Jimma University

School of Graduate Studies

Jimma Institute of Technology

Faculty of Civil and Environmental Engineering

Hydrology and Hydraulic Engineering chair

Assessment of small scale irrigation structure failures; case of Weira, Hadiya zone,SNNRP, Ethiopia

By

Handore Ertiro

A thesis submitted to the School of Graduate Studies of Jimma University Institute Technology in Partial fulfillment of the requirements for the Degree of Masters of Science in Hydraulic Engineering

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Main advisor: Dawud Tamam (PhD)

Co-advisor: Mr Abdata Wakjira (Msc.)

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Jimma

EXAMINERS' APPROVAL

The undersigned certify that the thesis entitled ; **Assessment of small scale irrigation structure failures; case of Weira, Hadiya zone,SNNRP, Ethiopia** is the original work of Hadore Ertiro and we hereby recommend for the acceptance by school of post graduate studies in Jimma University in partial fulfillment of the requirements for degree master of science in Hydraulic Engineering.

	Signature	Date
Main Advisor Dawud Tamam (PhD)	___________	__________
Co-Advisor Mr Abdata Wakjira (Msc.)	___________	_________

We, the under signed, members of the Board of Examiners of the final open defense by Hadore Ertiro have read and evaluated his thesis entitled "**Assessment of small scale irrigation structure failures; case of Weira, Hadiya zone,SNNRP, Ethiopia** and examined the candidate's oral open presentation. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree in Hydraulic Engineering.

Mr. Nasir Gabi Signature____________ Date___________
Chairperson
Dr. Adena Abebe Signature____________ Date___________
External Examiner
Miss. Desu Megra (Msc) Signature_____________ Date__________
Internal Examiner

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LIST OF ACRONYM/ABBREVIATIONS

DA	Development Agent
DEM	Digital Elevation Model
FAO	Food and Agricultural Organization
FGD	Focused Group Discussion
FSL	Full Supply Level
GE	Exit Gradient
GIS	Geographic Information System
GPS	Global Positioning System
HFL	High Flood Level
IWMI	International Water Management Institute
KI	Key Informant
MoWR	Ministry of Water Resource
NGO	Non-Government Organization
SNNPR	Southern Nation Nationality People Representative
SSI	Small Scale Irrigation
TEL	Total Energy Line
WARDO	Woreda Agricultural and Rural Development Office
WUA	Water User Association

ABSTRACT

Many irrigation structures have been designed and constructed in past years. But different researcher shows that most of design irrigation structures do not perform their proposed uses, because of different reasons. It has been observed in various researches that some of the schemes have failed to serve the purpose for which they are designed. Therefore it is important to deal with cause of failure of this structure. The assessment of problems of small scale irrigation scheme failure with the main objective to assess the problems of failure of small scale irrigation scheme in case of SNNP Region, Hadiyya zone in weira scheme was carried out. Failures of weira irrigation project were assessed by using primary and secondary data. Primary data were collected through field data measurement, questionnaires and observation. Secondary data were collected from metrological data, other official documents and design documents. Some of the problems attributed to the institutional, social and operational problems are lack of adequate community participation in planning and designing, absence of WUAs, proper handing over problem and lack of proper training and absence of proper maintenance, evaluation and monitoring issues. Moreover, hydrologic failure was evaluated by Soil Conservation Service (SCS) method. The findings related to hydrologic failure revealed that the peak flood used for the design of the weir is smaller than the result obtained in this investigation, for irrigation scheme structure of 50 years recurrent intervals were 118m3/s the calculated value of flood for 50 years 155.75m3/sec. Therefore, the other major causes for the damage of the irrigation structures and overtopping of the canals and weir were as the result of the flood that came from catchment areas of the streams. Hydraulic failures were evaluated by lacey's formula. Actual value of RL of the upstream and downstream bottom of scour hole (m), upstream and downstream anticipated scour, scour depth, manning's roughness coefficient, discharge in canal and velocity, were compared with the designed value. The finding of these scheme failure shows that there was main canal siltation, damage on scouring sluice gate, downstream scouring and damage on downstream apron.

Key words: *Hydrology, Waira SSIP, Frequency analysis, SCS Method, Design Discharge*

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1 INTRODUCTION

1.1 Background

Irrigation is one means by which agricultural production can be increased to meet the growing demands in Ethiopia (Awulachew et al. 2010).

One of the best replacements for achieving food security is expanding irrigation on various scales through river diversion, constructing micro-dams and water harvesting structures etc (pingale, 2017).

According to 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.

According to Kassa, (2019) the main limitations that contributed to the malfunctioning of the irrigation system in Ethiopia were sedimentation at the headwork, damage of intakes and sluice gates, clogging of intakes, damages of distribution systems and main body. Whereas accumulation of boulders, structural failures of diversion weir, damages of the intake gates and main canals, and absence of the under sluice were also observed in some schemes. Therefore, investigation on the causes of failure of Weirs will help for the irrigation schemes to perform better and efficiently to increase agricultural productivity in Ethiopia.

Ethiopia faces four key technical, socio-economic, institutional, and environmental challenges that must be overcome in order to meet this ambitious target: Behind-schedule scheme delivery, low-performance of schemes, constraints on scale-up Of irrigation projects, protecting irrigation development sustainability(IWMI, 2010)

In addition, institutional planning, social, operational and economic problems contributing to the failures of the schemes were also identified and remedial measures are suggested. Eventually, the present study serves to meet the requirements of the failed schemes with the best rehabilitation works.

According to the data from Hadiya zone water, mine and energy office the SSI developed in the Zone was 25 projects of which 18 are not functional, 2 under construction and 5 are semi-functional. For example in Hadiya zone SSI schemes are failed because of sedimentation of headwork, siltation of canals, seepage, and planning, institutional and socioeconomic problems. The productivity of these irrigation schemes fully depends on hydraulic functioning of intake weir and main canal structures (drop structure¸ stilling basins¸ cross structures¸ culvert¸ chutes etc). When this structure does not perform well for the intended design function, the performance of the structures would be damaging rather than productive. However, the follow up of these schemes, especially with regard to technical, social, operation and maintenance, water utilization and economic have not been well studied and documented in the study area and almost all documents socio-economic and geological parts are similar in all documents these indicates copy from one document and no proper survey data. Therefore this study would be aimed at assessing the failures of SSI schemes by investigation of site, hydrological and hydraulic on selected irrigation schemes which were found in Shashogo woreda of Hadiya zone, SNNPR Ethiopia. From the selected schemes Weira SSI scheme is gravity type which means head works are constructed across the rivers.

1.2 Statement problem

According to Headwork design manual of Japan International Cooperation Agency JICA (2013), headwork's usually function as the key facilities for irrigation projects, investigations should be carried out taking into consideration not only design and construction but also operation and maintenance aspects after construction.

The irrigation scheme is heavily damaged by siltation, overtopping and seepage in addition to lack of community participation. The above problems would cause total failure of the irrigation scheme, which eventually leads to failure on the irrigation scheme, upstream, downstream flooding and other related problems.

Investigations are the basis for securing functions of the scheme through planning, design, construction and operation stages, therefore, it is required to carry out efficient and effective investigation of scheme considering the relationship between planning and design, construction and operation &maintenance.

This research work is aimed to identify the major causes of failure of the irrigation scheme of Weira Irrigation Project and to recommend remedial measures. This will serve from minor repairs to complete replacement, either to maintain existing function, or to meet new requirements.

1.3 Objectives

1.3.1 General objective

The general objective of this study is to assess the causes of failures of existing small scale irrigation; case of Weira, Hadiya zone, SNNRP, Ethiopia.

1.3.2 Specific objectives

1. To investigate failure cases of small-scale irrigation schemes.
2. To assess the hydrological failures analysis of weira irrigation schemes.
3. To assess the hydraulic failure analysis and recommend the remedial measures for the schemes

1.4 Research question

1. What are the failure cases of irrigation structures in Weira small scale irrigation scheme?
2. What method to know the hydrological failure of irrigation scheme?
3. How is the failure of irrigation structures hydraulically and what measures should be adopted?

1.5 Significance of research

In this study the significance will be to identify the major cause of the failures of irrigation schemes. Conducting the study would have the contributions for the community, government body, NGO, decision maker and research center provide some information practice engaged in maintenance and improvement of existing irrigation schemes head works, so as to avoid mistakes and opportunity are not missed.

1.6 Scope of the research

The scope of this study is to examining the physical problems of small scale irrigation schemes in general and analysis. The study was considered only hydrologic and hydraulic problems of the selected schemes. This study didn’t address some important areas of the schemes like sediment load estimation and structural analysis because it takes more time, no detail design documents and insufficient data.

1.7 Organizations of the Thesis

The thesis is organized into five `chapters Chapter one deals with the background, problem statement, objectives, scope and limitation, significance of the study and organization of the thesis. Chapter two reviews related literature. Chapter three presents the description of study area and Methodological were presented in chapter four presents the results and discussions of the study. The final chapter includes conclusion and recommendations.

1.8 Limitation

Limitation of particular study concerns potential weakness that usually out of the research control and closely associated by chosen. The limitation which limit my research is thecovid-19, institutional problem which doesn’t responsible to give full data, awareness of community during field survey and questionnaires and the country southern part of security impact direct and indirect impact of my research.

2 LITERATURE REVIEW

2.1 Diversion Head work structures

Head works are hydraulic structure which supplies water to the off taking canal. It can also be defined as barriers across a river at the head of an off taking main canal. Head works can be either diversion head works or storage headwork. Diversion head works is a structure constructed across a river for the purpose of raising water level in the river so that it can be diverted into the off taking canals. It is also known as canal head works. Diversion head works are constructed at the head of the canal, in order to divert the river water towards the canal, so as to ensure a regulated continuous supply of relatively silt free water with a certain minimum head in to the canal (Asawa, 2008).

Diversion headwork provides an obstruction across a river, so that the level of the water is raised and water is diverted to the channel at the required level. The flow of water in the canal is controlled by the canal head regulator. The headwork serves the following purpose to raise water level at the headwork of the canal, to control the intakes of water into the canal, to control the entry of silt into the canal and to control deposit of silt at the head of the canal (Bibhabasu, 2012).

Head works are defined as the facility which diverts water from a river (lakes and marsh, excluding reservoir) in to a canal for irrigation or water supply. In the Multilingual Technical Dictionary on irrigation and drainage issued by the International Commission in Irrigation and Drainage (ICID), Head work are defined as “A collective term for all works (weirs or diversion dams, head regulators, upstream and downstream river training work and the irrigated structures) required at intakes of main or principal canals to divert and control river flows and to regulate water supplies into the main canals (Henock, 2016).

A research Lambisso, (2005) in Southern Nations Nationalities and Peoples Regional Government (SNNPR) was conducted aiming to evaluate the design practices and Performances of small scale irrigation structures on selected sites within the regional Investigating the causes of failure of existing small scale irrigation was the primary objective of the research. In this research work the available design documents of 15 sites from the 26 targeted sites obtained from the regional irrigation development authority was reviewed and practical field visits was also conducted on implementation and operation of the irrigation schemes of (26) sampled sites within the region The results of findings related to the prevalence of each observed failure and under performances with regard to possible causes of the targeted schemes for the research work have been Summarized in table 2.1

Table 2.1 type of structural failure of Irrigation Schemes and its prevalence in south region (Lambisso, 2005)

Type of structural failure or problem	Failure amount (%)
Main canal siltation	50
Head work failed by sedimentation	42
Main canal seepage loss	35
Foundation seepage loss	4
Damage or broken on intakes gates	3
Damage on sluice gate	27
Damage on weir body	4
Wing wall and bank eroded	8
Scouring to d/s bed level	19
Dried river floe condition	12

The other research used as a bench mark for this research is a research work by Nile Basin Capacity Building Network entitled design and operation of diversion. The objective of the research was to assess the existing river diversion systems in some countries of the Nile Basin and to identify categorically problems related to them.

The research work conducted by NBCBN also assesses the existing river diversion systems in some countries of Nile Basin and tries to identify categorically problems related to them. The objective of the research was to systematically compile and build a database of existing both traditional and modern diversion system, to review critically the existing design criteria and to identify the limitation in these criteria, to suggest remedial measures in a form of design and operation guidelines or procedure to improve the performance of existing conventional diversion systems and suggest to the use of alternative design. The results of findings related to the prevalence of each observed failure with regard to possible causes of the targeted 84 small scale irrigation Schemes inventoried for the research work in north, central and southern Ethiopia categorically have been summarized table 2.2

Table 2.2 Problem category and weight (NBCBN, 2005)

No	Problem category	Weight
1.0	Site selection	31%
1.1	Clogging of under sluice	1
1.2	Sedimentation problem on headwork	1
1.3	Change in river course	1
1.4	Damage on main canal and farmlands	1
1.5	Main canal siltation	1
2.0	Structure selection	25%
2.1	Sedimentation problem on headwork	1
2.2	Damage on weir proper	1
2.3	Clogging of under sluice and outlet	1
2.4	Prevalence of downstream scouring	1
3.0	Hydrology and sediment consideration	13%
3.1	Clogging of under sluice and outlet	1
3.2	Main canal siltation	1
4.0	Hydraulic design of weirs and components	69%
4.1	Prevalence of d/s scouring	1
4.2	Damage on main canal and farm land	1
4.3	Damage on downstream apron	1
4.4	upstream flooding	1
4.5	Damage on retaining wall	1
4.6	Seepage problem under the weir	1
4.7	Change in river course	1
4.8	Clogging of under sluice and outlet	1
4.9	Damage on d/s cutoff	1
4.10	Damage on sill (if any)	1
4.11	Damage on divide wall	1

5.0	Structural design of weir and components	25%
5.1	Damage on intake gate	1
5.2	Damage on scouring sluice gate	1
5.3	Damage on weir proper	1
5.3	Damage on divide wall	1
6.0	Scheme operation	13
6.1	Damage on intake gate	1
6.2	Damage on scouring sluice gate	1

The research work was aimed at assessing the existing river diversion systems in the Nile Basin Countries and at identifying the, hydraulic, hydrologic, and structural problems related to them. The specific objective of their research was to systematically compile and build a data base of existing both traditional and modern diversion systems to review critically the existing design criteria and identify limitation in these criteria, to suggest remedial measures in a form of design and operation guideline or procedures to improve the performance of existing conventional diversion systems and suggest the use of alternative design.

2.1.1 Weir and its failure types

Weirs can be adopted if the slope of the earth surface is relatively steep. A weir is un gated barrier across river to raise the water level in the river. It raises the water level in the river and diverts the water into the off taking canal situated on one or both of the river banks just upstream of the weir. Weirs are usually aligned at right angles to the direction of flow in the river. The following are the failure types of weir

2.1.1.1 Piping

Piping is caused when groundwater seeps out of the bank face. Grains are detached and entrained by the seepage flow and may be transported away from the bank face by surface runoff generated by the seepage, if there is sufficient volume of flow. The exit gradient of water seeping under the base of the weir at the downstream end may exceed a certain critical value of soil. As a result, the surface soil starts boiling and is washed away by percolating water (Lufira, 2021).

Piping is especially likely in high banks backed by the valley side, a terrace, or some other high ground. In these locations the high head of water can cause large seepage pressures to occur. Evidence includes: Pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones and layers; run-out deposits of eroded material on the lower bank (Tadesa, 2019).

2.1.1.2 Failure due to subsurface flow

According to Shiva Kumar Khaple, (2014) the water from the upstream side continuously percolates through the bottom of the foundation and emerges at the downstream end of the weir or barrage floor. The force of percolating water removes the soil particles by scouring at the point of emergence. As the process of removal of soil particles goes on continuously, a depression is formed which extends backwards towards the upstream through the bottom of the foundation. A hollow pipe like formation thus develops under the foundation due to which the weir or barrage may fail by subsiding.

2.1.1.3 Failure by surface flow

When the water flows with a very high velocity over the crest of the weir or over the gates of the barrage, then hydraulic jump develops. This hydraulic jump causes a suction pressure or negative pressure on the downstream side which acts in the direction uplift pressure. If the thickness of the impervious floor is insufficient, then the structure fails by rupture (Marsudi, 2021).

2.1.1.4 Riprap

Riprap is implemented where there is the concern that local scouring would occur on the riverbed, taking into account the condition of the riverbed and the resulting flow in both the upstream and downstream, as well as safe construction against flows and will have an energy dissipating effect. Riprap is consistently implemented on the downstream apron to prevent riverbed scouring. The scouring is due to the removal of deposited silt or the vertical inflow compensation of the local dissipation of water energy (Lufira, 2021).

2.2 Barrages:

A barrage and a weir are similar structures and differ only in a qualitative sense. The crest of the barrage is usually at a lower level and the pounding up of the river for diversion into off taking canal is achieved by means of gates. Barrages are considered better than weirs due to the following reasons (Asawa, 2008) Barrages offer better control on the river outflow as well as discharge in the off taking canal

2.3 Canals and its type

Canals are generally designed assuming steady and uniform flow. However, this situation is seldom found in a modern irrigation scheme. Modern irrigation schemes are increasingly demand oriented and require frequent operation of control gates that leads to unsteady and non-uniform flow. The design becomes more complicated in case the canal has an erodible boundary and carries water with sediment. Most schemes in this category require a large amount of maintenance due to unwanted deposition on or erosion of the canal bed and banks (Paudel, 2010)

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 (Laycock., 2007)

Lined canals According to Laycock, (2007) 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.

2.3.1 Failure of Canals

According to Dunbar (2008) 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 (Lambisso, 2005)

2.4 Social and institutional aspects of small scale irrigation

The survey on the Operation and Maintenance of the Awushoana and Kecha SSI schemes were conducted, and response from the selected target community was reported. The respondents confirmed that the irrigation schemes were completely damaged due to very poor O&M of the scheme. Failure to deliver sufficient water was found due to improper adjustment of the waste valves and insufficient delivery of water to the pump. More than 90% of the respondent confirmed that Operation and Maintenance activities of the schemes were not proper. The 16% Kecha and 22.2% Awushoana respondents revealed that poor Operation Maintenance activities were taken at these schemes (pingale, 2017).

According to Girma, (2005) the very important aspect of irrigation development has been left to the bureau of Agriculture development which is different institution with different job description, action plan and work programmed diametrically opposite to the former institution. Hence the weak institutional relationship created unbridgeable gap among the construction, extension and the beneficiaries which ends up in failure of the project performances.

According to Hoogesteger, (2013) Participation in collective action that is called upon by the WUOs is usually a part of the normative framework of irrigation systems. Non-participation in this collective action (be it for the perpetuation of the irrigation system or for broader issues of regional or national interest) usually leads to fines and in some cases also the temporary and/or definitive loss of the right to access water within the irrigation system.

2.5 Hydrologic analysis

In The hydrologic analysis, the peak discharge at the weir will be estimated. The determination of the peak flood is a major task in the hydrology part and Peak discharge estimates are often needed at ungauged sites where no observed flood data are available. The value obtained will be compared with value used for the design purpose and additionally will be used for redesign of the weir. The peak flood flow is the maximum expected flow at a certain location for a given frequency. peak flood flows depend on the catchment area, the slope of the main channel, the basin shape factor, the hydrologic region, and the return period (Kumar, 2010).

According to Ebissa, (2017) Potential extreme peak discharges are estimates of the highest peak discharges expected to occur at a certain location and, are explained mostly by the area of the corresponding catchment and by the hydrologic region where the catchment is located. Maximum design discharge is the peak river discharge that corresponds to a certain return period. The maximum design discharge QMax is used in the design to determine the back water curves results from constructing to the weir which enables to predict the highest water level that occurs average once every T years, where T is the selected return period of the discharge.

Hydrological stress produces high-magnitude flash floods and erosive events, often causing hydro geological instability and disruption (Sadeghfam, 2019).

2.5.1 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).

2.5.1.1 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 (ERA, 2002).

2.5.1.2 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.

a. 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.

$t_c = 0.01947\mathbf{L^{0.77}}\ S^{-0.386}$……………………………………………………………………………2-1

Where t_c is the time concentration, The maximum length of water travel (m) and S is average slope of the channel given as a fraction of the vertical elevation rise to the corresponding horizontal length. The time to peak (T_P) has been estimated from the tc values using US SCS method.

b. Determination of Curve Number

An important shortcoming of the standard CN method is that it does not take into account the effect of slope. In fact, the reference CN values provided in the standard SCS tables were mainly identified from small agricultural watersheds with mild slopes, considering that the rainfall-runoff transformation is only affected by the soil and land cover characteristics. However, in the general case, the relief characteristics also affect greatly the hydrological response of a watershed. Steep slopes cause a reduction of initial abstractions, a decrease in infiltration and a reduction of the recession time of overland flow, which in turn results in increased surface runoff. Today, it is accepted that the reference CN values are applicable for terrain slopes around and several researchers have proposed empirical formulae for adjusting the CN values to slope.

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.

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.

2.6 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.

2.6.1 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).

2.6.1.1 Normal distribution

For normal distribution, the frequency factor KT can be expressed by following equation

$K_T = \frac{XT - \mu}{\delta}$ ……………………………………………………………………………2.2

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

p = 1/p

Can be calculated by finding the value of an intermediate variable w:

$W = [\ln (1/p^2)]^{1/2}$ $0<p<0.5$……………………………………………………2.3

2.6.1.2 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 'X_T', at return period T, can be obtained from the relation

$X_T = \exp Y_T$……………………………………………………………………………….2.4

$Y_T = \tilde{y}\ (1+CvyK_T)$……………………………………………………………………….2.5

Where, Y is the mean and C vy is the coefficient of variation of Y

2.6.1.3 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………………………………………………………………………2.6

Log X = M + $K_T S$……………………………………………………………………..2.7

2.6.1.4 Gumbel distribution

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

$X_T = \ddot{x}\ (1+ Cv\ K_T)$ ……….……………………………………………………………..2.8

Where, X is Mean of the observed rainfall, CV is Coefficient of variation and KT is Frequency factor

2.7 Hydraulic analysis

According to pingale, (2017) In general, the hydraulic analysis consists of the estimation of the shape and height of the weir, clear waterway, discharge, and head over the weir, length of the weir, flood and energy level, afflux and scour depth.

According to Lacey (1929) the regime equations for canals carrying the full supply discharge, the near bankful discharge forming the channel geometry is relevant. Thus, the measurement of discharge and the corresponding river geometry should be confined to monsoon season.

P=4.75Q1/2 ………………………………………………………….……………….2-2

The indicators of failure of hydraulic structures are the following

A. passage of sediment

The design of the headwork must prevent the bed-load from approaching the intake and causing clogging of the intake causing significant structural damages to the headwork components. entry of silt in to canal which takes off from head- works can be reduced by constructed certain special worst called silt control work (Bibhabasu, 2005).

According to Bibhabasu Mohanty, (2012) entry of silt in to canal which takes off from head-works can be reduced by constructed certain special works called silt control works. These works may be classified in to the two types silt excluders and silt ejectors.

B. Seepage Analysis

Seepage of water is one of the major problems which have an effect upon hydraulic structures. Therefore, the seepage under the hydraulic structures can be considered one of the most important factors in the hydraulic structure's safety. The seepage usually occurs in the impervious soils because of the differential pressures due to differences in water level between upstream and downstream. Seepage flowing below the foundation of hydraulic structures founded on permeable soils exerts pressure on the structures and tends to wash away soil under it. Excessive uplift pressure and piping are often the main cause of damage of the stability of the structure and may cause its failure (Asawa, 2008).

C. Scouring

According to Henock, (2016) Scour is a natural phenomenon caused due to the erosive action of flowing stream on alluvial beds which removes the sediment around or near structures located in flowing water. These endanger stability of the structure by shearing. Scouring occurs during floods and when the water flow with very high velocity over the structure. These have been checked by integrating both physical observation and flow velocity measured on Fantale diversion headwork.

Several studies focus on scouring at the downstream of hydraulic structures, which may be divided into two groups: scouring induced by subcritical flows and those by supercritical flows. Scouring by subcritical flows have focused on the downstream of structures like bridge piers, drops, bridge abutment and submarine pipeline (Roushangar et al.).

Scour is a natural phenomenon caused by the erosive action of the flowing stream on the sediment beds. Local scour downstream of a hydraulic structure due to horizontal jet poses an immense problem in designing the foundation of the hydraulic structure (Guven, 2008).

D. Clogging of under sluice

The under sluices are openings provided at the base of the weir. These openings are provided with adjustable gates. Normally, the gates are closed. The crest of the under sluice portion of the weir is kept at a lower level (1-1.5) than the crest of the normal portion of the weir. The suspended silt goes on depositing in front of the canal head regulator (Asawa, 2008).

2.8 Design of protection work

2.8.1 Downstream protection work

According to pingale, (2017)There was no proper design of downstream protection works like a downstream impervious apron, cut-off wall; downstream block protection, launching apron and a downstream sheet of piles and their length and thickness were not properly designed this results in failure of downstream work.

Downstream scouring and damage of downstream apron from the hydraulic analysis, weir indicated that failure corresponds to the position of the hydraulic jump due to variation occurring in the bed of the river (Azamathulla, 2010).

2.8.2 Upstream protection works

According to pingale, (2017) on the upstream of the upstream cutoff wall, a stone block of minimum size has to be provided for a length of about 1.5D1 where D1 is the depth of the upstream cutoff from the upstream bed. A curtain walls between the stone block and the launching apron is provided to keep both structures as solid mass. It is provided in the upstream & downstream part of the weir.

Water depth at the upstream of the screen changes according to the inflow condition(Froude number of super-critical flow), screen porosity and the size of bed materials demonstrate the effect of tail water on the scouring pit so that the maximum scouring depth increases as the depth of the tail water increases (Sadeghfam, 2019).

2.8.3 Energy dissipation

The flow phenomenon on the Spillway is that the flow velocity is very high, with the flow condition being supercritical. Therefore, before the water flow is directed to the river, the flow should be slowed down and transformed into the sub-critical flow to prevent scouring that damages the geometry river at its bottom and the river cliffs (Lufira, 2021).

2.9 Sampling technique

According to Efriem Tariku (2019) purposely sampling method used for cross checking the causes and effects of the mal-functionality of the irrigation structures with district experts who have the irrigation profession and water committee.

2.10 Tools

2.10.1 ARCGIS

Arc GIS Terrain Preprocessing uses DEM to identify the surface drainage pattern. Once preprocessed, the DEM and its derivatives can be used for efficient watershed delineation and stream network generation (Ambaw, 2016).

2.10.2 SPSS

According to Arlfa Rahman, (2021) SPSS is capable of conducting all major tests required for quantitative data analysis in the field of social sciences. With all that being said, in today's time, realizing the need, it is not only the choice but in some cases considered essential for social researchers to use SPSS as their quantitative data analysis and representation tool.

It is mathematical and statistical purposive software which can easily represent the collected data in a analytical way within a short period of time which is very helpful for the user.

3 METHODOLOGY

3.1 General Description Weira Project

Irrigation schemes are described as small scale, large scale and medium according to their amount of area coverage. From these types of schemes small scales irrigation schemes are given to emphasis in my study. Weira Irrigation Based Development project is the project which is undertaken in the form of intervention measure by the regional government of South for the consequent food insecurity problem at the area related to reduce shortage of food, which exposed the residents of the surrounding area for food aid. It is among the large scale projects found in Hadiya zone in South region which has been designed and constructed by NGO i.e. (World vision) and .The type of weir adopted to raise the irrigation water for abstraction is a rock fill type of weir. Design of Weira diversion structure uses a design flood of 50 years' frequency for the purposes of design of items other than free board. For the design of free board, a design flood of 50years' flood has been adopted. The design floods 118 m^3/s Weira small scale irrigation scheme

The main source Weira small scale irrigation is Waara River which is the tributary of rift valley river basin. The topographic condition of the Weira river basin is hilly, undulating to rolling terrain. Slope has major implication for land use practices. Fairly level or slightly undulating soil tend to be located mostly in low lying areas and generally have deep and medium to heavy texture soils and its aim is the irrigation only. They have also less limitations for cultivation of agricultural crops and are generally easy to irrigate, but may have a drainage problem on some part of sub-catchments but majority can have well drained, but conservation based agricultural practice must be applied in order to control the erosion hazards with the vicinity.

Although in the downstream of the river course, on selected and constructed diversion weir site of the project which is destroyed by high flood/runoff/ there is deposition and accumulation of boulders are to be seen during the field visit. The basic causes of siltation and scouring problems are extensive farming, soil erosion.

3.1.1 Location of the study area

The study area Weira Irrigation Based Development Project is found in Hadiya zone

This is located at about 55 km from Hossana town and at about 130 km south west of Addis Ababa. Geographically the diversion study area located 38°2′25′′E to 38°5′25′′E and 7°15′25′′N to7°45′50′′N elevation 1902 m.a.l (Fig 3.1).

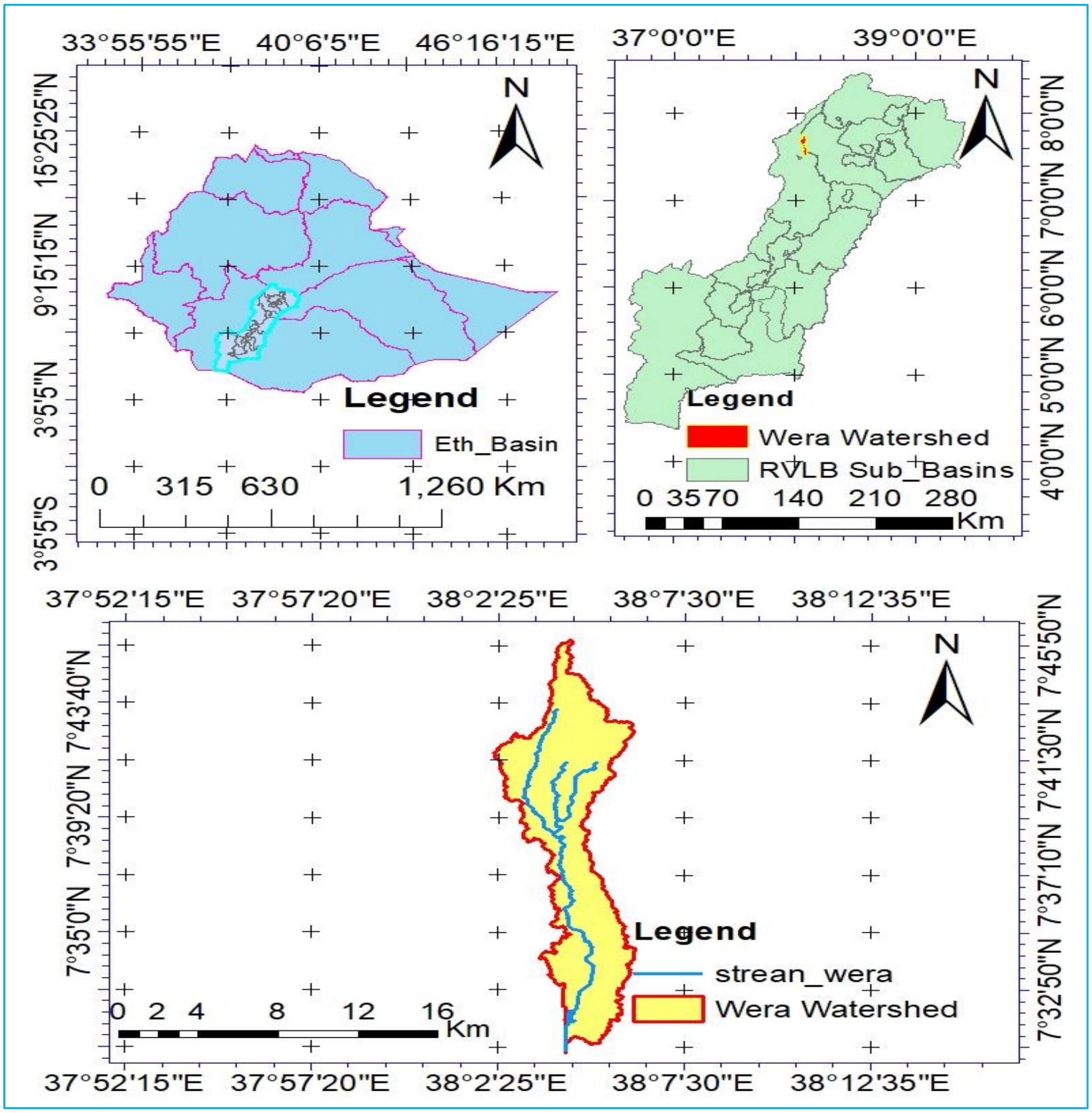

Figure 3.1 map of study area

3.1.2 Climate

Weira is tropical with a seasonal wet and dry season small rains (December-February) merge into the main rainy season which occurs from (June-August). The variations in climate during the year are largely associated with the macro-scale pressure changes and the monsoon flows related to these changes. The winter climate is influenced by the anticyclone over the Sahara and the range of high pressure extending into Arabia from the large high over central Asia. South-easterly flows dominate the region.

3.1.3 Land use land cover

Indicates the classification of the land of area different types of socio-economic uses; types of land use changes from time to time depending on socio-economic change of the study area. For instance, the grazing land, natural forest and fallow lands are decreasing from time to time while cultivated, manmade forest and residential lands are increasing. Accordingly, from the total area of the district the cultivated, lands (the lands covered by annual and perennial crops) represented. The vegetation covered land (grass land and shrub) accounted.

Table 3.1 land use classification of catchment

Land use type	Area coverage (km^2)	Percent (%)
Shrub land	218.24	89.1
Grass land	26.76	10.9
Total	245	100

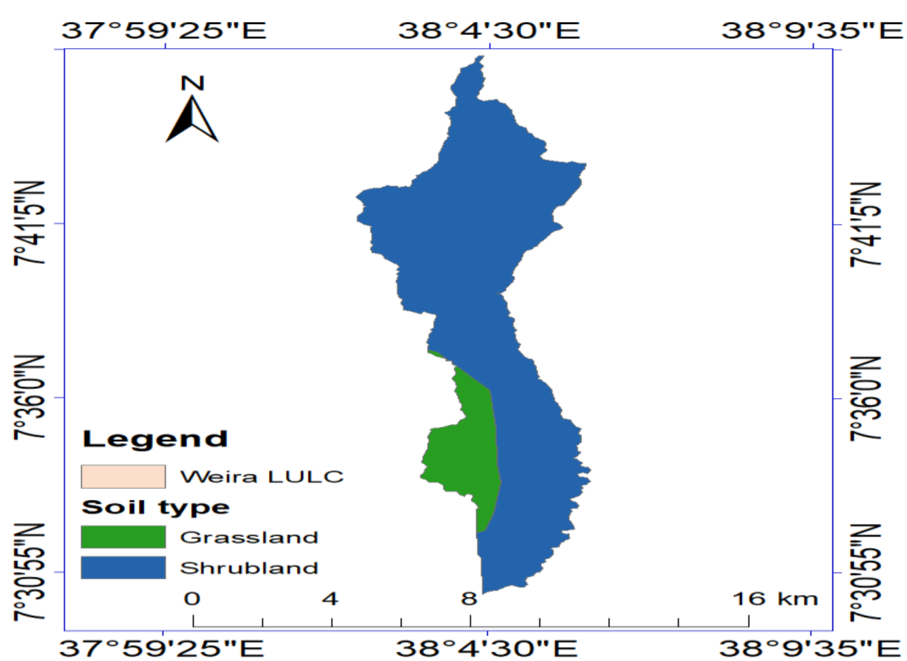

Figure 3.2 land use land cover

3.1.4 Topography

The altitude ranges from 1870m to 2117 m.a.s.l. The watershed has an agriculturally suitable land in terms of topography. Flood is a series problem in the flat topography areas.

3.1.5 Soil

Weira watershed soil classification system in the fig below, the most dominant soil in the area is Vitric andosols and orthic solonchaks covering in the Watershed area.

3.1.6 Population

The total population of the area is 116,287 people (CSA, 2007), it is one of the densely populated in hadiya zone.

3.2 Data Collection

The consultant on the project site, field measurements and observations are the primary sources of data for the study. In order to achieve the objectives of the study secondary data is also used. These data are obtained from the Ministry of Water, Irrigation and Electricity, southern design and supervision work enterprise and Ethiopian Meteorological Agency. In addition to these literatures, different project documents or proposals, project evaluation and completion reports are also refereed. On first stage previous studies, documents and papers related to diversion headwork have been revised and desk study have been undertaken to identify the key issues. Accordingly, the available relevant data on existing headwork is important to be collected from the concerned department.

Field visit surveys have been conducted for gathering out of data for the purpose of describing the nature of existing conditions and to compare existing conditions with the design standard of the diversion headwork. To assess the operation and functionality of constructed diversion head works as well as operations of each component parts of the head work, field visit survey coupled with primary data obtained by interviewing administrative officials at the project site have been conducted. The secondary data are also collected from southern Design and Supervision Works Enterprise design reports.

For the hydraulic analysis of the head work, the survey data's collected were of the river cross section, upstream and downstream elevation of the headwork's. Therefore, these sets of data were used for the hydraulic analysis of the previously proposed design by the consultant of the project.

3.2.1 Primary Data (field observation data)

The primary data was obtained from direct observation of events during many visits remunerated to the schemes. Comprehensive field survey such as transect walk was held through different components of the scheme to understand irrigation practices, sources of irrigation water, its water distribution system. Moreover, discuss with the focused group and key informants was undertaken to identify the root causes and effect of failed irrigation structures. Functionality of Irrigation schemes, site condition of the head works, upstream and downstream condition of the weirs, intake conditions, checking for upstream and downstream protection works and direct measurement of fields, problem of flooding, erosion, siltation of canal, weed growth in the canal, types of canals and other relevant data was observed at the field such as the photographs of the failed structures were taken and Grid coordinates was collected using GPS (Global Positioning System) for the purpose of preparing map of the study area.

3.2.2 Secondary Data

Secondary data used for this research were collected from responsible bodies and officials. These data include rainfall data, DEM, land use land cover, GIS data and design documents of the headwork components for the scheme and these help in identifying the sensitive parameters for the problems on the scheme.

3.2.2.1 Rainfall Data processing

The availability of a long and complete rainfall record is very important for carrying out a hydrological study successful. Precipitation data, which will be taken from National Meteorological Agency of Ethiopian most of time it has missing data therefore, the missing data, filled by using different methods from these Linear Regression methods.

Linear Regression (LR) Method: The correlation coefficients between the target station and each of the neighboring stations are initially calculated and then ranked. Then the missing data are estimated using a linear regression equation with the station that has the highest correlation. The correlation and the equation of the regression line are obtained using Microsoft Excel software. In order to be able to generate zero values together with non-zero values, the regression line is forced through the origin.

Checking the Consistency of Data

After the missing data filled then the consistency of the data checked. A small change may occur in and around a rain gage station; such a change occurring in a particular year will start affecting the rain gauge data, which is reported from that particular station. After several years, it may be felt that the data of station is not giving consistent rainfall values. In order to detect any such inconsistency, and to correct and adjust the reported rainfall values a technique, called double mass curve method used. It checks the consistency of rainfall record by plotting the cumulative annual rainfall at station x against the current cumulative values of mean annual rainfall for a group of surrounding station for the number of year of record.

3.2.2.2 DEM data Processing

A DEM is a sampled array of elevations for ground positions that are normally at regularly spaced intervals. The DEM is optimized through integration of the existing digitized drainage to obtain a final DEM. 30*30 DEM data is used as an input data for Arc-GIS software for catchment delineation and estimation of catchment characteristic.

3.3 Preparing research question

The necessary of questionnaires was the investigation of scheme failures and assess community participation during planning and design stage and awareness of community in operation and maintenance and interaction of WUA and government with community and it need to be addressed in order to complete the research objectives.

3.3.1 Method of sampling

Sampling means selecting the group that actually collect data from in my research. It must be cost effective and convenient that must be conduct define my target. The stages follow first stage sample area chosen and second stage sample of respondent selected within those areas by using non probability sampling techniques method use for my research purposive sampling techniques was purposely sampling method; it was used for cross checking the causes and effects of the mal-functionality of the irrigation structures. By using these methods selecting purposely user of irrigation only rather than the other and also analysis these sample selected data by descriptive analysis.

3.4 Hydrologic analysis

Hydrologic analysis should include the determination of several design flood frequencies for use in the hydraulic design. These frequencies are used to size different drainage structures to allow for an optimum design, that considers both risk of damage and construction cost. Consideration shall be given to what frequency flood was used to design other structures along a highway corridor.

The maximum design discharge was estimated using United States Department of Agriculture (USDA) Natural Resources Conservation Service Curve Number (NRCS-CN) method. This was used to determine the backwater curve results from constructing the weir in order to predict the highest water level for the 50 years return period. The maximum rainfall data was used for the hydrologic analysis to determine the maximum design discharge and checking the consistency of the structures constructed for the design period (Ebissa, 2017) or components and these help in identifying the sensitive parameters for the problems on the head work.

3.4.1 Rainfall Frequency Analysis weira weir site

There is the five nearest gauging station on the Weira small scale scheme irrigation from these all nearest station Fonko is more nearest than all gauging station Thus, it is preferred to base the flood analysis on rainfall data, which are better both in quantity and quality of data. The SCS hydrograph method is selected for the analysis of the rainfall runoff hydrograph and computation of the design flood.

3.4.2 Annual Highest Daily Rainfall Series

Data from SNNP Meteorological station has been used for determination of design rainfall. Frequency analysis of the annual maximum daily rainfall has been carried out to compute design 24-hour rainfall of various return periods. The maximum annual daily rainfall series for 30 years periods has been used for the Rainfall analysis of Fonko.

Table 3.2 annual maximum daily rainfalls at Fonko station of weir site rainfall analysis

Year	24hr duration Max. annual ppt (mm)	descending	Y-logr
1989	63.9	67.4	1.82866
1990	50.9	66.8	1.824776
1991	50.8	63.9	1.805501
1992	42.5	62.1	1.793092
1993	41.9	59.6	1.775246
1994	38.5	58.4	1.766413
1995	66.8	57	1.755875
1996	59.6	55.4	1.74351
1997	34.9	55.2	1.741939
1998	39.5	54.8	1.738781
1999	55.4	54.2	1.733999
2000	54.2	52.7	1.721811
2001	48.6	51.0947	1.708376
2002	67.4	51	1.70757
2003	50.4	50.9	1.706718
2004	42.8	50.8	1.705864
2005	49.4	50.7	1.705008
2006	50.7	50.4	1.702431
2007	54.8	49.4	1.693727
2008	47.7	49	1.690196
2009	57	48.6	1.686636
2010	45.6	47.7	1.678518
2011	49	45.6	1.658965
2012	52.7	42.8	1.631444
2013	58.4	42.5	1.628389
2014	51	41.9	1.622214
2015	36	39.5	1.596597

2016	62	38.5	1.585461
2017	55.2	36	1.556303
2018	51.0947	34.9	1.542825
Ave			1.701228
S.dev.			0.074184
C_s			-0.3689
K_s			-0.0615

3.4.3 Tests for Outliers

Outliers are data points which depart significantly from the trend of the remaining data. The observed annual daily maximum rainfall series was subjected to tests for high and low outliers. This test is conducted using the methodology specified in the US Army Corps of Engineers Manual on Hydrologic Frequency Analysis The following equation is used for detecting low and high outliers:

$$X_H = \bar{X} \pm K_{NS} \text{ -- } 3.1$$

Where, XH low/high outlier threshold in log units, X mean logarithmic of the test series S is standard deviation of the series, KN is outlier test value for a given sample size & level of significance, Lower Limit of low outlier is 10 ^ XL, Upper Limit of high outlier is 10 ^ XH

Table 3.3 Table showing outlier test

	Station	Fonko	Wulbarag	Hossana	Butajira	Alaba
G-B Test Value	XH	77.3509	90.693	115.642	192.246	97.0187
	XL	32.6581	18.2874	20.0102	9.2691	30.7898
	XL+/- 10%XL	32.6581	20.1162	22.0112	10.196	33.8688
Recorded RF	Max.	67.4	74.1	94.1	91.3	86
	Min.	34.9	23.9	14.408	10.2145	34.8
Outlier/Min		Not	Not	Yes	Not	Not
	Remove Outlier data for Hossana station					

3.4.4 Selection of Distribution

The observed data was tested using different statistical distributions. The most commonly distributions used to fit extreme rainfall events are four common statistical methods, namely, Normal, Gumbel max, Log-Pearson III (LP III), and Gen. Extreme value method

3.4.4.1 Normal Distribution

In statistics, normal distribution is a type of distribution where the data are characterized by a bell shaped curve. Discrete form and curve location are obtained by mean and standard deviation. As many natural phenomena fit into this; it is a highly significant probability distribution in statistics. This distribution illustrates how variable data are dispersed. The majority of annotations group about a central peak as it is symmetric and the probability is for data to shrink off uniformly in both directions away from the mean. The arithmetic mean of sample x1, x2…...xn typically represented by μ is the sum of the sampled value divided by item number (n):

Sample means (C= $\frac{\mathbf{X1+X2+X3 \ldots\ldots\ldots\ldots Xn}}{\mathbf{n}} = \frac{\mathbf{1}}{\boldsymbol{n}}\sum_{\mathbf{i=1}}^{\mathbf{n}} \mathbf{xn}$……………………………..……..3.2

3.4.4.2 Gumbel Max

Gumbel is a type of statistical distribution which began from extreme theory. Function in this distribution is unrestrained on whichever side, leading to negative flow calculation. This represents distribution of extreme values, either highest or lowest of samples, used in various distributions and for modeling distribution of peak levels. This is utilized for predicting earthquake, flood, and other natural hazards. It also models operational threat in managing threat and product life which wears out rapidly prior to a certain age. For the required return period (T), abridged variate (Yt) has been assessed by using the formula:

Yt = ln ($\mathbf{ln}(\frac{T}{T-1})$) --3.3

Kt = ($\frac{Yt-Yn}{Sn}$) ---3.4

3.4.4.3 Log-Pearson III

LP III is a statistical method of fitting frequency distribution values for predicting flood at a few sites of a specified river. Frequency distribution is built after calculating data related to statistics at a particular river site. Flood occurrence probability of different densities can be taken out from the curve. This particular method helps in extrapolating event data with return periods ahead of pragmatic occurrence of flood. After finding the actual discharge, we then calculate the natural logarithm of the actual discharges (Z) and find the standard logarithmic mean (μ) and standard logarithmic deviation (σ) of the calculated discharges for the respective seasons:

$Z = \log_{10} Q$ --3.5

$P = \frac{1}{T}$ ---3.6

$Kt = Z + (Z^2 - 1)K + \frac{1}{3}(Z^3\_6Z)K^2 + (Z^2\_1)K^3 + ZK^4 + \frac{1}{3}K^5$ ----------------------------------3.7

$K = \frac{Cs}{6}$ ---3.8

$Qp = \mu + Kt\alpha$ ---3.9

3.4.4.4 Generalized Extreme Value

Generalized extreme value is a continuous probability distribution developed within extreme value theory. It is a combination of Gumbel, Fréchet, and Weibull extreme value distributions and is a bounded distribution of standardized maxima of a series of autonomous and indistinguishable dispersed arbitrary variables. GEV is utilized as an estimate for modeling maxima of lengthy (limited) series of arbitrary variables. Significantly, while using this distribution, the upper bound is unidentified and hence has to be projected; when Weibull is applied, the lower bound is identified as zero.

Frequency factor for GEV distribution is:

$Kt = \frac{\sqrt{6}}{\pi}\left\{0.5772 + \ln(\ln(\frac{T}{T-1}))\right\}$ --3.10

$T = \frac{1}{1 - exp\left\{\exp[0.5772 + \frac{\pi Kt}{\sqrt{6}}]\right\}}$ --3.11

3.4.5 Goodness-of-fit test Graphical Method

The selection of best fitted statically analysis is an important point of any hydrologist before the estimation of peak magnitude of rain fall or stream flow of the given station.

There are two methods to select best fitted statically methods. These are graphical and analytical methods. Here for this study graphically method was used. Using this method LN3 statically method was selected since the site mean falls on it more than other methods.

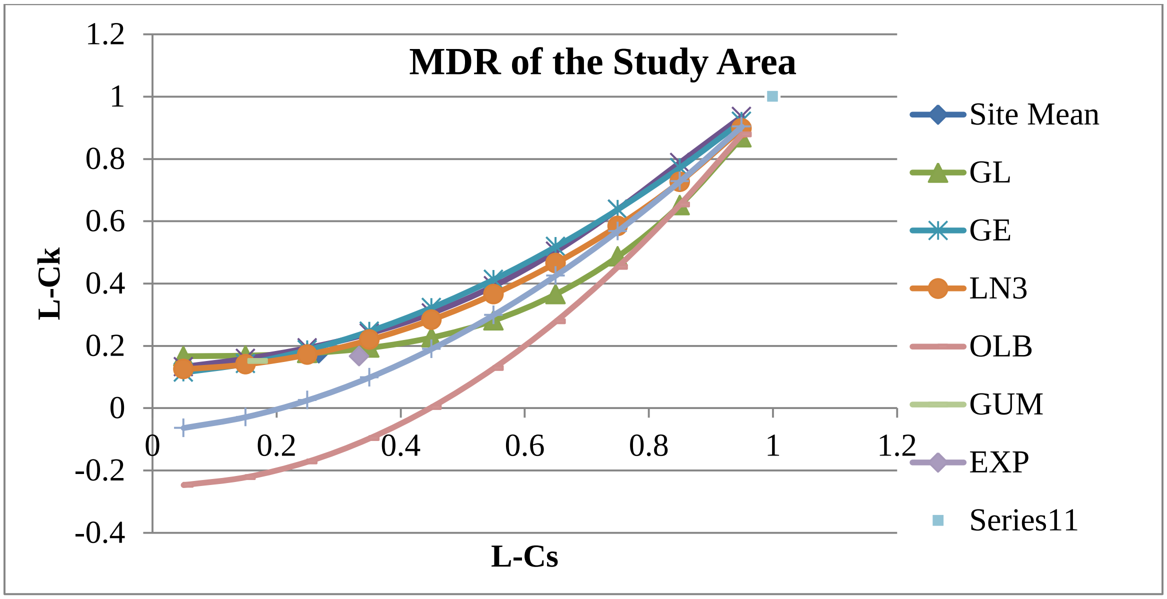

Figure 3.3 MDR of the study area

Table 3.4 Storm occurrence for different return periods

T	F	P	1/p^2	W	Z	KT	log(QT)	$Q_T=10^{logx}$
2	0.5	0.5	4	1.17741	-0.1600	-0.0991	1.69364	49.39035
5	0.8	0.2	25	1.79412	0.70094	0.72724	1.75496	56.8803
10	0.9	0.1	100	2.14596	1.1528	1.12587	1.78454	60.88914
15	0.93	0.067	225	2.32725	1.37812	1.31592	1.79864	62.8987
25	0.96	0.04	625	2.53727	1.63405	1.52489	1.81414	65.18493
50	0.98	0.02	2500	2.79715	1.94438	1.76861	1.83223	67.95651
100	0.99	0.01	10000	3.03485	2.22310	1.97864	1.84781	70.43933
200	0.99	0.005	40000	3.25524	2.47782	2.16340	1.86152	72.69839
500	0.99	0.002	250000	3.52550	2.78607	2.37800	1.87744	75.41326
100 0	0.99 9	0.001	100000 0	3.716922	3.00202 3	2.52258 3	1.88817	77.29931

Table 3.5 summery table for storm

Summary of 24hr Rain fall	
Return period	Using log-Pearson Distribution
T(Years)	QT=10^logx
2	49.39035
5	56.8803
10	60.88914
25	65.18493
50	67.95651
100	70.43933
500	75.41326
1000	77.29931

3.4.6 Estimation of design discharge

In general, three types of estimating flood magnitudes (namely: The Rational Method, SCS method and Gauged Data method) can be applied for ungauged catchments. Since, the catchment area of Waira diversion scheme is greater than 50 hectares; The US Soil Conservation Services (SCS) Method is preferred.

3.4.7 Estimation of Excess Rainfall/Runoff

A relationship between accumulated rainfall and accumulated runoff was derived by SCS (Soil Conservation Service). The SCS runoff equation is therefore a method of estimating

direct runoff from 24-hour or 1-day storm rainfall because the catchment area greater than 50ha and also simple to know runoff. The equation is:

Q = (P – Ia) 2 / (P – Ia + S) ---3.12

Where, Q is accumulated direct runoff in mm, P is accumulated rainfall (potential maximum runoff) in mm, Ia is initial abstraction including surface storage, inception, and infiltration prior to runoff in mm, S is potential maximum retention in mm. The relationship between Ia and S was developed from experimental catchment area data. It removes the necessity for estimating Ia for common usage. The empirical relationship used in the SCS runoff equation is:

$Ia = 0.2S$-- 3.13

Substituting 0.2S for Ia in equation, the SCS rainfall-runoff equation becomes:

$Q = (P - 0.2S)^2 / (P+0.8S)$ --3.14

S is related to soil and land cover conditions of the catchment area through CN and S is related to CN by:

$S = 254x\ [(100/CN) - 1]$ Conversion from average antecedent moisture conditions to wet conditions can be done by using tables or multiplying the average CN values.

3.4.8 Convoluting Excess Runoff using the SCS Unit Hydrograph

At the heart of the SCS UH model is a dimensionless, single-peaked UH. This dimensionless UH expresses the UH discharge, qt, as a ratio to the UH peak discharge, qp, for any time t, a fraction of Tp, the time to UH peak.

3.4.9 Determination of Curve Number

The curve number (CN) for the watershed is determined from the land use/land cover and soil data of the watershed. Secondary data from GIS sources (Ethio GIS and Woody Biomass) was used to extract the required information for the watershed.

$CN = \frac{As * Cs + Ag * Cg}{A\ tot}$---3.15

Where, CN is weighted curve number of catchment, As is area of shrub land in m^2 Cs is weighted curve number of shrub Ag is area of grass land in m^2 Cg is weighted curve number of grass land

3.4.10 Computation of Peak Floods

For the computation of the design flood using the SCS Synthetic Unit Hydrograph method, the catchment and the drainage network above the diversion site has been delineated from the 30m by 30m DEM using in the GIS. The GIS processing phase includes derivation of the important morphological characteristics that is used to derive the maximum time of flow concentration (tc), such as the longest flow length (L), the censorial flow length (Lc), the average slope.

3.5 Evaluating hydraulic failure analysis

Surface flow analysis means the determination of the- flow condition on upstream and downstream of the weir at different flow rates and to size different parts of the structure accordingly, so that the structure serves the purpose for which it is built Shape of the weir, Shape of canal, Weir height, Clear water way, Discharge and head over the weir, Scour sluice discharge, RL U/s and D/s scour hole, Depth of U/s and D/s sheet pile, U/s and D/s protection work, Length of the weir Scour depth, Canal width and depth, velocity and manning roughness coefficient.

$P=4.75\sqrt{Q}$--3-.21

$q=Q/L$---3-22

The regime scour depth is calculated from laces formula.

$R`=1.35(q^2/f)^{1/3}$---3-23

Where q is the unit discharge and f is the silt factor. The regime velocity and hydraulic radius, wetted perimeter and slope are calculated from the expression.

$V=\left(\frac{\mathbf{Qf^2}}{\mathbf{140}}\right)^{1/6}$ in m/s---3-24

$R=\frac{\mathbf{5}}{\mathbf{2}}(\mathbf{V^2}/f)$ ---3-25

$S=\frac{f^{5/3}}{3340Q^{1/6}}$ ---3-26

Where P is Flow perimeter in m. Q is Discharge in m3/s, R is Hydraulic radius in m, S is Stream bed slope (−), V is Average flow velocity in m/s f is Silt factor (no consistent unit), R` is Scour depth in m and q is discharge per unit width in $m^3/s/m$

Depth of u/s and d/s sheet piles are fixed based up on the maximum scour depth

Depth of u/s sheet of piles =1.5R--3-27

Depth of d/s sheet piles =2R--3-28

For canal by using lacey's formula

Analysis of silt along the canal has an advantage to maintain the irrigation canal and minimizing shortage of water due to silt accumulation. Therefore, this study used the lacey'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:

$$n=\frac{AR^{\frac{2}{3}}S^{\frac{1}{2}}}{Q} \qquad \text{3-29}$$

Where, Q is discharge of the channel in m^3/s, V is velocity in m/s, R is Hydraulic radius in m, A is Wetted cross-sectional area in m^2, S is bed slope (-), n is Manning's roughness coefficient (-)

3.6 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 below figure 3.2

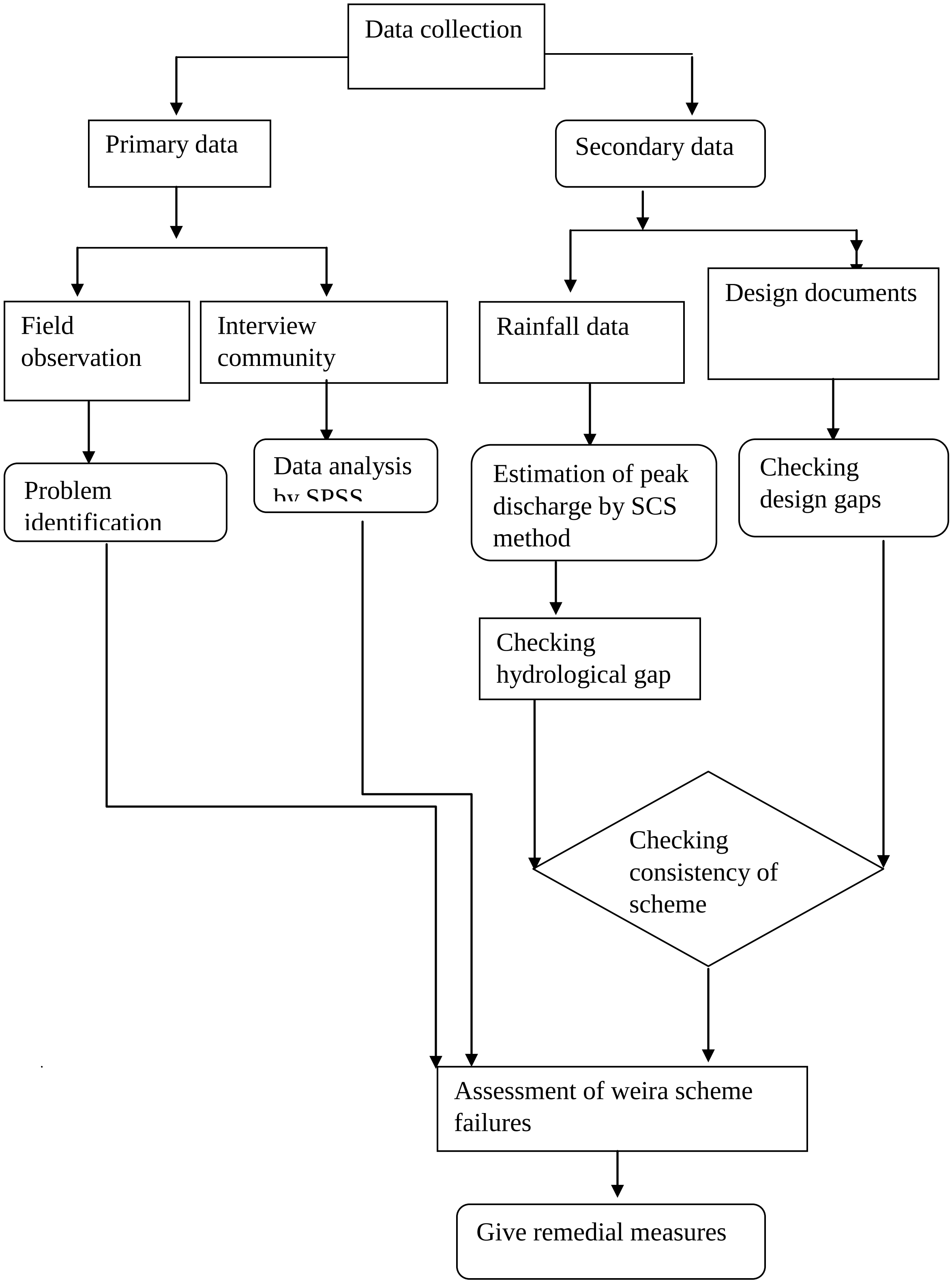

Figure 3.4 Conceptual Frame work of the study

4 RESULT AND DISCUSSION

According to Lambisso, (2005) number of such schemes have been designed and constructed in the previous years. However, while some schemes are performing successfully, it has been observed in various reports that some of the schemes have failed to serve the purpose for which they are intended.

Investigation of physical problems and rehabilitation method for the failed structures are the output of this study. This chapter presents the results of causes of failure problems for the selected small scale irrigation.

The causes for the failure may attribute to issued by field observation and from field interviews a number of irrigation beneficiaries are suffering from the problems associated with lack of adequately performed its duties and responsibilities of management body.

Hydrological failure of scheme shows the difference of the peak discharge obtained by computing is greater than design value that disturbs the irrigation scheme or destabilizes the size of the scheme these shows overtopping and erosion of scheme.

Hydraulic failures shows different part of scheme failed due to the size of scheme design is different from computed value and field measured value these shows highly scour and overtopping. Also the protection work of upstream and downstream value computed is different from design value, the depth of weir and cross-section of canal from field measure is different from designed value these leads hydraulic failures.

Therefore, the results of the problems with respect to investigate inventory on the failures case, hydrological, hydraulic of the schemes and give remedial measures for the schemes are given below:

4.1 Investigated inventory on the failure cases of small-scale irrigation schemes

During assessment of scheme field visit and interviewing questionnaires of scheme by community the following problem observed in weira small scale irrigation structures.

- ✓ Damage of main canal

- ✓ Damage of Sluice gate
- ✓ Damage on downstream apron
- ✓ Grass and silt accumulation.
- ✓ Location of sluice gates
- ✓ Key constraints related problems of scheme

4.1.1 Damage of main canal

The problem of main canal damage is observed in in field visit fig 4.1 schemes considered for the purpose of this study 43.3% of canal damaged. Pinpoint other causes observed during the field visit illegal off takes. These were produced by breaking the canal observed in most of the irrigations schemes resulted in uncontrolled water off take and loss. Upstream flooding and poor coordination of irrigators by the absence of WUA's for maintenance activities are observed. The poor coordination of maintenance resulted from absence of properly organized irrigation water users and follows up their contributions by the uses to conduct the maintenance work and help to sustain.

4.1.2 Damage of Sluice gate

Damage on sluices gate table 4.1 and shows is observed during field visit fig 4.2 and questionnaires 26.7% of sluice gate damaged. The cause for the problem of gates can be mainly attributed to improper scheme operation, improper hydraulic and hydrological design. Improper operation and maintenance is the factor of absence of WUA and poor operation and maintenance skills of community, lack of training give for community irrigation scheme operation and maintenance.

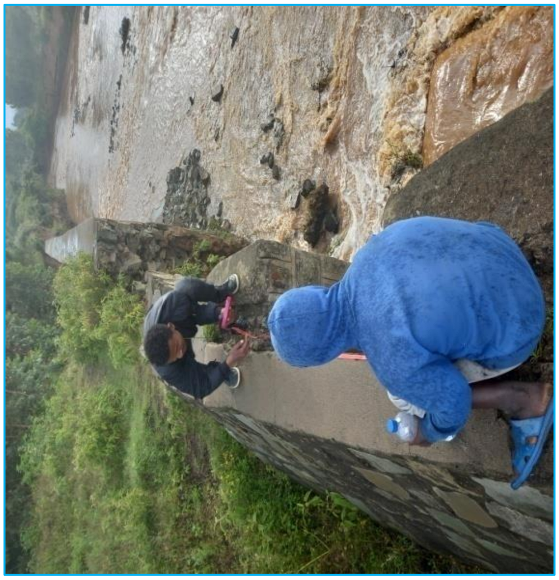


Figure 4.1 sluice gate

4.1.3 Damage on downstream apron

In case of weira irrigation scheme the provision of quality construction material reduce its ability to withstand erosive power of the energetic water and these will result in eroded apron surface these is the impact of improper hydrologic and hydraulic analysis. Therefore from table 4.1 and field visit fig 4.3 and questionnaires 28.3% damage on downstream apron.

Figure 4.2 damage of downstream apron

4.1.4 Grass and silt accumulation

In canal it is observed table 4.2 and during field visit fig 4.4 and questionnaires 38.3% damaged the problem of Grass and silt accumulation in canal on farm lands, especially canals are caused due to uncontrolled runoff from the watershed along the canals that initiated the canals bank erosion and consequently the bank collapse and 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 during field observation other animals dig holes in the canal this due to absence of WUA, lack of awareness of community and inadequate perform duties and responsibility and no formal handling of the project, have no sense of ownership of scheme and no formal management system for scheme.

Figure 4.3 grass and silt accumulation

4.1.5 Location of sluice gates

It's known that any irrigation schemes with sluice gate delivery system have gates with opening and closing mechanism. However if there is not proper operation mechanism of those gates clogging of gates once up on a time of their service life time is inevitable. And this problem is also prevalent in Weira where the project has a total no of 1 sluice gates with discharging capacity from the diversion to the command area of 4.95 m3/s whereas at the time of inspection at the site for this research work gate is completely damaged not clogged which means that the water is not currently being conveyed to the command area. Under sluices /scouring sluices are openings provided at the base of the weir or barrage but in Weira scheme sluice is not at the base of the weir it provided away the weir these is the main factor of the sluice gate damage at high flood season and upstream of the weir. The sluice gate is away from the weir controlling of water is difficult. However, if it is not controlled water the sluice gate and the command area is affected by flood and if it is controlled the upstream parts of the weir and the conveying structure from the weir to sluice gate structure is damaged.

Key constraints related problem of scheme

The constraints which are related to these selected schemes are not only the design and construction but also there are the main factors that affect the small scale irrigation.

4.1.4.1 Community involvement or participation

Table 4.1 community participation of scheme from respondent

		Frequency	Percent
Valid	Planning	6	10.0
	Design	4	6.7
	Construction	27	45.0
	all stage	6	10.0
	none stage	17	28.3
	Total	60	100.0

Accordingly, good performance of the schemes is directly related to the level of involvement of community members in the planning, designing and construction process. during field observation revealed that very poor participation of water user community in planning, designing, construction and evaluation of the irrigation schemes the above table 4.1 indicates the community participation in scheme is very low in design and planning these stage is the most important 93.3% and 90% are not participate respectively. The importance of participation of community is they know the environment condition, topography and flood which happen. For this reason highly participate community during design and planning stage. However, selected schemes which are built in the area was not expected level of participation of community observed.

4.1.4.2 Training given community and awareness

It is important to keep schemes sluice gate clogging, sedimentation and how to operate and maintain.

Table 4.2 training given for community

		Frequency	Percent
Valid	Irrigation development	32	53.3
	Irrigation structure operation and maintenance	2	3.3

	Crop cultivation activities	26	43.3
	Total	60	100

The training given for the community is focused only irrigation development and crop cultivation the training on irrigation structure operation and maintenance is very low in table 4.4 these affects the structure operation and maintenance. The skill community in operation and maintenance is the most important in irrigation scheme protection.

The survey on the maintenance and operation of the schemes (i.e., Weira SSI) has been conducted and response from the selected target community training given for community is irrigation development and crop cultivation the training in operation and maintenance is very poor. The results indicate 93.3% of community cannot operation and maintenance the irrigation schemes were damaged as the result of these poor operations and maintenance.

4.1.4.3 Establishment of WUA

Water user association (WUA) is a non-profit organization that is initiated and managed by the group of water users along one or more hydrological sub basin systems. The target of the formation of this legal status was equitable water distribution among the farmers regardless of their location, reliable water supply, quick dispute resolution, well maintained canal and overall efficient management of the schemes.

The interviews of the schemes respondents were conducted which indicates 83.3% (of the respondents said “not establishment of water use association” and 13.3% are not known. Beneficiaries had no role in scheme maintenance even the canal was damaged in Weira SSI. According to the DAs of the kebele there was no any cooperation between the farmers and the institution that implemented the Weira SSI and that is why the scheme became failed before its life span. According to the FGD of the schemes due to the absence of WUAs there was no regular supervision and monitoring purpose needed for the sustainability of the schemes as the result it became totally failed.

Table 4.3 Selected Farmer's response on the establishment of the WUA

Condition of WUA	Frequency	Percent
Established	2	3.3
Not established	50	83.3
Not known	8	13.3
Total	60	100.0

4.1.4.4 Institutional problem

The selected schemes which are built in the area the expected level of participation of these institutes was not observed. During field observation according to key informant interview (KI) interview of the Weira SSI which is implemented by this institution there was no proper hand over of the project. Handing over of irrigation systems to farmers, upon completion of construction, has been a standing procedure in small-scale irrigation development. It was based on a desire to decrease the resource burdens of the government for irrigation operation and maintenance. However, 96% of the respondent said that there was no formal handing over of the project to the beneficiaries. In addition of this, there was no the proper formation of WUA for the schemes. According to KI interview of the schemes the institutions did not consider more about the participation of the whole water user community (WUC) and there was no known or legal WUAs and they did not give any training concerned to the operation of this scheme.

4.2 Hydrological Failure analysis

Table 4.4 the following are the steps to calculate the peak discharge of the river

Catchment area	Km^2	245
Length of main river	M	28993.00
Time of concentration, Tc	Hr	7.43
Rain fall excess duration Duration(D = Tc/6)	Hr	1.24
Time to peak, Tp = 0.6 Tc + 0.5 D	Hr	5.08
Time to base, Tb = 2.67 Tp	Hr	13.56
Lag time, Te = 0.6 Tc	Hr	4.46

Peak rate of discharge p = (0.208* A) / Tp	m^3/sec/mm	10.04
Curve no. Condition two		70.73
Curve no. Condition three		84.75
Design Storm,	Mm	67.96
S = 25400/CN - 254, CN corresponding to AMC II		45.70

Table 4.5 composite hydrograph of study area

Time	H1	H2	H3	H4	H5	H6	SUM
0	0.00						
1.24	0.00	0.00					0.00
2.48	0.00	-0.24	0.00				-0.24
3.72	0.00	-0.48	4.30	0.00			3.82
4.96	0.00	-0.71	8.60	21.58	0.00		29.47
5.64	0.00	-0.84	10.94	33.41	11.22	0.00	54.73
6.20	0.00	-0.95	12.90	43.15	20.44	4.40	79.94
6.32	0.00	-0.94	13.31	45.25	22.44	5.34	85.40
7.56	0.00	-0.79	11.61	66.82	42.94	15.90	136.48
8.80	0.00	-0.66	9.92	53.3	63.34	24.85	155.75
10.04	0.00	-0.52	8.23	49.77	55.26	34.60	147.34
11.28	0.00	-0.34	6.51	41.25	47.18	30.19	124.79
13.56	0.00	-0.14	3.40	25.57	32.32	22.07	83.22
14.80	0.00	0.00	1.70	17.05	24.24	17.65	60.64
16.04	0.00		0.00	8.52	16.16	13.24	37.92
17.28	0.00			0.00	8.08	8.83	16.91
18.52	0.00				0.00	4.41	4.41
19.76	0.00					0.00	0.00

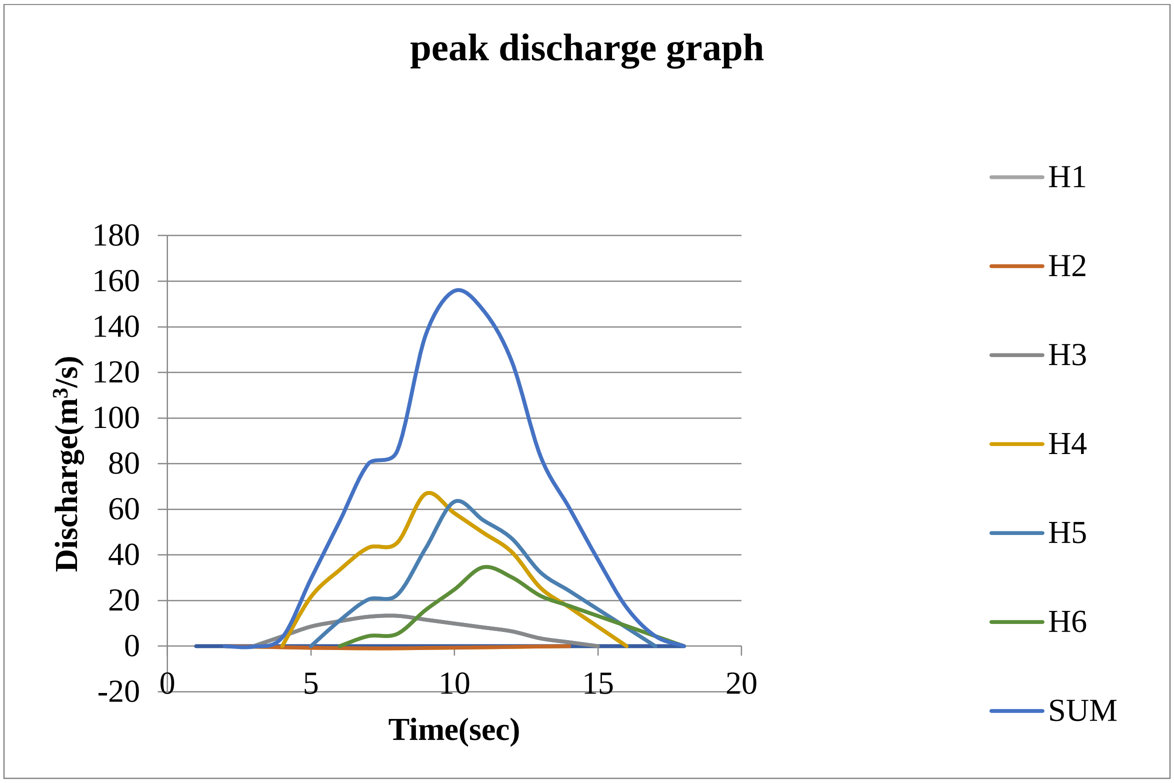

Figure 4.4 Peak Discharge

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

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 steepest descent direction.

The Flow accumulation grid is the core grid used in the stream delineation which was extracted from the flow direction grid. The flow accumulation records the number of cells that drain into an individual cell in the grid. Generating the longest flow path would help us to determine the slop and time of concentration. Based on this rationale was generated from Arch Hydro tools to determine the longest flow path of the watershed.

The estimation of flood magnitude is an important parameter for the design and analysis of structures to be constructed on the river. In the present study area, the flood estimation was not done accurately using long-term rainfall data. Therefore, rainfall data of the same station was collected and revised estimates of the peak annual rainfall and peak discharge were estimated using SCS method. Therefore, after the necessary goodness of the fit test was carried out and the peak rainfall (i.e. 67.8mm) was determined using the log-Pearson type III distribution method for the 50 years returns period. Using this method in table 4.7 and fig 4.10, the peak discharge was found to be 155.75m^3/s. However, there is a discharge difference of 37.75m^3/s which can destabilize the structure. The composite hydrograph also calculated by adding H1-H6 horizontally. Therefore, the Weira irrigation scheme fails by hydrological because the design discharge is $118m^3/$ and peak discharge $155.75m^3$/s.

When we compare 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 scheme. Therefore, underestimation of the catchment area of streams caused serious flood around the scheme the magnitude of the damage of the irrigation structure due to flood.

The drop structure of the scheme 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 scheme is lack of appropriately estimated hydrological design of the irrigation project.

Figure 4.5 overtopping of canal

4.3 Hydraulic failures Analysis:

According to NBCBN (2005) assess the existing river diversion structures in some countries of NileBasin and tries to identify hydraulic problem of scheme categories of prevalence of scouring, damage of main canal, damage of downstream apron, upstream flooding, damage on retaining wall, seepage of weir and damage on downstream cutoffs.

Hydraulic parameters change has a great effect on failure of irrigation scheme. In the

Hydraulic analysis, the Hydraulic design report is evaluated and the main factor which may facilitate the failure of the scheme is identified. Field observation has been made to identify problems and Factors which facilitate the failure of the weir is identified

In general, the hydraulic failure analyses consist of the estimation of these parameters Shape of the weir, Weir height, Clear water way, bed intake of the weir, passage of sedimentation, RL upstream and downstream scour hole, Depth of upstream and downstream sheet pile, Upstream and downstream protection work, Length of the weir, Scour depth, Canal width, depth, velocity and Manning roughness coefficient.

Table 4.5 hydraulic analysis of structures.

Hydraulic structure	Design	New calculated
Flow intensity ($m^3/s/m$)	2.28	2.63
Discharge through sluice scour (Qs)	4.95	4.95
Scour sluice height (m)	0.75	0.75
Sluice width (m)	1.2	1.2
No of gates	1	1
Sluice Flow intensity ($m^3/s/m$)	0.47	0.47
Scour sluice total width (m)	1.2	1.2

Canal And weir parameters	designed value	Computed value	Field measured cross section Canal
Scour depth (m)	0.27	0.72	-
Slope (-)	0.0028	0.002688	-
Depth of sluice gate	0.9		0.65
Sluice gate	1.5		1.2
Depth of weir(m)	3.75		3
Width of weir(m)	27.5		23

4.3.1 Shape of the weir

The shape of the weir is decided based on its practicability and economy of the structure whether to use masonry, rock fill and concrete weir for the construction of the weir. There the selected shape of the weir at the Weira river cross section must be reinforced concrete broad crested weir with upstream vertical face and downstream slopping face because the area is not rocky and steep slope. But previously the constructed weir was masonry type with poor wok man ship and poor construction materials. In addition to there was poor hydrological analysis and there was no proper geological survey which is resulted with collapsing of the weir and water pass below the foundation. The reason for selecting reinforced concrete weir the river is at the huge boulder stage.

4.3.2 Weir height

Considering the level of the land to be irrigated, the weir height should sufficient enough to divert the water to the required level. Hence the height of the weir is determined to be table 4.7 by measuring in 3m which design value be 3.75m which impact the structures during rainy season destabilize structures in downstream part because there is no strong energy dissipater and so these indicates the poor supervision the schemes fail.

4.3.3 Length of the water way

The shorter the weir the less will be the cost of the main structure but on account of the increase in discharge per unit run. Dynamic action on the river bed downstream will be sever, guard against which the thickness and the length of the impervious and pervious floor will have to be increased. Therefore in table 4.7 the length of the water way is determined from the lacey's wetted perimeter is 59.2m and the design water way is 51.6m. The discharge per unit length or the discharge intensity for this weir structure is estimated to be $2.63m^3/s/m$ and designed value $2.2m^3/s/m$ when these values are occurred at high flood conditions the finding outcome of the scheme is the collapse of wall and upstream flooding.

4.3.4 Regime scour depth

When the natural waterway of the river is contracted, the waterway scours the bed both on upstream and downstream of the structure. These phenomena table 4.7 occurred when there is high flow of water over the structure without the design of the hydraulic jump. The scour whole may form may progress towards the structure causing its failure and it is estimated to be 2.6m and design scour 2m and prevent the failure of this structure the piles should be provided at the upstream and downstream of the structure and estimated to be 4m and 5.26m respectively and the designed value to be 3.4m and 4m respectively. Therefore the difference of design value and computed value for these reasons the scheme fails by regime scouring these impacts upstream and downstream structure.

4.3.5 Passage of sedimentation

Sedimentation problem around the weir was not found in weira irrigation project. the consequence of these results increase the depth of the weir bed at the upstream and downstream of the scheme. The observation of sedimentation problem of the project bed of the weir regarding work resulted have been tabulated.

Table 4.6 Bed level lower of weir due to erosion and scouring

St name	ELV(m) design	ELV(m) current	ELV(m) difference
Bed level of upstream weir	1870	1869.37	-0.63
Bed level of downstream weir	1869.5	1868.75	-0.75

4.3.6 Bed intake of the weir

For project bed intake gates are very crucial phenomena for safe obstruction of the intended amount of flow throughout the project service period, through seasonal variation of flow without occurrence of elevation difference due to scouring and erosion of bed materials.

Weira project is found to be not safe against this currently as been realized from surveying work was lowering the elevation at bed levels of the concerned point which has been surveyed due to scouring or erosion. The average lowering of elevation is -0.69 m due to erosion and scouring.

4.3.7 Seepage through Foundation

Seepage failure of scheme the water is totally flowing beneath the foundation of the structure resulting in loses of water due to seepage of water under the foundation of the scheme. In addition to this, there was improper design of upstream and downstream impervious structures and improper hydrological analysis mean that the design discharge is less than current condition of discharge there is no sedimentation of scheme these results scouring of scheme.

4.3.8 Downstream Scouring

The problem in table below 4.8 is attributed to the improper structural selection for the site or hydraulic design. This problem was observed in Weira schemes. In case of Weira downstream scouring problems was intensified due to the resulting in proper dissipation of energy of flowing water on stilling basin. The problem is observed on the schemes and it is caused due to improper hydraulic design which arises from poor knowledge of the energy dissipation. The impervious floor is designed in all cases to reduce the surface flow action that causes scouring due to unbalanced pressure in the hydraulic jump trough. In the study area this problem occurs due to silting of stilling basin. The prolonged occurrence of abrasion and scouring of downstream portion of the structure may end up in the total collapse of the structures.

Table 4.7 depth of sheet pile scour

Upstream scour	Design document	New evaluated
Silt factor	1.1	1.1
Discharge intensity (m^3/s/m)	2.28	2.63
Depth of scour (m)	2.2	2.6
D/S scour (m)		
Discharge intensity 'q' (m^3/s/m)	2.23	2.63
Depth of scour (m)	2.2	2.6

4.3.9 Upstream and downstream protection works problem

The table 4.9 impervious floor of a weir and head regulator is normally protected on the upstream as well as downstream by loose aprons. Because of poor upstream and downstream protection work scour is expected and hence there is the difference of design and calculated protection works have on the design of Weira small scale irrigation weir these results scouring problem. Hence sheet pile used to protect scouring problem of scheme there is the difference of sheet pile upstream and downstream coppering design value of upstream and downstream sheet pile by computing, then the computing value is high because that resist the flow which comes and the design value of sheet pile is less that cannot resist the flow which comes then it eroded or scoured it seen during field survey.

Table 4.8 Protection works

Upstream protection work	Design documents	New calculated
Depth of scour 'R (m)	2.2	2.6
Anticipated Scour (m)	3.4	4.0

Upstream scour level (m)	1867.8	1866.77
Downstream protection work		
Depth of scour (m)	2.2	2.6
Anticipated scour (m)	4.56	5.26
Downstream scour level (m)	1867.3	1866.15

4.3.10 Poor Energy Dissipater

The high energy loss that occurs in a hydraulic jump has led to its adoption as a part of high energy dissipater system below a hydraulic structure. In the case of Weira small scale irrigation scheme design, as water flows by high energy down the sloping glacis and this maximum energy is not dissipated due the critical depth of design is less than computed value and absence of proper design energy dissipater, the energy will scour the downstream floor until the main weir and this may be taken as failure causes of weira small scale irrigation.

4.3.11 Irrigation Canal

Hydraulic parameters change has a great effect on failure of irrigation canals in table 4.7. 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 from design value on the main canal The finding decrease on the values of the calculated Manning's roughness coefficient (n) causes increasing of discharge and velocity from the design. The increase in velocity and discharge causes the overtopping the canal and scouring.

Overtopping the canal could usually causes distraction of the canal structures and scouring of the canal and erosion of farm land these leads to clogging of canal this clogging cause water pass into the secondary canal loss leads to decreasing discharge and velocity it causes silt deposition in the irrigation canal and weeds to grow along the canal. During field measure the design value of cross-section, the design value of the canal is the great difference of field measure value. These seen in above field survey weed growth, destruction of canal and sluice failures.

Table4.9. canal section during field visit and design

Canal	designed value	Computed value	Field measured cross section Canal
Discharge (m^3/s)	3.78	4.95	-
Velocity (m/s)	0.5	0.6	
Area (m^2)	7.56	8.4	0.54
Width (m)	7.12	8.6	1.2
Depth (m)	1.062	0.9	0.6
Wetted perimeter	9.24	10.56	2.1
Hydraulic radius	0.56	0.791	0.27
Manning roughness coefficient	0.07	0.022	

4.4 Remedial measures

Since the observed problems did not completely cease the schemes' operation and they reduce the efficiency of the schemes, some structures should be rehabilitated or renewed. Considering the research study findings to alleviate the problem of the case study, the following remedial measures are provided:

4.4.1 Conduct operational, planning and institutional problem

The gate and gate site must be kept clean and occasionally the gate will have to be dismantled and the accumulated sand and silt removed and nominating water committee which supervise irrigation user and collect contribution for maintenance and operation fee. At the planning and design actively participating community because the community knows the topography and runoff condition of environment, also community take the responsibilities and owner of structure during planning and design the institution strongly supervise the construction by design standard with regarding to cross-section, mix ratio and class of contractor.

4.4.2 Hydrological remediation

The occurrence of a certain peak flood of given return period during the life time of a project is a probability of a given flood that should be safely discharged from the diversion structures (weir and under sluices). Particularly the structural safety as well as economic design of hydraulic structures depends on the accuracy of the design flood which is a function of robustness of the method of estimation and the available data therefore peak discharge is 155.75m^3/s instead of 118m^3/s.

4.4.3 Downstream scouring and damage of downstream apron

Damage on the downstream apron was caused by excessive energy water carried by upstream water. The problem is common in other part of the country. In SSPN region 19% of the schemes are seen to be affected with this problem (Lambisso, 2005).

Accordingly, proper design of both impervious and pervious apron are required to control the excessive upstream energy and control structures should be provided to manage the problem of receding jump. Consequently, accessories to control the jump are usually installed in the basin. The main purpose of such control is to shorten the range within which the jump will take place and thus to reduce the size.

Prevent such problems additional thickness of the impervious floor is provided at the point where the hydraulic jump is formed to counterbalance the suction pressure. There is also proper design of launching apron should be provided.

4.4.4 Seepage through Foundation

To avoid such seepage condition providing cut-offs (sheet piles) in the riverbed at the upstream and downstream ends and, most commonly in the form of sheet piling (steel or concrete). The cut-offs extend the seepage path and reduce the hydraulic gradient that causes piping and the structure can be equipped with vertical cut-offs. They hinder the water flow along and underneath the structure. The cut-offs are part of a structure and can be driven into the bed and the embankments of a canal.

Table 4.10 Protection works

Upstream protection work	Value provided
Anticipated Scour (m)	4.0
Upstream scour hole (m)	1866.77
Downstream protection work	
Anticipated scour (m)	5.26
Downstream scour hole (m)	1866.15

4.4.5 Sluice Gate

To avoid such condition sluice gate therefore, it is possible to make flexible and movable using different smoothing materials and removal of sediments that are stored in front of sluice gates. In addition to this, presence of skilled water user associations (WUAs) has also resulted in organizing and assessing gate operators especially during high flood seasons. This operational problem can be improved by offering training to the users and carrying out proper handing over and follow up in operation of the schemes. If the users properly trained, and they are provided with maintenance tools, they can be able to effectively operate and maintain the damaged gates and also the cross section of gate must be sufficient to pass water into the main canal.

4.4.6 Canal siltation

The causes of the canal siltation scouring sluices are provided for exclusion of siltation of main canals and other way of silt removal should properly be provided.

Sediment control structures should be used at the intake from rivers to exclude or extract sediments mostly sand and gravels.

Avoid such canal siltation condition Sediment exclusion by closing the canal or by reducing canal flows during periods in which high sediment concentrations are transported. Sediment trapping in very large settling basins constructed at the head of the schemes resulting in very low velocities of flow.

4.4.7 Damage on farm lands canals

The causes of this problem were described in detail from the point view of the observed schemes. Provision of designs for controlling runoff from watershed by providing appropriate canal slope and canal structures which could pass the designed supply of water would prohibit further damage., the cut off trench should be properly designed to catch the water running from the watershed and Canal Bank Protection by Eradication of animals, protection of waste deposit from agricultural land into the canal and protecting the canal embankment by lining for which different materials.

4.4.8 Energy dissipater problem

The gabion energy dissipater is suitable for sites where the tail water depth is low and the rock in the downstream area is not good and doesn't resistant to erosion. Throws the jet at a sufficient distance away from the structure` where a large scour hole may be produced and the exit gradient should be checked

5 CONCLUSION AND RECOMMENDATION

5.1 Conclusion

The capacity to pass the estimated peak flood safely over the weir has been checked and accordingly, the estimated peak flood cannot safely flow over the weir it passes beneath of foundation.

The hydraulic design evaluation shows the poor of the main components of hydraulic design like upstream and downstream protection works, energy dissipation and plays a great role for the failure of Weira small scale irrigation scheme.

The analysis of problem revealed the major problems of the existing irrigation structures considered for this study includes: sedimentation of the headwork of main canal siltation, problem of seepage through foundation, scouring of downstream and upstream flooding.

Proper hydrologic and hydraulic analyses of the systems are very important factors that are contributed for the good hydraulic performance and sustainability of irrigation schemes and According to the field visit of the selected schemes the main causes of failures of these schemes are very poor participation of target community of the schemes in irrigation development stages, lack of proper training and absence of proper supervision, monitoring and evaluation of the schemes. Malfunctioning of the gates caused sediment deposition in front of the canal head regulator, the sediment deposited clogged the under -sluice this intern allowed entry of silt in to the main canal.

5.2 Recommendations

The following recommendations are believed to contribute for improving and reducing the problem s of small scale irrigation schemes of the study area.

Irrigation schemes are designed and constructed at headwork and main canal levels. Distribution systems should also be properly designed and constructed for proper management and utilization of water.

Community awareness should be created to develop sense of ownership and culture of understanding to remove agricultural wastes and cattle obstruction from canal. Therefore, farther research has to be done on the management and construction problems of Weira small scale irrigation scheme.

Proper hydrologic and hydraulic analyses are very important which contributes to the design objectives and sustainability of the irrigation scheme.

Supervision and evaluation of the head-work on the site should have to be carried out on a certain time interval to preserve the diversion structure from failure due to hydraulic. This will serve for proper implementation and functionality of the project.

Continuous monitoring and evaluation of irrigation schemes is necessary to provide feedback and information important for the future planning the management of new schemes.

The community should fully participate throughout the project planning, implementation and evaluation phases

Giving training for community about operation and maintenance of irrigation schemes. Therefore, farther research has to be done on operation and maintenance problem.

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7 APPENDIX OF THE THESIS

Appendix A respond of community during field visit and interview

Table 7.1 response of community failure section

		Frequency	Percent
Valid	damage of canal	26	43.3
	damage of sluice gate	16	26.7
	damage of downstream apron	17	28.3
	damage of upstream of weir	1	1.7
	Total	60	100.0

Table 7.2 of failure problem of irrigation scheme

	Problem	Frequency	Percent
1	Sedimentation problem	7	11.7
2	Drinking animals	1	1.7
3	Grass and silt accumulation in canal	23	38.3
4	Seepage from the headwork and/or canals	9	15.0
5	Breakage of gates by illegal users	8	13.3
6	Upstream and downstream flooding	12	20.0
	Total	60	100.0

7.3 community awareness of operation and maintenance

		Frequency	Percent
Valid	Good	6	10.0
	very good	4	6.7
	very poor	36	60.0
	Poor	14	23.3
	Total	60	100.0

Appendix B hydrological analysis

Table 7.4 time of concentration determination

Length of flow (m)	Elevation (m)		H (km)	Slope	Tc (hrs)
0.00	2117.00	0.00	0.00	0.00	0,00
10401.0	1968.00	10.40	149	0.01	2.06
16345.0	1889.00	16.35	79.00	0.00	4.44
130.00	1886.00	0.13	3.00	0.02	0.06
2117.00	1874.00	2.12	12.00	0.01	0.87
	Total L(m)	28.99		Tc(total)	7.43

Table 7.5 determination of incremental rainfall

Duration	50yr RT RF	Rainfall profile		Area to point ratio	Areal rainfall	Incremental rainfall	Descending order	Rank
Hr	Mm	%	Mm					
0	67.96	15.0	10.2	45.0	4.6	4.60	15.30	1
1.24		50.6	34.4	57.93	19.9	15.30	9.30	2
2.48		62.8	42.7	68.3	29.2	9.30	6.21	3
3.72		70.6	48.0	73.8	35.4	6.21	4.60	4
4.96		75.0	51.0	76.9	39.2	3.81	3.81	5
6.2		79.	54.0	77.7	42.0	2.79	2.79	6

Table 7.6 determination of cumulative runoff.

Design arrangement rank	50yrs incremental RF	Cumulative	Direct runoff	Time to begin	Time to peak	Time to end
6	2.79	2.79	1.02	0	5.08	13.56

4	4.6	7.39	0.07	1.24	6.32	14.80
3	6.21	13.60	1.40	2.48	7.56	16.04
1	15.3	28.90	8.06	3.72	8.80	17.28
2	9.3	38.20	14.40	4.96	10.04	18.52
5	3.81	42.01	17.86	6.2	11.28	19.76

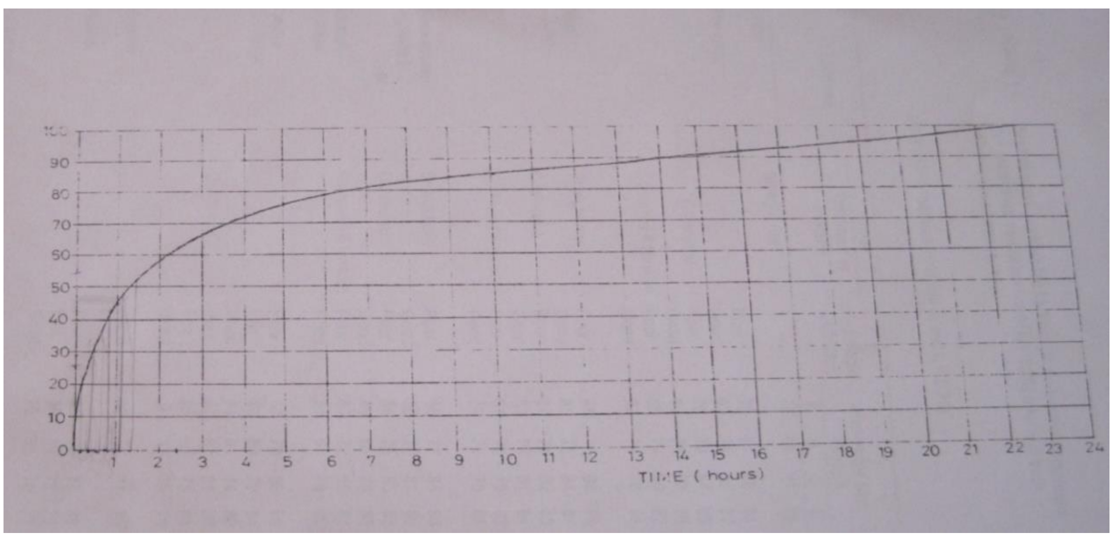
Table 7.7 determination of peak runoff

Duration (hr)	Cumulative Runoff	Incremental Runoff (mm)	Peak Runoff Incremental	Time of Begin (hr)	Time to Peak (hr)	Time to End (hr)	
0	1.02	0.00	0.00	0.00	5.08	13.56	H1
1.24	0.07	-0.95	-9.54	1.24	6.32	14.80	H2
2.48	1.40	1.33	13.36	2.48	7.56	16.04	H3
3.72	8.06	6.68	66.82	3.72	8.80	17.28	H4
4.96	14.40	6.34	63.40	4.96	10.04	18.52	H5
6.2	17.86	3.46	34.60	6.20	11.28	19.76	H6

Table 7.8 of area to point ratio

Table - IVA-1B.1 Areal to Point Ratio (%)

Area km²	Duration I(hrs)	0.50	1.00	2.00	3.00	4.00	5.00	6.00	9.00	12.00	15.00	18.00	21.00	24.00
25		88	78	82	85	87	88	88	91	92	93	93	94	94
50		61	71	78	82	84	85	87	89	90	91	92	92	93
75		57	67	75	79	82	84	83	87	89	90	91	91	92
100		54	65	73	78	80	82	83	86	88	89	90	91	91
125		52	63	72	76	79	81	82	85	87	88	89	90	91
150		50	61	70	75	78	80	61	84	86	88	89	89	90
175		48	59	69	74	77	79	81	84	86	87	88	89	90
200		46	58	68	73	76	78	80	83	85	87	88	88	89
225		45	57	57	72	75	77	72	82	85	86	87	88	89
250		44	55	66	71	74	77	78	82	84	86	87	88	88
275		42	54	65	70	74	76	78	81	84	85	86	87	88
300		41	53	54	70	73	75	77	81	83	65	86	87	88
325		40	53	63	58	72	73	77	80	83	84	86	87	87
350		38	52	63	68	72	74	76	80	82	84	85	86	87
375		39	51	62	68	71	74	78	80	82	84	85	86	87
400		38	50	61	67	71	73	75	79	82	83	85	86	87
425		37	50	61	67	70	73	75	79	81	83	84	85	86
450		36	49	60	66	70	72	74	79	81	83	84	85	86
475		36	48	60	66	69	72	74	78	81	83	84	85	86
500		35	48	59	66	69	72	74	78	80	82	84	85	86
525		34	47	59	65	68	71	73	78	80	82	83	85	85
550		34	47	58	64	68	71	73	77	80	82	83	84	85
575		33	46	58	64	68	71	73	77	80	82	83	84	85
600		33	45	57	63	67	70	72	77	79	81	83	84	85
625		32	45	57	63	67	70	72	76	79	81	83	84	85
680		32	45	56	63	67	69	72	76	79	81	82	84	84
675		31	41	56	62	66	69	71	76	79	81	82	83	84
700		31	44	56	62	66	69	71	76	78	80	82	83	84
725		31	45	55	62	66	69	71	75	78	80	82	83	84
750		30	43	55	61	65	68	71	75	78	80	82	83	84
						49								



Graph 7.1 of rainfall profile

Appendix C hydraulic analysis parameters.

Hydraulic structure	Design	New calculated
Downstream bed level of weir (m)	1869.5	1868.75
Upstream bed level of weir (m)	1870	1869.37
Peak discharge at 50 years (m^3/s)	118	155.75

Upstream water level (m)	1873.42	1874.39
Downstream water level (m)	1869.66	1870.63
Gates height (m)	0.75	0.75
Gates width (m)	1.2	1.2
Clear water way (m)	51.6	59.27

Calculations on withdrawal of water

The existing flow of water in to the main canal is taking place within only one gate opening which is opened about 0.75 m height . Using submerged orifice equation below the discharge in to the main canal can be estimated

$Q = Cd \times A \times \sqrt{2gh}$

Where, Cd is Discharge coefficient 0.64, A is Opening area (m^2), g is Gravity, (9.81 m/s^2), h is Upstream water level minus downstream water level (m).

$h = 1874.39 – 1870.63$

$h = 3.76$ m

$Q= Cd \times A\times\sqrt{2gh}$

$Q = 0.64 \times1.2\times0.75\times\sqrt{2\times9.81\times3.76}$

$Q = 4.95\ m^3/s$

Design document water way

$Q= 4.38\ m^3/s$

Appendix D research questionnaires for weira small scale irrigation

Questionnaires of the thesis from targeted community

Research Site: Region SNNPR

Zone Hadiya

Districts Shashogoworeda

Village …………..

Interviewer full name: ....Date of Interview....................age…....

1. What is the name of project?

2. When it was constructed?

3. What is the aim of the project?

4. Who initiate the construction of the scheme in your area?

1) Government 2) NGO 3) Community 4) 1&2 5) 1&3 6) All in collaboration

5. Were you happy when you first heard that an irrigation scheme was going to be constructed in your area? 1) Yes 2) No

6. If your answer to Q5 IS yes, why?

7. Has the scheme been constructed with consent and full participation of the target beneficiaries? 1) Yes 2) No

8. If your answer to 7 is yes at which stage of the intervention process you have

Participated? 1) Planning 2) Construction 3) Design 4)1&2 5)2&3 6)1&3 7) in all phases

9. Is there women involvement in irrigation activities before and after scheme development?1) Yes 2) No

10. If the answer is yes for Q 9 how they participated?

11. How do you manage your irrigation systems before intervention?

12. When was the construction completed?

13. Is there formal handing over of the project to beneficiaries? Yes/No

14. If not to whom did the project hand over the scheme

15 Is the management system put in place for developed scheme after intervention?

1) Yes 2) No

16.If your answer to Q 15 is yes, who manage the system now?

1) Community alone 2) WUA alone 3) NGO alone 4) Government alone

5) Traditional leader alone 6) All in collaboration (multiple answer is possible)

17. Is the management body adequately performed its duties and responsibilities?1) Yes 2) No

18. If your response to Q17 is No what do you think the reason?

19. Is the irrigation technology you use is simple to operate, maintain and manage?

1. Yes 2. No

20. If the answer to Q19 is No, what is beyond your ability?

21. Do you have sense of owner ship to the developed scheme? 1. Yes 2. No

22. What do you think should be done by the community to improve the sustainability of Scheme?

23. What is the source of water for your scheme?

1. River 2. Dam 3.Lake 4.Groundwater 5. Other specify

24. Which canal type will serve for long time?

1. Pipe system 2. Geomembrane lined

3. Traditional system 4. Others specify

25. Which section of scheme fail or damaged

1). damage of main canal. 2). damage of sluice gate. 3) .damage of downstream apron 4). damage of upstream weir

26. Do you know the cause of failure? 1. Yes 2 No

27. If yes, what are they?

1. Sedimentation problem 2. Drinking animals 3. Grass and silt accumulation in canal

4. Seepage from the headwork and/or canals 5. Breakage of gates by illegal users

6. Upstream and downstream flooding 7. Unavailability of water for irrigation

28. Types of training given to the target community after the implementation of the irrigation schemes

i.Irrigation development administration

ii.Irrigation structure operation and maintenance

iii. Crop cultivation and management activities

29. Did you ever took training on operation and maintenance of the structures and canals

1. Yes 2. No

30. if yes, your awareness of operation and maintenance of irrigation scheme?

Good 2) Very good 3) Very poor 4) Poor