. Combination inlets consist of both a curb-opening inlet and a grate inlet placed in a side-by-side configuration, but the curb opening may be located in part upstream of the grate (Debo and Reese, 2003)

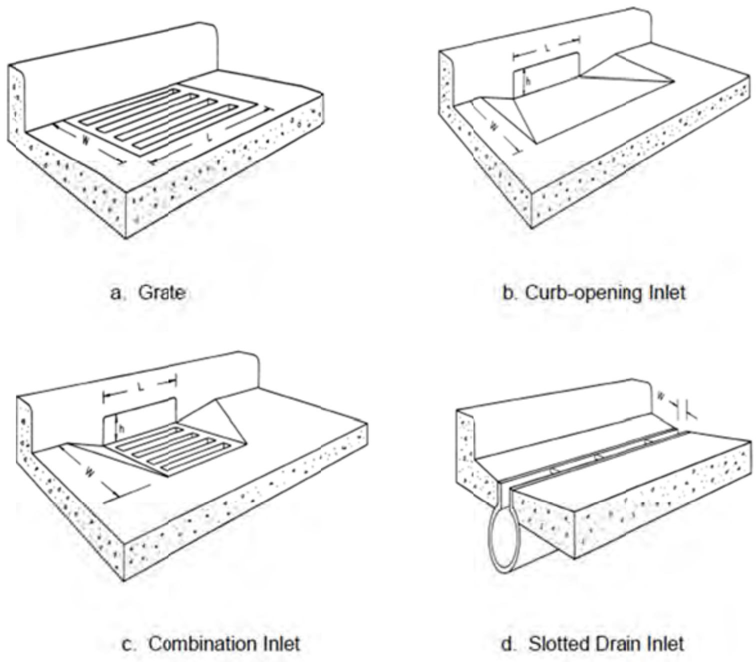



*Fig 2.7 Classes of storm drain inlets*

#### 2.4.5.2. Factors Affecting Inlet Interception Capacity and Efficiency on Continuous Grades

Inlet interception capacity, $Q_i$, is the flow intercepted by an inlet under a given set of conditions. The efficiency of an inlet, E, is the percent of total flow that the inlet will intercept for those conditions. The efficiency of an inlet changes with changes in cross slope, longitudinal slope, total gutter flow, and, to a lesser extent, pavement roughness. In mathematical form, efficiency, E, is defined by the following equation:

$$E = \frac{Qi}{Q} \qquad (2.11)$$

Where:

E = inlet efficiency
Q = total gutter flow, $m^3/s$
$Q_i$ = intercepted flow, $m^3/s$

Flow that is not intercepted by an inlet is termed carryover or bypass and is defined as follows:

$$Q_b = Q - Qi \quad (2.12)$$

Where:

$Q_b$= bypass flow, $m^3/s$

The interception capacity of all inlet configurations increases with increasing flow rates, and inlet efficiency generally decreases with increasing flow rates. Factors affecting gutter flow also affect inlet interception capacity. The depth of water next to the curb is the major factor in the interception capacity of both grate inlets and curb-opening inlets. The interception capacity of a grate inlet depends on the amount of water flowing over the grate, the size and configuration of the grate and the velocity of flow in the gutter. The efficiency of a grate is dependent on the same factors and total flow in the gutter. Interception capacity of a curb-opening inlet is largely dependent on flow depth at the curb and curbs opening length. Flow depth at the curb and consequently, curb-opening inlet interception capacity and efficiency, is *increased by the use of a local gutter depression at the curb-opening or a continuously depressed gutter to increase the proportion of the total flow adjacent to the curb.* Top slab supports placed flush with the curb line can substantially reduce the interception capacity of curb openings (FHWA, 2001)

#### 2.4.5.3. Interception Capacity of Inlets on Grade

**Grate Inlets**

Grates are effective highway pavement drainage inlets where clogging with debris is not a problem. When the velocity approaching the grate is less than the "splash-over" velocity, the grate will intercept essentially all of the frontal flow. Conversely, when the gutter flow velocity exceeds the "splash-over" velocity for the grate, only part of the flow will be intercepted. A part of the flow along the side of the grate will be intercepted, dependent on the cross slope of the pavement, the length of the grate, and flow velocity. Manufacturer of grates have investigate inlet interception capacity (Debo and Reese)

$$Q_i = EQ = Q[R_f E_o + R_s(1 + E_o)] \quad (2.13)$$

Where

$Q_i$ = intercepted flow, $m^3/s$

Q = total gutter flow, $m^3/s$

$R_f$= the ratio of frontal flow intercepted to total frontal flow ($Q_w$)

$R_s$ =the ratio of side flow intercepted to total side flow ($Q_s$)

E = inlet efficiency

$E_o$ = the ratio of frontal flow to total gutter flow (Q)

$$R_f = 1 - K_u(V - V_o) \quad (2.14)$$

Where:

$K_u$ = 0.295

V = velocity of flow in the gutter, m/s

$V_o$ = gutter velocity where splash-over first occurs, m/s

(Note: $R_f$ cannot exceed 1.0)

This ratio is equivalent to frontal flow interception efficiency. Chart 1in appendix III provides a solution for equation above which takes into account grate length, bar configuration, and gutter velocity at which splash-over occurs.

$$E_o = \frac{Q_w}{Q} = 1 - (1 - \frac{W}{T})^{2.67} \quad (2.15)$$

Where:

Q = total gutter flow, $m^3/s$

$Q_w$ = flow in width W, $m^3/s$

W = width of depressed gutter or grate, m

T = total spread of water, m

Chart 2 in appendix III provides solutions of $E_o$ for either uniform cross slopes or composite gutter sections.

$$R_s = 1 \Big/ \left(1 + \frac{K_u V^{1.8}}{S_x L^{2.3}}\right) \tag{2.16}$$

Where:

$K_u$ = 0.0828

$S_x$ = cross slope, m/m

V = velocity of flow in the gutter, m/s

L= grate inlet length

Chart 3 in appendix III provides a solution to the above equation

**Curb-Opening Inlets**

Curb-opening inlets are effective in the drainage of highway pavements where flow depth at the curb is sufficient for the inlet to perform efficiently, as discussed in above. Curb openings are less susceptible to clogging and offer little interference to traffic operation. They are a viable alternative to grates on flatter grades where grates would be in traffic lanes or would be hazardous for pedestrians or bicyclists. Curb opening heights vary in dimension; however, a typical maximum height is approximately 100 to 150 mm (4 to 6 in). The length of the curb-opening inlet required for total interception of gutter flow on a pavement section with a uniform cross slope is expressed by equation

$$L_T = K_u Q^{0.42} S_L^{0.3} \left(\frac{1}{nSx}\right)^{0.6} \tag{2.17}$$

Where:

$K_u$ = 0.817

n = manning coefficient (table 2.3)

$L_T$ = curb opening length required to intercept 100 percent of the gutter flow, m

$S_L$ = longitudinal slope

Q = gutter flow, $m^3/s$

The efficiency of curb-opening inlets shorter than the length required for total interception is expressed by equation

$$E = 1 - (1 - \frac{L}{L_T})^{1.8} \quad (2.18)$$

Where:

L = curb-opening length, m

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections can be found by the use of an equivalent cross slope, Se, in equation above in place of $S_x$. $S_e$ can be computed using equation below.

$$S_e = S_x + S'_w E_o \quad (2.19)$$

Where:

S'w = cross slope of the gutter measured from the cross slope of the pavement, Sx, m/m

S'w = a / [1000 W], for W in m; or = Sw - Sx

a = gutter depression, mm

Eo = ratio of flow in the depressed section to total gutter flow determined by the gutter configuration upstream of the inlet. Eo is the same ratio as used to compute the frontal flow interception of a grate inlet (Mays, 2004)

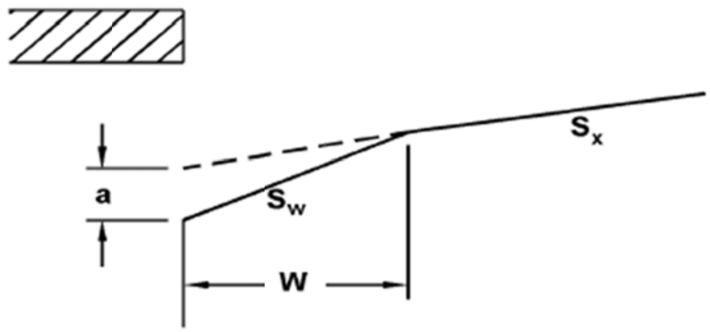



*Fig 2.8 Depressed curb opening inlet*

### 2.4.5.4. Inlet Locations

The location of inlets is determined by geometric controls which require inlets at specific locations, the use and location of flanking inlets in sag vertical curves, and the criterion of spread on the pavement. In order to adequately design the location of the inlets for a given project, the following information is needed:

Layout or plan sheet suitable for outlining drainage areas

- Road profiles
- Typical cross sections
- Grading cross sections
- Super elevation diagrams
- Contour maps

There are a number of locations where inlets may be necessary with little regard to contributing drainage area. These locations should be marked on the plans prior to any computations regarding discharge, water spread, inlet capacity, or flow bypass. Examples of such locations follow.

- At all low points in the gutter grade
- Immediately upstream of median breaks, entrance/exit ramp gores, cross walks, and street intersections., i.e., at any location where water could flow onto the travel way

- Immediately upgrade of bridges (to prevent pavement drainage from flowing onto bridge decks)
- Immediately downstream of bridges (to intercept bridge deck drainage)
- Immediately upgrade of cross slope reversals
- Immediately upgrade from pedestrian cross walks
- At the end of channels in cut sections
- On side streets immediately upgrade from intersection
- Behind curbs, shoulders or sidewalks to drain low area

In addition to the areas identified above, runoff from areas draining towards the highway pavement should be intercepted by roadside channels or inlets before it reaches the roadway. This applies to drainage from cut slopes, side streets, and other areas alongside the pavement. Curbed pavement sections and pavement drainage inlets are inefficient means for handling extraneous drainage (FHWA, 2001)

## 2.5. Storm drains

A storm drain is that portion of the highway drainage system which receives surface water through inlets and conveys the water through conduits to an outfall. It is composed of different lengths and sizes of pipe or conduit connected by appurtenant structures (Butlers and Davies, 2004)

### 2.5.1. Hydraulics of Storm Drainage Systems

#### 2.5.1.1. Flow Type Assumptions

The design procedures presented here assume that flow within each storm drain segment is steady and uniform. This means that the discharge and flow depth in each segment are assumed to be constant with respect to time and distance. Also, since storm drain conduits are typically prismatic, the average velocity throughout a segment is considered to be constant. In actual storm drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is a conservative practice to design using the steady uniform flow assumption (Debo and Reese, 2003)

#### 2.5.1.2. Hydraulic Capacity

The hydraulic capacity of a storm drain is controlled by its size, shape, slope, and friction resistance. Several flow friction formulas have been advanced which define the relationship between flow capacity and these parameters. The most widely used formula for gravity and pressure flow in storm drains is Manning's Equation (AACRA, 2003)

$$V = \frac{K_V}{n} D^{0.6} {S_o}^{0.5} \tag{2.20}$$

$$Q = \frac{K_Q}{n} D^{6.67} \cdot {S_o}^{0.5} \tag{2.21}$$

Where

V = mean velocity, m/s
Q = rate of flow, m3/s
$K_V$ = 0.397
$K_Q$ = 0.312
n = Manning's coefficient (refer table 7-1)
D = storm drain diameter, m
$S_o$ = slope of the hydraulic grade line, m/m

A nomograph solution of Manning's Equation for full flow in circular conduits is presented in chart 4 in appendix III.

The value of manning coefficient is indicated in table 2.4.

*Table 2.4 Manning's Coefficients for Storm Drain Conduits* (Larry W. Mays, 2004)

| **Type of culvert** | **Roughness corrugation** | **Manning’s n** |
|---|---|---|
| Concrete Pipe | Smooth | 0.010-0.011 |
| Concrete Boxes | Smooth | 0.012-0.015 |
| Spiral Rib Metal Pipe | Smooth | 0.010-0.011 |
| Corrugated Polyethylene | Smooth | 0.009-0.015 |
| Corrugated Polyethylene | Corrugated | 0.018-0.025 |
| Polyvinyl chloride (PVC) | Smooth | 0.009-0.011 |

#### 2.5.1.3. Energy Grade Line/Hydraulic Grade Line

The energy grade line (EGL) is an imaginary line that represents the total energy along a channel or conduit carrying water. Total energy includes elevation head, velocity head and pressure head. The calculation of the EGL for the full length of the system is critical to the evaluation of a storm drain. In order to develop the EGL it is necessary to calculate all of the losses through the system. The energy equation states that the energy head at any cross section must equal that in any other downstream section plus the intervening losses. The intervening losses are typically classified as either friction losses or form losses. The friction losses can be calculated using the Manning's Equation. Form losses are typically calculated by multiplying the velocity head by a loss coefficient, K. Knowledge of the location of the EGL is critical to the understanding and estimating the location of the hydraulic grade line (HGL) (AACRA, 2003)

The hydraulic grade line (HGL) is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube at any point along the pipe. The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. HGL, a measure of flow energy, is determined by subtracting the velocity head ($V^2/2g$) from the EGL.

When water is flowing through the pipe and there is a space of air between the top of the water and the inside of the pipe, the flow is considered as open channel flow and the HGL is at the water surface. When the pipe is flowing full under pressure flow, the HGL will be above the crown of the pipe. When the flow in the pipe just reaches the point where the pipe is flowing full, this condition lies in between open channel flow and pressure flow. At this condition the pipe is under gravity full flow and the flow is influenced by the resistance of the total circumference of the pipe. Under gravity full flow, the HGL coincides with the crown of the pipe.

The HGL and Water Surface level (WSL) must be below the surface level or inlet level at pits or structures, otherwise the system will surcharge. Generally the pipe system should be designed so that during the design storm the HGL is at least 150 mm below the level of inlet (Institute of Engineers Australia, 2001).

The pipe line design is most conveniently carried out by working upstream from the outlet. The estimated water surface level at the outlet must be determined for the design storm and the HGL downstream of the outlet assigned this level. Calculations are then carried out up the pipe system to determine the rise in HGL.

The head loss at the outlet structure is added to the elevation of the receiving water level. To this figure the calculated pipe loss is added to arrive at the HGL immediately downstream of the next structure. The process is then repeated for each structure and pipe up the line.

Checks need to be undertaken at structure to ensure the HGL immediately up stream of the pit is not higher than the ground surface level. If the HGL is higher the pit will surcharge and *larger pipes need to be specified* (FHWA, 2001)

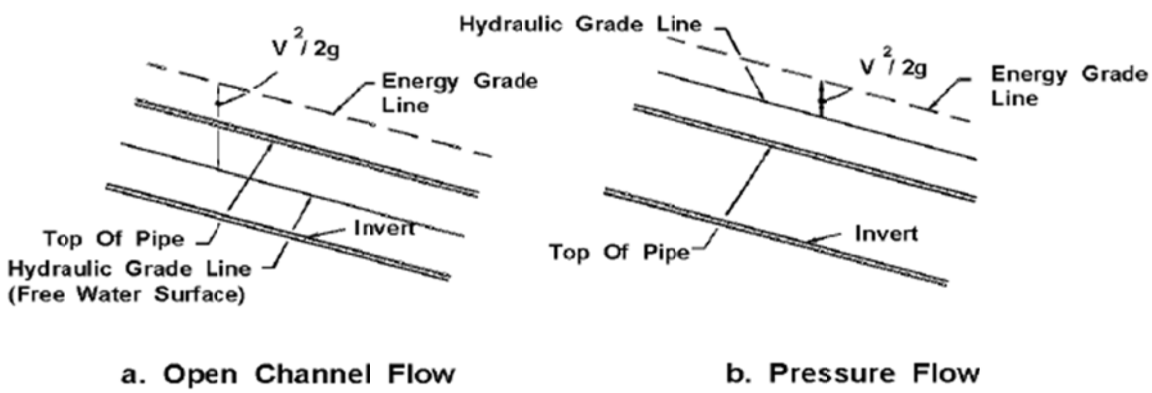



*Fig 2.9 Hydraulic and energy grade line in pipe flow.*

#### 2.5.1.4. Energy Losses

Prior to computing the hydraulic grade line, all energy losses in pipe runs and junctions must be estimated. In addition to the principal energy involved in overcoming the friction in each conduit run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access holes (American concrete pipe association, 2000)

**Pipe Friction Losses**

The major loss in a storm drainage system is the friction or boundary shear loss. The head loss due to friction in a pipe is computed as follows :( FHWA, 2001)

$$H_f = S_f L \tag{2.22}$$

Where:

$H_f$ = friction loss, m

$S_f$ = friction slope or hydraulic gradient, m/m

L = length of pipe, m

The friction slope in equation above is also the slope of the hydraulic gradient for a particular pipe run. As indicated by equation above, the friction loss is simply the hydraulic gradient multiplied by the length of the run. Since this design procedure assumes steady uniform flow in open channel flow, the friction slope will match the pipe slope for part full flow. Pip friction losses for full flow can be determined as follows: (FHWA, 2001)

$$S_f = \frac{H_f}{L} = \left(\frac{Qn}{K_Q D^{2.67}}\right)^2 \tag{2.23}$$

Where:

$K_Q$ = 0.312

n = Manning's coefficient (table 7-1)

D = storm drain diameter, m

**Losses due to bend, obstructions, or structures**

Losses due to bend, obstructions, or structures may be expressed as function of the velocity of flow in the pipe immediately downstream of the bend, obstruction or structure as shown below (AACRA, 2003)

$$h_s = KV_o^2/2g \quad (2.24)$$

Where

$h_s$= head loss at bend, obstruction or structure (m)

K = pressure change coefficient or pit loss coefficient (dimensionless)

$V_o$ = velocity of flow in the downstream pipe (m/sec)

g = acceleration due to gravity (9.807 m/sec)

$V_o^2/2g$ = velocity head (m)

# 3. Data Collection and Data Analysis

## 3.1. Data Collection

To attain the stated objectives, various data were collected from design drawings and by direct field survey. Primary data which are collected during field survey includes:

- Existing inlet type and spacing (table A – 24, A-25 and A-26 in appendix III)
- Condition of Inlets (table A – 24, A-25 and A-26 in appendix III)
- Size and existing condition of curbs (table A – 24 and A-25 in appendix III)

These data are collected twice in the research period. August 2011 and January 2012 are the two months when the data was collected.

In addition to the primary data, there are also secondary data which are collected during the research period in the month of September and October 2011. These data includes

- Land use map of the study area
- Contour map of the study area
- Cross slope and longitudinal slope of the road in the study areas (table A-24, A-25 and A-26) )
- IDF curve of the study area (fig 2.4 and fig 2.5)
- Existing pipe size (table A-24, A-25 and

Plan and typical section of the roads is shown in figure 3.1, 3.2 and 3.3. These figures clearly show the arrangement of the inlets and curb in a given section of road, cross slope of the road and existing inlet spacing of the study area.

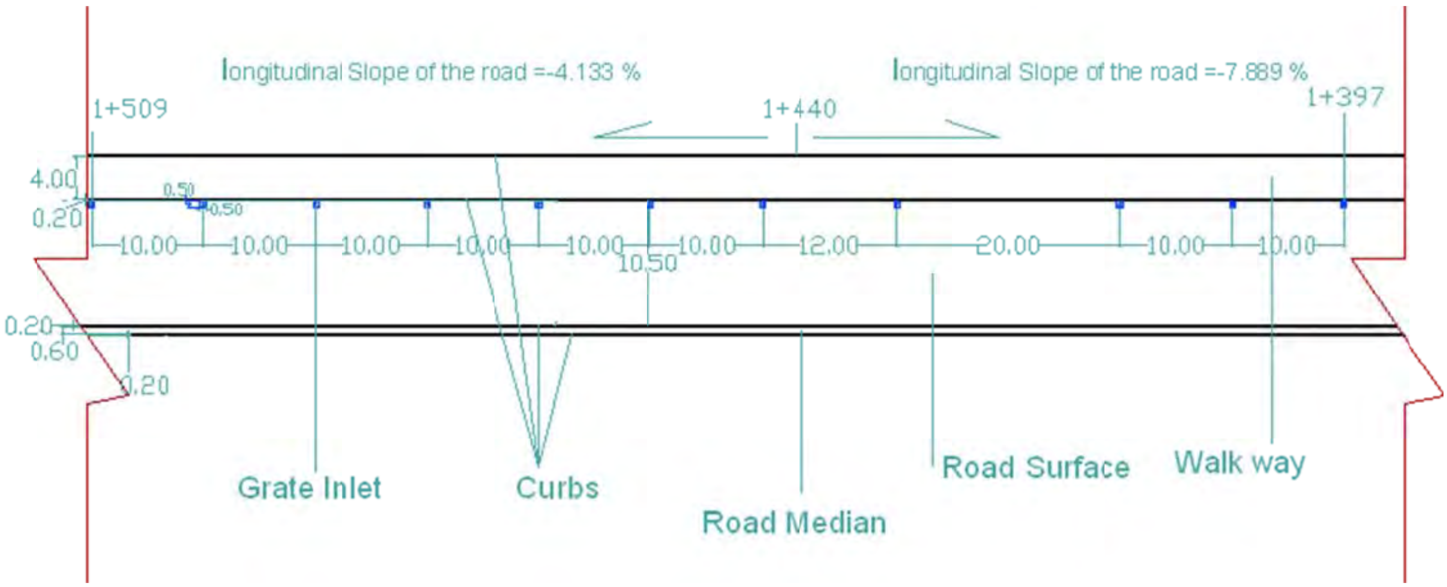
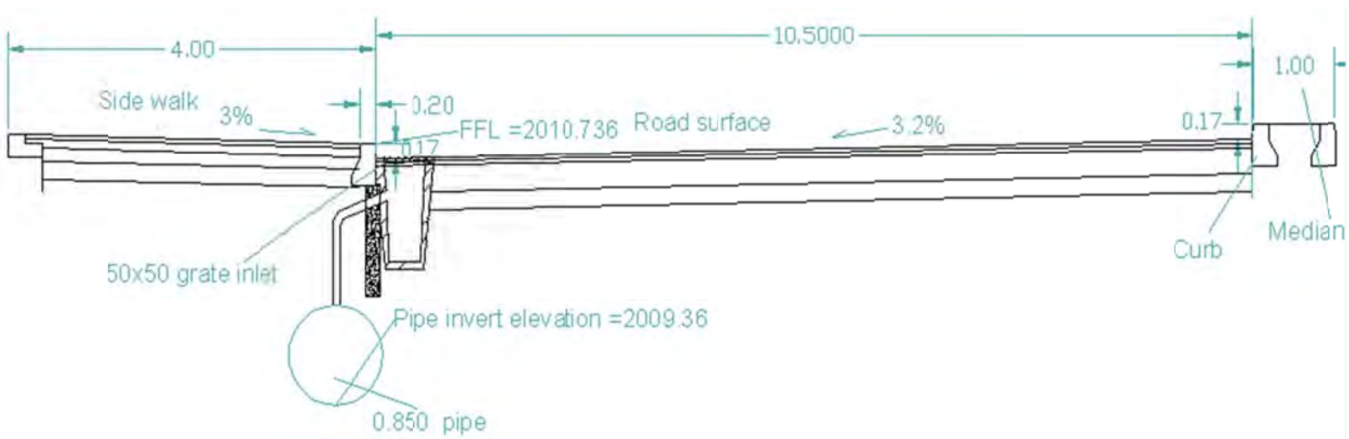



*Fig 3.1 a) Gotera – Wollo Sefer road horizontal profile with the existing inlet spacing (from Chainage 1+397 – 1+509)*

*Note: units are in m*




*Note: units are in m*

*Fig 3.1 b) Gotera - wollo Sefer road section at chainage 1+509*

*Fig 3.1 Gotera Wollo sefer road existing road data* (TCDSc, 2000)

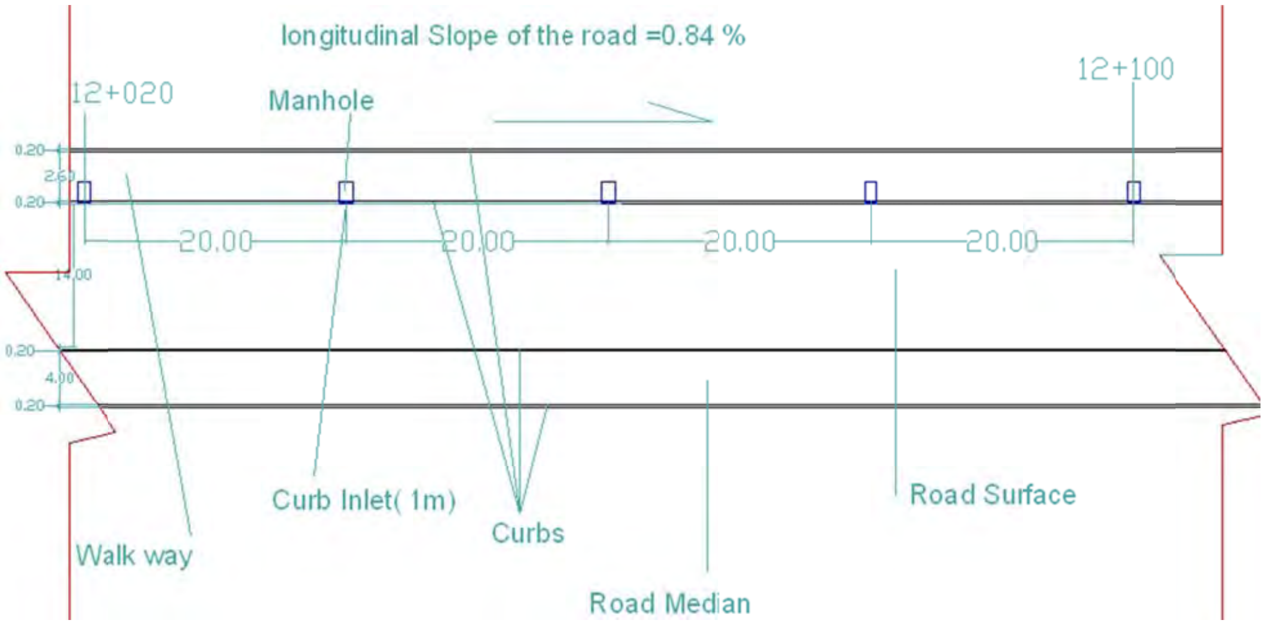
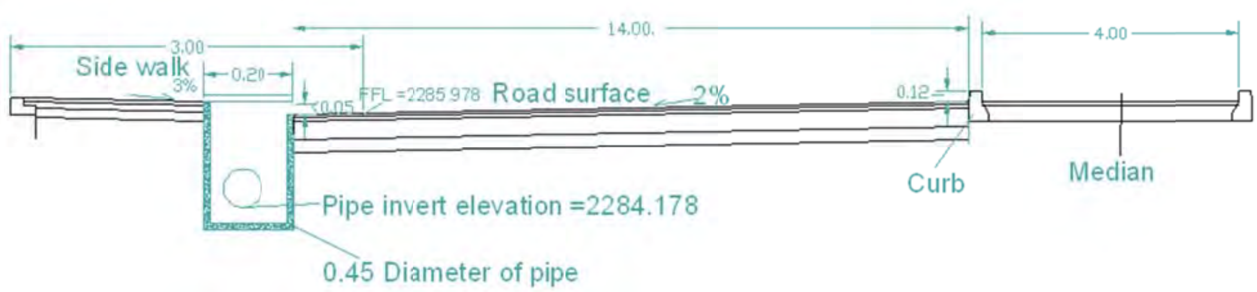



*Fig 3.2 a) Saris- Gotera road horizontal profile with the existing inlet spacing (from Chainage 12+020 – 12+100)*

*Note: units are in m*




*Note: units are in m*

*Fig 3.2 b) Saris – Gotera road section at chainage 12+140*

*Fig 3.2 Saris - Gotera road existing road data* (Engineer Zewdie Eskinder and PLC, 2004)

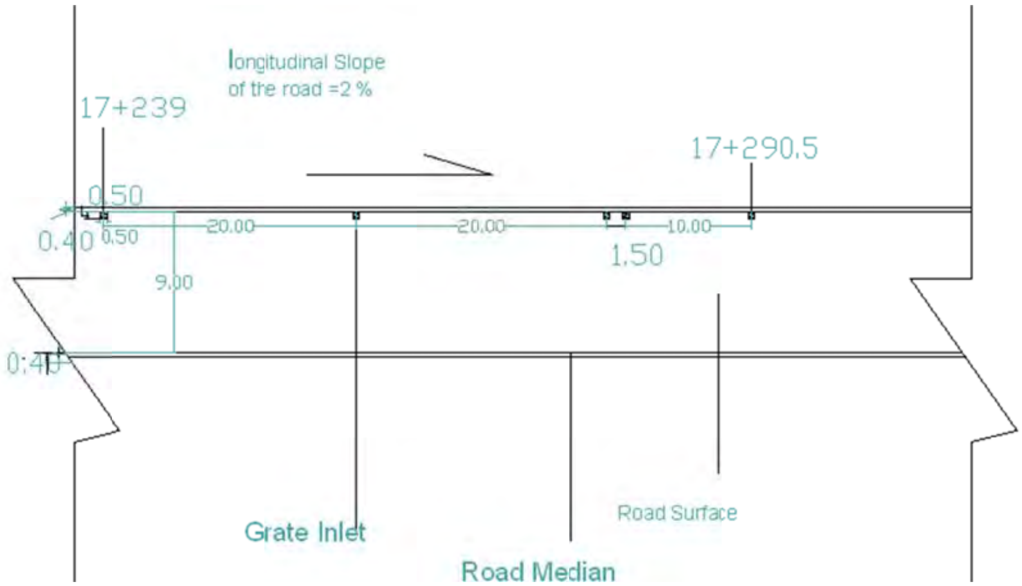
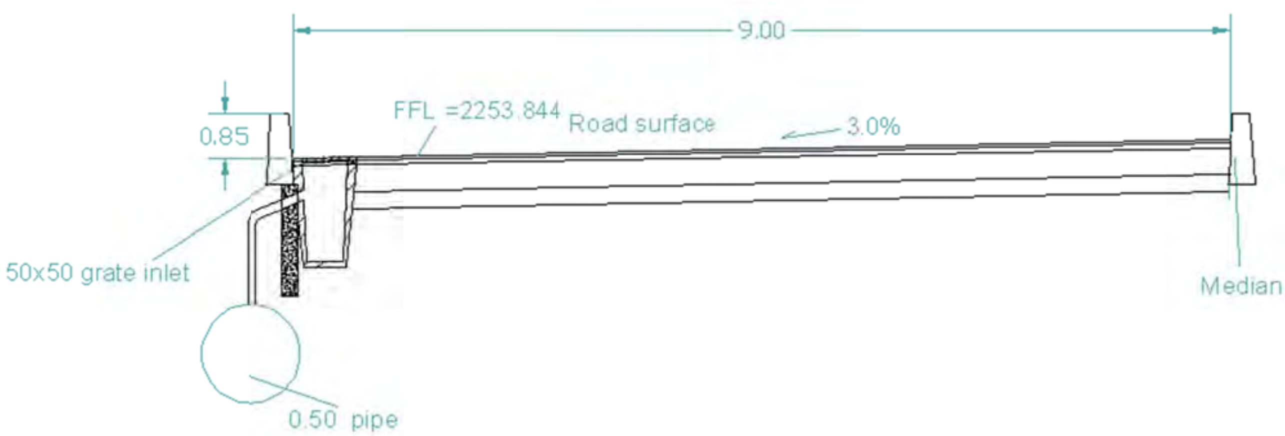



*Fig 3.3 a) Ring Road (Total Zenebework) horizontal profile with the existing inlet spacing (from Chainage 17+239 – 17+290)*

*Note: units are in m*




*Note: units are in m*

*Fig 3.3 b) Ring road section at chainage 17+249*

*Fig 3.3 Ring road existing road data* (Parkman Limited, 1997)

`

## 3.2. Data analysis

The above data collected are helpful to identify the cause of the problem which is going to be investigated on this research. Existing size of inlets and size of curbs data collected are used to evaluate the construction of the system by comparing with the original design. The existing conditions of curbs and inlets data are also used to evaluate the operational drainage management of the system. The basic data which are used to design the inlet spacing and pipe sizing of the system are cross slope of the road, longitudinal slope of the road, land use map, contour map and IDF curve of the nearest station.

Condition of inlets and curb data is summarized as shown in table 3.1, 3.2, 3.3, 3.4, 3.5 and 3.6.

*Table 3.1 Summary of Inlets Data (Gotera Wollo Sefer Road)*

| S.no | Description | Number | Remark |
|---|---|---|---|
| 1 | Inlets in good condition | 30 | Inlets function (operate) properly |
| 2 | Inlets without grate cover (removed), filled with silt and out of function | 50 | Inlets out function |
| 3 | Inlets with grate cover but totally filled with silt and out of function | 8 | Inlets out function |
| 4 | Inlet without cover (Removed) and operate with problem | 16 | Inlets function(operate) with problem |
| **Total** | | **104** | |

*Table 3.2 Summary of Curb Data (Gotera Wollo Sefer Road)*

| S.no | Description | Existing Length (m) | Remark |
|---|---|---|---|
| 1 | Curb in good condition | 1704 (0.17 height) | curbs function (operate) properly |
| 2 | Curb damaged | 510 | curbs out of function |
| **Total** | | **2214** | |

*Table 3.3. Summary of Curb Inlets (Saris – Gotera Road)*

| S.no | Description | Number | Remark |
|---|---|---|---|
| 1 | Curb inlets operate with problem | 38 | Due to in efficient opening |
| 2 | Curb inlets without function | 11 | due to blocked by garbage and silt and poor construction (pavement and cover overlap) |
| **Total** | | **49** | |

*Table 3.4 Summary of Curb (Saris – Gotera Road)*

| S.no | Description | Existing Length (m) | Remark |
|---|---|---|---|
| 1 | Curbs Constructed less than the design | 960 (0.12 height) | curbs function (operate) with problem |
| **Total** | | **960** | |

*Table 3.5 Summary of Inlets Data (Ring Road Total – Zenebework)*

| S.no | Description | Number | Remark |
|---|---|---|---|
| 1 | Inlets in good condition | 11 | Inlets function (operate) properly |
| 2 | Inlets without grate cover (removed), filled with silt and out of function | 10 | Inlets out function |
| 3 | Inlets with grate cover but totally filled with silt and out of function | 2 | Inlets out function |
| 4 | Inlet without cover (Removed) and operate with problem | 1 | Inlets function(operate) with problem |
| **Total** | | **24** | |

*Table 3.6 Summary of Inlets Data (Ring Road Addis Sefer – Abo )*

| S.no | Description | Number | Remark |
|---|---|---|---|
| 1 | Inlets in good condition | 22 | Inlets function (operate) properly |
| 2 | Inlets without grate cover (removed), filled with silt and out of function | 7 | Inlets out function |
| 3 | Inlets with grate cover but totally filled with silt and out of function | 6 | Inlets out function |
| 4 | Inlet without cover (Removed) and operate with problem | 6 | Inlets function(operate) with problem |
| **Total** | | **41** | |

During the data collection time, various photographs are taken to visualize the data collected clearly. Figure 3.4, 3.5 and 3.6 shows the condition of inlets and curb in the study area.

*Fig 3.4 a) Inlets filled with silts*

*Fig 3.4b) Inlets filled with silt and grate removed*

*Fig 3.4 c) Inlets filled with garbage*

*Fig 3.4d) Inlets operating properly*

*Fig 3.4 e) curb damaged due to ramp construction.*

*Fig 3.4 e) curb damaged due to existence of construction site*

*Fig 3.4 Condition of inlet and curs in Gotera – Wollo Sefer Road (January 2011)*

*Fig 3.5 a) Inlet blocked by silt and garbage*

*Fig 3.5 b) Inlet cover overlapped with the Pavement (no opening at all)*

*Fig 3.5 c) under designed curb height (12 cm) and small inlet opening*

*Fig 3.5 Condition of Inlets and curbs at Saris – Gotera road (January 2011)*

*Fig 3.6 a) small opening blocked by silt and garbage (Opening at concrete media)*

*Fig 3.6 b) Inlet covered by garbage*

*Fig 3.6 Condition of Inlet in ring road (January 2011)*

# 4. Methodology

Based on the data collected on the site and design report, the following methodology is adopted to investigate the cause of the problem. The construction and storm drainage operation of the system is evaluated by observing the existing construction and operation system of the storm drainage. To evaluate the design, first we have check the original design and we try to identify the success and the short coming of the design. By correcting the short coming, the system is re - designed and the result is compared with the existing system.

## 4.1. Hydrology and Hydraulics Evaluation

From the design report of the study area, we observe that rational method was used to estimate the peak flow of the study areas. The rational method is a reasonable formula for computing the peak discharge for small catchment. We need to evaluate the hydrology and hydraulics of the study areas because of the following reasons:

- While using the rational formula in Saris – Gotera road, the Kirpich formula was used in the original design but Kirpich formula was developed for rural areas and used for a well defined channel and steep slopes having a slope of (3% to 10 %). For evaluation we choose Federal aviation administration formula which is frequently used and recommended for urban basin(Chow, etal)

- In the design report of Saris – Gotera road, the area is calculated only by considering the road pavement and walk way but sheet flow up to 150 m can contribute to the peak flow. So we have to consider areas contributing to the inlet beyond the road surface.

- In the Ring road design report, the intensity of rain fall is calculated only by taking the observatory metrology station but Bole station should be considered.

- To check the efficiency of inlet used in the study area we have to re-design the system to get the parameters used in inlet efficiency calculation.

- In Gotera – Wollo sefer road, detail design report can't be obtained for inlet spacing computation.

**Inlet spacing**

Based on the concept in item 2.5.5.6, primary inlet are allocated along the alignment of the road and then using inlet spacing sheet in the appendix II inlets are spaced along the road. Spreads on the pavement and flow depth at the curb are often used as criteria for using the spread sheet. The basic parameters in the spread sheet are obtained as follows:

*Drainage area:* Using contour map, road alignment and cross section; the areas which contribute to a particular inlet is approximately calculated.

*Time of concentration*: time of concentration can easily be computed using federal aviation administration formula (1970) which is frequently used and recommended for over land flow in urban basin. (Chow, etal) See table A-4 in the appendix III.

$$\mathrm{Tc} = 1.8 * (1.1 - \mathrm{C}) * \mathrm{L}^{0.5} / \mathrm{S}^{0.33} \qquad (3.1)$$

Where:

C= Rational method runoff coefficient

L= Length of overland flow, ft

S= Surface slope, %

*Rainfall Intensity and flow (Q):* IDF curve which are found in AACRA design manual is used for determining rainfall intensity. The Curve was prepared using the data obtained from the two metrological stations (Addis Ababa Bole and Observatory). If the study area is nearer to one of the station, The IDF curve of that station is used for computation. Accordingly; For Addis Sefer

(Ring Road), Gotera – Wollo Sefer road and Saris - Gotera road we used the IDF curve of Bole station but for Total - Zenebework (Ring Road) we use observatory station.

The flow (Q), runoff coefficient(C), velocity(V), ratio of flow in depressed section to total gutter flow ($E_o$), intercepted flow($Q_i$), by pass flow($Q_b$) and efficiency(E) are calculate using equation 2.1, 2.2, 2.10, 2.15, 2.13, 2.12, 2.11(2.18 for curb inlets) respectively. But the ratio of side flow intercepted to total side flow (Rs) is calculated based on chart 1 in appendix III.

*Design frequency and spread*: The spread and design frequency of the roads selected based on table 2.2. Gotera – Wollo Sefer and Saris - Gotera roads are high volume roads having speed < 70 km/hr. So the design frequency selected for the design is 10 years and design spread becomes shoulder + 1m. Ring road is a type of road having high volume and a speed of >70 km/h, so the design frequency is 10 years and the design spread is taken as the shoulder width. For all roads at the sag point design frequency is taken as 50 years and the design spread is become shoulder + 1m.

The calculated spread according to equation 2.8 should be less than design spread written above. If the calculated spread exceeds the design spread, we have to decrease inlet spacing until we get safe design spread as well as the depth at the curb become less than the curb height.

**Pipe sizing**

The pipe sizing is done based on the computation sheet in the appendix II and like inlet spacing basic parameters are calculated as follows

*Drainage area*: The length of the contributing area can be taken the distance between the two consecutive inlets and the width can be taken from the top map of the area, so we can easily estimate the contributing area by multiplying the length with the width.

*Time of concentration*: Inlet time is obtained the same way as we used in inlet spacing spread sheet but to get the system time we have to add the time at which the storm pass through the conduit in the upstream to that of inlet time. During the initial time the system time and inlet time are equal. The time through the conduit can be calculated by dividing the conduit length with the velocity. The velocity for partial flow condition can be obtained by dividing the flow (Q) with Area of the partial flow (A). ( Debo and Reese, 2003)

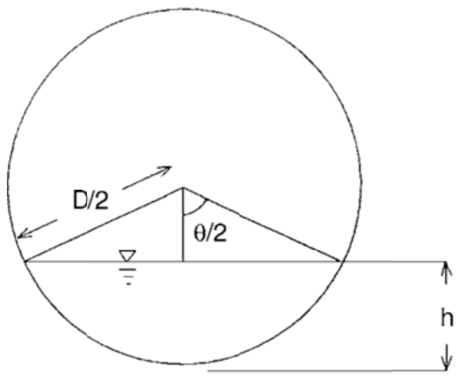



*Fig 4.1 partial flow in storm pipe*

$$K = Qn\, D^{-8/3}\, S^{-1/2} \tag{3.2}$$

$$A = D^2\, [(\theta - \sin\theta)/8]$$

$$\theta = 3\Pi/2\, \{1- [1- (\Pi K)^{1/2}]^{1/2}\}^{1/2}$$

$$h/D = 1/2\, [1 - \cos(\theta/2)]$$

Where $Q$ = the discharge ($m^3$/sec);

$K$ = a constant;

$h$ = the depth (m);

$D$ = the pipe diameter (m);

$n$ = Manning's roughness;

$S$ = the pipe slope (m/m); and

$\theta$ = the central angle.

*Pipe diameter:* The Diameter of the pipe is calculated using equation 2.21 but the value obtained from this equation may not be used as a design pipe diameter. Market availability and minimum pipe diameter for inspection and maintenance are the two reasons for not using the calculated value. The market available pipe diameters are 0.64 m, 0.75m, 0.8m, 0.9m, 1m and 1.2m. Therefore the diameter that we get from calculation should be transferred to the nearest market available pipe.

## 4.2. Storm Drainage Operation Management Evaluation

*Gotera – Wollo Sefer*: the drainage operation of Gotera – Wollo Sefer road is evaluated by direct observation of grate inlet condition, Disposal of solid and liquid waste which affected the storm drainage system, and the condition of curbs after construction. The grate inlet conditions in the area are categorized in to three. The first categories are inlets working without problem. The second categories are inlets whose covers are removed due to poor awareness and illegal people. Even if the second category of inlets operates properly, they are highly exposed to be filled with debris and silt. So, the researcher categories these inlets as inlets operate with problem. The last categories of inlets are inlets which are totally out of function due to blockage by debris and silt. The researcher tries to identify the three categories of inlet quantitatively.

*Saris- Gotera road:* The inlet for saris Gotera road are curb inlets and as it is known curb inlets are not easily blocked by debris and silt. But in this site it is observed that some of the inlets are blocked due to inadequate operation and we try to identify these inlets quantitatively.

*Ring Road*: the drainage operation of Ring road study area (Addis Sefer, Zenebework and Total) is evaluated by the same methodology as that of Gotera – Wollo Sefer Road.

## 4.3. Storm Drainage Construction Evaluation

Storm drainage construction of the study areas is evaluated by direct field data collection of Inlet spacing, inlet size and curbs heights. The data collected from the ground is compared with the design data in the drawing.

# 5. Result and Discussion

## 5.1. Gotera – Wollo Sefer Road

### 5.1.1. Storm Drainage Design Evaluation

**Inlet spacing**

From the data collection chapter, we can observe that 50x50 grate is used for collecting storm water from the gutter. For evaluating the inlet spacing and the efficiency of the inlet, we use inlet spacing computation sheet in FHWA drainage design manual. According to the methodology used, here is the computation and result of inlet spacing:

Initial inlet station = 0+010

$S_x$ (cross slope of the road) = 2%

$S_L$ (Longitudinal slope) = 0.24%

$Q_b$ (Previous by pass flow) = 0$m^3$/Sec, for initially

Length of sheet flow (L) = 120 (from contour map)

Run of coefficient (C) = 0.78 (from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 4.5% (from contour map)

Allowable spread = shoulder +1= 4.6m

Area (A) =$120 * 10 * 10^{-4}$ = 0.12ha

Time of concentration (equation 3.1)

$Tc = 1.8 * (1.1 - 0.78) * (3.28 * 120)^{0.5} / (4.5^{0.33})$ = 6.95min

From the IDF curve of Bole station I = 104mm/hr

Q (Total gutter flow, equation 2.2)

$Q = 0.78 * 104 * 0.12/360$ = 0.02704 $m^3$/Sec (equation 2.1)

T (Spread) = $= (0.0704 * 0.016/(0.376 * 0.02^{1.67} * 0.0024^{0.5}))^{0.375}$ = 2.8m < 4.6 m ok! (using equation 2.8)

d (depth at the gutter) $= 2.8 * 0.02 = 0.06$m < 0.17 m ok ! (using equation 2.9)

Velocity (V) = $0.752 * (0.0024^{0.5} * 0.02^{0.67} * \frac{2.8^{0.67}}{0.016})$ = 0.3m/sec (using equation 2.10)

Eo (ratio of frontal flow to that of total gutter flow) $= 1 - (1 - \frac{0.5}{2.8})$^2.67 = 0.40

$R_f$ (ratio of frontal intercepted to that of total frontal flow) = 1 (using chart 1)

Rs (ratio of side flow intercepted to total side flow) = 1/ (1+0.0828*0.3^1.8/(0.2*0.5^2.3) = 0.26 (using equation 2.16)

$Q_i$ (Intercepted flow) = $0.02704(1 * 0.40 + 0.26(1 + 0.40))$ = 0.02509 $m^3$ /sec (using equation 2.13)

$Q_b$ (by pass flow) = 0.02704 – 0.02509 = 0.00644 $m^3$/sec using equation (2.12)

E (efficiency) = (0.02509/0.02704)*100 = 76.15 %

This procedure will be repeated by assuming the next station of inlet and computing spread and depth of gutter. If the computed value is greater than the design spread (4.6m) and curb height (0.17m), we have to decrease the length of the next station and repeat the procedure until we get safe spread and depth of flow. The detail computation is shown in table A -12 in the appendix. Fig 5.1 also shows the result of inlet existing spacing and new inlet spacing obtained by evaluation.

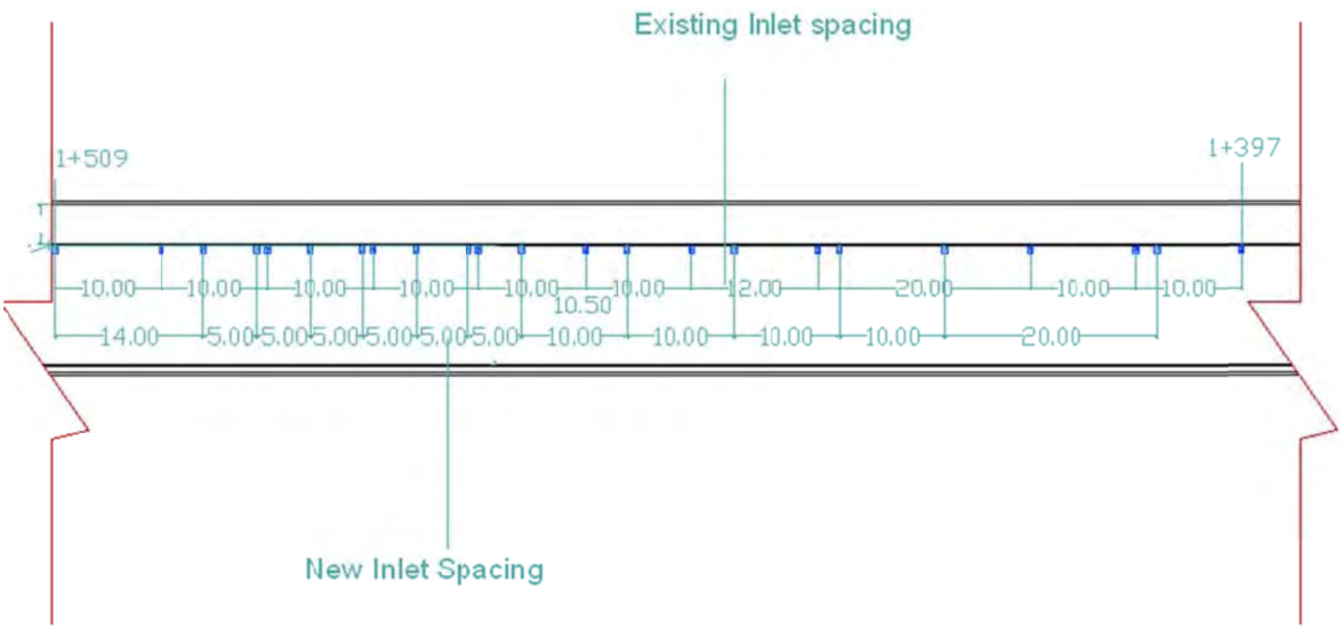



*Fig 5.1 Existing and new computed inlet spacing at Gotera – Wollo sefer road.*

The efficiency of the existing grate inlet is compared with that of curb inlet by computing the efficiency of the curb inlet without changing the other data. The computation is shown below;

$L_T$ (Curb opening required to intercept 100 % of the gutter flow)
$= 0.817 * 0.02704^{0.424} * 0.0024^{0.3} *$ (1/0.016*0.2)^0.6 = 4 m (using equation 2.17)

E (efficiency of curb inlet) = $1 - \left(1 - \frac{1}{4}\right)^{1.8}$ = 0.4354 =43.54% < 76.15%

$Q_i$ (intercepted flow) = E*Q = 0.4354*0.02704 = 0.01178m$^3$/sec

From this result we can see the efficiency of curb inlet is less than the existing efficiency of grate inlet. The summarized result of the comparison is shown in table A-3 in the appendix.

**Pipe sizing**

FHWA design manual is used as a guide to evaluate the pipe sizing of the storm drainage system. The following procedure shows the result of the computation:

Station 0+00 – 0+60 (pacing between two manholes from the existing design drawing)

Length $= 60 - 0 = 60m$

Length of sheet flow (L) = 120 (from contour map)

Run of coefficient (C) = 0.78 (from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 4.5% (from contour map)

Area (A) =$120 * 60 * 10^{-4}$ = 0.72ha

Time of concentration (equation 3.1)

Inlet time $= 1.8 * (1.1 - 0.78) * (3.28 * 120)^{0.5}/(4.5^{0.33})$ = 6.95min

From the IDF curve of Bole station I = 104mm/hr

$Q = 0.78 * 104 * 0.72/360$ = 0.1622 m$^3$/Sec (equation 2.1)

Slope of pipe line = 0.01 (initially assumed as the longitudinal slope of the road)

Pipe diameter (D) $= (0.1622 * 0.016/(0.312 * 0.01^{0.5}))$^0.375 = 0.364m ≈ 0.64m (minimum market available pipe size for storm drain and using equation 2.21)

Velocity (V) $= \frac{0.312}{0.016} * 0.64^{0.6} * 0.01^{0.5}$ = 2.265 m/sec (full velocity calculated using equation2.2)

Design velocity = 2.027 (using equation 3.2)

For the next station

0+60 – 0+105

Length $= 105 - 60 = 45m$

Length of sheet flow (L) = 135 (from contour map)

Run of coefficient (C) = 0.81(from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 4.0% (from contour map)

Area (A) $=135 * 45 * 10^{-4}$ = 0.6075 ha

Total area ($A_T$) = 0.6075 + 0.72 = 1.328 ha

Time of concentration (equation 3.1)

Inlet time $= 1.8 * (1.1 - 0.78) * (3.28 * 135)^{0.5}/(4.0^{0.33})$ = 6.95min

Section time $= (60/2.26)/60$ = 0.493 min (from previous station velocity and length)

System time = 0.493 + 6.95 = 7.44 min

From the IDF curve of Bole station I = 100mm/hr

Q $= 0.81 * 104 * 1.32/360$ = 0.2927 $m^3$/Sec (equation 2.1)

Slope of pipe line = 0.01

Pipe diameter (D) $= (0.297 * 0.016/(0.312 * 0.01^{0.5}))$^$0.375$ = 0.45m ≈ 0.64m (minimum market available pipe size for storm drain and using equation 2.21)

The detail computation sheet of pipe sizing is shown in table A – 11 in the appendix and also, the summarized existing and new pipe size is shown in table A -1 in the appendix.

### 5.1.2. Storm Drainage Operation Evaluation

From the data analysis it is observed that the operational management of Gotera – wollo sefer road is insufficient. Due to this 56 % inlets are out of function and 23 % curbs are damaged. Fig 5.2, 5.2 clearly shows the condition of inlets and curbs in the area.

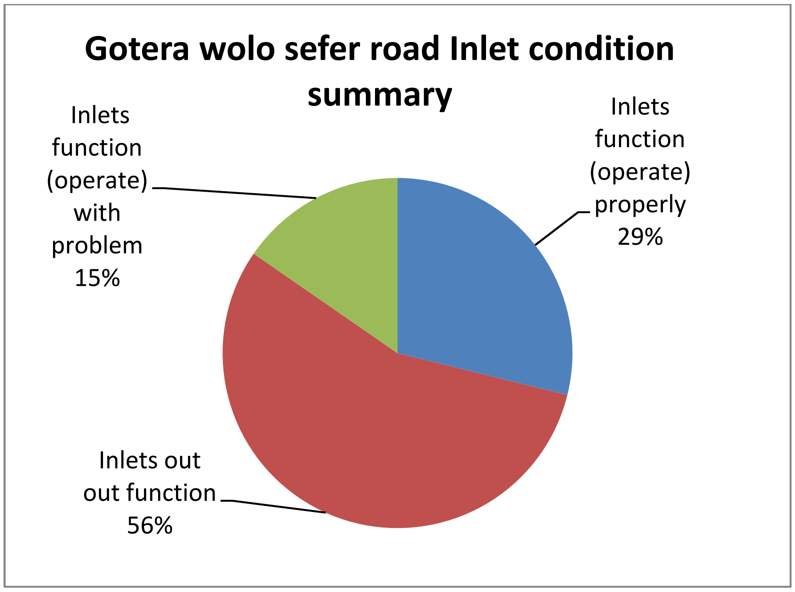



*Fig 5.2 a) Grate Inlet condition in Gotera – Wollo sefer road*

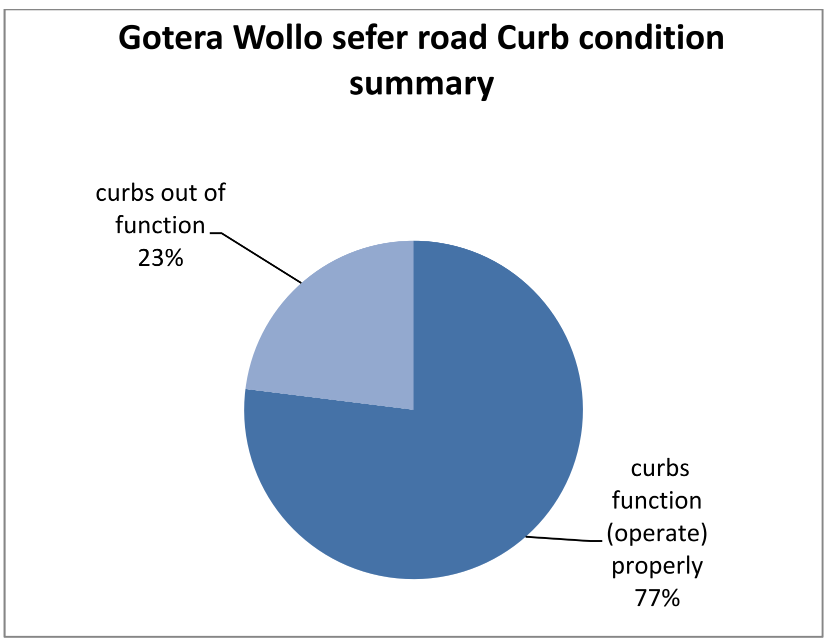



*Fig 5.2 b) Curb conditions in Gotera – Wollo sefer road*

*Fig 5.2) Curb and inlet conditions in Gotera – Wollo sefer road due to in adequate drainage management.*

#### 5.1.3. Storm Drainage Construction Evaluation

The design data in the study area shows grate inlet having a dimension of 50* 50 and a curb having a height of 0.17 m is used for the drainage system. Data collected on the site shows curbs in the road section are constructed in the right position and design dimension and also grate inlets are constructed with correct dimension.

From the above three evaluation it was observed that, the existing pipe size is relatively enough to carry the storm discharge. The existing inlet spacing is not enough to get a spread of less than the design especially at the sag point (near to Ethiopian Telecommunication Collage).

From the data analysis; inadequate drainage operation management is because of unawareness of community who dispose solid and liquid wastes in to the inlets and on the pavement of the road, removal of inlet cover and accumulation of silt for long period of time without inspection. In addition to this, due to the existence of construction sites in the vicinity of the road and construction of ramp for vehicles 23% of the curbs are damaged.

### 5.2. Saris Gotera Road

#### 5.2.1. Storm Drainage Design Evaluation

**Inlet Spacing**

From the data collection chapter, we can observe that curb inlets are used for collecting storm water from the gutter. For evaluating the inlet spacing and the efficiency of the inlet, we use inlet spacing computation sheet in FHWA drainage design manual. According to the methodology used, here is the computation and result of inlet spacing:

Initial inlet station = 12+950

Upper contributing station = 12+969

$S_x$ (cross slope of the road) = 2%

$S_L$ (Longitudinal slope) = 2.3%

$Q_b$ (Previous by pass flow) = 0m$^3$/Sec, for initially

Length of sheet flow (L) = 95 (from contour map)

Run of coefficient (C) = 0.70 (from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 3% (from contour map)

Allowable spread = shoulder +1= 4m

Area (A) = $95 * (12969 - 12950) * 10 * 10^{-4}$ = 0. 2ha

Time of concentration (equation 3.1)

$Tc = 1.8 * (1.1 - 0.70) * (3.28 * 95)^{0.5}/(3^{0.33})$ = 8.82 min

From the IDF curve of Bole station I = 100mm/hr

Q (Total gutter flow, equation 2.2)

$Q = 0.70 * 100 * 0.2/360$ = 0.0351 $m^3$/Sec (equation 2.1)

T (Spread) = $= (0.0351 * 0.016/(0.376 * 0.02^{1.67} * 0.023^{0.5}))^{0.375}$ = 2.07m < 4.0 m ok! (using equation 2.8)

d (depth at the gutter) $= 2.07 * 0.02 = 0.04$m < 0.17 m ok ! (using equation 2.9)

Cur opening length required to intercept 100% of gutter flow ($L_T$) $= 0.817 * 0.0351^{0.42} * 0.023^{0.38} * \left(\frac{1}{0.016*0.02}\right)^{0.6} = 8\ m$ (using equation 2.17)

Efficiency of curb inlet (E) $= 1 - \left(1 - \frac{1}{8}\right)$ ^1.8= 21.25% (using equation 2.18)

$Q_i$ (Intercepted flow) = 21.25%* 0.0351 = 0.00746 $m^3$ /sec (using equation 2.13)

$Q_b$ (by pass flow) = 0.0351 – 0.00746 = 0.0276 $m^3$/sec (using equation 2.12)

This procedure will be repeated by assuming the next station of inlet and computing spread and depth of gutter. If the computed value is greater than the design spread (4.0 m) and curb height (0.17m), we have to decrease the length of the next station and repeat the procedure until we get safe a spread and depth of flow. The detail computation is shown in table A -14 in the appendix. Figure 5.2 also shows the result of existing inlet spacing and new inlet spacing obtained by evaluation.

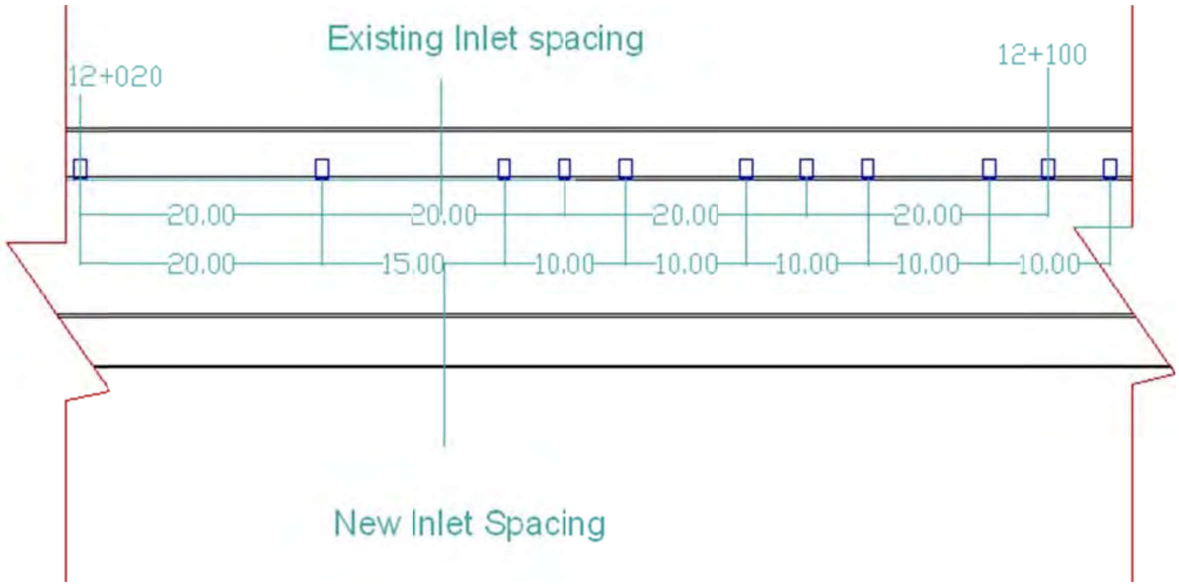



*Fig 5.3 Existing and new computed inlet spacing at Saris – Gotera road.*

In the literature review, we have seen that the efficiency of curb inlet can be increased by providing depression. The computation below shows how the efficiency of curb inlet is increased by providing depression.

Initial inlet station = 12+950

Upper contributing station = 12+969

$S_x$ (cross slope of the road) = 2%

$S_L$ (longitudinal slope) = 2.3%

Frontal width (W) = 1m

a (gutter depression ) = 30 mm

$Q_b$ (previous by pass flow) = 0m$^3$/Sec, for initially

Length of sheet flow (L) = 95 (from contour map)

Run of coefficient (C) = 0.70 (from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 3% (from contour map)

Allowable spread = shoulder +1= 4m

Area (A) = $95 * (12969 - 12950) * 10 * 10^{-4}$ = 0. 2ha

Time of concentration (equation 3.1)

$Tc = 1.8 * (1.1 - 0.70) * (3.28 * 95)^{0.5} / (3^{0.33})$ = 8.82 min

From the IDF curve of Bole station I = 100mm/hr

Q (Total gutter flow, equation 2.2)

$\mathrm{Q} = 0.70 * 100 * 0.2/360$ = 0.0351 $m^3$/Sec (equation 2.1)

T (Spread) =$(0.0351 * 0.016/(0.376 * 0.02^{1.67} * 0.023^{0.5}))^{0.375}$ = 2.07m < 4.0 m ok! (using equation 2.8)

d (depth at the gutter) $= 2.07 * 0.02 = 0.04$m < 0.17 m ok ! (using equation 2.9)

$E_o$ (ratio of flow in the depressed section to total flow) = $1 - \left(1 - \frac{1}{2.07}\right)^{2.67} = 0.86$ (using equation 2.15)

$S_e$ (equivalent cross slope) = $0.2 + \frac{30}{1000} * 0.86 = 0.05$

Cur opening length required to intercept 100% of gutter flow ($L_T$) $= 0.817 * 0.0351^{0.42} * 0.023^{0.38} * \left(\frac{1}{0.016*0.05}\right)^{0.6} = 5\ m$ (using equation 2.17)

Efficiency of curb inlet (E) $= 1 - \left(1 - \frac{1}{5}\right)$ ^1.8 = 33.69% (using equation 2.18)

$Q_i$ (Intercepted flow) = 33.69%* 0.0351 = 0.0118 $m^3$ /sec (using equation 2.13)

$Q_b$ (by pass flow) = 0.0351 – 0.0118 = 0.0232 $m^3$/sec using equation (2.12)

33.69% (efficiency of curb inlet with depression) > 21.25% (efficiency of curb inlet with depression)

The detail computation of curb inlet with depression is shown in table A – 16 in the appendix.

**Pipe sizing**

FHWA design manual is used as a guide to evaluate the pipe sizing of the storm drainage system. The following procedure shows the result of the computation:

Station 12 +950 – 12+970 (pacing between two manholes from the existing design drawing)

Length $= 12970 - 12950 = 20m$

Length of sheet flow (L) = 95 (from contour map)

Run of coefficient (C) = 0.70 (from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 3% (from contour map)

Area (A) $=95 * 20 * 10^{-4}$ = 0.19ha

Time of concentration (equation 3.1)

Inlet time $= 1.8 * (1.1 - 0.70) * (3.28 * 95)^{0.5}/(3^{0.33})$ = 8.84min

From the IDF curve of Bole station I = 100mm/hr

$Q = 0.70 * 100 * 0.19/360$ = 0.04 $m^3$/Sec (equation 2.1)

Slope of pipe line = 0.02

Pipe diameter (D) $= (0.04 * 0.016/(0.312 * 0.02^{0.5}))$^$0.375$ = 0.18m ≈ 0.64m (using equation2.21)

For the next station

Velocity (V) = $\frac{0.312}{0.016} * 0.64^{0.6} * 0.02^{0.5}$ = 3.24 m/sec (full velocity calculated using equation 2.2)

Design velocity = 1.57 m/sec (using equation 3.2)

12+930- 12+950

Length $= 12950 - 12930 = 20m$

Length of sheet flow (L) = 95 (from contour map)

Run of coefficient (C) = 0.70 (from land use map and equation 2.2)

Slope of sheet flow (S) = 3.0% (from contour map)

Area (A) $=95 * 20 * 10^{-4}$ = 0.195ha

Total area ($A_T$) = 0.19 + 0.19 = 0.38 ha

Time of concentration (equation 3.1)

Inlet time $= 1.8 * (1.1 - 0.70) * (3.28 * 95)^{0.5}/(3^{0.33})$ = 8.84min

Section time $= (20/1.57)/60$ = 0.21 min (from previous station velocity and length)

System time = 8.847 + 0.21= 9.06 min

From the IDF curve of Bole station I = 100 mm/hr

$Q = 0.7 * 100 * 0.38/360$ = 0.07 $m^3$/Sec (equation 2.1)

Slope of pipe line = 0.02

Pipe diameter (D) $= (0.07 * 0.016/(0.312 * 0.02^{0.5}))$^$0.375$ = 0.24m ≈ 0.64m (minimum market available pipe size for storm drain and using equation 2.21)

The detail computation is shown in appendix A -17 and the summary existing pipe size and new computed pipe size is shown in appendix A – 4.

### 5.2.2. Storm Drainage Operation Evaluation

The data collected shows that, the inlet in Gotera – wollo sefer road is not properly working due to inadequate drainage operational management. The silt and garbage at the entrance of the inlet blocked the entrance of runoff to the pipe drain system. From data analysis we can see 22.4 % of inlets are out of function due to inadequate drainage operational management.

### 5.2.3. Storm Drainage Construction Evaluation

The data analysis of the study area shows, the curbs constructed in the area have a height of less than the designed curb height which is 17 cm. The under design curb height has impact on small curb opening height. Even in some chainage the pavement and the inlet cover (Manhole cover) overlap each other, so the storm water unable to enter to the inlet properly. The Pi - chart in fig 5.4 shows that the curb inlets in the area are out of function or operating with problem due to inadequate operational management as well as construction problem.

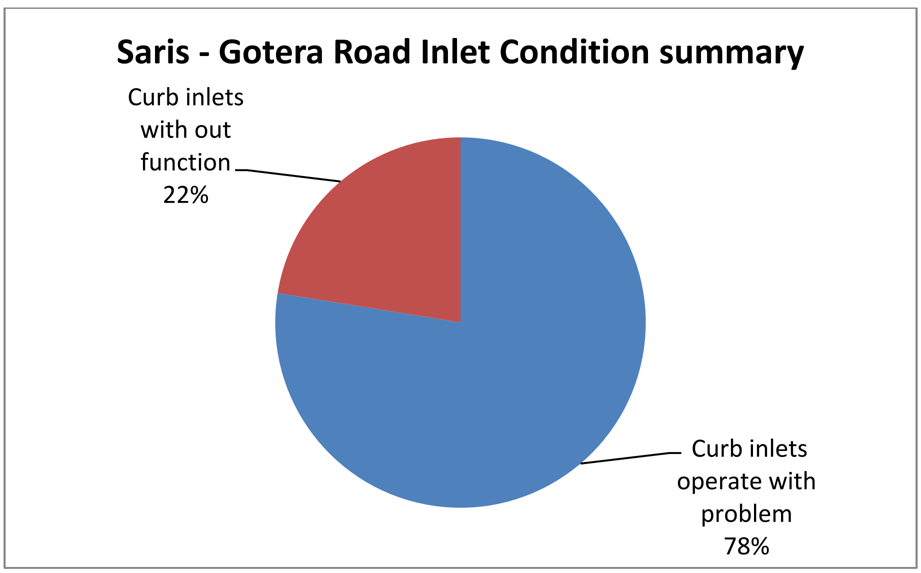



*Fig 5.4 a) Condition of curbs in saris – Gotera road*

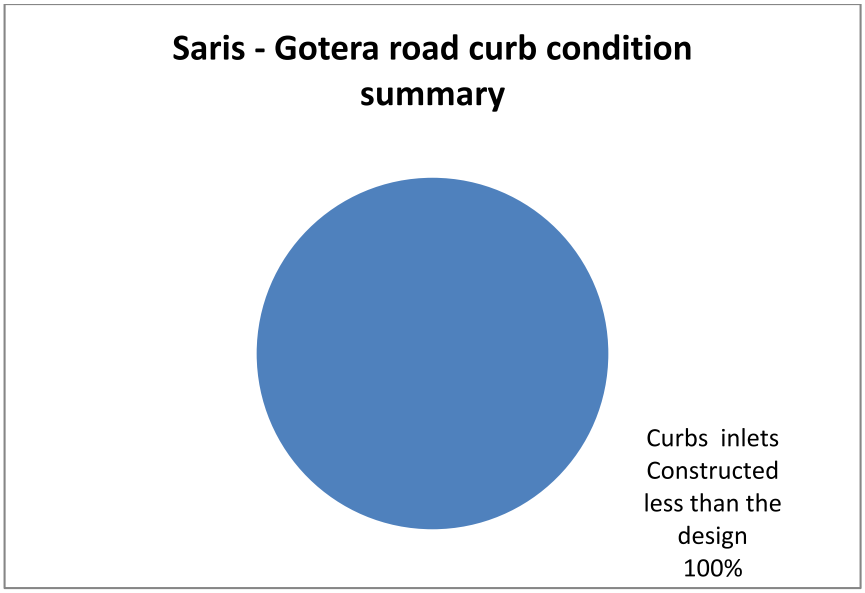



*Fig5. 4b) Curb condition in saris – Gotera Road*

*Fig 5.4) Curb and inlet condition in Saris – Gotera road.*

From the design analysis and field observation the problem in saris Gotera road is caused by the combination of design problem (Pipe sizing and inlet spacing), Construction problem and inadequate drainage management.

The pipe size implemented in this particular road is very small and the existing curb inlets are spaced uniformly in 20m which is different from that of the computed inlet spacing in the appendix I. So the existing spacing leads to spread of storm water greater than the deign spread which cause pavement drainage problem. In addition to this, small inlet opening and curb height with inadequate operational management also aggravate the problem in the area.

## 5.3. Ring Road

### 5.3.1. Storm Drainage Design Evaluation

**Inlet Spacing**

In ring road study areas 50x50 grate is used for collecting storm water from the gutter. For evaluating the inlet spacing and the efficiency of the inlet, we use inlet spacing computation sheet in FHWA drainage design manual. The design criterion here is the spread of water on the road pavement. Since study area in ring road is bounded with concrete blocks, runoff which comes out of the pavement is not contributing to the gutter. According to table 2.2 the design spread is only the shoulder of the road (i.e. 3m). According to the methodology used, here is the computation and result of inlet spacing at Total - Zenebework:

Initial inlet station = 16+400

Sx (cross slope of the road) = 2%

$S_L$ (Longitudinal slope) = 0.6%

$Q_b$ (Previous by pass flow) = 0 $m^3$/Sec, for initially

Length of sheet flow (L) = 10 (since only pavement runoff contributing the surface)

Run of coefficient (C) = 0.90 (from land use map and table A-20)

Allowable spread = shoulder = 3m

Area (A) =$(16400-16300)*10*10^{-4}$ = 0.1ha

Velocity of concentrated flow $= 1*0.619*0.6^{0.5}$= 0.4794 m/sec (using equation 2.4)

Time of concentration (equation 2.6)

$Tc = (\frac{16400-16300}{0.494}/60)$ = 3.57mi $\approx 5min$ (using equation 2.3)

From the IDF curve of Observatory station I = 150 mm/hr

Q (Total gutter flow, equation 2.2)

$Q = 0.9*150*0.1/360$ = 0.0375 $m^3$/Sec (equation 2.1)

T (Spread) = $= (0.0375*0.016/(0.376*0.02^{1.67}*0.006^{0.5}))^{0.375}$ = 2.75m < 3 m ok! (using equation 2.8)

Velocity (V) = $0.752*(0.006^{0.5}*0.02^{0.67}*\frac{2.8^{0.67}}{0.016})$ = 0.5m/sec (using equation 2.10)

Eo (ratio of frontal flow to that of total gutter flow) $= 1-(1-\frac{0.5}{2.75})$^2.67 = 0.41

$R_f$ (ratio of frontal intercepted to that of total frontal flow) = 1 (using chart 1)

$R_s$ (ratio of side flow intercepted to total side flow) = 1/ (1+ (0.0828*0.5^1.8)/ (0.2*0.5^2.3) = 0.14 (using equation 2.16)

$Q_i$ (Intercepted flow) = $0.0375(1 * 0.41 + 0.14(1 + 0.41))$ = 0.02315 $m^3$ /sec (using equation 2.13)

$Q_b$ (by pass flow) = 0.0375 – 0.02315 = 0.0143 $m^3$/sec using equation (2.12)

E (efficiency) = (0.02315/0.0375)*100 = 61.72 %

This procedure will be repeated by assuming the next station of inlet and computing spread and depth of gutter. If the computed value is greater than the design spread (3m), we have to decrease the length of the next station and repeat the procedure until we get safe spread. The detail computation is shown in table A -19 in the appendix. The summarized result of existing inlet spacing and computed new inlet spacing is shown in table A- 9 and A-10. Fig 5.5 also shows the result of inlet existing spacing and new inlet spacing obtained by evaluation.

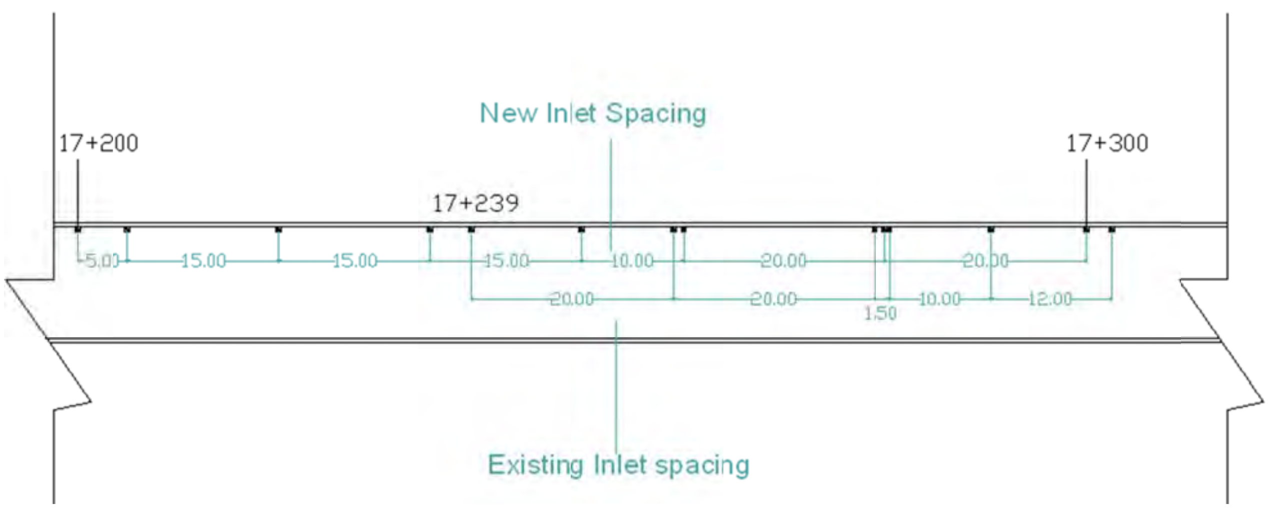



*Fig 5.5 Existing and new computed inlet spacing at Ring road (Total – Zenebework).*

**Pipe sizing**

FHWA design manual is used as a guide to evaluate the pipe sizing of the storm drainage system. The following procedure shows the result of the computation:

Station 16+300 – 16+330 (pacing between two manholes from the existing design drawing)

Length $= 16330 - 16300 = 30m$

Length of sheet flow (L) = 10 (from road map)

Run of coefficient (C) = 0.90 (from land use map, equation 2.2 and table A-20)

Slope of sheet flow (S) = 2% (from contour map)

Area (A) =$30 * 10 * 10^{-4}$ = 0.03ha

Time of concentration (equation 3.1)

Inlet time $= 1.8 * (1.1 - 0.90) * (3.28 * 10)^{0.5}/(2^{0.33})$ = 1.64min$\approx 5min$

From the IDF curve of Observatory station I = 150mm/hr

$Q = 0.90 * 150 * 0.03/360$ = 0.0133 $m^3$/Sec (equation 2.1)

Slope of pipe line = 0.032

Pipe diameter (D) = $(0.0133 * 0.016/(0.312 * 0.032))$^$0.375$ = 0.107m $\approx$ 0.64m (minimum market available pipe size for storm drain and using equation 2.21)

Velocity (V) $= \frac{0.312}{0.016} * 0.64^{0.6} * 0.032^{0.5}$ = 4.05 m/sec (full velocity calculated using equation 2.2)

Design velocity = 1.78 m/sec (using equation 3.2)

For the next station

16+330 - 16+400

Length $= 16400 - 16330 = 70m$

Length of sheet flow (L) = 10 (from road map)

Run of coefficient (C) = 0.90 (from land use map and equation 2.2)

Slope of sheet flow (S) = 2.0% (from contour map)

Area (A) =$70 * 10 * 10^{-4}$ = 0.07ha

Total area ($A_T$) = 0.07 + 0.03 = 0.10 ha

Time of concentration (equation 3.1)

Inlet time $= 1.8 * (1.1 - 0.90) * (3.28 * 10)^{0.5}/(2^{0.33})$ = 1.64min

Section time $= (30/1.78)/60$ = 0.28 min (from previous station velocity and length)

System time = 1.64 + 0.28= 1.92min ≈ 5 min

From the IDF curve of Observatory station I = 150 mm/hr

$Q = 0.90 * 150 * 0.10/360$ = 0.0375 $m^3$/Sec (equation 2.1)

Slope of pipe line = 0.039

Pipe diameter (D) $= (0.0375 * 0.016/(0.312 * 0.02^{0.5}))$^0.375 = 0.162m ≈ 0.64m (minimum market available pipe size for storm drain and using equation 2.21)

The detail computation is shown in appendix II table A -18 and the summary existing pipe size and new computed pipe size is shown in appendix I table A – 7 and A-8.

### 5.3.2. Storm Drainage Operation Evaluation

From the data analysis it is observed that the operational management of Ring road is insufficient. Due to this, 50 % inlets in Total – Zenebework and 32 % inlets in Addis sefer – Abo are out of function. Fig 5.6and 5.7 shows the summarized result of inlet condition in the study area of ring road.

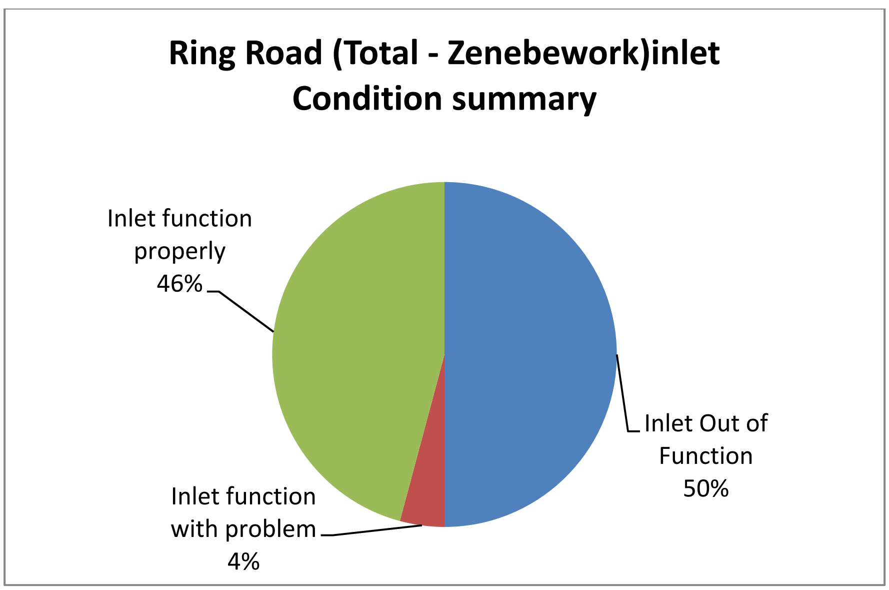



*Fig 5.5 Inlet condition at Ring Road (Total – Zenebework)*

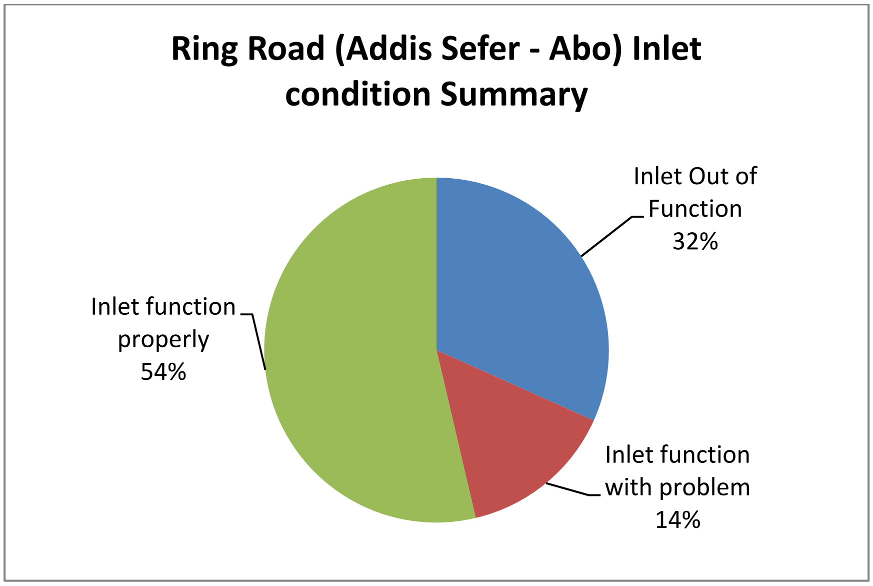



*Fig 5.6 Inlet condition at Ring Road (Addis sefer - Abo)*

### 5.3.3. Storm Drainage Construction Evaluation

The design data in the study area shows grate inlet having a dimension of 50* 50 is used to collect water from the gutter. Data collected on the site shows the gate inlets are constructed with the dimension and spacing shown in the design drawing.

The Result of the investigation shows that the main problem in the area can be categories as follows:

- The inlet spacing in the Ring road is properly designed except under designed spacing at the sag point.
- At super elevated area of the ring road the drain from the upper part of the road is drained to the other side of the road by providing opening through concrete median as shown in the figure below (Fig 3.6 a). Even though it is a good design approach, the openings are out of use due the small size (Diameter 10-15cm) of opening which makes the system easily blocked by silt and garbage. This restricts the flow of storm water on one side of

the road. Therefore the cumulative effect of this becomes one of the causes of flooding at the sag point of both Total - Zenebework and Addis - Sefer Abo roads.

To overcome inadequate drainage operation system in Gotera Wollo – Sefer, Saris - Gotera and ring road the following proposal is recommended (see table 5.1 and 5.2).

Before implementing the proposal, awareness training should be given to the community in “kebele” or “wereda” level. After training and awareness is given to the community, a maintenance activity should be done as a first stage to the drainage system. This stage includes installing the grate inlet cover that are removed illegally, anchoring the grate inlet cover to the pavement tightly with bolt or any other method, removing the silt and garbage which filled the inlets and providing sufficient inlet opening for curb inlets.

After a month of the first stage is completed, the second proposed stage will begin. In this stage we expect the community to be aware about the system, so the maintenance activity can be done with less crew to that of the first stage. If the result is not satisfactory, we have to continue the awareness and training activity parallel with the maintenance work.

In the last stage a continuous follow up will be done depending on the season. In dry season the inlets are less susceptible to be blocked by silt and garbage, so the follow up activity can be done once in a month. But during rainy seasons the inlets are highly blocked by silt and garbage so a follow up and action should be taken once in a week.

Using this proposal of drainage management system we can minimize the storm drainage problem caused by in sufficient drainage management.

*Table 5.1 Propose drainage management system during dry season*

| S.no | Description | Team | Number | Period | Remark |
|---|---|---|---|---|---|
| **Primary stage (Maintenance)** | | | | | |
| 1 | **Grate Inlet** | | | | |
| | For maintaining one k.m drainage system ( In average 30 - 40 inlets exist) | Inspector | 1 | Two days | |
| | | Welder | 2 | | |
| | | Helper | 4 | | |
| | | Daily laborers | 6 | | |
| 2 | **Curb Inlets** | Inspector | 1 | Two days | |
| | For maintaining one k.m drainage system ( In average 30 - 40 inlets exist) | Mason | 2 | | |
| | | Helper | 4 | | |
| | | Daily laborers | 4 | | |
| **Secondary stage (Evaluation)** | | | | | |
| 1 | **Grate Inlet** | | | | |
| | For Evaluation one k.m drainage system ( In average 30 - 40 inlets exist) | Inspector | 1 | One day | once in a month |
| | | Welder | 2 | | |
| | | Helper | 4 | | |
| | | Daily laborers | 6 | | |
| 2 | **Curb Inlets** | Inspector | 1 | One day | Once in a month |
| | For managing one k.m drainage system ( In average 30 - 40 inlets exist) | Mason | 2 | | |
| | | Helper | 4 | | |
| | | Daily laborers | 4 | | |
| | **Third stage (follow up)** | | | | |
| 1 | **Grate Inlet** | | | | |
| | For managing one k.m drainage system ( In average 30 - 40 inlets exist) | Inspector | 1 | One day | Once in a month |
| | | Daily laborers | 6 | | |
| 2 | **Curb Inlets** | Inspector | 1 | One day | Once in a month |
| | For managing one k.m drainage system ( In average 30 - 40 inlets exist) | Daily laborers | 4 | | |

*Table 5.2 Propose drainage management system during rainy season*

| S.no | Description | Team | Number | Period | Remark |
|---|---|---|---|---|---|
| | **Third stage (follow up)** | | | | |
| 1 | **Grate Inlet** | | | | |
| | For managing one k.m drainage system ( In average 30 - 40 inlets exist) | Inspector<br>Daily laborers | 1<br>6 | One day | Once in a week |
| 2 | **Curb Inlets** | Inspector | 1 | | |
| | For managing one k.m drainage system ( In average 30 - 40 inlets exist) | Daily laborers | 4 | One day | Once in a week |

*Table 5.3 proposal schedule for drainage management*

| s.n | Description | Month | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Se | Oc | No | De | Ja | Fe | Ma | Ap | Ma | Ju | Jl | Au |
| 1 | Training and awareness | | | | | | | | | | | | |
| 2 | First stage (Maintenance activity with the proposed crew) | | | | | | | | | | | | |
| 3 | Second stage (Evaluation) | | | | | | | | | | | | |
| 4 | Third Stage (Follow up) | | | | | | | | | | | | |

N.B:

- The follow up stage can be done immediately after a month of the maintenance stage started and continue throughout depending on the season.
- Dry Season in Addis
  - ✓ From October – February + may
- Rainy season in Addis
  - ✓ June – September + March and April

***Drainage profile***

The pipe invert elevations profile which are computed in pipe size computation sheets is shown in figure 5.7, 5.8 and 5.9. The figures are sample profile of the problem area, and the invert elevation of the whole section of the road surface can easily referred from the pipe size computation sheet table A-11, A-17 and A-18 in the appendix.

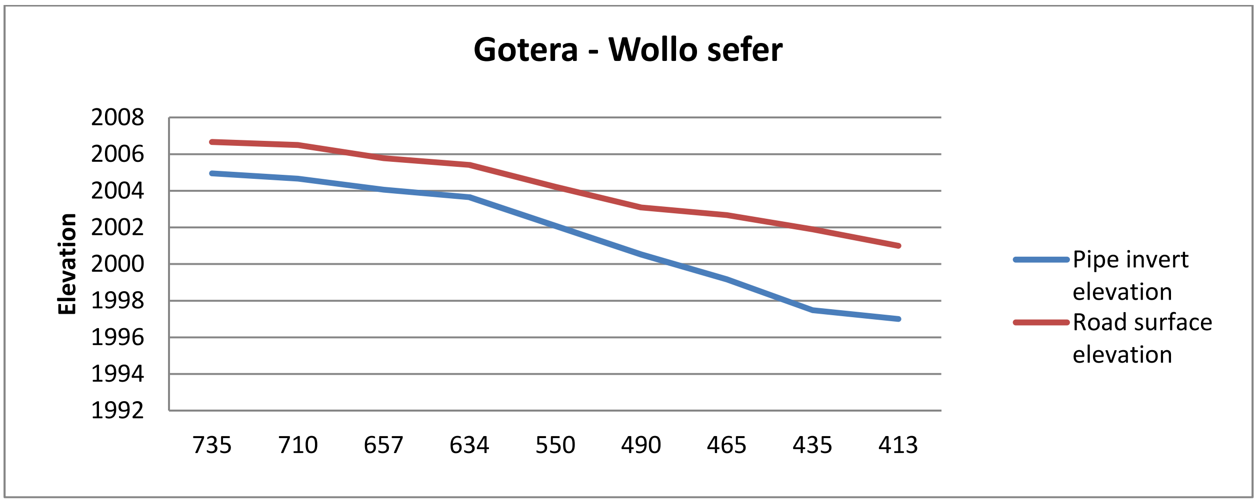



*Fig 5.7 Drainage profile of Gotera - Wollo sefer road (from station 0+318 -0+735)*

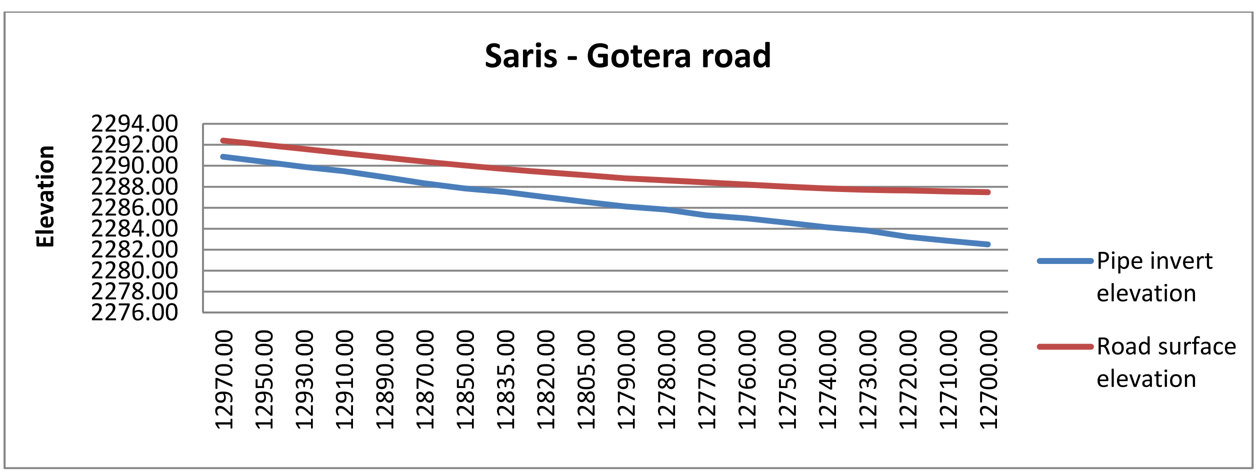



*Fig 5.8 Drainage profile of Saris- Gotera road (from station 12+700 -12+870)*

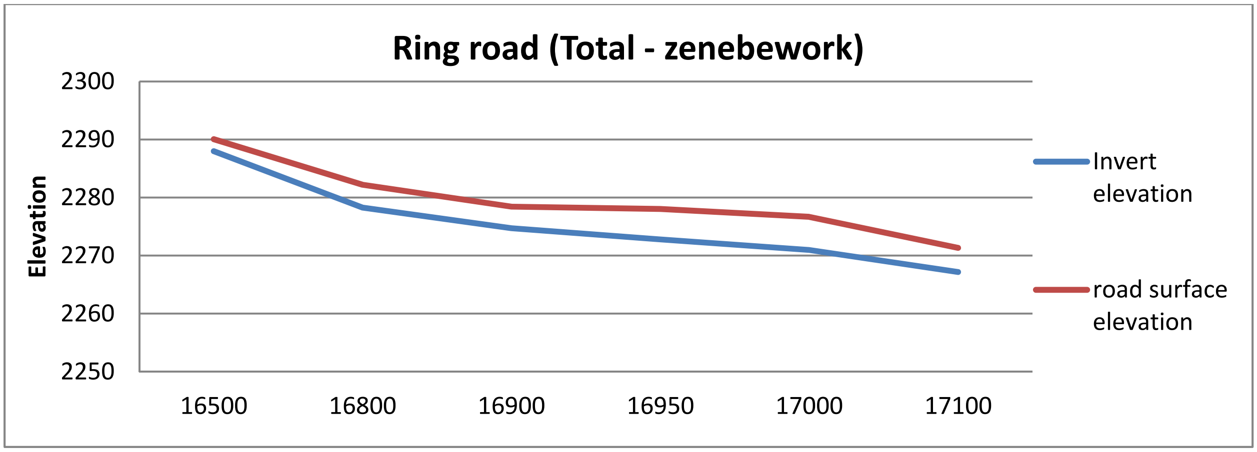



*Fig 5.8 Drainage profile of Ring road (from station 16+500 – 17+100)*

# 6. Conclusion and recommendation

In the study area it is observed that drainage problem is a cause of flooding on pavement, congested traffic flow and difficulty on day to day activity of people. To investigate the cause of the problem, we try to evaluate the design by redesigning the system in the problem areas and site investigation is done by collecting direct field data to assess the storm drainage construction and operation management problem.

From the result section we can easily observe that the problem in the study area caused by design problem, construction problem and in adequate drainage operation management. Based on the result obtained, the following conclusion and recommendation are listed:

- Large inlet spacing is a cause over spread of storm water in the pavement of the road. This can be minimized by providing extra inlets especially on the sag point.
- The minimum curb inlet opening due to construction problem is also a cause storm drainage problem in Saris - Gotera road. This can easily be corrected by re constructing the manhole cover to providing sufficient inlet opening (100 mm – 150 mm).
- Inadequate drainage operational management is also a cause of inlet not to work properly which aggravate the problem in the study area, so scheduled follow up and maintenance activity should be done by the concerned organization.
- The efficiency of curb inlets can also be improved by providing inlet depressions in the road cross section so new roads which are designed in the future should have inlet depression.
- From the analysis we can see the efficiency of grate inlets is better than that of curb inlet but on the other hand grate inlets are easily susceptible by garbage. Therefore if we design gate inlet with good management system, the drainage system of a road will work with good efficiency and less problem.
- The liquid waste which joins the storm drainage system of Gotera – Wollo Sefer road (at station 1+829) is a basic problem in Gotera Wollo Sefer road especially during rainy season. So a separate system should be provided to avoid the problem.

# 7. References

AACRA, 2003: Drainage design manual

American Concrete Pipe association, 2000: Concrete pipe design manual

David Butler and John Davies, 2004: Urban drainage

Engineer Zewdie Eskinder & PLC, 2004: Akaki Bridge – Meskel square Road up grading project

ERA, 2002 Drainage design manual

FHWA (U.S Department of Transportation Federal highway admission), 2001: Urban drainage design manual

Larry W. Mays, 2004: Storm water collection system design hand book

Parkman Limited, 1997: Addis Ababa Ring road project design report

TCDSc, 2000: Gotera Wollo Sefer design report

Thomas N. Debo. Andrew J. Reese, 2003: Municipal storm water management

Ven Te Chow, etal, Applied Hydrology

# Appendix I

(Design Computation summary)

From page 78 – 90 See attached excel

Table A1

Table A2

Table A3

Table A4

Table A5

Table A6

Table A7

Table A8

Table A9

Table A10

# Appendix II

(Detail Design Computation Sheet)

From page 92 – 129 See attached excel

Table A11

Table A12

Table A13

Table A14

Table A15

Table A16

Table A17

Table A18

Table A19

# Appendix III

(Table and Chart)

Table A-20 Runoff Coefficients for Rational Formula

| Type of Drainage Area | Runoff Coefficient, C* |
|---|---|
| Business: | |
| Downtown areas | 0.70 - 0.95 |
| Neighborhood areas | 0.50 - 0.70 |
| | |
| Residential: | |
| Single-family areas | 0.30 - 0.50 |
| Multi-units, detached | 0.40 - 0.60 |
| Multi-units, attached | 0.60 - 0.75 |
| Suburban | 0.25 - 0.40 |
| Apartment dwelling areas | 0.50 - 0.70 |
| | |
| Industrial: | |
| Light areas | 0.50 - 0.80 |
| Heavy areas | 0.60 - 0.90 |
| | |
| Parks, cemeteries | 0.10 - 0.25 |
| Playgrounds | 0.20 - 0.40 |
| Railroad yard areas | 0.20 - 0.40 |
| Unimproved areas | 0.10 - 0.30 |
| | |
| Lawns: | |
| Sandy soil, flat, 2% | 0.05 - 0.10 |
| Sandy soil, average, 2 - 7% | 0.10 - 0.15 |
| Sandy soil, steep, 7% | 0.15 - 0.20 |
| Heavy soil, flat, 2% | 0.13 - 0.17 |
| Heavy soil, average, 2 - 7% | 0.18 - 0.22 |
| Heavy soil, steep, 7% | 0.25 - 0.35 |
| | |
| Streets: | |
| Asphaltic | 0.70 - 0.95 |
| Concrete | 0.80 - 0.95 |
| Brick | 0.70 - 0.85 |
| | |
| Drives and walks | 0.75 - 0.85 |
| | |
| Roofs | 0.75 - 0.95 |
| *Higher values are usually appropriate for steeply sloped areas and longer return periods because infiltration and other losses have a proportionally smaller effect on runoff in these cases. | |

Table A-21 Manning's Roughness Coefficient (n) for Overland Sheet Flow

| Surface Description | n |
|---|---|
| Smooth asphalt | 0.011 |
| Smooth concrete | 0.012 |
| Ordinary concrete lining | 0.013 |
| Good wood | 0.014 |
| Brick with cement mortar | 0.014 |
| Vitrified clay | 0.015 |
| Cast iron | 0.015 |
| Corrugated metal pipe | 0.024 |
| Cement rubble surface | 0.024 |
| Fallow (no residue) | 0.05 |
| Cultivated soils | |
| Residue cover ≤ 20% | 0.06 |
| Residue cover > 20% | 0.17 |
| Range (natural) | 0.13 |
| Grass | |
| Short grass prairie | 0.15 |
| Dense grasses | 0.24 |
| Bermuda grass | 0.41 |
| Woods* | |
| Light underbrush | 0.40 |
| Dense underbrush | 0.80 |
| *When selecting n, consider cover to a height of about 30 mm. This is only part of the plant cover that will obstruct sheet flow. | |

Table A-22 Values of Manning's Coefficient (n) for Channels and Pipes

| Conduit Material | Manning's n* |
|---|---|
| Closed Conduits | |
| Asbestos-cement pipe | 0.011 0.015 |
| Brick | 0.013 - 0.017 |
| Cast iron pipe | |
| Cement-lined and seal coated | 0.011 - 0.015 |
| Concrete (monolithic) | 0.012 - 0.014 |
| Concrete pipe | 0.011 - 0.015 |
| Corrugated-metal pipe - 13 mm by 64 mm (½ inch by 2 ½ inch) corrugations | |
| Plain | 0.022 - 0.026 |
| Paved invert | 0.018 - 0.022 |
| Spun asphalt lines | 0.011 - 0.015 |
| Plastic pipe (smooth) | 0.011 - 0.015 |
| Vitrified clay | |
| Pipes | 0.011 - 0.015 |
| Liner plates | 0.013 - 0.017 |
| Open Channels | |
| Lined channels | |
| Asphalt | 0.013 - 0.017 |
| Brick | 0.012 - 0.018 |
| Concrete | 0.011 - 0.020 |
| Rubble or riprap | 0.020 - 0.035 |
| Vegetal | 0.030 - 0.400 |
| Excavated or dredged | |
| Earth, straight and uniform | 0.020 - 0.030 |
| Earth, winding, fairly uniform | 0.025 - 0.040 |
| Rock | 0.030 - 0.045 |
| Unmaintained | 0.050 - 0.140 |
| Natural channels (minor streams, top width at flood stage <30 m (100 ft)) | |
| Fairly regular section | 0.030 - 0.070 |
| Irregular section with pools | 0.040 - 0.100 |
| *Lower values are usually for well-constructed and maintained (smoother) pipes and channels. | |

Table A-23 time of concentration formula

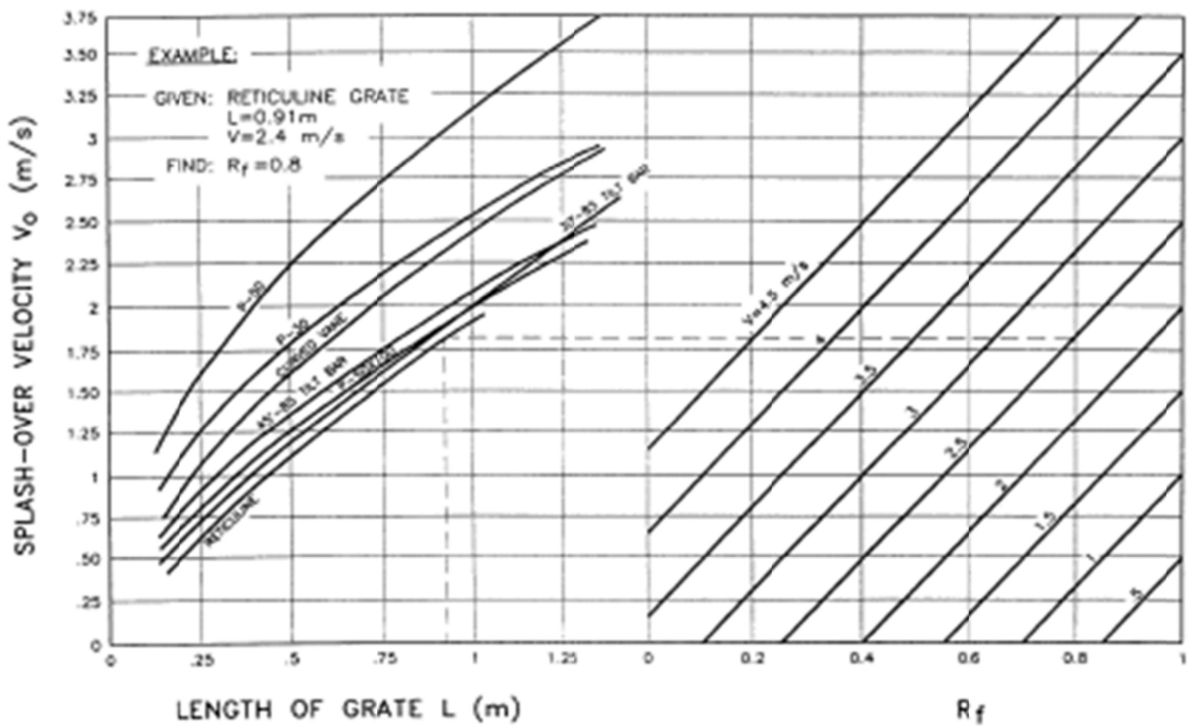
| Method and Date | Formula for $t_c$ (min) | Remarks |
|---|---|---|
| Kirpich (1940) | $t_c = 0.0078L^{0.77}S^{-0.385}$<br>$L$ = length of channel/ditch from headwater to outlet, ft<br>$S$ = average watershed slope, ft/ft | Developed from SCS data for seven rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%); for overland flow on concrete or asphalt surfaces multiply $t_c$ by 0.4; for concrete channels multiply by 0.2; no adjustments for overland flow on bare soil or flow in roadside ditches. |
| California Culverts Practice (1942) | $t_c = 60(11.9L^3/H)^{0.385}$<br>$L$ = length of longest watercourse, mi<br>$H$ = elevation difference between divide and outlet, ft | Essentially the Kirpich formula; developed from small mountainous basins in California (U. S. Bureau of Reclamation, 1973, pp. 67–71). |
| Izzard (1946) | $t_c = \dfrac{41.025(0.0007i + c)L^{0.33}}{S^{0.333}i^{0.667}}$<br>$i$ = rainfall intensity, in/h<br>$c$ = retardance coefficient<br>$L$ = length of flow path, ft<br>$S$ = slope of flow path, ft/ft | Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surfaces; values of the retardance coefficient range from 0.0070 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf; solution requires iteration; product $i$ times $L$ should be $\leq 500$. |
| Federal Aviation Administration (1970) | $t_c = 1.8(1.1 - C)L^{0.50}/S^{0.333}$<br>$C$ = rational method runoff coefficient<br>$L$ = length of overland flow, ft<br>$S$ = surface slope, % | Developed from air field drainage data assembled by the Corps of Engineers; method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins. |

Table A-23continued

| Method and Date | Formula for $t_c$ (min) | Remarks |
|---|---|---|
| Kinematic wave formulas Morgali and Linsley (1965) Aron and Erborge (1973) | $t_c = \frac{0.94L^{0.6}n^{0.6}}{(i^{0.4}S^{0.3})}$ <br> $L$ = length of overland flow, ft <br> $n$ = Manning roughness coefficient <br> $i$ = rainfall intensity in/h <br> $S$ = average overland slope ft/ft | Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces; method requires iteration since both $i$ (rainfall intensity) and $t_c$ are unknown; superposition of intensity–duration–frequency curve gives direct graphical solution for $t_c$ |
| SCS lag equation (1973) | $t_c = \frac{100\ L^{0.8}[(1000/CN) - 9]^{0.7}}{1900\ S^{0.5}}$ <br> $L$ = hydraulic length of watershed (longest flow path), ft <br> CN = SCS runoff curve number <br> $S$ = average watershed slope, % | Equation developed by SCS from agricultural watershed data; it has been adapted to small urban basins under 2000 acres; found generally good where area is completely paved; for mixed areas it tends to overestimate; adjustment factors are applied to correct for channel improvement and impervious area; the equation assumes that $t_c = 1.67 \times$ basin lag. |
| SCS average velocity charts (1975, 1986) | $t_c = \frac{1}{60}\sum\frac{L}{V}$ <br> $L$ = length of flow path, ft <br> $V$ = average velocity in feet per second from Fig. 3-1 of TR 55 for various surfaces | Overland flow charts in Fig. 3-1 of TR 55 show average velocity as function of watercourse slope and surface cover. (See also Table 5.7.1) |

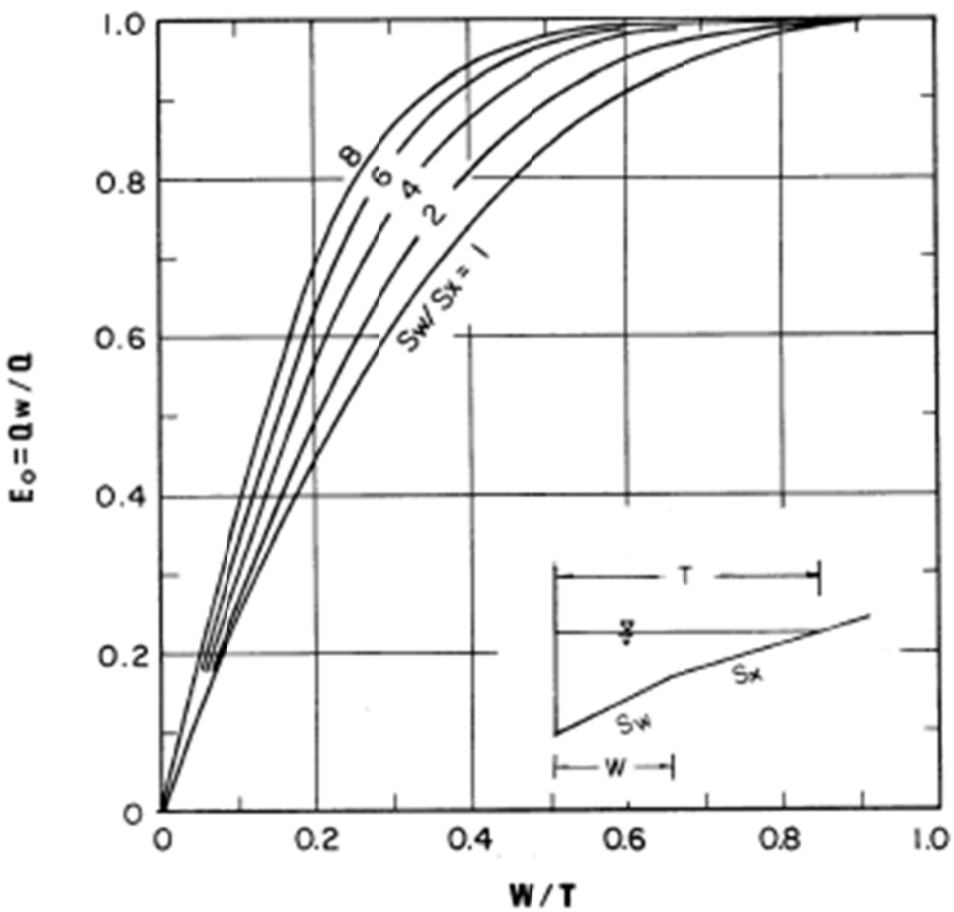
Chart 1: Grate inlet frontal flow Efficiency ($R_f$)

A - 10



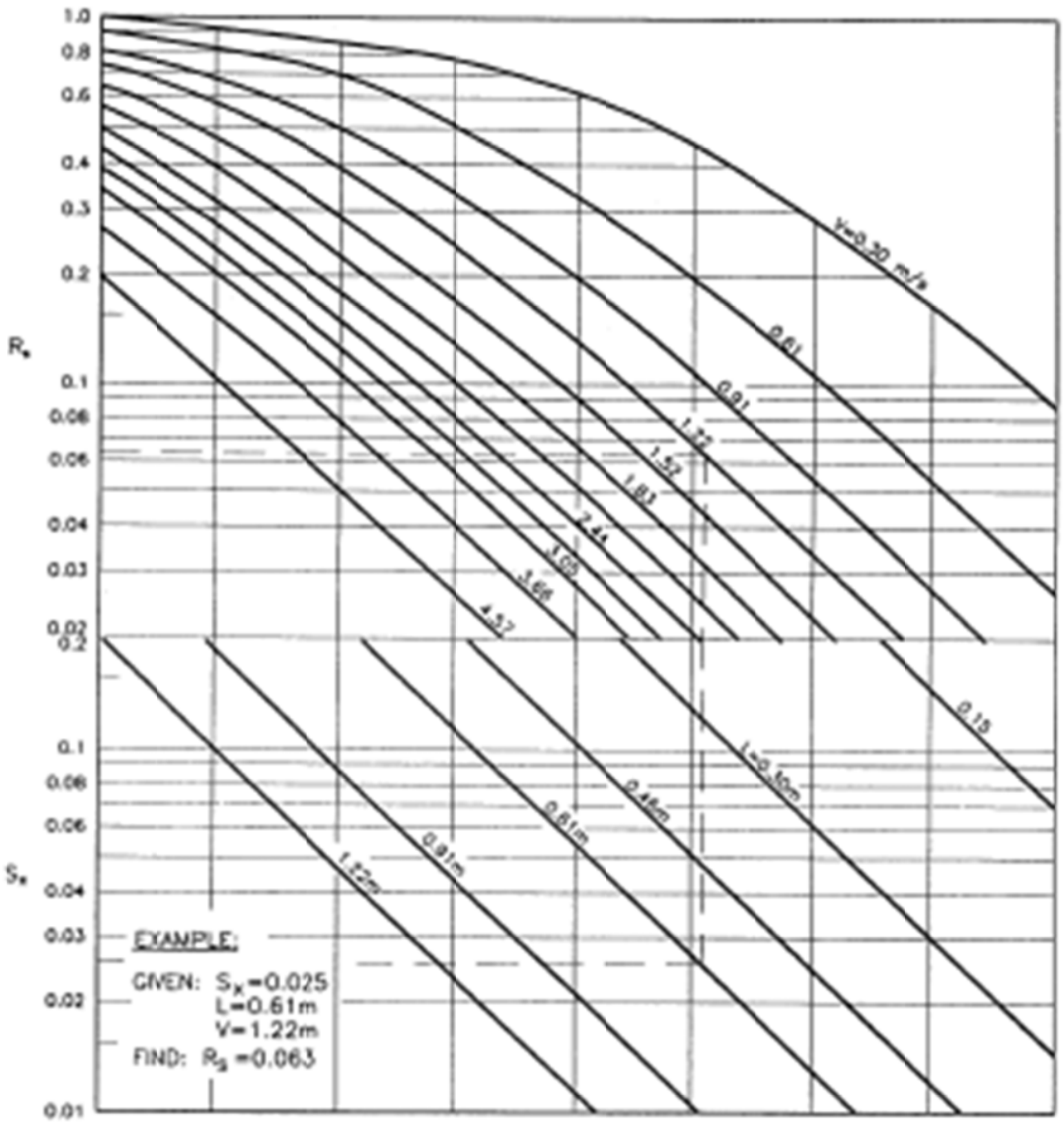

Grate Inlet Frontal Flow Interception Efficiency

Chart 2: Ratio of frontal flow to total gutter flow ($E_o$)



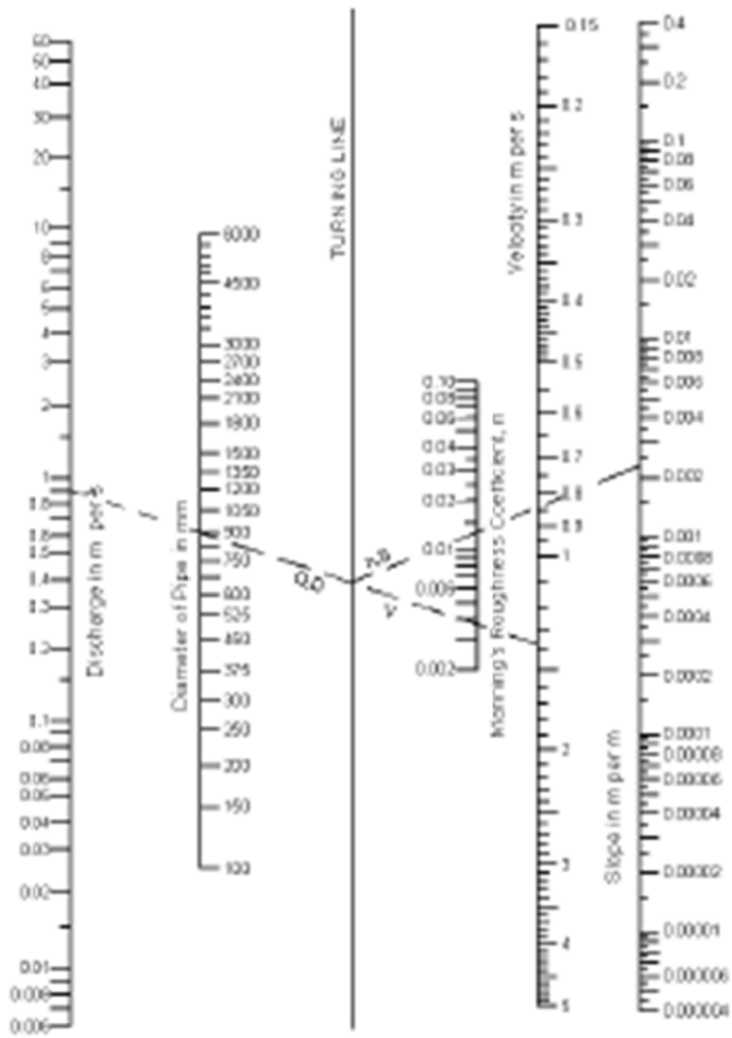

Ratio of frontal flow to total gutter flow.

Chart 3 Grate Inlet side flow interception efficiency ($R_s$)




Grate Inlet Side Flow Intercept Efficiency.

Chart 4 Solution of manning equation for flow in storm drain




Solution of Manning's Equation for flow in Strom Drains.

From page 142 – 149 See attached excel

Table A24

Table A25

Table A26