Assessment on Causes of Failure of Irrigation Canals and Its Remedial Measures: Case of Fentale Irrigation Project

Sintayehu Getahun

A Thesis Submitted to
The School of Civil and Environmental Engineering

Presented in Partial Fulfillment of the Requirements for the Degree of Master of Science (Hydraulic Engineering)

Addis Ababa University
Addis Ababa, Ethiopia
December 2015

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Declaration

I, the undersigned person, declare that this thesis is my original work and that all sources of materials used for this thesis have been duly acknowledged.

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Signature _________________

Place Addis Ababa University

Date of submission December 2015

Addis Ababa University
School of Graduate Studies

This is to certify that the thesis prepared by Sintayehu Getahun, entitled: Assessment on Cause of Failure of Irrigation Canals and Its Remedial Measures: Case of Fentale Irrigation Project and submitted in partial fulfillment of the requirements for the degree of Master of Science (Hydraulic Engineering) compiles with the regulations of the University and meets the accepted standards with respect to originality and quality.

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Acknowledgements

My most sincere and heartfelt thanks goes to my adviser Dr. Daniel F/Selassie for his constructive advice all along my activities. My heartfelt thanks goes to Dr.Yenesew Mengiste for her unreserved, professional and scholastic approach at any time I need her help. Had it not been for their genuine support the successful completion of this thesis could be in doubt.

I am also grateful to ERA for sponsoring my education and Oromia Irrigation Development Authority and west Arsi Zone Irrigation Development office for their material and transportation support during my frequent data collection periods.

I would also like to thank my friends Ato Roba and Ato Duguma who helped me during data collection and my sincere gratitude should also do to my lovely and caring family Chalachew, Mekoya ,Tsehaye, Fenta, Huluagersh, Adamainesh ,Eilsabet ,Aster,Endalamaw, Gebeyehu and Melkamu for their inspiring advise and moral support to pursue my study at higher education.

My very special appreciation and thanks goes to my lovely and wonderful husband, Endalamaw Teka, our beloved children Beab Endalamaw and Eferata Endalamaw and my beloved mam Kebedech Gudera for their unfailing support, encouragement, prayers as well as for their sacrifice and willingness in allowing me the time and space to complete this thesis.

Finally, but most importantly, I would like to thank my God who gave me endurance and strength in the face of all challenges.

ABSTRACT

Like most developing countries, the economy of Ethiopia is highly dependent on Agriculture. Rain fed agriculture contributes by far the largest part of the economy and it is currently growing on average 5% per year. In Ethiopia 85% of the population is dependent on rain fed agriculture. In spite of being relatively rich in water and land resources, the country is unable to produce reliable food supplies due to highly uneven rainfall distribution and droughts. However, Ethiopia has an estimated 5.3million hectare of irrigable land, yet only about 200,000 hectare is presently irrigated and only provides approximately 3%of the country"s food crop requirements. Similarly Oromia region regardless of its relatively better natural setting, most part of the region is suffering from food insecurity, particularly the low land area of the region, where majority of the population in the area were pastoralists.

Oromia regional government has constructed different irrigation projects. However, most irrigation projects, including Fentale have failed to achieve the planed objectives due to various reasons. Therefore, this study is intended to assess the major causes for the hydraulic and hydrologic failures of Fentale irrigation project and to recommend possible solutions.

Hydraulic and hydrologic failures of Fentale irrigation project were assessed by using primary and secondary data. Primary data were collected through field data measurement and observation. Secondary data were collected from designee document, other official documents and reports. Hydraulic failures were evaluated by manning"s formula. The actual value of manning"s roughness coefficient, discharge and velocity, were compared with the designed value. Moreover, hydrologic failure were evaluated by Soil Conservation Service (SCS) method.

The findings revealed that the values of the calculated *n* for different sections on the main canal were between 0.039-0.072. However, the designed values range from 0.017-0.03 for the same canal on which *n* was calculated. The calculated velocities for different sections on the main canal were between 1.048-0.62 m/s.

The measured velocity range from 0.566-0.337 m/s for the same canal on which velocity was calculated. Similarly, the designed and measured discharge capacity within the canal stretch showed great difference. An increase on the values of the calculated manning"s roughness coefficient (**n**) and the decrease in velocity and discharge causes the accumulation of silt along the canal.

The findings related to hydrologic failure revealed that the values of design flood for cross drainages structure of 50 years recurrent intervals were between 6.07-51m^3/sec and the calculated value of flood for cross drainages structure of 50 years recurrent intervals were between 22.44- 358.96m^3/sec. Therefore, the other major causes for the damage of the irrigation structures and overtopping of the canals were as the result of the flood that came from catchment areas of the streams.

Hydraulic performance of irrigation canal can be enhanced by appropriate design through use of adequate hydraulic parameters such as area, perimeter, velocity, discharge, and roughness coefficient and the hydrologic failure caused by underestimation of the cross drainage should be redesigned, constructed and maintained. However, maximum and efficient utilization of the Fenatle Irrigation Project can be achieved by not only on the major causes of failure, but also on examining the construction and management feature of the irrigation project in detail. Therefore, construction and management problems of Fentale irrigation project should be studied for efficient utilization.

Key Words: Hydrologic Failure, Hydraulic Failure. Fentale Irrigation Awash River Basin

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List of Acronyms and Abbreviations

BC	Branch Canal
DBL:	Design Bade Level
ERA	Ethiopian Road Authority
FAO	Food and Agricultural Organization
FSL	Full Supply Level
HFL	High Flood Level
IWMI	International Water Management Institute
ITCZ	Inter Tropical convergence Zone
MOWR	Ministry of Water Resources
MSF	Metahara Sugar Factory
ODOT	Ohio Department of Transport
OIDA	Oromia Irrigation Development Authority
PET	Potential Evapotranspiration
SCS	Soil Conservation Service
SSIP	Small Scale Irrigation Project
MSIP	Medium scale irrigation project
LSIP:	Large Scale Irrigation project
USDA	United States Department of Agriculture
WUA	Water Use Association
JICA	Japan International Cooperation Agency

1 Introduction

The majority of population of Ethiopia is dependent on rain fed agriculture. However, estimated crop production is not close to fulfill the food requirements of the country. Ethiopia has excessive potential in irrigation. According to the study made by Awulachew,et.al (2007) on Ethiopia‟s major river basins and corresponding surface irrigation potential the country has 5.3 million hector excessive irrigation land, yet only about 200,000 hectare is presently irrigated and only provides approximately 3%of the country‟s food crop requirements. The irrigation potential in Ethiopia can enables to increase the frequency of crop production and alleviate the water shortage caused by insufficient rainfall, and or dry seasons, and helps to produce sufficient food that meets the requirements of the fast growing population of the country. If successful, irrigation in Ethiopia can contribute up to ETB 140 billion to the economy and can secure food for 6 million households in the country (Awulachew et .al, 2010). According to Clark et.al (2012) and Japan International Cooperation Agency JICA (2004) irrigation can promotes sustainable economic growth of a nation and plays a significant role in poverty reduction through enhancing the productivity and profitability of irrigated agriculture.

Oromia, despite its relatively better natural setting, most part of the region is suffering from food insecurity, particularly the low land area of the region where majority of the population in this area were pastoralists. Since one third of the area of the region is lowland with prone to drought (Oromia Irrigation Development Authority (OIDA), 2011).
Irrigation is the best alternatives to overcome the problem. Based on this, The Oromia Irrigation Development Authority is a governmental organization actively engaged in development of small scale to large scale irrigation based agricultural projects in different parts of the region.

Among this, Fentale irrigation project has been designed and constructed on Awash River and mainly located in Fentale district, East Shewa zone of the Oromia Regional State, and a limited part of Boset district in the same zone and adjacent to some part of Fentale district is also incorporated in the project.

The major causes for failure of irrigation canals would be bad design, bad construction and/or poor management (Laycock, 2007). As Food and Agricultural Organization (FAO) (1993) bad design and or bad construction are the cause of sub optimal functioning of a scheme. A canal may be too small to supply enough water to irrigate the area served by the canal and if the discharge needed is supplied to such a canal it can be excessive and water can be overtop and the over top can erode the canal bank Food and Agricultural Organization (FAO) (1992). Also the water level in a canal may have been wrongly determined and if it is too low water may not enter the fields by gravity. Munir (2011) stated bad design could cause sediments. Sediments tend to deposit in irrigation canals and become a serious problem in canal operation and maintenance, which requires frequent de silting campaigns to keep the water free to move.

And the delay in canal maintenance affects their hydraulic performance Munir (2011).
As key part of the irrigation project, canals do fail to serve the purpose for which they are intended. Accordingly, at Fentale irrigation project the hydraulic and hydrologic failures, lack of maintenance and lack of awareness of the community hinder the irrigation project to provide the expected result. The study conducted by Geleta(2011) on Fentale irrigation project also showed the limitation of design construction quality and weak environmental impact mitigation strategy of Fentale irrigation project. The Fentale irrigation canal, which fails to serve the design purpose, has not been studied carefully. This research is therefore, tries to identify the hydrologic and hydraulic reasons behind this failure and recommend possible solutions

1.1 Statement of the Problem

A canal, which fails to serve their designed objectives, is common in Fentale large scale irrigation project. Canals failed due to improper design, faulty construction, and lack of proper maintenance and/or poor management. These reasoning could also be considered as generalized failure causes. Improper design by itself is very broad as far as failure is concerned. Design includes decision on the hydraulic and hydrologic studies and parameters to be used in the canal design.

The main cause for failure of irrigation canal would be hydraulic and hydrologic. Among hydraulic failures siltation is the common one. Siltation would be occurred due to design problem, construction or poor management and also one of the main causes for hydrologic failure would be over topping. Over toping can be caused by flood that inter in to the canal and accumulation of silt that causes the water to flow over .Therefore, overtopping can be both hydraulic and hydrologic failure Therefore, this study was intended to assess the major causes of the hydraulic and hydrologic failure of Fentale irrigation canals

1.2 Objectives

1.2.1 General Objective

- ❖ Assess the hydraulic and hydrologic cause of failure and recommend remedial measure of irrigation canals in Fentale irrigation project.

1.2.2 Specific Objectives

- ❖ Identify hydraulic failure of the Fentale irrigation project
- ❖ Identify the hydrologic failure of the Fentale irrigation project
- ❖ Recommend remedial measures related to hydraulic and hydrologic causes of failure of the canals in Fentale irrigation project.

1.3 Research Questions

- Is failure of irrigation canals problem in Fentale irrigation project?
- What are the main causes of failure?
- Are hydrologic and hydraulic failures common?
- What measures should be adopted/taken to alleviate these problems

1.4 Organization of the Thesis

The content of the thesis is outlined briefly in this section as follows

Chapter 1: has introduced the general introduction, statement of the problem, research objectives and research questions of the thesis and states the whole over of the thesis

Chapter 2: deals about literature review and gives detail information about irrigation, irrigation development in Ethiopia, irrigation potential in Ethiopia, purpose of canal, canal design, canal control structure, failure of irrigation canal, hydraulic failure of irrigation canal, hydrologic failure of irrigation canal estimation of flood and discharge measurement structures

Chapter 3: deals about the general descriptions of the study area, and the research methodology that is the research methods, the research methods used, and the data collection, analysis and interpretations.

Chapter 4: elaborates about the result and discussions of the thesis in light of the specific research question raised and research objectives

Finally, chapter 5 deals about the conclusion and recommendation resulted from the study work and suggest topics for further research.

2. Literature Review

2.1 Irrigation

According to Ali (2010) Irrigation is the application of water to meet the crop water demand at the proper time in the proper way. Irrigation is one means by which agricultural production can be increased to meet the growing food demands of the fast growing population of the country (Food and Agricultural Organizations(FAO),(2003). Increasing food demand can be met in one or a combination of three ways: (i) increasing agricultural yield, (ii) increasing the area of arable land, and (iii) increasing cropping intensity by growing two or three crops per year using irrigation (Belete.et.al, 2011).

2.2 Irrigation Development in Ethiopia

Ethiopia has a long history of traditional irrigation systems. Simple river diversion still is the dominant irrigation system in Ethiopia. As Belete,et.al (2011) traditional irrigation has been practiced in the highlands for centuries, particularly for producing subsistence food crops; it was only in the early 1950s that modern irrigation technologies were introduced to Ethiopia by a Dutch company in the Upper Awash Valley with the objective of producing industrial crops such as large-scale sugarcane plantations. The potential irrigable land of the country is estimated to be about 3.7 million hectares only about 10- 12% of the total potential is under production using traditional and modern irrigation schemes (Belete,et.al,2011).

Also Awulachew,et.al (2007) Irrigation is practiced in Ethiopia since ancient times producing subsistence food crops. However, modern irrigation systems were started in the 1960s with the objective of producing industrial crops in Awash Valley. Private concessionaires who operated farms for growing commercial crops such as cotton, sugarcane and horticultural crops started the first formal irrigation schemes in the late 1950s in the upper and lower Awash Valley. In the 1960s, irrigated agriculture was expanded in all parts of the Awash Valley and in the Lower Rift Valley. The Awash Valley saw the biggest expansion in view of the water regulation afforded by the construction of the Koka dam and reservoir that regulated flows with benefits of flood control, hydropower and assured irrigation water supply

As Awulachew,et.al (2007) tried to survey the total estimated area of irrigated agriculture in the country is 107,265.65 hectares out of which 20,038.39 hectares are from small-scale, 30,291.26 hectares is from medium-scale and 56,936 hectares is from large scale.

2.3 Irrigation Potential in Ethiopia

According to Awulachew,et.al (2007) Ethiopia has vast cultivable land (30 to 70 million hectares (Mha), but only about a third of that is currently cultivated (approximately 15 million hectares (Mha), with current irrigation schemes covering about 640,000 ha across the country and total irrigable land potential in Ethiopia is 5.3 million hectares assuming use of existing technologies, including 1.6 million hectares through RWH and ground water. This means that there are potential opportunities to vastly increase the amount of irrigated land. The distribution of the surface water potential breakdown by size is 5% Small Scale Irrigation (SSI), 9% Medium Scale Irrigation (MSI) and 86 % Large Scale Irrigation (LSI).

The Ministry of Water and Energy has identified 560 irrigation potential sites on the major river basins. The total irrigable land in Ethiopia is estimated to be around 3.7 million hectares (without considering the groundwater potential and gently sloping areas). The area under irrigation development to-date is estimated to range between 160,000-200,000 hectares for the entire country (Awulachew,et.al 2007)

2.4 Purpose of Canals

According to Laycock (2007) canal is defined as an artificial watercourse, a duct or passage that conveys fluids. Most irrigation schemes distribute their water through an open canal network. Water flows at a rate which is governed by the canal size, roughness and longitudinal slope Usually for irrigation we are trying to keep the water delivery point at as high an elevation as possible by minimizing the canal longitudinal slope. A wide variety of structures can be inserted in the canal to control water levels, discharges, turbulence, sediment content and velocity, to convey water around or across obstacles, and to measure the flow. Although irrigation distribution is by far the commonest use of canals, there are many other applications including drainage, power generation, navigation, flood relief, industrial and domestic water supply and sewerage.

2.5 Canal Design

According to Garg (2005) the entire system of the main canals, branched canals distributaries and minor canals is to be designed properly for a certain realistic value of peak discharge that must pass through them, so as to provide sufficient irrigation water to the command areas.

The design of a channel involves the selection of channel alignment, shapes, size, and bottom slope and whether the channel should be lined to reduce seepage and/or to prevent the erosion of channel sides and bottom. Since a lined channel offers less resistance to flow than an unlined channel (Ajiboye, 2010).

As Muya et al (2009) the canal dimensions and longitudinal slope, whether for irrigation or drainage can be calculated through trial and error with the Manning formula. This formula is derived from the continuity equation and the equation for unsteady flow. These equations have been simplified by assuming steady uniform flow in the canal, assuming that the canal is long with constant cross section and slope.

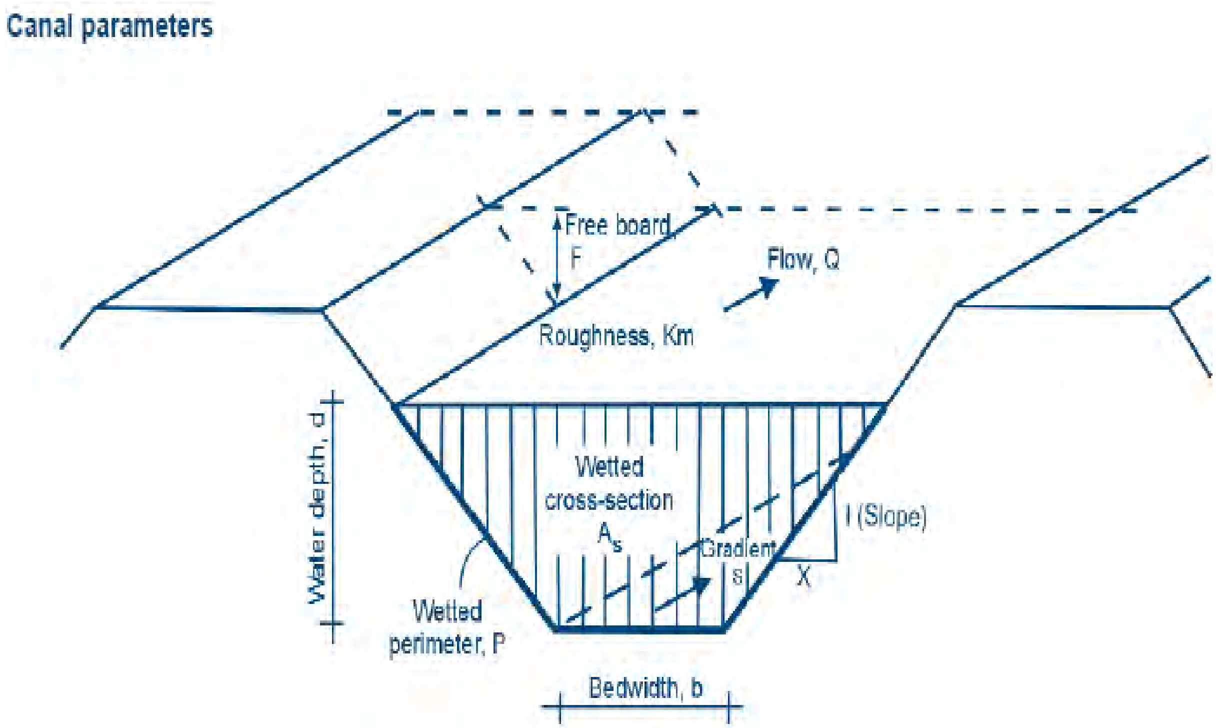


Figure 2.1Canal parameter for trapezoidal cross sectional shapes

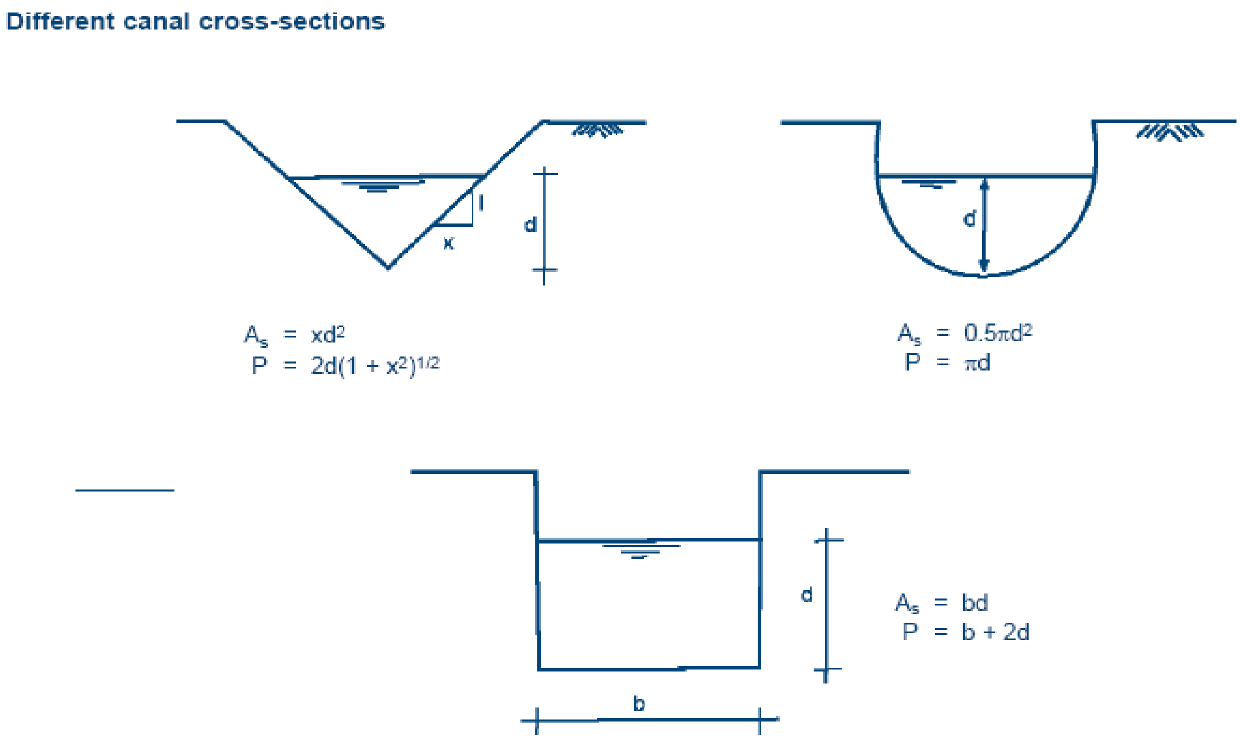


Figure 2.2 Canal parameter for different cross sectional shapes

Different canal parameters A and P and thus R in the Manning formula, can be expressed in d, b and X (Figure 2.2).

Where,

d = Water depth (m)

b = Bed width (m)

X = Side slope = horizontal divided by vertical

For a trapezoidal canal, A is the sum of a rectangle and two triangles.

The cross-sectional area of a rectangle is:

$$A = (b \times d) \tag{2.1}$$

The cross-sectional area of A although the trapezoidal canal shape is very common, other canal shapes, including V-shaped, U-shaped, semicircular shaped and rectangular shaped canals, can also be designed.

Area of triangle, $A = \frac{1}{2}(base \times height) = \frac{1}{2}(Xd \times d)$ (2.2)

Thus, the wetted cross-sectional area A of the trapezoidal canal is

$$A = b \times d + 2\left(\frac{1}{2} \times Xd \times d\right) = b \times d + Xd^2 = d(b + xd) \tag{2.3}$$

Thus the wetted perimeter for the trapezoidal canal section is:

$$P = b + \{2(d^2 + (dx)^2)\}^{1/2} = b + 2d(1 + X2)^{1/2} \quad (2.4)$$

The hydraulic radius R is:

$$R = d(b + Xd)/(b + 2d(1 + X^2)^{1/2} \quad (2.5)$$

Canals with narrower beds and higher water depths have a smaller wetted perimeter, and thus a higher discharge, than canals with larger beds and lower water depths, for the same cross sectional area. This is due to the fact that the hydraulic radius R (= A/P) increases if the wetted perimeter decreases, while keeping the wetted cross-sectional area the same (Muya, 2009).

Thus, there are several formulae for canal design, but the one in most common use is that of Manning, which came into general use in the early years of the 20th century. Manning gives velocity in terms of channel geometry, slope and roughness (Laycock, 2007; Garg, 2005)

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad (2.6)$$

Or, in terms of the discharge

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \quad (2.7)$$

Where,

Q = discharge of the channel (m^3/s)

V =velocity (m/s)

R = (A/p) = Hydraulic radius (m)

A = Wetted cross-sectional area (m^2)

S = bed slope (-)

n = Manning‟s roughness coefficient

2.5.1 Unlined Canals

The design of unlined canals needs a lot of care in order to make them stable. The purpose is to determine such values of depth, bed width, side slopes and longitudinal slope of the canal which produce a non-silting and non-scouring velocity for the given discharge and sediment load.

When the velocity of flow is such that there is no silting or scouring action in the canal bed then that velocity is known as critical velocity**.** It is denoted by „Vo‟. The value of Vo was given by Kennedy according to the following expression (Garg, 2005)

$$Vo = 0.546 D^{0.64} \quad (2.8)$$

Where,

D = Depth of water

The ratio of mean velocity „V" to the critical velocity „Vo" is known as critical velocity ratio (CVR). It is denoted by m i.e.

$$CVR(m) = V/Vo \tag{2.9}$$

When m = 1, there will be no silting or scouring.

When m > 1, scouring will occur
When m < 1, silting will occur
So, by finding the value of m, the condition of the canal can be predicted whether it will have silting or scouring.

Critical velocity ratio is not applicable in lined canals but the possibility of silting cannot be neglected. Hence, the critical velocity ratio should be aimed at higher than unity or by any other method it should be ensured that silting will not take place in the lined canal (Muya, 2009)

Maximum water velocities
The maximum permissible non-erosive water velocity in earthen canals should be such that on the one hand the canal bed does not erode and that on the other hand the water flows at a self-cleaning velocity with minimum or no deposition of soil.

2.5.2 Lined canals
Design of lined irrigation canals is relatively simple as there is not a certain restriction on higher or lower values of the flow velocities. As long as the Manning"s n is estimated correctly for the given lining material, the canal works as per design.

Factors affecting canal discharge
Canal gradient or longitudinal slope of the canal the steeper the gradient, the faster the water flow and the greater the discharge. This is substantiated by the Continuity Equation. Velocity increases with an increase in gradient or longitudinal slope. It therefore follows that a canal with a steeper gradient but with the same cross-section can discharge more water than a canal with a smaller gradient (Muya, 2009).

Canal roughness
As Muya (2009) the canal roughness, as depicted by the Manning roughness coefficient, influences the amount of water that passes through a canal. Unlined canals with silt deposits and weed growth and lined canals with a rough finish tend to slow down the water velocity, thus reducing the discharge compared to that of a clean canal with a smooth finish. Canals that slow down the movement of water have a low Km or a high n. It should be understood that the higher the roughness coefficient (Km) or the lower n, the higher the ability of the canal to transport

water, hence the smaller the required cross-sectional area for a given discharge. Manning coefficients (n) are often assumed relatively high during the design phase compared to what they actually will be during scheme operation due to deterioration of the canals. The result is an increased wetted cross-sectional area of the canal during scheme operation with the danger of overtopping the canal banks. This in turn means that the canal discharge has to be reduced to below the design discharge, in order to avoid overtopping. There is therefore a need for regular and proper maintenance of canals.

2.6 Canal Control Structures

Water flowing in the canal is controlled in several ways by structures of various kinds which perform one or more specific functions.

2.6.1Cross Drainage Structure

According to Jordaan, (2011) Cross drainage is necessary to conduit storm runoff water safely across the canal and avoid such water carrying rock and debris from entering and contaminating clean water supplied by the canal, such as water should also not run in drainage channels next to and parallel to the canal sides, as under cutting of the lining would result causing collapse

Across drainage work is a structure which construct at the crossing of a canal and natural drain so as dispose of drainage water without interrupting the continuous canal supplies (Garg, 2005 and Radhan, 1989).

Types of Cross Drainage Works

According to Garg, (2005) and Anyemedu, (2007) Depending on the relative bed levels and water levels of the canal and the drain and their relative discharges the cross-drainage works may be of the following types:

i. Cross-drainage works carrying the canal over the natural drain. This may be accomplished by the following types of cross-drainage works:

   a) Aqueduct

Aqueduct is just like a bridge in which instead of road or a railway, a canal is carried over a natural drain. An aqueduct is constructed where the bed of the canal is well above the High Flood Level (H.F.L). of the drain. The canal water is taken across the drain trough supported on piers. The drain water flows under the canal such that there is sufficient headway available between the High Flood Level (H.F.L). of the drain and the underside of the canal trough. The drain, therefore, flows at atmospheric pressure under the work (Garg, 2005 and Anyemedu, 2007).

b) Siphon aqueduct

Siphon aqueduct is constructed where the water surface level of the drain at high flood is higher than the canal bed. The drain water flows under pressure through the barrels, which act as inverted siphons, and hence this cross drainage work is known as siphon aqueduct. In siphon aqueduct, the drain bed is generally depressed and provided with impervious floor. On the u/s side, the drainage bed may be joined to the impervious floor either by a vertical drop (when drop is in the order of 1 m) or by a glacis of 3:1 (when drop is more than 1 m. The d/s rising slope should not be steeper than 5:1 (Garg, 2005)

ii. Cross drainage works carrying the natural drain over the canal, such as

a)Super passage

In this type of cross-drainage work, the natural drain runs above the canal, the canal under the drain always having a free surface flow. Thus, it is a reverse of an aqueduct. It is constructed where the drain bed is well above the canal F.S.L. In this case the drain water is taken across the canal in a trough supported on piers. The canal flows at atmospheric pressure as it has sufficient head way .An inspection road is not provided (not possible) along the canal. (Garg, 2005)

b)Siphons (or canal siphons)

iii. Cross-drainage works admitting the drain water into the canal. In this type of works the canal water and the drain water are allowed to intermingle with each other. This may be achieved by the following works:

(a) Level crossing

Level crossing is provided when the beds of the canal and the drains are practically at the same level. This is more frequently used if either of the flows occurs for a short period (e.g. flash flood in the drains) in addition the mixing of the two bodies of water must also be acceptable (quality considerations)(Novak et.al,2007) The canal regulator regulates its flow with the drain regulator kept closed. Whenever the flash flood occurs, the canal gates are closed and drainage gates opened to let the flood flow pass. Drain crest level is at Full Supply Level (F.S.L).of the canal. When the drain is not carrying water the drain regulator is closed and the cross regulator of the canal is kept fully open so that the canal flows without interruption. When the drain is carrying water, it spills over the crest into the canal and a corresponding discharge is passed through the drain regulator into the drain. The supplies in the canal are controlled accurately by the canal regulator (Garg, 2005)

(b) Inlet and outlet

When the drainage flow is small it may be absorbed into the canal through inlets. The flow in the canal may be balanced, if necessary (in the case of small canals), by providing suitable outlets (or escapes). Such a structure is adopted when the drainage discharge is small and the drain crosses the canal with its bed level equal to or slightly higher than the canal Full Supply Level (FSL) When the drainage discharge is high or if the canal is small, so that the canal section cannot take the entire drainage water, an outlet is constructed to escape out the additional discharge at a suitable site, a little d/s along the canal. It is not necessary that the escaped discharge should be equal to the admitted discharge (Garg,2005) and (Novak et.al ,2001).

Selection of a Suitable Type of Cross Drainage Work

According to Garg (2005) the relative bed levels, water levels, and discharge of the canal and the drainage are the primary factors which govern and dictate the type of cross drainage work that may prove to be most suitable at a particular place. For example, if the bed level of the canal is sufficiently above High Flood Level (HFL) of the drain, an aqueduct is the first choice. But, if the bed level of the drain is sufficiently above the canal Full Supply Level (FSL), a super passage may be constructed. Similarly, when a canal carries a small discharge compared to the drain, the canal may be taken below the drain by constructing a siphon, as against a siphon aqueduct which is adopted when the drain with smaller discharge can be taken below a large canal.

However, in actual field, such ideal conditions may not be available and the choice would then depend on many other factors, such as

a. Suitable channel alignment
b. Nature of available foundation
c. Suitability of soil for embankment
d. Permissible head loss in canal
e. Economy

The relative bed levels of the canal and the drainage may be changed and manipulated by suitably changing the canal alignment, so that the point of crossing is shifted u/s or d/s of the drainage. For example, if the canal alignment is such that sufficient headway is not available between High Flood Level (HFL) of drain and bed of canal, a siphon aqueduct has to be normally adopted. But, if other conditions are not favorable for the construction of a siphon aqueduct, the canal alignment may be changed so that the crossing is shifted to the d/s where drainage bed is low and thus sufficient headway becomes available for constructing an aqueduct in place siphon aqueduct. The canal alignment is, therefore, finalized only after finalizing the cross drainage works.

Compared to an aqueduct, a super passage is inferior and should be avoided whenever possible. Similarly, a siphon aqueduct (unless large drop in drainage bed is required) is superior to a siphon (Garg, 2005 and Novak et.al 2001).

Design of Cross Drainage Structures

According to Garg, (2005) the following steps may be involved in the design of an aqueduct or syphon-aqueduct. The design of super passage and a syphon is done on the same lines as for aqueducts and syphon aqueducts, respectively, since hydraulically there is not much difference between them, except that the canal and the drainage are interchanged each other.

i) Determination of maximum flood discharge

The high flood discharge for small drains may be worked by using empirical formulas; and for large drains other reliable methods such as hydrograph analysis rational formula etc. may be used.

ii) **Fixing the waterway requirements for aqueducts and syphon aqueducts**

An approximate value of required waterway for the drain may be obtained by using Lacey"s equation, as

$$P = 4.75\sqrt{Q} \tag{2.10}$$

Where

P = wetted perimeter

Q =Total discharge

For wide drains, the wetted perimeter may be approximately taken equal to the width of the drain and hence, equal to the waterway required. However, no extra provision is generally made for the space occupied by piers. Hence, if the total waterway provided is equal to P, the effective or clear waterway will be less than P by as much extent as is occupied by pier widths. For smaller drains, a smaller figure for the waterway than that given by Lacey"s regime perimeter may be chosen. The maximum permissible reduction in waterway from Lacey"s perimeter is 20%. Hence, for smaller drains, the width of the waterway provided should be so adjusted as to provide this required perimeter (minimum value = 0.8 P) (Garg, 2005).

Size of the barrels: After having fixed the waterway width & number of bays, the height of the drain barrels has to be fixed. In case of an aqueduct, the canal trough is carried clear above the drain High Flood Level (HFL) and drain bed is not to be depressed. Hence, the height of bay openings is automatically fixed in aqueducts, as equal to the difference between High Flood Level (HFL) and Design Bead Level (DBL) of drain.

However, in syphon aqueducts, the required area of the drainage waterway can be obtained by dividing the drainage discharge by permissible velocity through the barrels. The velocity through the barrels is limited to 2 to 3 m/s. The waterway area is then divided by the decided waterway width of the drain openings, to compute the height of the openings, and the extent of depressed floor.

iii) **Afflux and Head Loss through Syphon Barrels**

The head loss, h, through syphon barrels and the velocity, V, through them are generally given by

$$h = \left[1 + f_1 + f_2 \frac{L}{R}\right] \frac{V^2 - V_a^2}{2g} \tag{2.11}$$

Where

L =Length of the barrel

R =Hydraulic mean radius of the barrel

V =Velocity of flow through the barrel

Va =Velocity of approach and is neglected

f_1 = Coefficient of head loss at entry =0.505 for unshaped mouth =0.0.08 for bell mounted

f_2 =is a coefficient such that loss of head through the barrel due to surface friction is given by $f_2, \frac{L}{R}, \frac{V^2}{2g}$ where f_2 is give as $f_2 = a\left(1 + b/R\right)$ (2.12)

iv. **Head loss and bed levels at different sections**

The total head loss consists of three losses;

a) Entrance loss = $f_1\left(\frac{V^2}{2g}\right)$ (2.13)

b) Friction loss = $f_2\left(\frac{LV^2}{2gR}\right)$ (2.14)

c) Exit loss = $\frac{V^2}{2g}$ (2.15)

v. **Design of Transitions**

The channel width at any section x-x, at a distance x from the flumed section is given

$$B_x = \frac{B_n B_f L_f}{L_f B_n - X\left(B_n - B_f\right)} \tag{2.16}$$

Where

B_n =Bed width of the normal channel section

B_f = Bed width of the flume channel section

B_X = Bed width of at any distance x from the flume section

L_f =Length of transition

2.6.2 Diversion Structures

Divisors (diversion structures) are used to separate a flow into two or more parts. These structures simply divide the flow in a ditch in two the desired measure or proportional part.

2.6.3 Stilling Basins

A stilling basin is a basin with protected walls and floor and which is filled with water. Its function is to convert the energy of fast flowing water into turbulence, so that the flow enters at low speed into the canal downstream of the basin. Stilling basins are required downstream of structures where flow velocities can be high, such as intake structures, off takes, culverts, weirs, or drop structures. Flow velocities in intake structures, for instance, may be high when water flows through a pipe before entering the main canal of an irrigation scheme, and the velocity of water which spills over a weir may be high. Fast flowing water is highly erosive and can easily damage a canal bank or bed, and so the energy of this fast flowing water must be dissipated in order for it to flow smoothly into the downstream canal section (Laycock, 2007 and Novak et al, 2001).

2.6.4 Spillways

A spillway is a structure that guides excess water safely to the drainage system. Water levels in irrigation canals are seldom constant. Depending on the inflow and outflow of the canal section concerned, the water level changes. The water level may rise if the gate of an intake structure is open instead of being closed, or if field intakes are closed instead of open. This can happen even in well managed schemes. When rising, water may pass the free board level reach the crest of the canal embankment, and start over topping. If water reaches the crest of a canal bank and overtops, this can result in destruction of the bank. To avoid this problem, a small section of the canal bank is lowered and is reinforced with concrete or with masonry. As this is the lowest part of the canal bank, rising water will spill over here. This water will be guided to the drainage system (Laycock, 2007 and Novak et.al ,2001).

2.6.5 Drop Structures

Irrigation water may need to be transported over steeply sloping land. If a canal had the same slope as the surrounding steeply sloping field, the flow velocity in the canal would be very high. In these cases the canal is given a slope which is less than the field slope in order to avoid unacceptably high flow velocities (Novak et.al, 2001).

The function of drop structures is to convey water from a higher to a lower elevation and to dissipate excess energy resulting from this drop. A canal along this same terrain would ordinarily be steep enough to cause severe erosion in earth canals or disruptive flow in hard surface lined canals. The water must therefore be conveyed with a drop structure designed to safely dissipate the excess energy. Different kinds of drops that may be used are vertical, baffled apron, rectangular inclined, and pipe drops (Novak et.al, 2001).

2.6.6 Freeboard

According to Muya (2009) freeboard is a depth supplement that is adopted in canals in order to take into account any unexpected events and to provide greater security against overflow. The causes involved in the adoption of the freeboard widely vary and have very different degrees of importance

2.7 Failure of Irrigation Canals

According to Laycock (2007) failure of canals can be distinguished under three major indicators. These are "either water doesn't flow, or it overflows, or it disappears, or the drains are running."

Irrigation canals have failed due to Bad Construction, poor management, social problems, or wrong strategies during design (Robel,2005 and Laycock,2007).More specifically Laycock (2007) explained the following major pointes on failure of irrigation canals

2.7.1 Bad Construction

Construction tolerances

Tolerances are a bone of contention on construction sites. The tolerance is the permissible deviation in dimensions or levels between that specified on the design drawings and what is actually constructed.(Laycock, 2007)

Supervision

Poor supervision or no supervision at all, is quite usual on small, low cost projects. It is a common mistake on the part of funding agencies not to spend enough money on supervision. Inadequate supervision is inevitable when large numbers of small contractors are required to be supervised by a small number of engineers. The paperwork gets out of hand and the engineers don't have any time in the field. This often happens on big projects if it is a government strategy to generate employment for small contractor. (Robel, 2005).

Bad workmanship

Many engineers may go through their entire career not knowing how to recognize good or bad concrete, all because they never had the right back up from their seniors when they first went out on site

Poor soil compaction

The process of compaction rearranges the soil particles to form a dense mass, which binds together for strength and restricts the seepage of water through it. Different types of soil compact at different densities, permeability and strengths. All soils are critically dependent on moisture

content to achieve optimum compaction. Too dry and no amount of comp active effort will work. Too wet. And the soil turns to slurry and the machine gets stuck, or it cracks on drying out.
Poor compaction results not only in leakage but in soil movement. It may settle under its own weight. The easy passage of water through the soil can create rapid changes of moisture content. As it dries out it may shrink, and large cracks may form (Laycock,2007)

2.7.2 Bad Design

Sedimentation

According to Vabre (1995) The problem of sediment being particularly severe in irrigation canals, on the one hand water surface levels must be kept constant to deliver water to different off take devices in the other hand water discharge regularly decrees along any irrigation canal due to farmer off takes this both induces low velocities and sediment deposition and as the result of the silt deposit could usually causes weeds to grow along the canal. Potential difficulties caused by silt deposition need to be fully appreciated at the design stage. Extract it, eject it, exclude it, or flush it through the canals, or else be prepared to dig it out.

Oversizing

Oversize canals give a long response time. It is a common mistake of inexperienced engineers to be over conservative in estimating the roughness coefficient of a canal. The canal ends up larger than it needs to be and response times are increased. When canals are built in deep cut in some cases, there were all kinds of problems with landslips in the rainy season and siltation because of sheer inaccessibility for cleaning (Laycock, 2007).

Unsuitable local materials

Mud mortar and laterite blocks may be suitable for houses but they are no good at all for hydraulic structures

Not enough turnouts

Inadequate provision of outlet structures will result in farmers resorting to their own devices. The result is that there are never enough, because the standard tertiary turnout spacing is determined by government rules, and not by farmer needs. And nobody ever thought to question them since they are printed in government manuals (Laycock, 2007).

Regulators and division structures

These structures are often adaptations of standard undershot turnouts, with the principles of hydraulic flow ignored in the interests of simplifying construction. This results in square division boxes that are too large and have internal sharp corners where flow velocity is reduced and sediment collects (Laycock, 2007)

Design for short life

Durability of canal lining is not always an important factor in present design philosophies. Early failure of lining is sometimes accepted as unavoidable and even desirable in order that future work in reconstruction may be assured.

2.7.3 Poor Water Management

Poor management covers a multitude of sins. But there are some fundamental causes here, which should be considered.

Lack of training

Many schemes particularly small ones suffer because there are no firmly established rules of operation, and incomplete training of Canal Operators. Operating rules and maintenance schedules should be considered an essential part of detailed design. (Laycock, 2007)

Improper gate operation

A canal with a lot of manually-operated gates can be a nightmare to control. No matter how many rules might be laid down, spillage is common. And spillage in one place means a water shortage somewhere else.

2.8 Hydraulic Failure of Irrigation Canal

According to Food and Agricultural Organization (FAO) (1992) Overtopping of a canal section is caused by an excessive discharge in that section in relation to the actual canal capacity. Canal banks which are frequently overtopped are very probably eroded and lowered, and thus the actual capacity will be less than the original capacity for which the canal has been designed.

Sedimentation

The deposition of soil and debris can affect the functioning of a structure. For instance, a stilling basin collects soil deposits the available water mass diminishes and energy dissipation will be less effective. Similarly in the case of soil deposits in a flow division box, the division of the flow will be less accurate due to changes in flow velocities and water levels (Food and Agricultural Organization (FAO), 1993).

According to Munir (2011) sediment control strategies start with the selection of a proper point for the diversion and the choice of appropriate structures at river intakes in order to prevent unwanted sediment entry into the irrigation canals. Then the entered sediments into the canals are ejected through different means, by structures or sometimes sediments are deposited in the oversized canal sections, settling basins, at the head of the canals and then periodically removed. Further, the canals are so designed that the hydraulic conditions during canal operation allow neither sediment deposition nor scouring in the canal prism. The off taking structures are designed for maximum withdrawal of the sediments from the parent canal depending upon the command areas.

Canal erosion

The sides and bed of an unlined canal are sometimes badly attacked by scouring water. This process is called erosion. Canal bends and sections downstream of structures in particular are susceptible to erosion, since local flow velocities can be very high and the direction of flow changes suddenly, causing turbulence (Food and Agricultural Organization (FAO), 1992).

2.8 Hydrologic Failure of Irrigation Canal

According to Laycock. (2007)either because of gates being wrongly operated upstream or downstream, too much water coming in at the head works a blockage downstream or excess rainwater flooding in during the rainy season, the canal will overflow. Overflows are controlled by a spillway or waste weir, often called escapes when used in a canal. Side escapes are located near to or integral with cross drainage structures, and as close as practicable to potential points of control such as cross regulators or siphon intakes which are liable to cause a backwater effect. Tail escapes are naturally enough located at the tail end of canals, usually integral with a flushing sluice gate for draining the canal for maintenance.

2.9 Estimation of Flood

Many hydrologic methods are available to estimate flood. The following alternative methods are used for estimation of flood: Rational method, Soil Conservation Service (SCS) methods, Empirical Method, Unit Hydrograph Method and Flood-frequency Method. The choice of a method for estimation of the flood primarily depends upon the importance of the work and available data (Ethiopian Road Authority(ERA),(2002;)Thus, in this thesis the Soil Conservation service (SCS) method are used and also the rational method is also explained for farther understanding of the literate in the area.

2.9.1 Rational Method

The Rational Method estimates the peak rate of runoff at a specific watershed location as a function of the drainage area, runoff coefficient, and mean rainfall intensity for duration equal to the time of concentration and Rational Method shall not be utilized for drainage areas greater than 50 ha(Ethiopian Road Authority(ERA), (2002).

The rational formula is expressed as:

$$Q = 0.00278CIA \qquad (2.20)$$

Where

Q = maximum rate of runoff, m^3/s

C = runoff coefficient representing a ratio of runoff to rainfall

I = average rainfall intensity for a duration equal to the time of concentration, for a selected

return period, mm/hr.
A = catchment area tributary to the design location, ha

Runoff Coefficient

Ohio Department of Transport (ODOT) (2005) stated that the ground cover and a host of other hydrologic abstractions considerably affect the coefficient. The rational equation in general relates the estimated peak discharge to a theoretical maximum of 100% runoff. The Values of C vary from 0.05 for flat sandy areas to 0.95 for impervious urban surfaces, and considerable knowledge of the catchment is needed in order to estimate an acceptable value. The coefficient of runoff also varies for different storms on the same catchment, and thus, using an average value for C, gives only a rough estimate of Qp in small uniform urban areas. If the basin contains varying amount of different land cover or other abstractions, a coefficient can be calculated through areal weighing as shown in equation below

$$WeightedC = \sum \frac{C_x A_x}{A_{total}} \qquad (2.21)$$

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

T**he rainfall intensity (I)** is the average rainfall rate in/hr for a selected return period that is based on a duration equal to the time of concentration (tc). Once a particular return period has been selected for design and a time of concentration has been calculated for the drainage area, the rainfall intensity can be determined from rainfall-intensity-duration (Ohio Department of Transport (ODOT), 2005).

2.9.2 SCS Method

As stated by United States Department of Agriculture (USDA) (1986) for calculating rates of runoff Soil Conservation Service (SCS) method requires the same basic data as the Rational Method: like catchment area, a runoff factor, time of concentration, and rainfall.
The Soil Conservation Service (SCS) runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad (2.22)$$

Where
Q = accumulated direct runoff (mm)
P = accumulated rainfall or potential maximum runoff (mm)

Ia = initial abstraction including surface storage, interception, evaporation and infiltration prior to runoff (mm)

S = potential maximum soil retention (mm)

$$S = \frac{25400}{CN} - 254 \qquad (2.23)$$

CN= SCS curve number

An empirical relationship used in the Soil Conservation Service (SCS) method for estimating Ia is

$$I_a = 0.2S \qquad (2.24)$$

Substituting 0.2S for Ia in Equation 3-1 the Soil Conservation Service (SCS) rainfall-runoff equation becomes Equation 2.22

Peak flow is then estimated with the following equation

$$Q_P = Q_U * A * Q_d$$

Where

Q_p =Peak flow (m^3/s)

Q_d =Runoff depth (mm)

A =Basin area km^2

Q_U = Unit peak flow (m^3\s\km^2

OPW (2012) Unit peak flow determined by

$$Q_u = 0.000431 * 10^{C_o + C_1 Log t_C + C_2 (\log t_c)^2})$$

t_c = Time of concentration

Time of concentration

Time of concentration (TC) is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet. Time of concentration is generally applied only to surface runoff and may be computed using many different methods. Time of concentration will vary depending upon slope and character of the watershed and the flow path.

The kirpich equation for time of concentration can be expressed as

$$t_c = 0.01947 L^{0.77} * s^{-0..386} \qquad (2.25)$$

Factors Affecting Runoff

Size

The size (area) of a drainage basin is the most important watershed characteristic affecting runoff. Determining the size of the drainage area that contributes to flow at the site of the drainage structure is a basic step in a hydrologic analysis regardless of the method used to evaluate flood flows. The drainage area, expressed in hectares or square miles, is frequently determined from field surveys, topographic maps, or aerial photographs.

Shape

The shape, or outline formed by the basin boundaries, affects the rate at which water is supplied to the main stream as it proceeds along its course from the runoff source to the site of the drainage structure. Long narrow watersheds generally give lower peak discharges than do fan or pear shaped basins.

Slope

The slope of a drainage basin is one of the major factors affecting the time of overland flow and concentration of rainfall Steep slopes tend to result in shorter response time and increase the discharge while flat slopes tend to result in longer response time and reduce the discharge.

Land Use

Changes in land use nearly always cause increases in surface water runoff. Of all the land use changes, urbanization is the most dominant factor affecting the hydrology of an area. Land use studies may be necessary to define present and future conditions with regard to urbanization or other changes expected to take place within the drainage basin.

A criterion of good drainage design is that future development and land use changes, which can reasonably be anticipated to occur during the design life of the drainage facility, be considered in the hydraulic analysis and estimation of design discharge.

2.10 Design Rainfall Analysis

For the computation of the design rainfall, the 24 hour annual maximum is taken to determine maximum rainfall for different return periods.

Probability distributions

There are many distributions, which could be used in hydrology.

The probability distribution functions most commonly used to estimate the rainfall frequency are normal, log-normal, log-Pearson type-III and Gumbel distributions (Singh et.al 2012).

Normal distribution

For normal distribution, the frequency factor K_T can be expressed by following equation (Chow, 1988)

$$K_T = \frac{X_T - \mu}{\delta} \tag{2.26}$$

This is the same as the standard normal variant z. The value of z corresponding to an exceedence of p

$(P = 1/T)$ Can be calculated by finding the value of an intermediate variable w:

$$W = \left[Ln\left(\frac{1}{P^2}\right) \right]^{1/2} \quad (0 < P \leq 0.5) \tag{2.27}$$

Log-normal distribution

For log-normal distribution, it is assumed that Y= ln X is normally distributed [the value of variant 'X' (rainfall) is replaced by its natural logarithm]. The expected value of rainfall '*XT*', at return period T, can be obtained from the relation

$$X_T = \exp Y_T \tag{2.28}$$

$$Y_T = \bar{y}(1 + C_{vy} k_T) \tag{2.29}$$

Where Y = is the mean and

C_{vy} = is the coefficient of variation of Y.

Return period

Return period was calculated by Weibull's plotting position formula (Chow et.al, 1988) by arranging one day maximum daily rainfall in descending order giving their respective rank as:

$$T = \frac{N+1}{R} \tag{2.30}$$

Log-Pearson type-III

In log-Pearson type-III distribution, the value of variant 'X' (rainfall) is transformed to logarithm (base 10). The expected value of rainfall '*XT*' can be obtained by the following formulae

$$X_T = Anti \log X \tag{2.31}$$

$$Log X = M + K_T S \tag{2.32}$$

Gumbel distribution

In Gumbel distribution, the expected rainfall '*XT*' is computed by the following formula

$$X_T = \overline{X}(1 + C_V K_T) \tag{2.33}$$

Where,

$\overline{X}$ =Mean of the observed rainfall,

CV =Coefficient of variation and

KT = Frequency factor

Testing the goodness of fit of probability distribution

As stated in Singh et.al (2012) to determine the goodness of fit of probability distribution, the expected values can be compared with the observed values by using chi-square equation.

$$X^2 = \sum_{i=1}^{k} \frac{\left(Q^i - E^i\right)}{Ei}$$

Where,

Oi = the observed rainfall and

Ei =the expected rainfall and

The best probability distribution function can be determined by comparing Chi square values obtained from each distribution and selecting the function that gives smallest chi-square value

2.11 Discharge Measuring Structures

There are so many structures used to measure discharge in the literature. The descriptions of following structures are taken from (Laycock, 2007).

Sharp-edged weirs

Sharp-edged weirs are used for accurate flow measurements, either rectangular or trapezoidal in profile.

Flumes

Flumes are a good type of measuring structure because they are not easily blocked with debris and they do not need much head to operate. They are a constriction in the canal, either at the sides (a throated flume) or in the bed (a sill) that causes the water to accelerate through its critical flow state (Anyemedu, 2007).

The height of water level upstream is measured, and this is related to the discharge which can be read off rating curves.

There are several common types including cut-throat, Parshall, venturi, H flumes and WSC flumes.

Cut-throat flumes

Cut-throat flumes are a simple but effective structure which is useful for indicating discharge in small canals. It is a low-head, critical flow structure and is not susceptible to blockage by debris. It is suitable for pre casting in concrete and can also be prefabricated in steel and backfilled with concrete,

Current Meters

Meters are measuring instruments that determine flow velocity or volume over time, usually by counting the revolutions of a rotating propell

3 Methodology used

3.1 General Description of the Study Area

3.1.1 The Awash River Basin

Awash River basin is the most utilized river basin in Ethiopia, and it has a catchment area of 112,696km^2, flowing 1200Km long and serves as home to 10.5 million inhabitants. It originates from Central West part of Ethiopia, on the High plateau near Ginchi town west of Addis Ababa in Ethiopia and flows along the rift valley into the Afar triangle, and it ends in the large depression of Lake Abbe on the border with Djibouti Awulachew,et.al (2007). Awash covers parts of the Amhara, Oromia, Afar, Somali regional states, and Dire Dawa, and Addis Ababa city administrative states of the country. The River basin has a lowest elevation of 210 m and a highest elevation of 4195 m. The total mean annual flow from the River basins is estimated to be 4.9 Billion Meter Cube (BMC) (Oromia Irrigation Development Authority (OIDA),2011).

3.1.2 Fentale Irrigation Project

Fentale is found at a distance of 200 km from Addis Ababa and 100 km from Adama town on Asphalt road. Part of the project area located in Boset Woreda (Kawa and Huluko) is located 40-45 km off the asphalt road to east of (Fenetale) Metehara town. Generally, both woredas are low land agro-ecological zone characterized by low annual precipitation and high temperature and geographically in the main rift-valley system in east Showa zone (Oromia Irrigation Development Authority (OIDA), 2011).

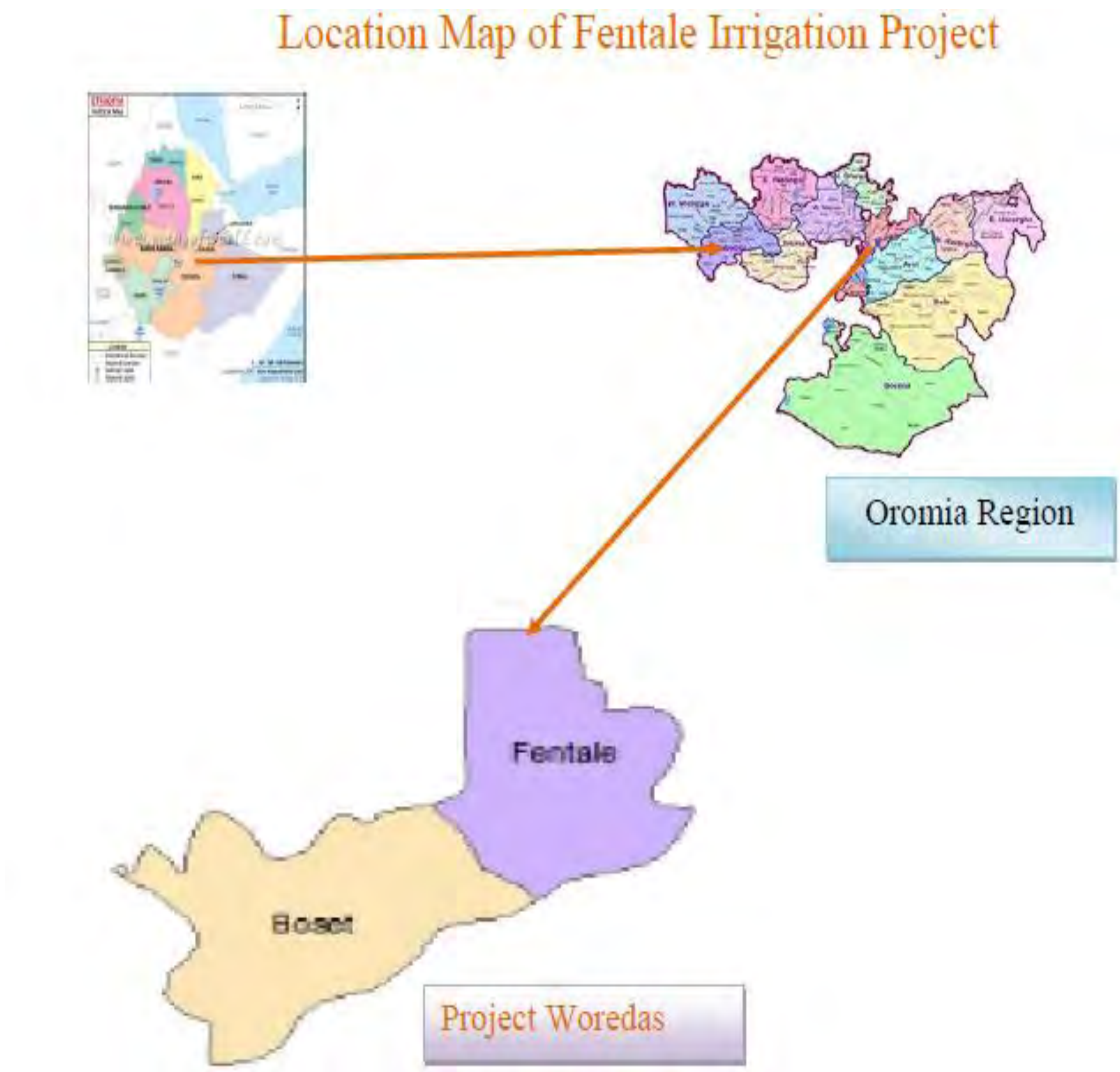


Figure 3.1 Location map of the study area

The command area is situated west of Metehara town and it is bound to the foot slopes of mount Fentale on the north, Lake Beseka on the east, Rift valley escarpment on the west and rocky land (young lava flow) on the south. More precisely it falls in between 80 50' and 90 04' N latitude and 390 41' and 390 52' E longitude. The study has covered gross area of about 27,000 hectares the altitude ranging from 970 to 1020 m above sea level (Oromia Irrigation Development Authority (OIDA), 2011).

Topography and project boundary

The total area covered was about 27,000 hectares out of which areas for the expansion of Metehara Sugar Factory (MSF) covered 11,000 hectares. The project area covers Kawa and Huluko from Boset district; and Gidara, Turo Bedenota and Tututi kebeles from Fentale district (Oromia Irrigation Development Authority (OIDA), 2011).

Accessibility

The Addis Ababa Harar asphalt road, which passes through the project area and through the nearest major town, Metehara, provides the main access to the project area. The project area is also accessed by the Addis Ababa to Djibouti railway line. The all-weather road to Sabober hill in the northeast of the project area and a rough road to Berehet in the northwest of the project area provide additional access to the interiors of the project area. Both roads branch out from the main asphalt road at a point very near to the intersection of the asphalt road and railway line, within the project area. Owing to the flat topography and scattered trees, almost all parts of the project area are accessible. The project area covers Kawa and Huluko from Boset district; and Gidara, Turo Bedenota and Tututi kebeles from Fentale district. The diversion site on Awash River and the Kawa command area can be accessed via Metehara Sugar Factory (MSF) 25 km on all-weather road and 5 km dry weather access road (Oromia Irrigation Development Authority (OIDA), 2011).

Climate

The mean monthly minimum and maximum temperature in the project area varies from 10 ^{0}C to 38.3 ^{0}C respectively. The mean minimum temperature is recorded in December and the maximum is recorded in May. May and June are the hottest months of the year. Annual rainfall in the project area is about 500 mm. Mean monthly rainfall varies from 4.8 mm to 124 mm. The highest rainfall occurs in July and August; while the lowest rainfall occurs in December. Mean monthly relative humidity varies from 32% to 49%.

Soil

According to the design document of Oromia Irrigation Development Authority (OIDA) (2011) the texture of the soils in the study area is loam to sandy loam, the dominant being sandy loam.

The coarser soil texture may necessitate frequent application of irrigation water due to its relatively low water holding capacity and the soils in the study area have moderately developed blocky structure. The available water holding capacity of the soils on horizon basis ranges from 8 to 25 cm/m and the aggregate water holding capacity of the soils by profile vary from 10 to 15 cm/m. This indicates that the water holding capacity of the soils is low to medium. The low water holding capacity of the soils requires frequent application of irrigation and best be by water saving technologies. The infiltration and hydraulic conductivity of the soils in the study area is in the range of 0.7 to 3.6 cm/hr and 0.2 to 1.8 m/day, respectively Oromia Irrigation Development Authority (OIDA) (2011).The acceptable infiltration and hydraulic conductivity rates for surface irrigation are in the range of 0.5 to 7 cm/hr and 0.2 to 1.4 m/day, respectively. Therefore, the soils of the study area have acceptable hydraulic conductivity rate for surface irrigation.

The command area

The command areas of the project are scattered along the main canal and divided as south and north blocks. Hence, the areas that establish under south block are Kawa, Huluko, Burka Direseden, Bomisa, Ammuma and South Illala. Similarly under North block are North Illala, Alaka, Gelcha and Haro-arba. Consequently, the areas that mentioned on the first block are found between headwork and asphalt Road/ Highway Djibouti to Addis Ababa. And the area of the second Block are lay under Mountain Sebebor and Fentale including the area that located between the highway and Railway and beyond Rail way both right and left side of the main and branch canals. Therefore, in all Command areas irrigation water through whole canal system can flow by gravity except Kawa command area, which is irrigated by pump.

Water Resource and Diversion

The water source for the whole command area of the project is the Awash River. Irrigation water for abstraction is raised by weir at the headwork of the project and conveyed through conveyance main canal of 49.3 km and two branch canal 38 & 31 km with pertinent structures and pump including electro mechanical equipment for Kawa command area.

3.2 Methodological Framework

The methodological framework that was used in this study is outlined in Figure 3.2. The details of the data collection methods, software and mathematical equations and method of analysis used to understand the major cause of failure of irrigation canals are specified. More specifically understanding hydraulic and hydrologic failures of irrigation canals and exploring its remedial measures have been done. Brief description of the methods used in this study is presented in Figure 3:2.

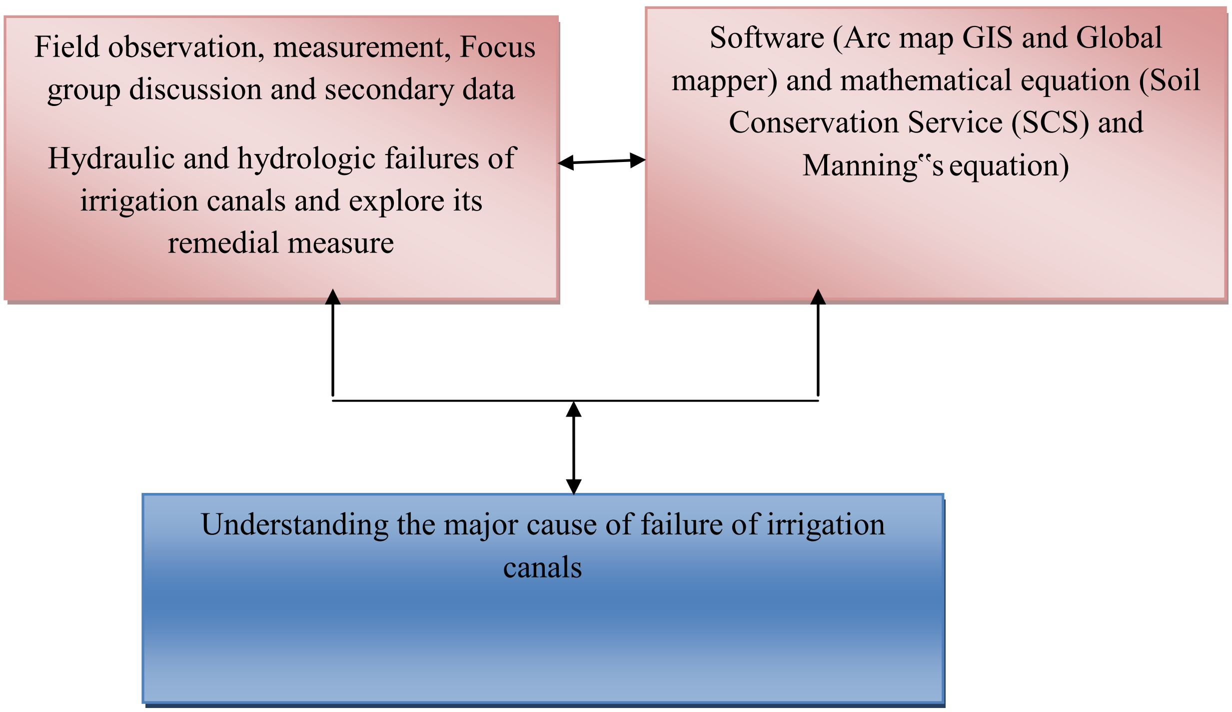


Figure 3.2 Methodological framework

3.3 Data Collection

Primary and secondary data were collected through employing different data collection instruments. Data were collected through observation; field data measurement and focused group discussion. Focused group discussion with community representatives and concerned government professionals were made to gather information to have a general insight of the magnitude of the flood during summer session, the miss use of the geo membrane and the illegal water abstraction. Field observation was made to visualize the magnitudes of the damaged occurred on the irrigation structures like drop, culvert and cross drainage and also it helped to decide where canal discharge measurements would be taken. Field data measurements were made using meter and current meter to measure the canal discharge. The major data collected basically included rainfall data, Digital Elevation Model (DEM) data, XY coordinate of the canal root, depth of water, bottom width of the canal and velocity.

3.3.1 Primary Data Collection

Primary data were collected through observation and field measurement .Frequent field observation at Fentale irrigation projects were made to identify where the canal fails and also pictures that show the magnitudes of the failure of the irrigation canals of the scheme were captured using digital camera. Above all, the observation was used to judge place where the canal discharge measure would be taken. Then, after carefully selecting the place where the canal discharge would be measured, current meter was used to measure the velocity at several points in a cross section and then the velocities were multiplied by respective subsection areas to obtain flow rate (discharge).The points of measurement taken were at street reaches of the canals. The measurement using current meter was made in collaboration with an expert from Oromia Irrigation Development Authority (OIDA).

3.3.2 Secondary Data Collection

Full information that include design discharge, velocity, curve number, design flood for cross drainage, shape of the designed canal, xy coordinate of the canal root and general descriptions of the project were collected from the design document of Oromia Irrigation Development Authority (OIDA). Rainfall data were collected from National Meteorological Services Agency and Digital Elevation Model (DEM) 30 m x 30 m data spacing with the Universal Transverse Mercator (UTM) projection were collected from ministry of water and irrigation GIS department. A Digital Elevation Model (DEM) is a raster representation of a continuous surface, usually referring to the surface of the earth. The accuracy of Digital Elevation Model (DEM) data is determined primarily by the resolution (distance between sample points).Grids that are derived from the Digital Elevation Model (DEM) are flow direction (flow network), flow accumulation (drainage area), and flow length

3.4 .Hydraulic Failure of Irrigation Canal

Analysis of silt along the canal has an advantage to maintain the irrigation canal and minimizing shortage of water due to silt accumulation (Munir 2011).Munir recommends the used of Manning‟s formula to determine the roughness coefficient of irrigation canal. Therefore, this study used the Manning‟s formula to analyze the hydraulic failure of the irrigation canal. The actual value of Manning‟s roughness coefficient, wetted perimeter, velocity and discharge were compared against the designed value. Accordingly the roughness coefficient of the irrigation canal was determined by the following Manning‟s formula:

$$Q = \frac{1}{n} A R^{2/3} S^{1/2} \tag{3.1}$$

$$n = \left(\frac{A R^{2/3} S^{1/2}}{Q} \right) \tag{3.2}$$

$$V = \frac{1}{n} R^{2/3} S^{1/2} \tag{3.3}$$

Where,

Q = discharge of the channel (m^3/s)
V =velocity (m/s)
R = (A/p) = Hydraulic radius (m)
A = Wetted cross-sectional area (m^2)
S = bed slope (-)
N = Manning‟s roughness coefficient (-)

3.5 Hydrologic Failure of Irrigation Canal

Hydrologic failure of irrigation canals would be caused by underestimations of peak flood that crosses the canals. The area of drainage basin is the most important watershed characteristics affecting runoff. Determining the size of the drainage area that contributes to flow at the site of the drainage structure is a basic step in hydrologic analysis to estimate flood flows Therefore in this study arc map GIS and Global Mapper were used to determine the catchment area of the stream that the canal crosses. And then Soil Conservation Service (SCS) method was used for this study to determine flood. According to Ethiopian Road Authority (ERA) (2002) of different possible hydrologic methods only the Rational and Soil Conservation Service (SCS) methods are applicable to the whole country. The hydrologic methods approved by ERA stated that rational method shall be used only for catchment areas less than 50 hectares and Soil Conservation Service (SCS) methods shall be used for catchment areas greater than 50 hectares. Therefore, SCS method was used, since the catchment area of Fentale irrigation project that cross the main canal is more than 50hectares.

Arc Map GIS

Arc Map GIS software, version 10.1 was used. From Arc Map GIS software, the following Arc hydro Tools were used. And from this tool terrain preprocessing and watershed processing were used to determining the catchments area, the longest flow path and the slop.

Figure 13.3 Arc Hydro tools

DEM Processing

A DEM is a sampled array of elevations for ground positions that are normally at regularly spaced intervals. The DEM is clipped to the area of interest using Global mapper software and imported to GIS database. The DEM is optimized through integration of the existing digitized drainage to obtain a final DEM.

Terrain Preprocessing

Terrain Preprocessing were used (Digital Elevation Model) DEM to identify the surface drainage pattern. The (Digital Elevation Model) DEM and its derivatives were used to delineate watershed and generate stream network of the study area. The steps used in the terrain preprocessing menu were DEM manipulation, flow direction, flow accumulation, stream definition, stream segmentation, catchment grid delineation, catchment polygon processing, drainage line processing and ad joint catchment processing.

Watershed Processing

Watershed processing was used after the Terrain Preprocessing to allow fast delineation of the watershed. The steps used in the watershed processing were batch watershed delineation and longest flow path watershed

Global Mapper

In this study Global Mapper was used to determine the canal root and which was extracted to Arc Map GIS

SCS method

The SCS runoff equation was used to estimate direct runoff from 24-hour or 1-day storm rainfall. The following equation was used to calculate the accumulated direct runoff.

$$Q_d = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{3.4}$$

Where

Q_d = accumulated direct runoff (mm)

P = accumulated rainfall or potential maximum runoff (mm)

Ia = initial abstraction including surface storage, interception, evaporation and infiltration prior to runoff (mm)

S = potential maximum soil retention (mm)

The relation between Ia and S was developed by analyzing the rainfall and runoff data from experimental small watersheds and is expressed as Ia=0.2S.The SCS method was represented as

$$Q_d = \frac{(p-0.2S)^2}{P+0.8S} \tag{3.5}$$

The potential maximum retention storage S of watershed was related to a CN, which was a function of land use, land treatments, soil type and antecedent moisture condition of watershed. The CN was dimensionless and its value varies from 0 to 100. In this study I used the curve number from design document The S-value was obtained from CN by using the relationship

$$S = \frac{25400}{CN} - 254 \tag{3.6}$$

The accumulated rainfall or potential maximum runoff was determined from maximum 24-hour rainfall. In this study, I used Gumbel (extreme value type I) distribution method for annual daily highest rainfall analysis of determined return period T years. For the good reason, Gumbel (extreme value type I) was the best distribution fit to the actual data. Therefore, the following equation was used to calculate maximum value of event corresponding to return period T.

$$X_T = \overline{X}(1 + C_V K_T) \tag{3.7}$$

Where,

X_T = maximum value of event corresponding to return period T
$\overline{X}$ = mean of the observed rainfall,
CV = coefficient of variation and
KT = frequency factor

The frequency factor

The frequency factor was depends upon the return period T and the assumed frequency distribution. The expected value of annual maximum daily rainfall for the same return periods were computed for determining the best probability distributions **(**Singh et al 2012). Accordingly frequency factor equation used in this study was simplified as follows:

$$K_T = -\frac{\sqrt{6}}{\prod}\{0.5772 + \ln[\ln(\frac{T}{T-1})]\} \tag{3.8}$$

Testing the goodness of fit of probability distribution

In the study the expected values of maximum rainfall were calculated by two probability distributions, log-Pearson type-III and Gumbel distribution at different selected probabilities.

Among these two distributions, the best fit distributions were decided by chi-square test for goodness of fit to observed values. The chi-square test statistic was given by the following equation

$$X^2 = \sum_{i=1}^{k} \frac{\left(Q^i - E^i\right)}{Ei} \quad (3.9)$$

Where,

Oi = observed rainfall
Ei = expected rainfall
The best probability distribution function was determined by comparing Chi-square values obtained from each distribution and selecting the function that gives smallest chi-square value

As discussed above Peak flow was then estimated as

$$Q_P = Q_U * A * Q_d \quad (3.10)$$

Where

Q_p =Peak flow (m^3/s)

Q_d =Runoff depth (mm)

A =basin area km^2

Q_U =Unit peak flow (m^3\s\km^2)

The unit peak flow was determined by

$$Q_u = 0.000431 * 10^{C_o + C_1 Log t_C + C_2 (\log t_c)^2} \quad (3.11)$$

Where

t_c = Time of concentration

C_0, C_1, $C_{2=}$ coefficients, these were determined from a function of the 24 hour rainfall distribution type and $I_{a/P}$ ratios

Time of concentration

Time of concentration (tc) is the time required for runoff to travel from the hydraulically most distant point in the watershed to the outlet. The hydraulically most distant point is the point with the longest travel time to the watershed outlet, and not necessarily the point with the longest flow distance to the outlet. Kirpich equation were used as

$$t_c = 0.01947 L^{0.77} * s^{-0..386} \quad (3.12)$$

Where

L = longest flow path of the catchment

S =slope of the catchment

3.6 Data Analysis and Interpretation

The analysis of data is made both during and after data collection. All the information collected from field observation, field data measurement focused grouped discussion and documents were used to explain the major cause of failure of Fentale irrigation canal. The method of analysis carried out in this study comprises both qualitative and quantitative analysis. Qualitative information obtained from field observation and discussions were analyzed through qualitative descriptions. The quantitative data obtained from field measurement and documents were analyzed using mathematical equations like manning"s and Soil Conservation Service(SCS).And also, software like arc map Geographic Information system (GIS) and global mapper .was used for the quantitative analysis. The data were analyzed using explanatory correlation design to explain the change observed in one variable is reflected in changes in the other.

4 Results and Discussions

4.1 General Description of the Main Canal

Fentale irrigation project was designed and constructed to take maximum of 18m^3/sec of water from the river by Diversion Weir about 4km upstream of Bole-district. The diverted irrigation water is conveyed through the Main Canal of total length about 49.3km. At 34.650km a Branch Canal (BC -I) is bifurcates towards the Haro-arbo area and the other Branch Canal-II (BC-II) going towards Galcha (North Block) after 49.3km. Irrigation water from Awash River was conveyed and distributed to the farms through a network of canals and pipes by gravity for all of the command areas except Kawa. The major conveyance systems were summarized in Table 4.1

Table 4.1 The major conveyance system

Item No.	Canal Name	Chain age (km)	
		From	To
1	Main Canal before Asphalt Road	0+000	42+740
2	Main Canal after Asphalt Road	42+740	49+300
3	Branch Canal 1	0+000	17+758
4	Branch Canal 2	0+000	31 857*
5	Branch Canal 2-1	0+000	17+325*

Source: - Design Document of Oromia Irrigation Development Authority 2011

Geology of main canal before asphalt road (42.7km)

The Geological formation of the canal rout of 43km investigation is made based on the excavated trench. The geology and soil conditions of the area vary in a short distance laterally and vertically, significantly due to variability in mode of eruption of volcanic events at different levels of topography before these events. Material description along the main canal line were summarized as in annex 3

Geology of main canal after asphalt road

The geology and soil conditions of this part of the main canal stretch has similarity with the stretch upstream of the asphalt road, where the geology vary in a short distance, significantly due to variability in mode of eruption of volcanic events at different levels of topography

Type and shape of the canal section

In areas covered by soil units, the hydraulic design of the canal differ in sections, trapezoidal shaped with side slope of 1.5 H x 1 V and 1.0 H x 1.0 V is adopted. For stretches covered by sound rocks difficult to excavate, rectangular shaped retaining wall cross section is designed. Hence the hydraulic design has considered three types of section. Canal stretches of permeable soil formation; geo-membrane lining with a protection of boulders covered with geo-mesh is adopted on selected parts.

Discharge

The main canal off-taking from the diversion weir and serving the command is designed for 24 hour continuous flow. The total length covers 49.3km.

For the purpose of canal capacity, the canal up to Asphalt road has been divided in to three reaches based on the discharge capacity:-

- From the head regulator to about 17.525km in Gidara PA
- From 17.525km to 26.9km at Bomisa
- From 26.9km to 42.7km Asphalt Road at Ilala
- From 42.7km to 49.3km up to division box to Alaka and Gelcha command areas.

Discharge capacity varies within the canal stretch, 18m^3/sec, 17m^3/sec, 15m^3/sec and 7.22m^3/sec for different sections within 49.3km to covey sufficient irrigation water to the command areas located in both south and north farms of the project. However, according to the measured discharge, the discharge released at each point showed different result. The measured discharges during the research at the above points were 6.065m^3/sec, 5.321m^3/sec3.525m^3/sec and 0.902m^3/sec respectively. Thus, as the discharge decrease the command area will decrease .That is why the total estimated command area is 6000ha while the planed hectare was 16,000ha.

Figure 4.1Main canal discharge measurement at Fentale irrigation project

According to the collected data from frequent field visit, the major causes for decrease of the discharge are canal silt up; illegal water abstraction; growing of weeds along the canal and the slues get (which helps to remove the silt) at the head work was damaged. For the sake of clarity and sequence the aforementioned problems are discussed.

4.1. Failure of Irrigation Canals in Fentale Irrigation Project

As the results of the data collected from field observation, document analysis and discussion with community there are serious failure of irrigation canals in Fentale irrigation project. During field observation, it was possible to noticed serious damage of irrigation structure like drop; canal silt up, weed growing along the canal, over toping of canal; illegal water abstraction along the main canal and misuse of the geo membrane along primary canal are the major failures of Fentale irrigation project.

Moreover it was possible to notice that there was serious illegal water abstraction at different place along the main canal and primary canal Figure 4.2. According to the result obtained from analysis of the design document of Oromia Irrgation Developemt Authority (OIDA)(2011), filed observation and discussion with the community, the major causes for the illegal water abstraction by the community was lack of adequate division box and the major cause for the misuse of the geo membrane Figure4.5 was lack of community awareness.

Figure 4.2 Illegal water abstractions at Fentale Irrigation project

Figure 4.3Discussion with the community at Fentale irrigation project

Figure 4.4 Silt up and weed growing along the canals

Figure 4.5 Miss use of Geo-membrane

4.2 Causes of Failure

Failure of Fentale irrigation project is clearly indicated in Figure 4.4 and 4.5.The main causes for the failure of Fentale irrigation canals are problem of design, construction and lack of community awareness. Although the emphasis of this study is on hydraulic and hydrologic failure of the canal it is better to pinpoint other causes observed during the field visit.

4.3 Hydraulic Failure of the Irrigation Canal

Hydraulic parameters change has a great effect on failure of irrigation canals. Among these parameters, manning"s roughness coefficient (*n*) is the major one. In this study, the hydraulic failure of irrigation canals was manipulated on manning"s roughness coefficient (*n*).

The values of the calculated *n* for different sections on the main canal by using Manning"s formula were between 0.039-0.072. However, the designed values on Table 4.2 range from 0.017-0.03 for the same canal on which *n* was calculated. This is as result of the decreasing discharge, velocity and the irregularity of the area. Other study conducted by Tefera (2013) on hydraulic effect of conveyance efficiency of Fentale irrigation project has also showed an increase in value of manning"s roughness coefficient (*n*) values of the calculated *n* for different sections on the main canal according to Tefera were between 0.0226-0.0637.

According to Mohamed (2014) the presence of many decreasing of flow along canals and irregularity of its cross sections were the main reasons of Manning„s roughness coefficient high values. The difference on the value of calculated n with the current calculated value is as the result of miner silt removed from the canal by Oromia Irrigation Development Authority (OIDA).

ARC-ILI(2003)The maximum and minimum value of manning"s roughness coefficient is between 0.01-0.16.

The summary of comparison of calculated and designed value for *n* can be seen in the Table 4. 2.

Table 4.2 Comparison of calculated and designed value for *n* for Fentale

Canal section	Bed slope	Bed width (m)	Water depth (m)	Side slope	Area (m2)	Wetted perimeter (m)	Hydraulic Radius (m)	Flow discharge (m3/s)	Roughness coefficient **n** calculated	Designed roughness coefficient **n**	Velocity measured
From the head regulator to bout17.525 Km	0.0012	8.5	1.061	1.5	10.71	12.325	0.869	6.065	0.056	0.02	0.566
From 17.525 Km to 26.9 km	0.001	10.15	0.931	1.5	10.75	13.507	0.796	5.321	0.055	0.03	0.495
From 26.9 km to 42.7 km	0.001	12.75	0.75	1.5	10.406	15.454	0.677	3.507	0.072	0.02	0.337
From 42.7 km to 49.3 km	0.001	8.25	0.3	1.5	2.61	9.33	0.28	0.902	0.039	0.017	0.346

The manning"s *n* values are very important reference resource for engineering of canal construction design, especially in calculating velocity (Chang.et.al, 2010) On Table4.3 flow discharge and velocity were considered at different sections of the canal and showed considerable decreasing result. In comparison with table 4.2 above there is inverse proportion of the manning"s roughness coefficient to velocity and discharge.

Table 4.3 calculated velocities for different sections on the main canal were between 1.048-0.62 m/s However, the measured velocity range from 0.566-0.337 m/s for the same canal on which velocity was calculated Similarly, designed discharge capacity varies within the canal stretch were, 18m^3/sec, 17m^3/sec, 15m^3/sec and 7.22m^3/sec for different sections along the canals. However, according to the measured discharge, the discharge released at each point showed different result. The measured discharges during the research at the above points were 6.065m^3/sec, 5.321m^3/sec3.525m^3/sec and 0.902m^3/sec respectively. According to the findings, an increase on the values of the calculated manning"s roughness coefficient (**n**) causes decreasing of discharge and velocity. The decrease in velocity and discharge causes the accumulation of silt along the canal.

Accumulation of silt along the canal could usually causes weeds to grow along the canal and other major effect of silt deposition in an irrigation canal is the reduction in flow carrying capacity of the canal and raise in water levels. This reduction in the flow carrying capacity results in inadequate and inequitable water supply to off taking canals and outlets. The accumulation of silt in the canal could also causes overtopping. The overtopping that come duo to the silt deposit has eroded the back fill of the canal structure. Therefore, this all confirmed the presence of serious hydraulic failure of Fentale irrigation projects.

Table 4.3 Decreasing of flow discharge decreases the velocity of the main canal

Canal section	Bed slope Designed	Bed width (m)	Water depth (m) Designed	Water depth(m) calculate	Side slope	Area (m2) Designed	Area (m^2) Calculated	Wetted perimeter (m)	Hydraulic Radius (m)	Flow discharge measured (m^3/s	Flow discharge designed (m3/s)	Velocity designed(m/s)	Velocity calculated
From the head regulator to bout17.525 Km	0.0012	8.5	1.2	1.58	1.5	12.36	17.17	14.19	1.21	6.065	18	1.45	1.048
From 17.525 Km to 26.9 km	0.001	10.15	1.44	1.9	1.5	17.7	24.7	17	1.453	5.321	17	0.96	0.688
From 26.9 km to 42.7 km	0.001	12.75	1.31	1.36	1.5	10.53	20.14	17.66	1.14	3.525	15	1.42	0.62
From 42.7 km to 49.3 km	0.001	8.25	0.69	0.9	1.5	7.86	8.64	2.96	2.625	0.902	7.2	0.9	0.833

4.4 Hydrologic Failure of the Irrigation Canal

To identify the hydrologic failure of the irrigation canal GIS, Global Mapper and Digital Elevation Model (DEM) data were used to determine the hydrologic parameters such as area, longest flow path, slop, and time of concentration. The DEM is exported to the selected area using Global mapper software and imported to GIS database. The grids derived from the DEM are flow direction, flow accumulation, drainage area and longest flow path.

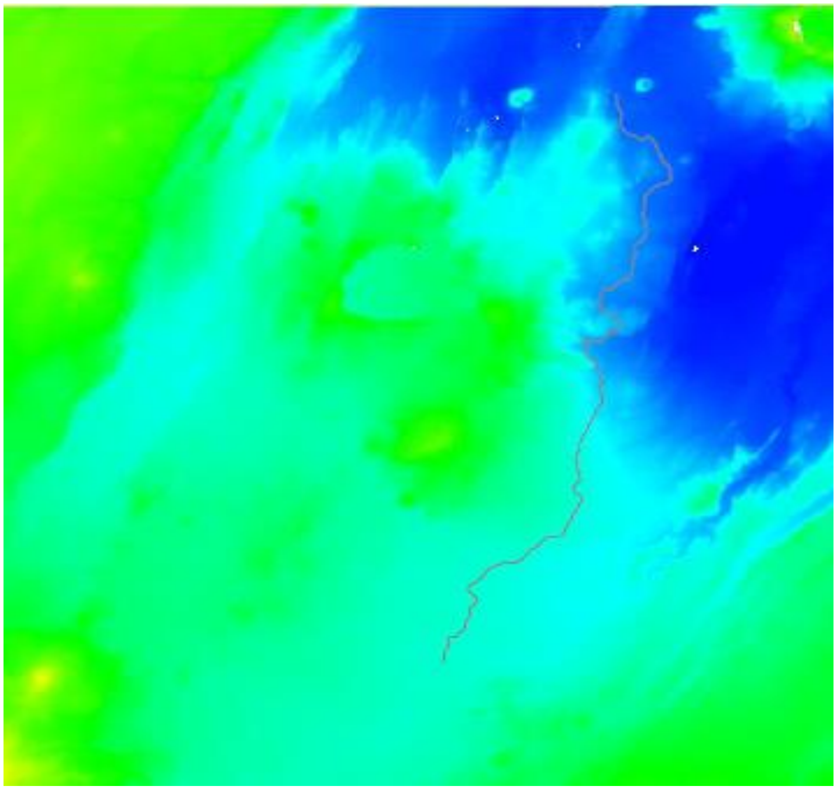
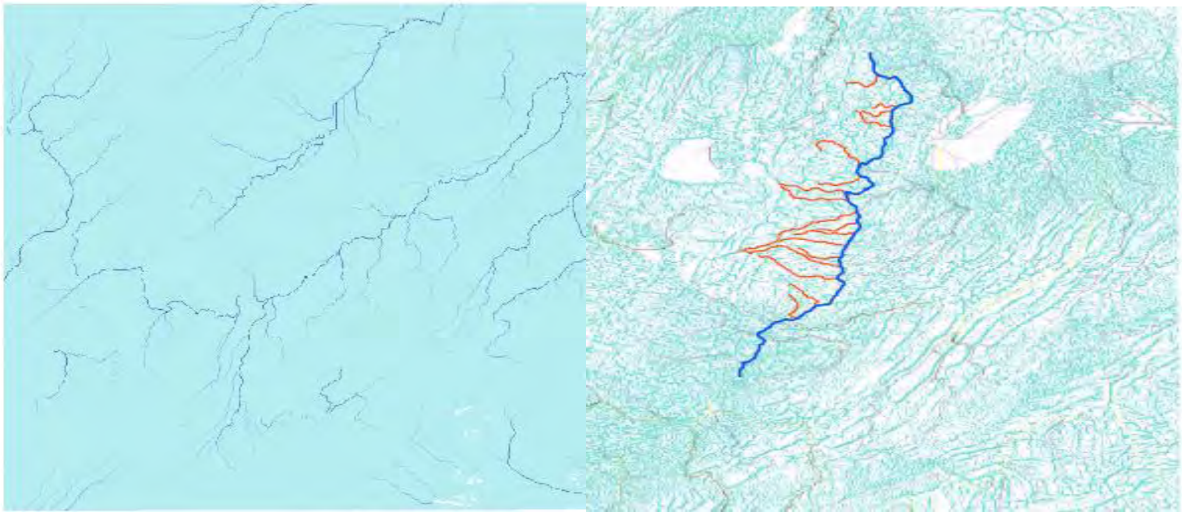


Figure 4.6 Digital Elevation Model (DEM) Fentale Irrigation Project

Flow Direction of the study area

The flow direction of the study area was extracted from the raw Digital Elevation Model (DEM) using the arc hydro tool. The values in the cells of the flow direction grid indicate the direction of the steepest descent from that cell. Accordingly, water in a given cell can flow to one of its eight steepest descent direction as can be seen in Figure 4.7(a)



a) b)

Figure 4.7 a Flow Direction of Fentale Irrigation Project. b) Longest flow path or streams that cross the main canal

Flow Accumulation

The Flow accumulation grid is the core grid used in the stream delineation.Firure4.8 was extracted from the flow direction grid. The flow accumulation records the number of cells that drain into an individual cell in the grid.

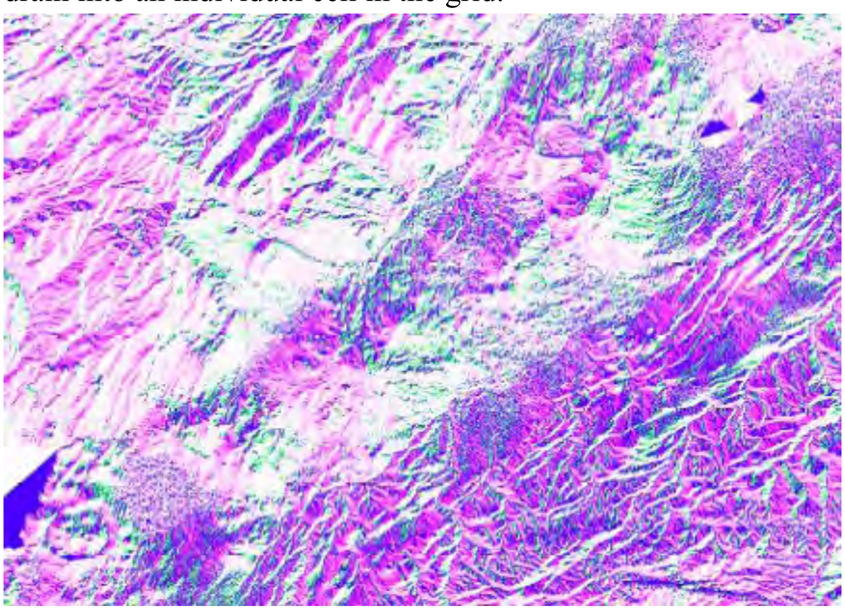


Figure 4.8 Flow Acommulation of Fentale Irrigation Project

Longest Flow Path and Slope Determination

Generating the longest flow path would help us to determine the slop and time of concentration. Based on this rationale, Figure 4.7 (b) was generated from Arch Hydro tools to determine the longest flow path of the watershed

Adjoint Catchment Processing

As stated in the Arc Hydro tools Adjoint Catchment Processing generates the aggregated upstream catchments from the"Catchment" feature class. For each catchment that is not a head catchment, a polygon representing the whole upstream area draining to its inlet point is constructed and stored in a feature class that has an "Adjoint Catchment" tag as in Figure4:9 This feature class is used to speed up the point delineation process.

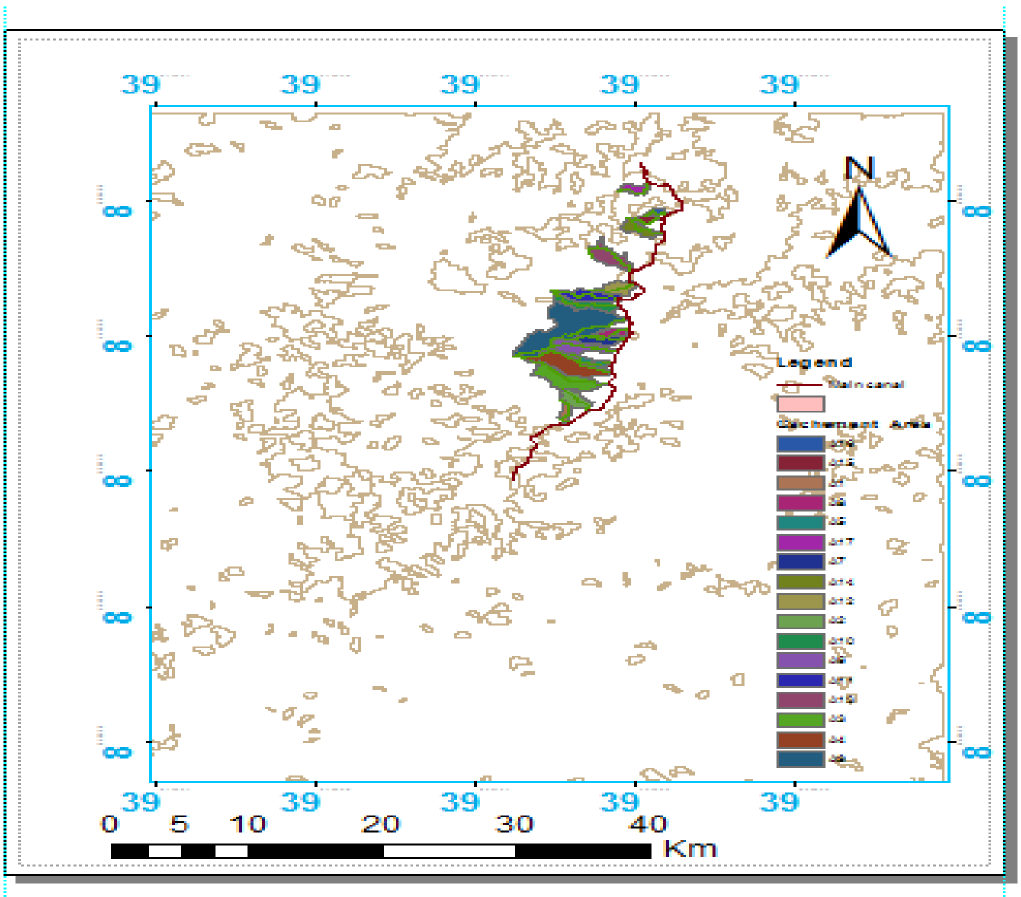


Figure 4.9 Catchment area of the stream that crosses the main canal

From the above steps, GIS database are developed to estimate watershed parameters, which are catchment area, length of longest flow path, slope of longest flow path and time of concentration.as presented in Table4.4

Table 4.4 Detailed data Analysis of Cross Drainages Calculated

No	Area (ha)	Length (m)	Area (Km^2)	H1 (m)	H2 (m)	H1-H2	S %	tc
1	148.0642	3683.79	1.48	1238.189	1176.216	61.973	1.682318	8.875893
2	333.1445	3595.32	3.33	1266.726	1176.469	90.257	2.5104	7.464204
3	769.7657	7131.75	7.70	1463.334	1172.285	291.049	4.081032	10.48503
4	894.2742	8384.37	8.94	1601.653	1162.545	439.108	5.237221	10.78612
5	179.1914	3841.71	1.79	1305.428	1154.626	150.802	3.925384	6.610155
6	470.2722	5347.24	4.70	1356.417	1134.982	221.435	4.141108	8.352546
7	286.8744	4983.62	2.87	1313.187	1101.872	211.315	4.240194	7.839789
8	178.3501	3493.83	1.78	1238.176	1085.134	153.042	4.380356	5.889592
9	2041.772	10981.33	20.42	1709.81	1068.401	641.409	5.840904	12.72943
10	422.3195	5957.81	4.22	1527.823	1065.885	461.938	7.753481	7.125858
11	478.6849	5945.34	4.79	1560.666	1064.192	496.474	8.350646	6.913499
12	328.9381	3871.84	3.29	1248.132	1046.659	201.473	5.203551	5.964449
13	516.5422	6223.76	5.17	1216.325	1047.632	168.693	2.710468	11.05685
14	300.3348	4066.41	3.00	1137.552	1031.259	106.293	2.61393	8.079469
15	117.7784	2775.79	1.18	1131.163	1025.626	105.537	3.802047	5.210596
16	92.54013	2389.91	0.93	1109.607	1021.737	87.87	3.676701	4.703849
17	189.2866	3512.82	1.89	1227.194	1011.695	215.499	6.134642	5.193193

Table 4.5 Detailed data Analysis of Cross Drainages design

No	A(ha)	L(m)	A(Km^2)	H1(m)	H2(m)	H1-H2	S%	tc
1	113.36	3467.16	1.1336	1240	1180	60	0.02	0.82
2	178.414	1470.89	1.78414	1200	1170	30	0.02	0.4
3	202.52	5200	2.0252	1530	1250	280	0.05	0.938
4	330.128	7120	3.30128	1490	1180	310	0.04	1.407
5	97.32	973.96	0.9732	1310	1170	140	0.14	0.14
6	192.7	5263.38	1.927	1400	1170	230	0.04	0.8
7	59	1723.14	0.59	1300	1160	140	0.08	0.27
8	50.3	1470.39	0.503	1300	1160	140	0.10	0.22
9	56.05	1208.08	0.5605	1240	1150	90	0.07	0.21
10	152.3	4616.42	1.523	1350	1125	225	0.05	0.69
11	20.493	1189.19	0.20493	1190	1095	95	0.08	0.2
12	82.18	1316.56	0.8218	1275	1100	175	0.13	0.18
13	280.15	6120.94	2.8015	1700	1080	620	0.10	0.65
14	125.806	4660.06	1.25806	1440	1075	365	0.08	0.58
15	134.5	4992.43	1.345	1510	1130	380	0.08	0.62
16	49.68	396.39	0.4968	1190	1080	110	0.28	0.05
17	33.21	282.18	0.3321	1190	1080	110	0.39	0.04

The Peak discharges for return periods of 5, 10, 20, 25 and 50 years are calculated by using Soil Conservation service (SCS) method.as showed in Table4.5 And the comparison of the design document and the calculated flood estimation of the returned period of flood can be seen from table 4.5 and Table4.6.

Table 4.5 Values of Calculated Flood for Cross Drainages.

Stream Design Discharge	Q5	Q10	Q15	Q20	Q25	Q50
stream1	15.20385	19.55	21.65	23.7832685	24.8318	29.7307
stream2	36.2243	46.58	51.58	56.6653872	59.1637	70.8355
stream3	74.50672	95.81	106.09	116.550273	121.689	145.696
stream4	85.65948	110.16	121.97	133.996447	139.904	167.504
stream5	20.23104	26.02	28.81	31.6472483	33.0425	39.5612
stream6	49.29325	63.39	70.19	77.1090422	80.5087	96.3914
stream7	30.70442	39.48	43.72	48.0306883	50.1483	60.0416
stream8	20.82779	26.78	29.66	32.5807445	34.0172	40.7281
stream9	183.5689	236.06	261.39	287.155333	299.816	358.963
stream10	46.59635	59.92	66.35	72.890305	76.1039	91.1177
stream11	53.31211	68.56	75.91	83.3957118	87.0725	104.25
stream12	38.27534	49.22	54.50	59.873812	62.5136	74.8462
stream13	49.02316	63.04	69.81	76.6865477	80.0676	95.8633
stream14	31.82981	40.93	45.32	49.7911237	51.9864	62.2422
stream15	14.22661	18.30	20.26	22.2545685	23.2357	27.8197
stream16	11.47901	14.76	16.35	17.956517	18.7482	22.4468
stream17	22.8846	29.43	32.59	35.7981985	37.3765	44.7501

Note: Q_5 is peak discharge for return periods (m^3/sec).

Table 4.6 Values of Design Flood for Cross Drainages

Streams/Design Discharge	Q_5	Q_{10}	Q_{15}	Q_{20}	Q_{25}	Q_{50}
Stream 1	11.64	13.80	15.06	16.00	16.74	19.07
Stream 2	19.40	23.00	25.10	26.67	27.89	31.78
Stream 3	19.6	23.3	25.4	27	28.2	32.2
Stream 4	31.6	37.4	40.8	43.4	45.4	51.7
Stream 5	10.97	13.00	14.19	15.08	15.77	17.96
Stream 6	20.19	23.94	26.13	27.76	29.03	33.08
Stream 7	6.31	7.48	8.16	8.67	9.07	10.33
Stream 8	5.86	6.95	7.59	8.06	8.43	9.61
Stream 9	5.04	5.97	6.52	6.93	7.24	8.25
Stream 10	16.73	19.84	21.65	23.00	24.06	27.41
Stream 11	3.71	4.39	4.80	5.10	5.33	6.07
Stream 12	9.57	11.35	12.39	13.16	13.76	15.68
Stream 13	26.63	31.58	34.47	36.62	38.29	43.63
Stream 14	13.89	16.47	17.98	19.10	19.97	22.76
Stream 15	15.87	18.82	20.54	21.82	22.82	26.00
Stream 16	6.25	7.41	8.09	8.59	8.99	10.24
Stream 17	3.81	4.52	4.93	5.24	5.48	6.24

Source: - Oromia Irrigation Development Authority (OIDA) 2011

When we compare Tables 4.5 and Table 4:6 above, the estimated flood in the design document was by far less than the values of calculated design flood for cross drainages. This implies that the flood that came from catchment areas of the streams is above the capacity of the cross drainage structure of the irrigation canals that crosses the main canal. Therefore, underestimation of the catchment area of streams that crosses the main canal during the design caused serious flood around the canals The magnitude of the damage of the irrigation structure duo to flood can be seen in Figure 4.10

Figure 4.10 Structures that damaged by flood in Fentale irrigation project

From Figure4.10 the drop structure of the canals and the culverts were seriously damaged by high flood that comes from the catchment area of the stream. Water is also following out of the designed root and it did not reach the command area it ought to be.Thus, it is possible to conclude that the major causes for the damage of the irrigation structures and overtopping of the canals were as the result of the flood that came from catchment areas of the streams. Therefore, the major cause for the hydrologic failure of the canal is lack of appropriately estimated hydrological design of the irrigation project.

5. Conclusion and Recommendations

Results of this study revealed that hydraulic and hydrologic failures are common in Fentale irrigation canals, which hinders the maximum utilization of the project. Attempt has been made to identify the main causes of hydraulic and hydrologic failure of the irrigation project by giving great emphasis to hydraulic parameters and flood of the cross drainage. In this section, conclusion and recommendations on these results are presented.

5.1 Conclusion

Hydraulic performance of irrigation canal can be enhanced by appropriate design. Hydraulic parameters change (Area, manning‟s roughness coefficient (**n**), velocity, depth, discharge) has a great effect for hydraulic failure and/or success of irrigation canals. Accordingly, in Fentae irrigation project significant difference has been observed between the design and the calculated findings using the aforementioned parameters.

In this study the research tries to test the effect of the manning‟s roughness coefficient (**n**) on the hydraulic failure of the canal. Accordingly, the result shows that the change of the manning‟s roughness coefficient which leads decreasing of discharge and velocity. The decrease in velocity and discharge causes the accumulation of silt along the canal. And the accumulation of silt in the canal could usually causes overtopping. The overtopping that come due to the silt deposit has eroded the back fill of the canal structure. Therefore, these all confirmed the presence of serious hydraulic failure at Fentale irrigation project.

With the discussion of field experts the observed cross drainage structure may cause due to underestimation of the peak discharge that possible passing through the drainage structure. The estimation of the peak discharge by its nature depended on catchment area frequency analysis (selection of problem distribution), estimation of daily maximum rainfall, the return period selected etc. However, due to the limitation of time this research focus to evaluate the effect of catchment area. Although, our point of catchment delineation not certain with the previous design document, the number of cross drainage area defines based on field observation along the main canal. Accordingly the area and cross ponding peak discharge computed and its show that the previous estimation underestimate it.

Moreover, as a result of the flood, the irrigation structures were seriously damaged and overtopping was also occurred in the canal. The over topping has also caused failure of drop structures at the primary canals. Thus, hydrologic failure has been also clearly observed at Fentale irrigation project.

Finally, lack of awareness of the community (illegal water abstraction and miss use of geo membrane) and construction problem have been also observed in Fentale irrigation project which may need farther study in the area

5.2 Recommendations

- Considering the maximum flood that come from the catchments area of the streams, the cross drainage structure should be re constructed and maintained. Unless this corrective measure was taken immediately, gorge will be created and the problem would be more difficult to utilize the water at the command area.
- Frequent maintenance and silt removal should be done in order to increase the discharge.
- The slues get at the head work which helps to remove the silt that entered in to the canal should be maintained and operational during off season.
- Watershed management of the basin is recommended in order to decrease the amount of silt that deposited in the canal.
- Community awareness should be created to develop sense of ownership and culture of understanding to minimize the illegal water abstraction and miss use of the geo membrane .Therefore, farther research has to be done on the management and construction problems of Fentale irrigation canal.
- To satisfy the scientific investigation on the case of failurity, further research on the different hydraulic and hydrologic parameter to be essential.

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Annex

Element: Highest rainfall in mm
Region: Shewa
Station: Nura Hira

Annex 1 Maximum 24-hour rainfall data of Nura era

S/N	Year of Recorded	Highest rainfall(mm/d)
1	1986	29.8
2	1987	82.4
3	1988	40.2
4	1989	58.1
5	1990	53.2
6	1991	54.5
7	1992	48.1
8	1993	89.4
9	1994	52.7
10	1995	48.6
11	1997	36.2
12	1999	33
13	2000	53.7
14	2001	35
15	2002	31
16	2003	55
17	2004	56.4
18	2005	32.2
	Mean	49.42
	St.dev.	16.58
	Cv	0.34
	Cs	1.04

Source: National Meteorological Services Agency

Annex 2 Coefficients or SCS peak Discharge Method

Rainfall type	Ia/p	C_0	C_1	C_2
I	0.1	2.3055	-0.51429	-0.1175
	0.2	2.23537	-0.50387	-0.08929
	0.25	2.18219	-0.48488	-0.06589
	0.3	2.10624	-0.45695	-0.02835
	0.35	2.00303	-0.40769	-0.01983
	0.4	1.87733	-0.32274	0.05754
	0.45	1.76312	-0.15644	0.00453
	0.5	1.67889	-0.0693	0
IA	0.1	2.0325	-0.31583	-0.13748
	0.2	1.91978	-0.28215	-0.0702
	0.25	1.83842	-0.25543	-0.02597
	0.3	1.72657	-0.091	0.02633
	0.5	1.63417	-0.61512	0
II	0.1	2.55323	-0.61512	-0.16403
	0.3	2.46532	-0.62257	-0.11657
	0.35	2.41896	-0.61594	-0.0882
	0.4	2.36409	-0.59857	-0.05621
	0.45	2.29238	-0.57005	-0.02281
	0.5	2.20282	-0.51599	-0.01259
III	0.1	2.47317	-0.51848	-0.17083
	0.3	2.39628	-0.51202	-0.13245
	0.35	2.35477	-0.49735	-0.11985
	0.4	2.30726	-0.46541	-0.11094
	0.45	2.24876	-0.41314	-0.11508
	0.5	2.17772	-0.36803	-0.09525

Annex 3 Material description along the main canal Fantale Irrigation Project

Chainage		Material Description	Remark
From (km) 0+000	To (km) 2+080	Along option II canal line up to Nura Hira main canal line Not yet excavated But contains Awash River flood plain deposit and recent vesicular scoraceous basalt flows	Nura Hira canal crossing is located at 2+080
2+080	3+730	Scoraceous vesicular basalt, intact rock is fresh and very strong shows massive appearance but at places lava flow structure like curved cleavage is observed.	Pockets of soil deposits of variable color and thickness are encountered
			possibly indicating old planation or lake surfaces are observed, easy for excavation
3+730	6+280	Dark brown silty clay soil, the top 50 cm slightly stiff and sticky under lain by yellowish grey silty soil associated with gravel size calcret materials	Soil cut slope to be followed
6+280	**9+924**	Scoraceous vesicular basalt, intact rock is fresh and very strong shows massive appearance but at places flow structure is anticipated	At places not easily dozable
9+924	11+240	0-0.5 m thick, dark brown silty clay soil, underlain by calcret rich light grey silty clay soil	Homogeneous and uniform stable at 80^0 cut slope in dry condition, irregular tensional crack on the wall of the canal
11+240	12+320	Top 0.5-m is brownish grey silty clay soil under lain by 0.5m thick horizontally layered sheet like trachyte flow again under lain by gravel size	Under cutting by erosional process observed at the base of calcrete formation

		light grey calcrete rich pumaceous agglomerate. Trachyte layer is variable in thickness and locally thin out and disappeared	indicating that susceptible to solutions process
12+320	14+ 440	Top 0.5m brownish grey silty clay soil under lain by light grey to yellowish grey carbonate cemented pumaceous agglomerate and again underlain by calcrete coated basalt.	At the base of the canal, locally fresh basalt is not easily dozable, and requires explosives.
14+440	15+420	Trachyte, pinkish grey characterized by horizontally layered sheet like flow structure highly fractured moderately weathered, underlain by pinkish grey to dark grey scoraceous and obsidian rich basalt locally intercalated by trachyte, basalt and agglomeratic tuff layers.	Vertical Section at15+380indicating association of reworked volcanic flows and requires care during slope preparations for lining
15+420	16+410	Ignimbrite, greenish grey to light grey characterized by thinly layered horizontally oriented flow cleavage, moderately weathered weak to medium strong locally overlain by dark brown 0-40 cm thick silty clay soil. At places the top part is covered by thin light grey to yellowish grey loose and friable gravely silty soil of calcret rich.	16+190 to16+250 thin layer of yellowish grey gravely silty soil friable, loose and calcrete rich is overlying the ignimbrite
16+410	17+040	Variegated color pyroclastic agglomerates composed of reworked material cemented with calcrets	Easily dozable
17+040	18+640	Ignimbrite, greenish grey to light grey, moderately weathered, fractured weak rock	Dozable but not easily
18+640	18+880	Basalt, dark grey vesicular, slightly weathered appears as blocks strong to very strong	Not easily dozable
18+880	19+080	Silty clay soil, brownish grey friable loose, easily erodible needs to be protected	surface drainage should be protected not to reach to the main canal

19+080	19+240	Basalt, dark grey vesicular, slightly weathered appears as blocks strong to very strong	Not easily dozable
19+240	19+440	Silty clay soil, brownish grey friable loose.	easily erodible to be protected
19+440	20+300	Basalt moderately weathered	ignimbrite exposed at the base of the canal not easily dozable
20+300	20+440	Sandy silty clay soil, light grey to brownish grey loose friable.	easily erodible alluvial deposit
20+440	20+620	Basalt dark grey, occurs as boulders, vesicular and the cavities are filled with calcite	Not easily shaped for canal lining
20+620	20+880	Ignimbrite, variegated color, the fragments contain basalt, pumice. At the base of the main canal moderately weathered basalt exposure is observed.	The formation is dozable
20+880	21+600	Silty sandy clay soil loose and friable with basalt rock fragments on top part	. Easily dozable
21+600	21+780	Silty sandy clay soil loose and friable,	Basalt is exposed at the base of the canal
21+780	22+180	Silty sandy clay soil, dark brown to light grey, loose and friable	Easily dozable
22+180	22+280	Basalt dark grey vesicular, moderately weathered, weak.	Dozable
22+280	23+600	Gravely silty clay soil, light grey to yellowish grey loose and friable derived from volcanic ash	Possibly lacustrine deposit. Easily dozable
23+600	23+730	Basalt mixed with gravel and boulder size angular to sub rounded rock fragments of pumice and ash, cavities are generally filled with carbonate,	Dozable but no bank stability, requires retaining wall
23+730	24+180	Pumaceous ignimbrite with lithic fragments and fiamea light grey soft formation, dusty locally calcret rich with lithic fragments of rhyolite and	The occurrence of Blister structure located 10m offset to the east of the

		basalt, sheet like structure horizontally layered	canal indicates the presence of volcanic conduit which is favorable for passage of canal water ,lining to be in especial care
24+180	25+280	Basalt, dark grey vesicular fragmented appears as boulders locally sheet like structure with horizontal layering flow cleavage. At places calcite cemented basaltic fragments and scoria of different shape and size at the base of the canal	Since the material is highly permeable .Should be protected not to have contact with surface water Retaining wall and material fill till required height of the wall
25+280	26+130	Gravely sand clay soil mixed with boulder size sub angular rock fragments of ignimbrite, basalt, obsidian .Generally the soil part is loose and friable	possibly colluvial in origin
26+130	27+320	Gravely silty clay soil yellowish grey, poorly sorted loose friable, easily dissolve in water and get in to suspension.	Possibly fluvio-lascustrine sedimentary origin
27+320	27+720	Ignimbrite greenish grey moderately weathered intercalated with layered obsidian fragments	Canal wall should be protected by stepped benching structure
27+720	28+080	Silty sandy soil, brownish grey to light grey up to 4m exposed thickness of alluvial deposit .At 27+960 blister structure formed of ignimbrite partly eroded is exposed at the center of the stream bed	Forms 90^0 natural slope at the bank of the stream having 8 to 10m thick exposure
28+080	31+160	Intercalated fragments of trachyte, obsidian flow and ignimbrite, variegated color, fragments are locally cemented by carbonate	The canal crosses at the margin between alluvial deposit and foot of the Birenti

			mountain chain
31+160	31+940	Colluvial deposit, yellowish grey, gravely silty clay loose and friable intercalated with angular rock fragments of trachyte .Fragments are generally coated with calcite	Rectangular shaped retaining wall to be constructed.
31+940	32+600	Silty clay soil, brownish grey, poorly sorted friable in dry condition and slightly sticky in wet condition, irregular cracks on the wall of the canal	Mesiba basin alluvial deposit homogenous and uniform no defined layering
32+600	35+380	Recent lava flows, dark grey vesicular scoraceous slightly weathered strong to very strong generally occurs as thin lava flow. Cavities are filled with carbonate	Not easily dozable
35+380	35+530	Lacustrine sediment, dark grey, silty sandy soil	easily dozable
35+530	35+550	Basalt, slightly vesicular, underlying the lacustrine sediments very strong, at places pockets of lacustrine deposits composed of silty clay is observed which is yellowish grey in color loose and friable.	Not easily dozable
35+550	37+846	Alternating basalt and lacustrine sediment, thin lacustrine soil of gravely silty clay is over lain by basalt slightly scoraceous and vesicular.	The canal passes at the margin between lava flow and sediment
37+846	38+280	Silty clay soil, light grey over lain by 0.5-1m thick dark brown silty clay soil. Gravel size calcrete fragments are randomly distributed.	Generally Stands 85^0 soil cut slope in dry condition
38+280	43+690	Pumaceous lacustrine sediment composed of gravely Silty clay soil formed in fluvio-lacustrine environment, variegated color and texture indicating different depositional episodes. At places loose friable soft light weight, easily erodable soil	Vertical section indicating an alternating soil layer of variable texture and color originated from lacustrine deposition however

		generally cemented with carbonate under lain by gravel sized sediments of similar compositions.	it forms 85^0 soil cut slope in dry condition. Easily erodable and dispersive.

Source: - Design document Oromia Irrigation Development Authority (OIDA, 2011)

Annex 4 Flow measurement from head regulator to 17.525 km

Distance from initial point (m)	width (m)	depth	velocity	Area	Discharge
0	0.2	edge			
0.4	0.5	0.92	0.352	0.46	0.16192
0.8	0.5	1.38	0.442	0.69	0.30498
1.2	0.5	1.41	0.364	0.705	0.25662
1.7	0.5	1.38	0.43	0.69	0.2967
2.2	0.5	1.37	0.382	0.685	0.26167
2.7	0.5	1.4	0.382	0.7	0.2674
3.2	0.5	1.48	0.523	0.74	0.38702
3.7	0.5	1.43	0.671	0.715	0.479765
4.2	0.5	1.46	0.702	0.73	0.51246
4.7	0.5	1.45	0.842	0.725	0.61045
5.2	0.5	1.45	0.723	0.725	0.524175
5.7	0.5	1.39	0.68	0.695	0.4726
6.2	0.5	1.4	0.762	0.7	0.5334
6.7	0.5	1.35	0.87	0.675	0.58725
7.2	0.5	1.34	0.504	0.67	0.33768
7.7	0.5	0.8	0.432	0.4	0.1728
8.5	0.3	edge	9.061	10.705	96.998005
			0.5663125		6.0623753

Annex 5 Flow measurement from 7.525 km to 26.9 km

Distance from initial point (m)	width (m)	depth	velocity	Area	Discharge
0	0.5	Edge			
1	0.5	0.6	0.323	0.3	0.0969
1.5	0.5	0.9	0.4	0.45	0.18
2	0.5	0.98	0.535	0.49	0.26215
2.5	0.5	1.18	0.48	0.59	0.2832
3.5	1	1.32	0.601	1.32	0.79332
4.5	1	1.42	0.523	1.42	0.74266
5.5	1	1.48	0.65	1.48	0.962
6.5	1	1.32	0.572	1.32	0.75504
7.5	1	1.28	0.585	1.28	0.7488
8.5	1	1.12	0.485	1.12	0.5432
9.5	1	0.98	0.29	0.98	0.2842
10.15	0.65	edge	5.444	10.75	5.320272727
	10.15		0.4949091		

Annex 6 Flow measurement 26.9 km to 42.7 km

Distance from initial point (m)	width (m)	depth	velocity	Area	Discharge
0	0.3	Edge			
1	0.7	0.46	0.213	0.322	0.068586
1.5	0.5	0.72	0.321	0.36	0.11556
2	1	0.84	0.334	0.84	0.28056
3	1	0.96	0.34	0.96	0.3264
4	1	0.88	0.327	0.88	0.28776
5	1	1	0.39	1	0.39
6	1	1.1	0.402	1.1	0.4422
7	1	1.14	0.372	1.14	0.42408
8	1	1	0.385	1	0.385
9	1	0.86	0.351	0.86	0.30186
10	1	0.85	0.365	0.85	0.31025
11	1	0.66	0.289	0.66	0.19074
12	0.75	0.58	0.29	0.435	0.12615
12.75	0.5	Edge	4.379	10.407	
	12.75		0.33684615		3.5055579

Annex 7 Flow measurement from 42.7 km to 49.3 km

Distance from initial point (m)	width (m)	depth	velocity	Area	Discharge
0	0.2	edge			
0.4	0.5	0.2	0.285	0.1	0.0285
0.8	0.5	0.26	0.322	0.13	0.04186
1.2	0.5	0.28	0.364	0.14	0.05096
1.7	0.5	0.32	0.385	0.16	0.0616
2.2	0.5	0.34	0.382	0.17	0.06494
2.7	0.5	0.32	0.385	0.16	0.0616
3.2	0.5	0.48	0.383	0.24	0.09192
3.7	0.5	0.5	0.401	0.25	0.10025
4.2	0.5	0.48	0.387	0.24	0.09288
4.7	0.5	0.36	0.38	0.18	0.0684
5.2	0.5	0.35	0.371	0.175	0.06493
5.7	0.5	0.35	0.362	0.175	0.06335
6.2	0.5	0.32	0.325	0.16	0.052
6.7	0.5	0.3	0.285	0.15	0.04275
7.2	0.5	0.25	0.279	0.125	0.03488
7.7	0.25	0.2	0.247	0.05	0.01235
8.5	0.3	edge	5.543	2.605	14.4395
	8.25		0.34644		0.90247