APPRAISAL OF DESIGN PRACTICE AND FAILURE OF RIVER DIVERSION FOR IRRIGATION SCHEMES: A CASE OF WADLA WOREDA NORTH WOLLO, ETHIOPIA

M.Sc. THESIS

KUMNEGER DESALEGN

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INSTITUTE OF TECHNOLOGY, SCHOOL OF GRADUATE STUD IES,

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FEBRUARY, 2017

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## DECLARATION

I hereby declare that this M.Sc. Thesis entitled, ‘‘Appraisal of Design Practice and Failure of River Diversion for Irrigation Schemes: A Case of Wadla Woreda, North Wollo, Ethiopia’’ My own original work, and has not been presented for a degree in any other university, and all sources of material used in this thesis has been correctly acknowledged.

Name: kumneger desalegn

Signature: __________

Date: __________

SCHOOL OF GRADUATE STUDIES

ARBA MINCH UNIVERSITY

ADVISOR'S THESIS SUBMISSION APPROVAL

This is to certify that the thesis entitled ''Appraisal of Design Practice and Failure of River Diversion for Irrigation Schemes: A Case of Wadla Woredas, North Wollo, Ethiopia'' In partial fulfillment of the requirements for the degree of Master of Science with specialization in Irrigation and Drainage Engineering, the Graduate Program of the Department of Water resources and Irrigation Engineering, and work has been carried out under my supervision. Therefore, I recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department of defense.

DR. Abdella Kemal ____________ ______________________

Name of Advisor Signature Date

**APPROVAL PAGE**

This thesis entitled with ‘‘Appraisal of Design Practice and Failure of River Diversion for Irrigation Schemes: A Case of Wadla Woredas, North Wollo, Ethiopia.” has been approved by the following examiners in partial fulfillment of the requirements for the degree of Master of Science in Irrigation and Drainage Engineering.

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**ABBERVATIONS**

| | |
|---|---|
| DA | development agency |
| WE | woredas expert |
| SPSS | statistical package for social science |
| NGO | nongovernmental organization |
| O&M | Operation and maintenance |
| GOs | Governmental Organizations |
| GPS | Geographical Position System |
| D/S | Downstream |
| U/S | Upstream |
| NBCBN | Nile Basin Capacity Building Network |
| f | lacey's silt factor |
| $^{\gamma}w$ | weight of water |
| WUAs | water user association |
| SCS | soil conservation system |
| Qp | peak runoff for incremental |
| FSL | full supply level |
| L | lacey's regime width |
| TEL | total energy line |
| HFL | high flood level |
| Hs | seepage head |
| C | Bligh's creep coefficient |
| $^{\gamma}m$ | weight of masonry |

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## ABSTRACT

Wadla woreda is located in south western parts of north wollo zone at latitude of 11°36'35.03" N and longitude of 38°55'54.06"E. This study appraises the design practice and failure of the river diversion structure for irrigation in the study area and identifying, analyzing and categorizing of the main problems were focus points. This study considered by selecting five existing river diversion structures in different Kebeles of the woreda to understand the causes of the major failures of the structures by considering different aspects such as pre- and post-construction, institutional aspects, planning problems, social and operational problems and initial design documents. In order to identify and characterized the existing irrigation schemes field observation and measurements, group discussions, interview and questionaries' were used to achieve the objectives of this study and analyzed by SPSS statistical software. Arc-GIS 10.1 was applied to delineate the watershed of the study area, pick discharge estimated by Soil conservation system (SCS) and Auto CAD -2007 used for design purposes. The design practice of the studied area is very poor. In these study areas most of the implemented schemes are suffering from problems attributed to design, construction, and operation & maintenance that lead to major failure of the structures. From the analysis the maximum value of hydrology and sedimentation consideration, hydraulic design of weir and structural design of weir problems showed that 31%, 49% and 20%, respectively. Based on the questioner, interview and group discussion, results the maximum value indicates absence of project idea of the community, water user association and lack of community participation on construction are 100%, 96% and 100% respectively. Finally, failures were due to the problems in the hydrologic, hydraulic and structural design of diversion systems in addition to improper management of the schemes.

Key words: SCS, Wadla, Arc GIS, Irrigation Scheme, River Diversion Failure

# CHAPTER ONE

## 1. Introduction

### 1. 1. Background

The diversion of water for agricultural production often involves the use of formal irrigation schemes with extensive permanent infrastructural facilities as well as traditional flood recession practices under limited water control systems (Underhill, 1990).

A research work which is aimed at assessing the existing river diversion systems in the Nile Basin countries revealed that 69 % of the main problems that are obtained and extracted from the inventory data on the status of existing projects are related to the design of weirs and its components (Nile Basin Capacity Building Network, 2005).

Small scale irrigation structures, owing to their relatively small investment cost, ease of construction, simplicity of operation and maintenance have been a strategic target of the country for achieving sustainable food security and self-sufficiency. For countries like Ethiopia where the principal component of project development, i.e. finance, is a constraint to development storage dams for irrigation, small scale irrigation can be an alternative solution to enhance production (Girma Asfaw, 2006).

Development of small scale irrigation through river diversion, constructing micro dams, water harvesting structures, etc. may be considered as a pragmatic approach in the current Ethiopia for ensuring food self-sufficiency. A number of schemes have been designed and constructed in the previous years. In line with this, recent study report for the Amhara region (Asfaw Afera, 2004) has been used as a benchmark to conduct related studies in the southern region.

There are numerous failure cases of diversion schemes in developing countries. In such countries there is, in general, a lack of properly compiled code of practice for planning, design and operation of such schemes. Unlike other types of structural design, almost all types of hydraulic structures need to represent the unique features of each project and are well known for defying a general standardization of the design procedure. This case applies mostly for river diversion structures, which must be carefully planned to take local situations into consideration. Otherwise, any shortcomings of the project will bring about undesirable far reaching consequences, (Boeiru, 2003).

The Amhara region as part of the country has been implementing such schemes since 1970‟s. Despite remarkable achievements in expanding irrigation development, most of the implemented schemes are suffering from problems attributed to design, construction, and O & M that lead to major failure of the structures (Seid Shimeles, 2012)

The study area dominated by the small scale irrigation diversion structures in different parts of the woreda district kebeles .In the study area totally thirty diversion structures available. Out of this 12 structures giving service fully and 10 structures serving partially whereas, 8 structures fail completely. Generally the previous document indicates that the potential irrigable land is 1682 ha and the actual irrigated land is about 1299. 065 ha (wadla woreda water resource and development office)

## 1. 2. Statement of the Problem

A number of schemes are designed and constructed in the previous years in the woreda. While some of the schemes are performing successfully, it has observed that some of the schemes have failed to serve the purpose for which they are intended. Therefore, it is essential to capitalize on the success stories of schemes and the failure case should be viewed. Hence, both the success and the failure stories help to generate knowledge and information on the extremely important parameters of design, construction and maintenance.

Generally, the extent of irrigation development, the locations of developed schemes, their functionality are not well known. Though few number of irrigation schemes have been constructed and are in operation, a comprehensive assessment and evaluation of the performance of diversion structures has not been done yet. Other than the extent of the encountered problems, the diversion structures are facing problems related technical, social, and operational. Major failures are observed in some of the schemes requiring frequent maintenance and some of them are even left unused due to technical design as well as construction problems and lack of ownership feeling by the beneficiaries. This motives for considering the design practice and failure of river diversion structures as one of the tasks to be planned besides new irrigation developments so that future development of projects will incorporate the findings and lessons drawn.

### 1. 3. Objectives

The General Objective of the Study is to Appraise Design Practice and Failure of River Diversion for Irrigation Schemes in Wadla woreda Amhara region.

Specific Objectives of the Study Were:

- ❖ To assess the failures of river diversion structures in relation to design practices, management practices and social aspects.
- ❖ To identify the most sensitive hydraulic, hydrology and structural problems for the failure of river diversion structures
- ❖ To categorize the river diversion structures problem based on similar case they face.

### 1. 4. Research Questions

The following questions were addressed:

- ➢ What are the causes of failures for river diversion structures in relation to irrigation management practices and social aspects?
- ➢ Which hydraulic, hydrological and structural are most sensitive for failure of river diversion structures?
- ➢ How does the river diversion structures problem categorized based on similar case they face?

### 1. 5. Significance of the Study

The study can play a vital role in different ways of the diversion schemes because of this study so many problems listed out for the future. So to know the filer and their problems of the studied irrigation schemes used for Policy makers to understand the causes of the major problems in relation to the design, management and social aspects of the different component of the structure and identify the gap in knowledge between the current design practices and failures of the structures and for Researchers those interested to deal with this issue, it was provided know how about the schemes. Generally the farmers and those recipients from the schemes were benefit.

# CHAPTER TWO

# 2. LITRATURE REVIEW

## 2.1. River Diversion Structures

Diversion structures are used to divert water from an existing natural water course into a water supply conveyance system. For large diversions, such as the head works for an irrigation main canal system that normally require a head pond, the diversion structure can include a weir, sluiceway, intake, and fish way. Provisions for allowing the river to return to near natural levels in winter (i.e. in general, water is not diverted into the conveyance system during the winter period), permitting boat access, or for passing floating logs may also be required (Boeriu, 1999).

## 2. 2. The Layout of River Diversion

A river diversion project includes the intake, the diversion dam, the approach channel and its training works, the tail water channel and appurtenant training works. The layout design defines: a diversion site in the river, the relative position of the intake and the diversion dam in the river, the geometry of the approach and downstream channel and the dam outlets, gated spans, spillway (Boeriu, 1999).

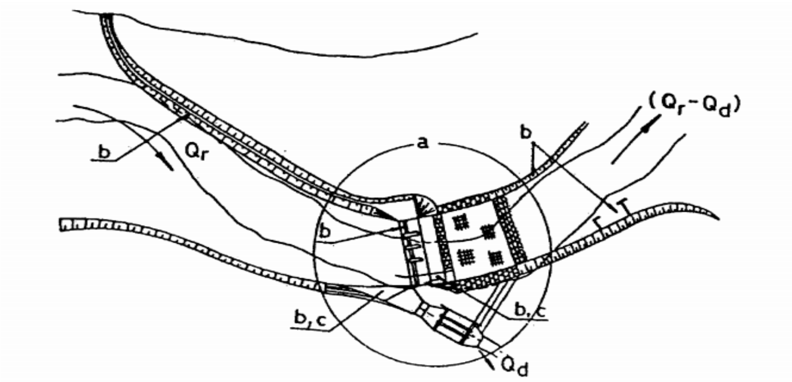


**Figure 2.1 Layout of river diversion structure (Boeriu, 1999)**

The diversion structure is located within a stable channel. Therefore, a channel of straight or moderate curvature is preferred overdone that has an actively progressing meander since floods may cause the latter type of channel to erode and bypass the structure. The foundation conditions at the proposed site for the structure should preferably consist of competent soils or rock with adequate bearing capacity and relatively low permeability.

Asfaw (2004), studied to appraise the current design practices for river diversion structures in Amhara region. Those focused a case study of problems and a comparative success of 33 existing river diversion works in the region was assessed.

To identify the knowledge gap between the current design practices and similar experiences in the other countries and GIS ArcView based database showing all features of the diversion structures that were operational between the years 1971-2000 was prepared.

The main problems observed in the existing schemes, problems of sedimentation at the head work and main canal, downstream scouring, upstream flooding, damage on cut-off and apron, clogging of the intake, scouring sluices outlets and improper operation and management of the schemes are found to be critical. The major finding in the investigation and analysis of the observed problems shown that the design of diversion structures is mostly adopted, hardly standardized of the existing design practice that were said to be the causes for most of the existing problems during that time While reviewing, hydraulic and structural analysis was not done to justify the existence of knowledge gap in the current design practice besides qualitative description.

Structural analysis of the damaged head work component was not performed to see their competency for the encountered problems beyond reporting the observed problems. It can be seen that investigating the cause of the failures of the schemes needs further assessment, and thus it is hardly enough to arrive at the conclusion for the causes of reported problems based on the qualitative description and field observations only. Robel (2005), Assessment of Design Practices and Performance of Small Scale Irrigation Structures in South Region a case study considering 26, existing small scale irrigation

Works was carried out in this research work and to understand the causes of the major problems that relate to the design consideration of the different components of the structure and identify the gap in knowledge between the current design practices and performance of the structures and analysis the pre and post construction aspects.

The physically observed problems of the existing irrigation structures considered for analysis are (downstream bank, drying of rivers, damage to impervious and flexible apron, and change of the river course, damage of under sluices and damage on CD works, some of the planning, institutional & operation problems) are conversed. The methodology used in this study was inventoried and field data collection of the existing small scale irrigation schemes, investigating the causes for the major problems occurring in the structure, and identify the knowledge gap and compile set of recommendations for planning, design, implementation and operation of irrigation structures.

Girma (2006) evaluated failures and design practice of river diversion structures for irrigation in Oromia region. A case of 36 diversion schemes for the head work physical problem and 17 for planning, socio- economic, institutional and operational on 105 informants interviewed on the matter were considered in this study.

The methodology used in this study was inventoried and field data collection of the existing small scale irrigation schemes, investigating the causes for the major problems occurring in the structure, and identify the knowledge gap and compile set of recommendation for planning, design, implementation and operation of irrigation structures and Statistical analysis using spreadsheet was used to investigate the frequency of each problem. Review of design practices was carried out to evaluate the design for minimizing the main canal siltation, provision of sufficient apron length, design of scour sluices and gates, head work sedimentation and upstream flooding.

Detail hydraulic analysis was carried out with the aim to compute the impervious apron length and downstream cut-off depth and hence to compare it with the already provided dimension. It was concluded that there were knowledge gaps in the current design practices.

Seid (2012) assessment and evaluation of the performances of diversion structure for small scale irrigation schemes (a case of Amhara region). This study considered a case of 41 existing river diversion structures to understand the causes of the major failures of the structures in relation to different aspects. This study is the same way to the above research works (Asfaw Afera (2004), and Robel Lambisso (2005), Girma Asfaw (2006)). Different aspects at the beginning of the design process have to be considered to divert a certain quantity of water from a river, such as:

- The flow rate in the river has to be assessed as a function of time and compared with the demand which is also a function of time.
- The diversion demands have to be decided taking into account a multitude of interacting factors of technical, environmental, political, aesthetic nature, etc.

By river diversion works, or intake works, we mean all facilities implemented to obtain water meeting quantity and quality requirements.  From the earliest days of water diversion engineering, the designers have been faced with the problem of sediment entering the channels and water conveyance systems (Baban, 1995).

The complexity of the problems rapidly increases with the proportion of flow that is diverted. Various components of a water project such as channels conduits, transition works, some treatment plant components, may be standardized many times. Only the diversion project which directly interferes with the river may be hardly standardized. Any shortcomings in the design of such projects can have far reaching consequences, too often of a divesting nature Boeriu (1999).

Boeriu (1999) has developed different ways of classifying diversion structures.  Among these the most important are according to:

a) The location of water diversion (Lateral intakes, Frontal intakes, Bottom intakes, Floating intakes)

b) The source (River intakes, Intakes from reservoirs and lakes)

c) The slope of the river they are on, which usually also indicates the size of the intake and the sediment size carried by the river, that is, boulders, gravel, sand and silt, respectively:

- Mountain intakes on steep rivers with slopes greater than about 1:1000
- Intakes on plain rivers with slopes 10-3<S<10-4
- Intakes on large rivers with slopes less than 1:10,000

d)  The measures taken against sediment entering into the intake system:

- Preventive, which is designed to exclude the sediment from entering the diversion channel.

**2. 3. Different Units of River Diversion Headwork's**

Diversion head works mainly consist of a weir (or barrage) and a canal head regulator. A weir has a deep pocket of under sluice portion upstream of itself and in front of the canal head regulator on one or both sides. The under sluice bays are separated from other weir bays by means of a dividing wall. In addition, river training structures on the upstream and downstream of the weir, and sediment excluding devices near the canal head regulator are provided. A weir is an undated barrier across a 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. Such weirs will have minimum length and normal uniform flow through all the weir bays thereby minimizing the chances of shoal formation and oblique flow (Asawa, 2008). Weirs are relatively low-level dams constructed across a river to raise the river level sufficiently or to divert the flow in full, or in part, into a supply canal or conduit for the purposes of irrigation, power generation, navigation, flood control, domestic and industrial uses, etc. (Boeriu, 1999)

Weirs are also used to divert flash floods to the irrigated area or for ground water recharging purposes. They are also sometimes used as flow-measuring structures. Weirs are usually aligned at right angles to the direction of a flow in the river. Such alignment ensures lesser length of the weir, better discharging capacity and lesser cost. Sometimes, the weir may be aligned at an oblique angle to the direction of the river current, and thereby, obtaining more safe and better foundations. In such a case, the weir will be of greater length will have less discharging power and will be costlier. Moreover, due to non-axial flow, cross-currents may be developed, which may undermine the weir foundation.

## 2. 4. Types of Weirs

The weirs may be divided into the following three classes (Asawa, 2008): such as masonry weirs with vertical drop, rock fill weirs with sloping aprons and concrete weirs with sloping glacis. The selection among the different types of the weirs depends on the availability of construction materials, foundation condition, hydraulic requirements, and construction technology to be followed.

### A. Masonry weirs with vertical drop

Consist of a horizontal floor and a masonry crest with vertical or nearly vertical downstream face. The raised masonry crest does the maximum pounding of water, but a part of it, usually, done by shutters at the top of the crest. The shutters can be dropped down during floods, so as to reduce the afflux by increasing the waterway opening. The stability of the crest should be examined for the following conditions (Asawa, 2008):

I. The water level in the upstream side is up to the top of the shutters with no flow on the downstream side and all the water is diverted into off taking canal. The overturning moment caused by the water pressure on the upstream side must be resisted by the weight of the crest without any tension at its upstream end. The stability of the crest against sliding due to water pressure should also be examined.

II. When the shutters are dropped down, water flows over the crest and the overturning moment is reduced due to the lowered water level on the upstream and the presence of water on both sides of the crest. However, there will be some loss of weight (and hence, the resisting moment) of the crest due to floatation because of the crest not being completely impervious. It is impossible to determine the amount of this loss of weight accurately. The reduced resisting moment is calculated on the basis of full weight of the masonry above the downstream level and submerged weight below the downstream level. The safety of the crest is examined for different stages of the discharge up to the maximum flood discharge. At all such stages, the resisting moment must be more than the overturning moment and there should be no tension at the upstream end of the crest.

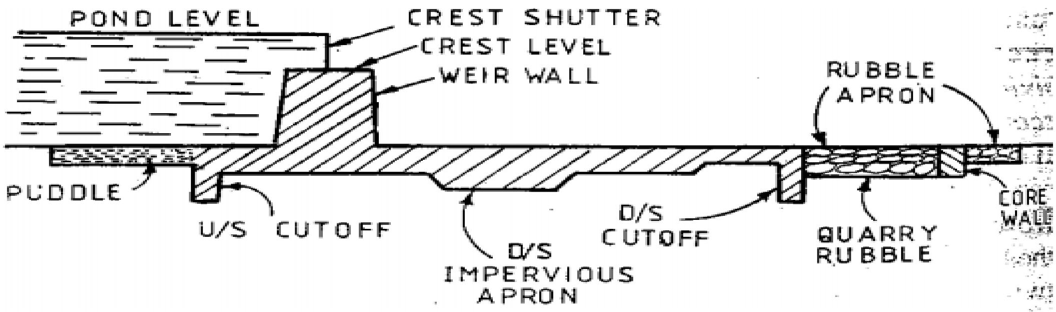


**Figure 2.2 Masonry weirs (garg, 2005)**

**B. Rock fill weirs with sloping aprons**

Such a weir is also called-‘Dry Stone Slope Weir’. A typical cross-section of such a weir is shown below. This type of weir is the simplest one, but requires a large quantity of stones for construction as well as maintenance. As such, this type of weir is suitable in areas where a large quantity of stones is available in the vicinity of the site and where labor is cheap (Asawa, 2008).

Also, it is suitable for fine sandy foundations. The stability of such a weir is not amenable to theoretical treatment. However, with the development of concrete glacis weirs, the above type is also becoming obsolete (Garg, 2005).

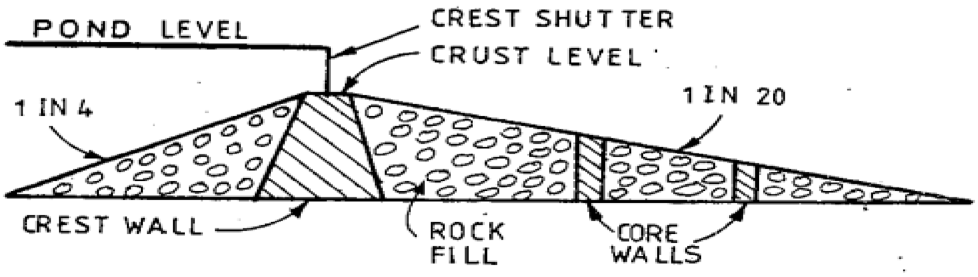


**Figure 2.3 Rock fill weir, (garg, 2005)**

**C. Concrete weirs with a sloping glacis**

According to Garg (2005), weirs of this type are of recent origin and their design is based on modern concepts of sub-surface flow (i.e., Khosla's Theory). In this type of weir, the excess energy of overflowing water is dissipated by means of a hydraulic jump which forms near the downstream end of the glacis. On pervious foundations, only concrete weirs are constructed these days.

These detailed designs require knowledge of:

i) the maximum flood discharge and corresponding level of the river at and near the selected site for weir,

ii) the stage discharge curve of the river at the weir site, and

iii) The cross-section of the river at the weir site. Based on the site conditions, general and economic considerations, and other data, the designer decide a) the afflux, b) the pond level,

a) the minimum waterway (or the maximum discharge per meter length of the weir), and the weir crest level (Asawa, 2008)

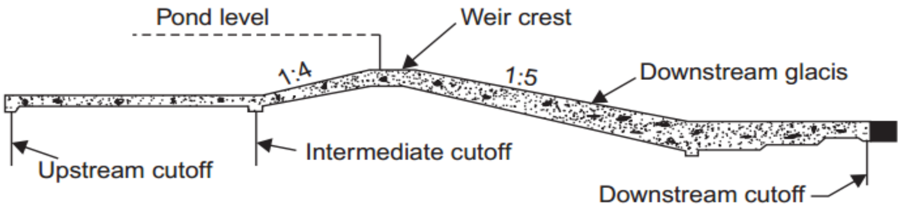


**Figure 2.4 Concrete weirs (asawa, 2008)**

## 2. 5. Design Process

The designing process of a river diversion works is often very complex. Therefore a wide range of disciplines including hydraulics, civil engineering, mechanical engineering, electromechanical instrumentation and control are involved (Boeriu, 1999).

The design process has to be based on well-defined principles. Cooperation between specialists in determining the overall lines of the project and fixing the final design and dimensions are indispensable.

Generally Boeriu (1999) outlined three groups that must be involved in setting up the overall project:

I. The group of operating team: who are the most familiar with the real operating limitations

II. The group of hydraulic design and laboratory tests or model studies, if applicable. If necessary, the various phases of the river diversion project should be studied using a physical model. The purpose of hydraulic design is to produce the following results:

I. Characteristic water levels and elevations of the structure: (Normal water level NWL, ,Maximum water level, MWL, Elevation of the weir crest, the gates operation deck and minimum elevation of the bridge crossing, etc.).

II. Weir dimensions (Crest length and height, Top and bottom width, Length of impervious apron and Cut-off depth )

III. Intake sizes (Entrance openings: width, height and elevation and Hydraulic losses before the conveyance entrance section)

IV. Hydraulic energy dissipation

V. Seepage through (Earth fill and it control and Below around concrete structures and foundations)

VI. The approach and tail channel geometries

VII. The length of training dikes and levees

VIII. The radii of curvature of the channel’s alignment

IX. Forecast of the morphological changes after the river diversion

The available means for carrying the hydraulic design are the following:

a) Theoretical: involving mathematical equations of fluid dynamics and empirical relations.

The available mathematical equations are the continuity equation, flow equation (or momentum) and the energy equation, expressed in terms of average values of the parameters involved.

b) Professional experience of the personnel involved in the design process: It is an invaluable asset. Using the previous solutions must be accompanied by a check on the behavior of these solutions under operational conditions.

III. The group of manufacturers of the hydro-mechanical equipment

Generally, the required data bases for a new project design are:

**A. Stream flow records**

It's essential to the design of diversion schemes. These data will provide the river flow hydrograph Q= f (t) and also indications about the annual peak, mean and minimum flows. They are also indicated the time periods when high flows may be expected related particularly to the needs of the installations supplied by the diversion scheme. Using stream flow records the 10-year, 50-year floods which the diversion weir must be able to pass, may be deducted.

B. **Topographic data**

The diversion site should be surveyed in detail, including the intake levels, roads and other existing physical features must be mapped. To calculate the backwater effects, the survey must extend to the entire length of the backwater. For hydraulic computations, survey scales of 1:1000 – 1:2000 are acceptable for the detailed structural design, scales of 1:100 – 1:200 are required.

- **Foundation behavior of material characteristics**

Information about soil stratification at the site of the structure is necessary. This information is important in relation to several factors such as the design of coffer damming system, load bearing capacity, seepage, uplift pressure, and scour undermining

Hydraulically structures, even if relatively small, should never be constructed without first drilling test bore holes. The history of a river valley may be very complex and recently deposited coarse alluvia may well hide deeper deposits of lacustrine origin. The investigations should be extended to a depth which is twice the design head acting on the structure and should also include the river bank areas surrounding the main structures. Information about construction materials available in the site neighborhood is also valuable.

**A. Project data**

Among the project data the main required data are includes water demand, location of the upstream and downstream users, water quality parameters, the required water levels at the entrance to the conveyance system and information about other river projects in the vicinity, roads, electric supply at the site, etc.

Over-all, the design of a river diversion is a process involving a large number of considerations. For this reason, it is important to begin with well ahead of time with the collection of the data. One of the problems which make difficult the design process, several times, is the scarcity of the field data mainly hydrological data. For most cases, data concerning sediment transport are totally non-existent. In these situations, carefully checked data from similar sites or results of theoretical formulae are used. A field trip and observations of the river and its catchment area are helpful in the correct selection of the empirical coefficients entering into any hydraulically computation. Therefore taking into account the above considerations, water intakes should be designed on the basis of generous factors of safety. The safety margin of the design must be increased when reliable data are missing. The safety factor has to be well defined by investigation program me.

## 2. 6. Failures of Weir Foundations

Piping or undermining the soil under the foundation can cause collapse of the apron and eventual overturning of the structure. A weir can fail when the uplift pressure creates an overturning moment in excess of the superstructure's balancing moment. To avoid this happening, the uplift pressure must be estimated correctly and the structure dimensioned properly. Detail discussion and its remedial measurement shown below. Failure of weirs on permeable foundations occurs as a result of one or more of the following:

### i) Subsurface flow actions

The exit gradient is the hydraulic gradient of the seepage flow under the base of the weir floor. The rate of seepage increases with the increase in exit gradient, and such an increase would cause ‘boiling’ of surface soil, the soil being washed away by the percolating water. The flow concentrates into the resulting depression thus removing more soil and creating progressive scour backwards (i.e. upstream). This phenomenon is called ‘piping’, and eventually undermines the weir foundations (P. Novak, 2007).

The piping phenomenon can be minimized by reducing the exit gradient, i.e. by increasing the creep length. The creep length can be increased by increasing the impervious floor length and by providing upstream and downstream cut-off piles.

### ii) Surface flow actions

This is caused by scouring of the downstream floor of the structure. It is due to unbalanced pressure in the hydraulic jump. The base of the impervious floor is subjected to uplift pressures as the water seeps through below it. The uplift upstream of the weir is balanced by the weight of water standing above the floor of the pond, whereas on the downstream side there may not be any such balancing water weight. The design consideration must assume the worst possible loading conditions, i.e. when the gates are closed and the downstream side is practically dry. The impervious base floor may crack or rupture if its weight is not sufficient to resist the uplift pressure. Any rupture, thus developed in turn reduces the effective length of the impervious floor (i.e. reduction in creep length), which increases the exit gradient.

The provision of increased creep lengths and sufficient floor thickness prevents this kind of failure. Excessively thick foundations are costly to construct below the river bed under water. Hence, piers can sometimes be extended up to the end of the downstream apron and thin reinforced concrete floors provided between the piers to resist failure by bending (P. Novak, 2007).

Analysis of uplift pressure under structures built on impervious foundations is simplified by the fact that the head dissipates by friction when the water percolates through cracks and fissures in the foundation. The uplift pressure is usually assumed to vary linearly from the upstream head to the tail water. Weirs constructed on impervious foundations are rare since most irrigation projects locate at or near the alluvial stage of rivers. In the subsequent sections some approaches for assessing the uplift pressure in pervious foundations and limitations on their application are discussed.

### 2. 7. Procedures in Estimating Peak discharge Using SCS-CN Method

This technique was developed by the United States soil conservation service. The method used to drainage area, runoff factor, time of concentration and rainfall depth. The rainfall-runoff relationship is used to separate total rainfall into direct runoff, retention and initial abstraction utilizing the following equations:

$$Q = \frac{[P - I_a]^2}{[P - I_a + S]}$$

$$I_a = 0.2S$$

$$Q = \frac{[P - 0.2S]^2}{[P + 0.8S]}$$

$$S = 25.4\frac{[1000 - 10]}{CN}$$

$$AMCIII = \frac{23 * AMCII}{10 + 0.13 * AMCCII}$$

Where

Q = accumulated direct runoff (mm)

P = accumulated rainfall (mm)

S = potential maximum retention (mm)

Ia = initial abstraction (mm)

CN = runoff curve number

- Design storm rainfall (EVII)

$$XT = \bar{X} + KTS$$

$$K_T = -\frac{\sqrt{6}}{\pi}\left\{0.5772 + ln\left[ln\left(\frac{T}{T-1}\right)\right]\right\}$$

- Time of concentration, rainfall excess duration and time to peak

The time of concentration can be computed by

Duration of excess rainfall,

If Tc<3hr, d=0.5

D ≈ 1hr if Tc > 3hr

Time to peak Tp = 0.5D+0.6*Tc

Time of base of hydrograph Tb = 2.67*Tp

Lag time, Tl =0.6 *Tc

- Peak runoff for 1mm of runoff:

Qp = 0.21*A/Tp Where

Qp = peak runoff (m3/s/mm)

A = Area of the catchment

Tp = Time to peak

Kirpich (1940) calibrated two equations for computing the time of concentration (min) for small watersheds (hydrologic analysis and design second edition, 1998)

$$t_c = 0.0013\, L_c^{0.77}\, S_c^{-0.5}$$

$$t_c = 0.0078\, L_c^{0.77}\, S_c^{-0.385}$$

The length (ft) and slope (ft/ft)

**2. 8. Planning, Institutional, Social & Economic Problems**

The above problems with such schemes are not only attributed to problems of design and construction. The software aspects of planning, institutional social & operational and economic problems are also crucially important. In the following section the highlights of each problem are presented:

- **Planning Problems**

The planning process in the development of irrigation projects can be viewed in the light of community willingness and participation. Accordingly, good performance of the schemes is directly related to the level of involvement of community members in the planning process Asfaw Afera (2004). In line with this, the schemes in the region can be categorized into two: Schemes implemented with due involvement of stakeholders and Schemes implemented without(less) participation of stakeholders

❖ **Institutional Problems**

In the implementation of irrigation schemes, various institutions are involved in the process of planning, design, implementation and operation & evaluation. However, in some of the schemes built by NGOs and GOs the expected level of participation of various institutions is not observed (Ibid, 2004)

❖ **Social and Operational Problems**

The planning and institutional problems can also be reflected in the proper operation and utilization of the implemented schemes. Establishing WUA is a task to be carried out during the planning process or right at the beginning of implementation (Ibid, 2004).

## 2. 9. Categorization of Problems

Categorization among problems occurs in the schemes is considered to be one of the output of this research. Bearing specific failure condition, observes in the schemes is not in exactly similar fashion, the following main problem categories are considered under this Subheading.

Problems related to site selection (Clogging of the under sluice, head Sedimentation problem on work, Change in river course, Damage on main canal and farmlands, Main canal siltation, Damage on retaining wall and Upstream flooding). This problem is organized via give the weight of 31%.

Problems related to structure selection covers the weight of 25% by including the problems such as head work Sedimentation, Damage on weir proper, Clogging of the under sluice and head work, Prevalence of d/s scouring, Change in river course and Upstream flooding and Damage on main canal and farmlands.

Problems related to Hydrology and sedimentation consideration 13% (Clogging of the under sluice and outlet, Main canal siltation and Upstream flooding)

Problems related hydraulic design of weir and components such as Prevalence of d/s scouring, Damage on main canal and farmlands, Damage on d/s apron, Upstream flooding, Damage on retaining wall, Change in river course, Clogging of the under sluice and outlet, Damage in d/s cut off and Damage on sill (if any) and Damage on divided wall the problem coverage is 59%.

Problems related to structural design of components 25% (Damage on intake gate, Damage on scouring sluice gate, Damage on weir proper, Damage on divided wall and retaining wall.

Problems related to scheme operation and planning, institutional, social, and operational problems such as

Planning (Area development under planned, Beneficiaries benefited under planned, Lake of project idea by the community, Lack of community participation on construction, Water shortage, Lack of benefit from the system, Lack of additional income, Lack of experts (DA) support the scheme and Lack of credit),

Social (Lake of project idea by the community, Lack of community participation on construction and Lake of project idea by the community),

Institutional (Lack of community participation on construction, Lack of experts (da) support the scheme, Lack of supply of improved seeds, Lack of selective crop production system, Lack of utilization of pesticides, Lack of cooperative, Lack of credit, Lack of WUA established, Lack of timely hand over, Lack of storage facility, Market problem (access problem for vehicle),

Economic (Lack of land redistribution, Lack of benefit from the system, Lack of additional income, Lack of credit) and

Operational (Lack of supply of improved seeds, Lack of selective crop production system and Lack of utilization of pesticides).

# CHAPTER THREE

# 3. MATERIALS AND METHODS

## 3.1. Description of the Study Areas

The study area is located at Wadla Woreda in North Wollo Administrative Zone. It is situated at latitude of 11°50'0" to 11°30'0" N and longitude of 38°50'0" to 39°10'0" E. The study area is bounded to the North with Mekiet, South with Dawunt, West with Delanta and the East with Mekiet woredas. The Woreda altitudinal variation ranges from 1501-3600m above sea level. The agro ecology of the Woreda is about 3.8% Kola, 53.9% Dega, 34.69% Woyna Dega, and 7.7% Wurich. The amount of rainfall is about 800-1200mm. The Woreda is frequently affected by drought in the past and even currently. The previous document indicates that the potential irrigable land is 1682 ha and the actual irrigated land is about 1299. 065 ha (Wadla Woredas Finance and Economics Bureau, 2016)

**Table 3.1 Land use classification of study area (Wadla Woreda Finance and Economics Office, 2016)**

| No | Land use type | Area coverage (ha) |
|---|---|---|
| 1 | Cultivated land | 9715 |
| 2 | Forest land | 330 |
| 3 | shrub land | 137.5 |
| 4 | Water body | 52.3 |
| 5 | Urban and rural construction | 3493.7 |
| 6 | Bare land | 11592.3 |
| Total | | 25320.8 |

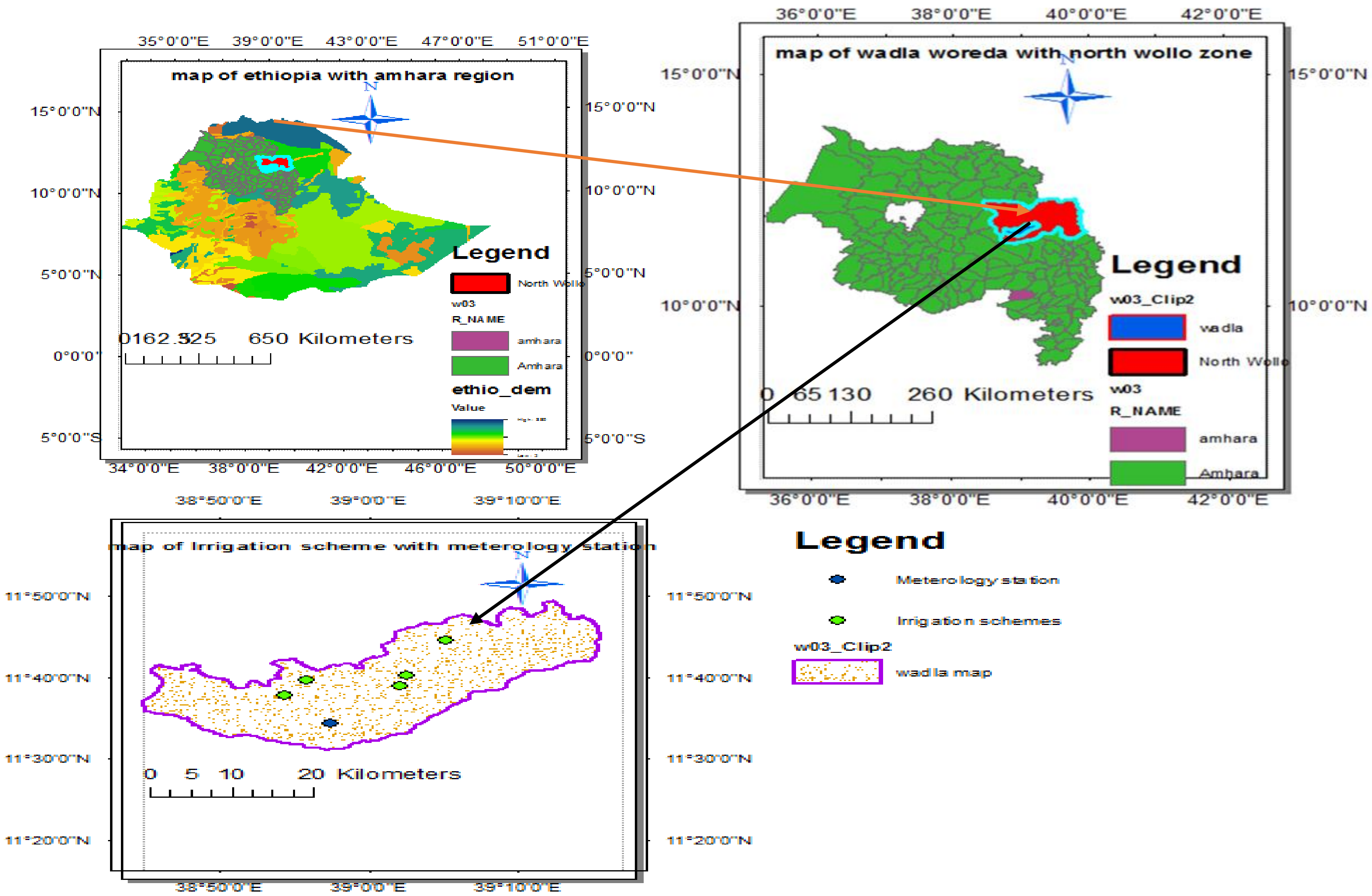


**Figure 3.1 Location of the Study Area**

The irrigation schemes are described as small scale, large scale and medium according to their amount of area coverage. From these types of schemes small scale irrigation schemes are given to emphasis in my study. The selected five small scale irrigation schemes are masonry type weirs. As the filed observation viewed that four weir diversion schemes are fully non- functional and one scheme is partial functional.

**Table 3.2 Explanation of the study area diversion structures (Wadla Woreda Water Resource and Development Office, 2008)**

| No | Keble | Name | Irrigable area (ha) | | Year of Completion | Implemented by | Number of beneficiaries (HH head) | Area of water shed (km^2) | Location of the structure | | | Pick discharge in m^3/s |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Irrigated | Planned | | | | | X | Y | z | |
| 1 | 13 | Yogi | 9.4 | 27 | 1996 | OXFAM | 45 | 79.2 | 492571 | 1289126 | 2877 | 85.8 |
| 2 | 1 | Yenja | 5 | 15.5 | 2000 | Fi/mew | 30 | 87.96 | 489906 | 1285589 | 2834 | 389.99 |
| 3 | 8 | Fidiango-2 | 28 | 37 | 2002 | ORDA | 82 | 24.85 | 503816 | 1287912 | 2992 | 290.85 |
| 4 | 18 | Kendbersh | 18 | 36 | 1997 | ORDA | 60 | 12.586 | 504603 | 1290165 | 3222 | 103.32 |
| 5 | 10 | Fidiango-1 | - | 50 | 2006 | AWO | 92 | 43.43 | 504625 | 1290145 | 3021 | 121.64 |

### 3.3. Sources and Procedure of Data Collection

#### A) Primary Data

The primary data was obtained from field observation, interview and focused group discussion. Moreover functionality of selected diversions, structural design systems, damage of weir components, problem of upstream flooding, siltation of the head work, siltation of the main canal and other relevant data's were observed in the field survey. The technique of the selected candidates for interview purpose are by using purposive sampling techniques.

#### a) Key Informants Interview

Specifically the interview focused on planning problems, institutional problems, social and operational problems of the schemes. For interview purpose community leaders, extension agents and selected farmers were participated.

#### b) Questionnaire

The questionnaire focused on project idea, community participation during construction, institutional problems, social and operational problems of the schemes. For this study a total of 125 farmers had been selected on which 25 households were selected in each five irrigation schemes.

#### c) Field Observation

The filed observation for the selected schemes were focused on functionality of the irrigation schemes, structural design system, site condition of the head work, upstream and downstream condition of the schemes, the stability of the river bank, intake condition, the upstream and downstream protection works and their length, damaged structures, flooding problems, siltation of main canal and head works, proposed total irrigable areas and irrigation practice. Likewise, Grid coordinate and altitude was collected by using GPS (Global positioning system) for the purpose of preparing maps of the study area.

#### B).Secondary Sources of Data

Secondary data were also gathered from different institutions. A ten and eighteen years Minimum and maximum rainfall data were gathered from Kone and Estaysh Metrological stations respectively. And also design document data from the Wadla Woreda ORDA office, finance office, water resource and development office and zone water resource office collected.

### 3.4. Materials

Table 3.3 Used materials and their function

| s.no | Types of used materials | Function |
|---|---|---|
| 1 | GPS | To recorded grid coordinate system and elevation reading |
| 2 | Tape meter | To measure weir appurtenances |
| 3 | Digital camera | To take photos of damaged structure |
| 4 | ArcGIS10.1 software | To delineate watershed area |
| 5 | AutoCAD | For the design purpose of the weir structures |
| 6 | SPSS Software | For the analysis of the occurred problems |
| 7 | SCS method | To check the consistence of the hydrological data and estimation of the peak discharge of the river |
| 8 | Excel software | To use estimation of mathematical equations |

### 3.5. Methodology and Data Analysis

The collected primary and secondary data were analyzed by using different software's and mathematical equations. Therefor; the data that's collected from field survey ( site selection, hydrology and sedimentation problem, structural selection, hydraulic design of weirs and components, structural design of weirs and components and scheme operations) were analyzed via, mathematical equations. Likewise the interview and group discussion data from the community is analyzed by SPSS Soft wares in terms of percentage. In addition to these the photograph of the damage structures was recognized for the supportive purpose of the analysis. By assessing the design reports and other allied documents and interviews with the engineers that were involved in the full design and construction process.

- ❖ **Hydrological Analysis**

Peak discharge at the weir site is usually calculated as return period of 50 years, for this design of discharge required a long flow data in the gauged river sites, but most of the rivers are not gauged, so different methods are used for their computation. There for in this study area the river was ungagged and there are no recorded required flow data. In the study area the daily maximum rainfall data were gathered from the Ethiopian meteorological agency of

Kombolcha branch, from this take the daily maximum rainfall data from Kone and Estaysh meteorological stations. The analyses of the hydrological data were used by comparing different methods such as Normal, Log Pearson type III, log normal, Pearson type III, Gumbal EVI and Gumbal method and selected the best value of the pick discharge of the rivers. The method of calculating these data is by soil conservation service (SCS) method and this analysis method discussed detail in Appendixes -1

- **Hydraulic Analysis**

The analysis of the design of weir structures all external forces acting on the structure must be calculated. Those forces; includes uplift pressure, soil and water pressure can be evaluated. In general, for the analysis of the weir hydraulic structures including parts of the structure; shape of the weir, weir height, length of the waterway, discharge and head over the weir, length of the weir, flood and energy level, afflux, scour depth and fixing various dimensions of the structure. By using mathematically the following formula is estimated

The length of waterway (L) is calculated from Lace's regime formula (garg, 2005)

$$L= 4.75\sqrt{Q} \qquad (1)$$

The value of length (L), after doing estimate the discharge (q), per unit width of the river in the relation of next formula

$$q = Q/L \qquad (2)$$

The regime scour depth is calculated from laces formula

$$R= 1.35(\frac{q^{\wedge}2}{f})\ {}^{\wedge}1/3 \qquad (3)$$

The regime velocity and velocity head are calculated from the expression

$$\text{Regime velocity, } V =\frac{q}{R} \qquad (4)$$

$$\text{Velocity head, } ha =\frac{V^2}{2g} \qquad (5)$$

The total energy level and high food level are given as follows

$$D/s\ TEL=D/s\ HFL+ ha \qquad (6)$$

$$U/s\ TEL= D/s + afflux \qquad (7)$$

$$U/s\ HFL=U/s\ TEL- ha \qquad (8)$$

Crest level of weir=U/s HFL- hd (9)

Weir height=crest level- river bed center (10)

Pond level =FSL of off taking canal +head loss through the head regulator (11)

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

Depth of u/s sheet of piles =1.5R (12)

Depth of d/s sheet piles =2R (13)

❖ **Structural Analysis**

Once the dimensions of structures are fixed using the abovementioned approaches, its structural Stability was checked as follows:

I. Stability analysis of the weir body should be checked against sliding, overturning and tension for both pond level and overflowing condition of which dimension of weir with a higher factor of safety is to be taken. The method of calculation is done on appendix (III)

II. Stability analysis of retaining and divide wall: Once their heights are determined by adding some free board, 0.3-0.5m, to the high flood level for upstream and downstream conditions. The stability is analyzed for trial top and bottom widths against overturning, sliding and tension.

❖ **Planning, Social and Operational Problems Analysis**

This statistical analysis method is the basic method for this thesis analysis results. Therefore, after interviewing the irrigation water users, development agency, and woredas experts, based on the interview given to the value one (1) for the problem occurred in the diversion structure and given to zero (0) for the problem not occurred in the diversion structures. For these data collection systems in the interview and group discussion 125 participants included. Those peoples selected by purposive sampling thickness for each studied 5 schemes, at each individual schemes 25 persons were selected and the total some of selected persons are 100. Out the total participants100 persons are males and the other remained participants (25) female. After this data composed the frequency percentage of the problem was analyzed by using SPSS software.

### 3. 2. Frame Work of the Study

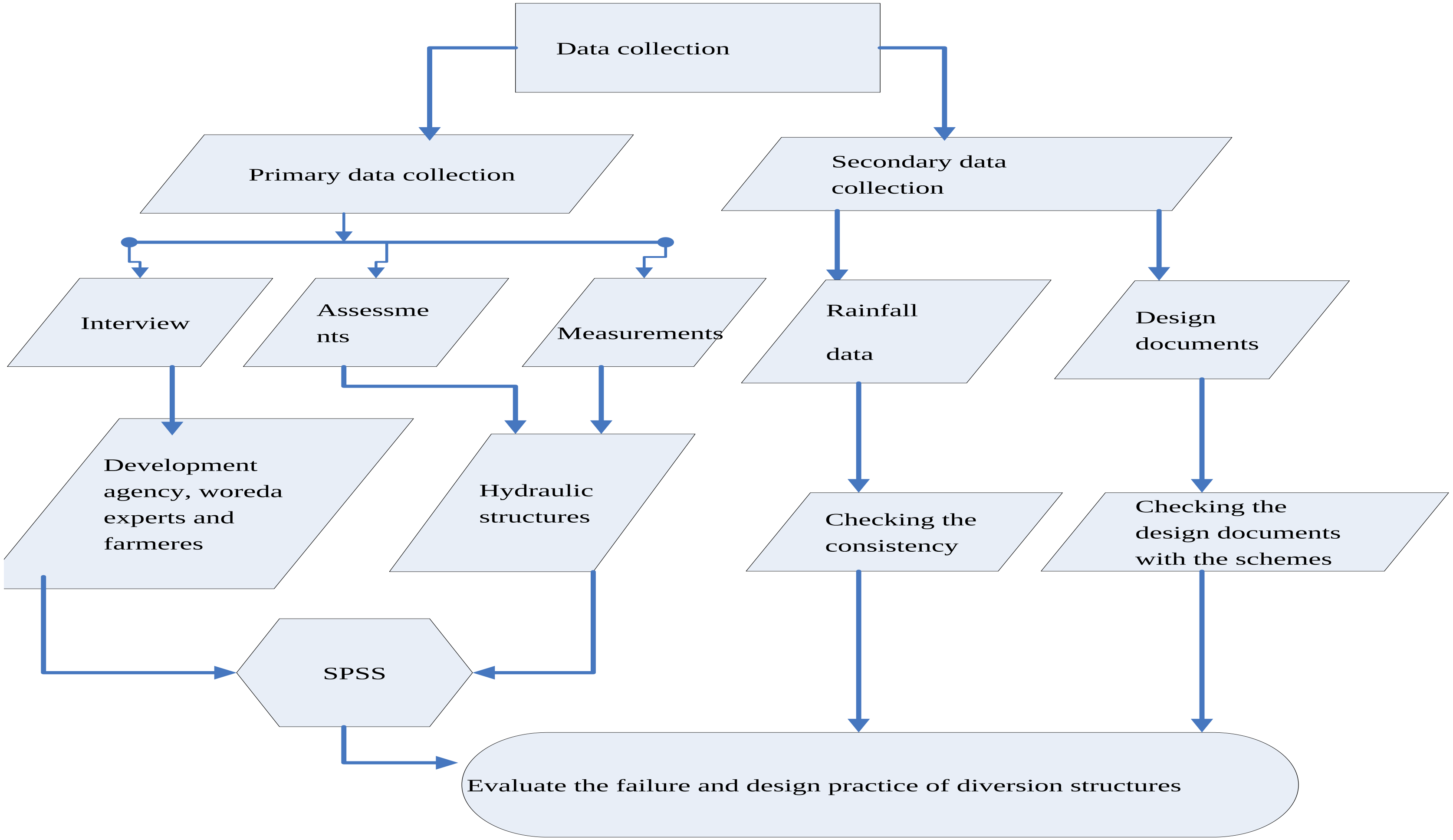


**Figure 3.2 conceptual frame works**

## CHAPTER FOUR

## 4. RESULTS AND DISCUSSIONS

### 4.1. Failure and Design Practice of River Diversion Structures

Based on the observation, most of the hardware irrigation structures (Gates, under sluice, head work, intakes, main canal, downstream and upstream protection works etc.) facilities likely to fail; because improper planning, designing, construction and operation of the schemes. In addition to these, field interview revealed that a number of irrigation beneficiaries were suffering from problems associated with improper planning, institutional problems like failure to hand over schemes, failure to establish a water user's association (WUA) and offer proper training and operation problem like conflict due to water right.

### 4.1.1 Improper Design of Structure

Before a specific project, there is no implemented and planned in a basin master plan for the use of water resources in the basin has to be established. In this study area usually the site investigation and planning activity were worked by the team of agriculture and rural development experts. The design practice in some of the studied schemes there is no include all part of the structures, like retaining wall, cut off, under sluice, divided wall and stream basins. Those problems occurred do to conflict, selected site is very wide and flat, lack of knowledge, and a large stone around the cross section of the weir.

**Figure 4.1 Kendbersha and Fiadiango_1 weirs constructed without under sluice and divide wall**

- **Site Selection**

The design and construction of any river diversion structure design practice one of the main works is site selection. In my study area the constructed schemes are failed as the causes of site selection problem. The selected sites of some schemes are very wide river crossing and the foundations are alluvial soils which are highly pervious and easily scour when the high velocity water passes over the structure.

- **Location of the Weir**

Initially, to decide on the location of the proposed structure without having topographic maps of the project area and layout of the river course. However, by walking along the river up and downstream of the location where the existing intake is or where the farmers believe it is an appropriate location; it is possible to identify a few places for the proposed structure.

- **Location of the Irrigated Area**

The site is too close to the proposed land, some of the area in the upper reach of the main canal cannot be commanded, and there is no sufficient irrigable land. In addition to this the scheme is constructed, but there is no irrigable land totally. Situation like this, the design engineer should not carry out the economic analysis.

- **The Design Discharge**

While designing a weir provision must be in the flood that is likely to occur during the lifetime of the structure. However, one can neither choose a very high nor a very low flood magnitude for the design. Taking a very high flood results in a costly structure and On the other hand, if a low flood magnitude is chosen, it will result in failure. Therefor in the study area the design discharge estimation practice is very poor, which are estimated the design discharge by thanking only one dray month of the year. In addition to this the estimated some river pick discharge by using rainfall data is not relate to the watershed area of the studied schemes.

**Table 4.1 Design Cooperation with Re-design of diversion structures**

| no | Weir structure components | Name of river diversion schemes | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Ydogite | | Fiadiangoa _1 | | Yneja | | Kendbersha | | Fiadiangoa _2 | |
| | | Design | Require | Design | Require | Design | Require | Design | Require | Design | Require |
| 1 | Weir Height (m) | 2.2 | 1.8 | 2.5 | 2.1 | 2.3 | 1.5 | 4.4 | 1.9 | 2.9 | 2.4 |
| 2 | Length of river cross section(m) | 20 | 44 | 29 | 52.4 | 33.1 | 93.8 | 44.7 | 48.3 | 30 | 81 |

- **Weir Height**

The comparisons of the study schemes based on designed (constructed) with re-design (required) of the diversion scheme are more exult different. This difference affects the downstream part of the schemes, because of the high water pressure flow to the downstream parts in detail as shown Table 4.1

- **Length of River Cross Section**

In the studied diversion structures the cross section of the weir is not properly designed or constructed. This design of the weir length is less than the re- design of the weir diversion structures. The cause of this differs is to change the river flow, damaged the rating wall and the abutment of river diversion structures. This difference estimated by direct measurement of the structures.

## 4. 2. Investigation of Observed Problems and Analysis

In the field survey viewed and checking, a number of irrigation schemes are suffering from the problems associated with handing over of incomplete schemes, failure to establish as well as weak irrigation water user association (IUA) with improper training, and water shortages due to reduced river flow and water conflict in u/s and d/s sides. The observed problems with the diversion structure are discussed next in detail.

### 4. 2.1 Planning, Institutional, Social and Operational Planning Problem

- **Planning Problems**

The planning procedure in the development of irrigation projects can be viewed in the light of community readiness and participation. Accordingly, the sustainability of the schemes is directly related to the level of contribution of community members in the planning procedure. As observed during field visits, the situation for those schemes the users had no involvement with the planning of the project and a category of convinced development or top down development was observed there. This problem has been found critical on all of the schemes, which consist of both old and new schemes. Even in recent schemes like fiadiangoa_2 diversion scheme.

By way of the interview and discussion of the beneficiaries, they have repeatedly shown their interest to be involved in the planning and implementation process. But, the design team as well as constructs the body did not give attention to their concern. It is the usual scientific procedure to start the planning of a diversion irrigation scheme by involving the local farmers and collecting the traditional farming/irrigation practices. Moreover, the farmers should be consulted about the site selection, flood condition and structure type selection. This creates a favorable condition for the design engineer to systematically collect pertinent field data in order to see different options of head work types and irrigation system.

- **Institutional Problem**

In the application of irrigation schemes, institutions should involve in the process of preparation, design, implementation and operation and maintenance. Lack of proper involvement of institutions was observed in all of the study schemes. The schemes are built by NGOs and Woredas Agriculture Office, the expected level of the participation of the institutions is not observed. Schemes like Kendbersha and Yneja can be cited as projects having no design and handing over documents. Preparation of proper design is the outstanding of the responsibility of the implementing institution, and the regional water resource authority. The absence of the proper design document also resulted in creating the problem in showing the level of completed works that lay concrete on away for handing over of the schemes to beneficiaries or concerned institutions.

In some schemes, the implementing organization is a private contractor and may not be around on completion of the agreement period. If the schemes are not given over to the responsible body with the necessary O&M documents and the active involvement of the stakeholders/respective woredas offices, the responsibility is lie on the farmers and the schemes are run like the traditional ones. This is the result of improper operation and may lead to major failure on the schemes.

It is clearly seen that the handing over problem largely results because of failure of concerned institutions to discharge their outstanding responsibilities. Likewise, the co-operative promotion office as an institution at zone or woredas level has a responsibility to facilitate conditions for organizing and forming the irrigation water users association. Moreover, the mandates include follow-up the collection of periodic contributions of the users to conduct the maintenance works and help sustainable extension of the project.

- **Social and Operational Problem**

The social and operational problems can be reflected in the partial operation and utilization of the implemented schemes. These problems occurred on one (1) scheme out of the total schemes. Starting irrigation water user association (IUA) is a task to be carried out before or right at the beginning of implementation. This important activity is carried out by the implementing institutions (Client and Contractors), woredas Agriculture Office, and the respective co-operative promotion office. Failure to establish legally instituted irrigation user association and selected leaders parallel to the construction of irrigation scheme results in weak transfer of the schemes to the beneficiaries and lack of responsible body that will handle the operation of schemes. In irrigation scheme observed, social problems like conflict were observed due to water rights.

The schemes are becoming unable to generate the required amount of water to the beneficiaries throughout the year. This is because among other factors downstream traditional river diversions are significantly increasing from year to year. The conflict with upstream settlers is mainly due to diversion of same river or tributary for other irrigation or some other purpose. The “first come, first served″ approach does not seem to work well in such conditions. The conflict with downstream settlers mainly occurs during low flow seasons when irrigators completely divert the stream flow to fields.

**Table 4.2 Response of target community in categorized problems of irrigation schemes**

| s.no | Interview problems | Fiadiangoa_ 2 | | Fiadiangoa_1 | | Kendbersha | | Ydogite | | Yneja | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Frequency | Percent (%) | Frequency | Percent (%) | Frequency | Percent (%) | Frequency | Percent (%) | Frequency | Percent (%) |
| 1 | Absence of project idea by the community | 14 | 56 | 19 | 76 | 17 | 68 | 25 | 100 | 23 | 92 |
| 2 | Community participation on construction | 19 | 76 | 13 | 52 | 18 | 72 | 25 | 100 | 20 | 80 |
| 3 | Cooperative problem | 9 | 36 | 16 | 64 | 22 | 88 | 23 | 92 | 5 | 20 |
| 4 | WUA problem | 20 | 80 | 22 | 88 | 21 | 84 | 24 | 96 | 10 | 40 |
| 5 | Credit service | 10 | 40 | 19 | 76 | 15 | 60 | 18 | 72 | 11 | 44 |
| 6 | Market problem | 11 | 44 | 15 | 60 | 17 | 68 | 12 | 48 | 16 | 64 |

- **Absence of Project Idea of the Community**

In the field survey of the study schemes, physical observation of the failure of the constructed structures, observation of the upstream and downstream part of the scheme, interview of the other farmers, and group discussion with users and woredas experts was the main work in this study. Therefor the interview result of the project idea by the community in that studied irrigation schemes four (4) schemes like Ydogite irrigation schemes are about100% of the community was not get fuel orientation about the constructed irrigation schemes. So, via in case of this problem that diversion scheme are failed.

- **Community Participation on Construction**

The proses of construction at the starting work to the end work for all the studied schemes done by governments and non-governmental organizations. This construction system was the cause for the failure of the structures because, the community has not participated in labor, many and any local materials, and these cases wasn't to give motivation for taking the responsibility for constructing irrigation schemes. As the result the irrigation schemes such as Fiadiangoa_2 (76%) and Ydogite (100%) weir diversion structures were the community is not participated.

- **Water User Association Problems**

The interviews of the schemes respondents which indicates the diversion scheme Fidiango_1 (90%) and Ydogite (88%) of the respondents said "not establishment of water use association" (Table 4.2). It also reveals that from the two schemes more than 92% of the respondents agreed that they were not the establishment of the water user associations. This discussion includes the water user associations and keble agricultural experts, some of the issues standout that time there is no skilled artesian and contractors the time of construction, the design of the structure is not complete, the communities do not understand the uses of the construction and there is no take responsibilities of the constructed structures.

**Figure 4.2 Group discussion of selective communities**

- **Credit service**

Farmers need credit for purchase of agricultural inputs (fertilizer, improved seed and insecticide), fattening and rearing of animals and promote petty trading. Lack of long term and short term credit provision affects the production of the diversion scheme. However, 76% of Fiadiangoa_1 irrigation scheme water user farmers told that due to shortage of credit, supply of inputs during irrigation season is very small at required time. This leads that, fertilizer application to irrigated plot is not a common practice in the scheme and this leads its decreasing of its productivity. Hence, the sustainability of the diversion scheme is not give attention by the farmers.

- **Market Problems**

Lack of market and marketing facility were another issue. Although not directly related to the functioning of irrigation systems, the market was considered as one of the main problems in the study area. However, the study area marketing system did not always facilitate outcomes desired by farmers. One reason was the similarity of products and marketing patterns; onion and tomato were the dominant crops, often harvested by farmers at the same time, which leads to a high availability and low prices during the main marketing period. Farmers also perceive that market intermediaries are not pricing products fairly, which suggests reduced returns and less incentive to invest in the use of irrigation.

### 4. 3. Sensitive Problems for Failure of Weir Diversion

In the studied irrigation schemes the design of a river diversion is not involving a large number of considerations when checking to the design document and field observation. This reason occurred do to improper collection of the data, incomplete design process, incorrect construction, lack of contractors and lack of knowledge of the designer experts.

The sensitive problems of the diversion structure failures in the studied schemes includes, Hydrology and sedimentation consideration, Hydraulic design of weirs, and Structural design of weir are the main portions. That sensitive problems was analyzed by based on the bench mark of the Nile Basin Capacity Building Network (NBCBN). This bench mark estimation is depend on the problems to give weight for each types of occurred problems. Hence the studied area is found under the Nile basin area. So, deepened on via this similar basin and the given weight of the bench mark, to give the weight of each problems of the studied diversion structures like 31%, 49% and 20% respectively and to divide each percentage by sub-divided problems.

**Table 4.3 Sensitive portions that cause for the failure of weir diversion**

| No | Problem category | Weight (%) | Percentage of observed problems causes of failure | | | | |
|---|---|---|---|---|---|---|---|
| 1 | Hydrology and sedimentation consideration | 31% | Fidiangoa_ 2 | Fidiangoa_1 | Kendbersha | Ydogite | Yneja |
| | Head work Sedimentation | 13 | 41.94 | 41.94 | 41.94 | 41.94 | |
| | Main canal siltation | 7 | | 22.58 | 22.58 | 22.58 | 22.58 |
| | Clogging of the under sluice and outlet | 5 | | | | 16.13 | 16.13 |
| | Upstream flooding | 6 | 19.35 | 19.35 | | 19.35 | 19.35 |
| 2 | Hydraulic design of weirs | 49% | | | | | |
| | Prevalence of d/s scouring | 12 | | | 24.49 | | 24.49 |
| | Damage on d/s apron | 9 | | | 18.37 | | 18.37 |
| | Damage on retaining wall | 15 | | 30.61 | | 30.61 | |
| | Damage in d/s cut off | 5 | | | 10.20 | | 10.20 |
| | Change in river course | 8 | 16.33 | 16.33 | | 16.33 | |
| 3 | Structural design of weir | 20% | | | | | |
| | Damage on intake gate | 5 | | 10.20 | 10.20 | 10.20 | 10.20 |
| | Damage on scouring sluice gate | 9 | | | | 18.37 | 18.37 |
| | Damage on main canal | 6 | | 12.24 | 12.24 | 12.24 | 12.24 |

### 4.3.2. Hydrology and Sedimentation Consideration

The hydrological data are the base for designing and analysis of the weir diversion structures. Under this condition the head work sedimentation, Clogging of the under sluice and outlet (16.13%), Main canal sedimentation and Upstream flooding (19.35%) are the main occurred problems. As the field survey of the studied schemes observed and estimation that the cause of the failure of the existing structures giving the weight about 31% of the problems covered by hydrology and sedimentation effects.

While the data collected from the meteorology station of Kone and Estaysh stations and checked the consistency of the collected data. Then via using soil conservation service (SCS) method the diversion structure peak discharge was estimated, each studied irrigation schemes, from those re-design discharge estimations except fiadiangoa_ 1 irrigation schemes all are the design discharge was much less than the re-design discharge. This incorrect estimation affects the constructed diversion structures.

**Table 4.4 Design discharge and Re-design discharge compression**

| s.no | Name of diversion schemes | Design discharge (m^3/s) | Re-design discharge (m^3/s) | Watershed area (km^2) |
|---|---|---|---|---|
| 1 | Fidiangoa_1 | 155.28 | 121.64 | 43.43 |
| 2 | Fidiangoa_2 | 239.54 | 290.85 | 24.85 |
| 3 | Ydogite | 46.975 | 85.8 | 79.2 |
| 4 | Kendbersha | 66.43 | 103.32 | 12.586 |
| 5 | Yenja | 293.84 | 389.99 | 87.96 |

- **Clogging of the Under Sluice and Outlet**

These problems were stand up on the 2 schemes from the total studied schemes. Here, it is observed that the gates are totally damaged. These problems occurred in case of the silts deposited upstream part of the schemes, upstream flooding; the farmers do not give attention to the sluice gate and less knowledge about its impact on canal siltation.

- **Headwork Sedimentation Problem**

Head work sedimentation refers to the overall damaged of the wing walls, partial silt-up weir body of above the weir crest, and complete covering of u/s aprons. Since the phenomenon will result in uncontrolled and undefined flow through the river course, the main canal lying on the river bank is completely washed away by the flood water like Kendbersha and Ydogite diversion schemes.

The problem of sedimentation involves the diversion of head work. As stated above the other function of diversion headwork is to control entry of silt into the canal. The consequently indicated that 41.94% of a diversion head-work faces accumulation of high silt load at the head regulator, high sediment entry to the main off taking canal. The bed load consists of solids such as fine sand, gravel with a small diameter of up to about 2mm, or coarse material (gravel, stones of various sizes).

**Figure 4.3 Silted-up head work structure Kendbersha and Fiadiangoa_1 schemes**

- **Main canal Sedimentation Problem**

While as the field observation the main canals were highly charged with sediment (22.58%) like, Kendbersha, Ydogite and Yneja irrigation schemes are to suffer the problem. The silt load is observed to come either along with the river water (suspended and bed load) or as a run-off from the upstream nearby catchment. A lined canal Ydogite and Kendbersha diversion irrigation scheme completely filled with sediment (Figure, 4.4), the hidden canal section has insufficient space for farmers to move through it and clear the deposited sediment. So, maintaining the canal requires first routing the buried canal and then clearing the sediment. Though there were efforts started, it has not been successful since it demanded to mobilize both financial and human resources for proper understanding and improving the problem.

**Figure 4.4 Silted-up canals Ydogite and Kendbersha irrigation schemes**

Some canals do not have a proper design slope. In addition to these the canal in this reach passes in a deep cut, the soil bank immediately beside the canal is not stable. It is vulnerable to be washed by the rainwater and deposited into the canal. Hence, deposition of large soil mass should not be allowed to break beside the canals, but adequate edges should be provided.

- **Upstream flooding**

The problem of upstream flooding comprises of damage of main canals and damage of weir proper. During the field observation of the four schemes, there were the siltation of headwork and damage of main canal via, the case of upstream flooding. This problem was attributed due to improper site selection and no integrated watershed management approach. This characteristics supplemented with meandering of river terrain insisted d to high run-off /flooding/with silt and sediments deposition at lower river basin and downstream river course, on selected and constructed diversion weir site of the project which is destroyed by high run-off. In addition, the main causes of weir failure were wrong hydrological analysis, weir site foundation problem, and instability of the side banks at the upstream of the weir. From previous design document, during the feasibility study of the project, there was no rough survey on the geological situation of the catchment area which will help to be sure of the presence of erodible boulders which may affect the structure during the flood time.

**4. 3.3. Hydraulic Design of Weirs**

The sensitive problems for the failure of river diversion structures based on hydraulics are: Prevalence of downstream scouring, Damage on d/s apron, Damage on retaining wall, Damage in d/s cut off, Change in river course. The overall those problems are the main causes of the failure of the studied schemes

- **Prevalence of Downstream Scouring**

When the natural waterway of the river is contracted, the waterway scours the bed both on upstream and downstream of the structure. These phenomena occurred when there is a high flow of water over the structure without the design of the hydraulic jump. The scour whole forms progress towards the structure cause its failure. These problems occurred were 24.49% of Kendbersha and Yneja irrigation schemes. Failure of weirs on permeable foundations occurs as a result of the scouring of the downstream floor of the structure. It is due to unbalanced pressure in the hydraulic jump. Weirs constructed on impervious foundations are rare since most irrigation projects locate at or near the alluvial stage of rivers. In the subsequent sections some approaches for assessing the uplift pressure in pervious foundations and limitations on their application are discussed. In addition the above discussion these structures is have not constructed stream basin and under sluice parts. Therefore to prevent the failure of this structure the piles should be provided at the upstream and downstream of the structure.

**Figure 4.5 Downstream scoring Kendbersha irrigation scheme**

- **Damage on downstream protection work**

From the five selected schemes damage on downstream protection works became very serious problem observed in Kendbersha and Yneja diversion structures. During the field observation, the downstream work of those scheme are totally failed (Fig. 4.5). The reason is that there was no proper design of downstream protection works like downstream impervious apron, cut-off wall, downstream block protection, launching apron and downstream sheet of piles and their length and thickness were not properly designed. The problem was also attributed to improper structural selection for the site and hydraulic and structural design during the planning phase. This problem was caused due to improper hydraulic design which arises from poor knowledge of energy dissipation and impact of sediment on the structure (Novak et al., 2001). The impervious floor is designed in all cases to reduce surface flow action that causes scouring due to unbalanced pressure in the hydraulic jump tough. The stilling basin is seldom designed to confine the entire length of a free hydraulic jump on paved apron, because such a basin would be too expensive. Consequently, accessories to control the jump are usually 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 and the cost of stilling basin.

❖ **Damage on Retaining Wall**

Damage on the head work refers to those damages to the weir body, including the downstream bed, retaining walls and protection works. Damage on the weir retaining walls is observed in 2 of the total schemes. As field observed that 30.61% of collapsed wings and eroded protection works. In the diversion schemes where this problem succeeds, cut and scouring of the impervious floor are commonly observed. In addition, flexible aprons (protection works) are seen to be completely washed away by the energy of flowing water. The prolonged occurrence of cut and scouring of downstream portion of the structure may end up in the total collapse of the structures. This could be prevented by providing cut-off wall just at the end of the protected work.

Figure 4.6 Damage on the retaining wall of Fiadiangoa_2 and Ydogite schemes

- **Change in river course**

The river course changed from the first natural way in cause of the damaged of the upstream abutments and the damaged of the rating wall of the diversion structures. This problem occurred by the three diversion structures from the selected studied structures.

- **Seepage Problem**

The seepage problem can be categorized into Main canal seepage and Weir body/ foundation seepage. The problem of main canal seepage is observed on 3 of the total studied schemes considered for the analysis. It is the most common problems observed in most of the schemes. The seeping water is seen to flood out under of the soil and to the sides. Hence, significant quantity of irrigation water is lost before arriving to the distributing canals. By the loss of valuable diverted water, the seepage moisture is creating water logging of the adjacent lands. The cracks formed in the canal act as a conduit to pass the water into and from the irrigation canal depending on the water level in the canal. The following picture shows the seeping water into the canal.

Figure 4.7 Yneja diversion irrigation scheme

Though the river is accomplished of commanding the designed irrigable area, full irrigation has been difficult. The farmers in the head reach get the relatively best amount of irrigation water than the farmers in the tail reach. This created difference among the farmers and result lack of interest to participate in the yearly maintenance work. In water shortage season, selecting those crops that demand water in longer time interval is rather appropriate solution for the farmer.

### 4. 3.4. Structural Analysis of Diversion Weirs

Diversion weirs are constructed from a variety of materials. The most commonly used materials are reinforced concrete, masonry, and gabions. However, whatever materials are used, for the construction structures were checking field observation remains almost the same. It has been observed over the years that diversion weirs collapse, initially not because of the unbalanced moment, but mainly due to the foundation scouring. The stability analysis becomes important where the structural filer analysis.

- **Damage to Intake and Sluice Gates**

The cause for the problem of damaged gates is mainly improper scheme operation and the others are in case of high flood around the construction. The design document and filed survey work critically viewed that the design practices of the existing weir structures. Therefor in some schemes like Kendbersha and Fiadiogoa_1 irrigation schemes are the parts of under sluice and divided wall are not constructed, by these case during the high water flow of the river the canal outlet gates are damaged.

**Figure 4.8 Damage on intake gate and under sluice Ydogite schemes**

- **Damage on Main Canal**

This damage occurred in 4 schemes out of the total considered schemes for the analysis. During the rainy season river flows accomplished large amount of sediment for the cause of scouring and loses the formation of the foundation for the canal. The main canal running within the river course until it leaves the course is highly affected by the unstable geological condition of the river bank. The causes for this problem are associated with the diversion site selection and the absence of the protection works aligned along the canal.

Also lack of consideration in the hydraulic design to incorporate cut-off wall d/s of the diversion structure leaves open space subjected to progressive scour by the flowing water.

**Figure 4.9 Damage on main canal Ydogite and Kendbersha**

As a result of the silt deposition around the cut-off wall, relatively the river bed elevation rises up and the river tends to flow away from it. The problem is attributed to mainly the quality of construction work that is reflected to serious cracks in the canal. Besides, lack of design consideration for those canals passing through black clay soil that swells and shrinks with the moisture condition. Due to this, tension is developed under the canal and has to be absorbed and become minimal. This can be possible by the provision of canal bedding composed of granular materials and other selected materials on such critical canal reaches. Moreover, improper operation leads to abrasion and consequent cracks in the structure.

### 4. 4. Categorization

Categorization of the observed problems occurred in the schemes is the result of explicit identification of each problems and arranging them according to their impacts on the scheme. Similar problems are placed in the same category, but the extent of the problem occurring in one scheme will not be the same as that of the other scheme and does not necessarily have the same impact. The following categories are believed to describe the observed problems in general terms and help to see the main causes of the problems.

**Table 4.5 Categorization of observed problems on the scheme**

| No. | Problem Category |
|---|---|
| 1 | **Hydrology and sedimentation consideration** |
| | Head work Sedimentation |
| | Main canal Sedimentation |
| | Clogging of the under sluice and outlet |
| | Upstream flooding |
| 2 | **Hydraulic design of weirs** |
| | Prevalence of d/s scouring |
| | Damage on d/s apron |
| | Damage on retaining wall |
| | Damage in d/s cut off |
| | Change in river course |
| 3 | **Structural design of weir** |
| | Damage on intake gate |
| | Damage on scouring sluice gate |
| | Damage on main canal |
| 4 | **Planning, Social and Institutional problems** |
| | Absence of projects handing over |
| | Lack of community participation |
| | Cooperative problem |
| | WUA problem |
| | Credit service problems |
| | Market problem |

# CHAPTER FIVE

# 5. CONCLUSIONS AND RECOMMENDATIONS

## 5.1 Conclusions

This research has aimed to appraise the failures and design practices of river diversion structure for small scale irrigation schemes.

- The shortcomings of the current design practice with regards to site and structure selection, hydrology, hydraulic and sediment consideration could be attributed to many factors. Though the schemes are acting under the difficult conditions, the observed problems on the main canals and the head work are related to the construction quality.
- The current design practice approach fully ignores the consideration of the stream basin, retaining wall, divided wall and partially the hydrologic data of the rivers.
- Very poor participation of target community of the schemes in irrigation development stages.
- The problems related to improper hydrologic and sediment consideration was adopted.
- The intake and under sluice gates are totally damaged.
- Some of the irrigation structures are affected by the sedimentation problem.
- The rating wall and main canal of some schemes was failed.

## 5.2 Recommendations

- The design of diversion schemes are must be include the complete hydrological data and sedimentation problems.
- For the future scheme development and sustainability the community participation and mobilization in all aspects of scheme must be done from starting to maintenance of the schemes.
- The current design practice approach must be consider the sustainability and quality of the structures.

- The design document is very necessary for the research study and other relevant information's, so it is must be set properly in the future.
- For the scheme construction time monitoring and evaluation is to give attention as the future.
- Irrigation user association is the main backbones for the sustainability of the constructed schemes, so it is must be organize in proper way.
- Watershed conservation is the most important for the control of the sedimentation and the upstream flooding problems.
- The design practice at the starting time for the planning stage deeply analyses the irrigated area and the discharge of the rivers.

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## APPENDIXES I

Daily heaviest rainfall data availability and checking

Table 1 Rainfall data for Kone station (KMA, 2008)

| S.no. | Year | Max. RF |
|---|---|---|
| 1 | 2004 | 328.3 |
| 2 | 2005 | 340.7 |
| 3 | 2006 | 335.3 |
| 4 | 2007 | 297.5 |
| 5 | 2008 | 342.9 |
| 6 | 2010 | 311.3 |
| 7 | 2011 | 410 |
| 8 | 2012 | 258.4 |
| 9 | 2013 | 401 |
| 10 | 2014 | 361.6 |

Table 2 Rainfall data for Estaysh station (KMA, 2008)

| S.no. | Year | Max. RF |
|---|---|---|
| 1 | 1991 | 602.8 |
| 2 | 1992 | 304.4 |
| 3 | 1993 | 305.2 |
| 4 | 1994 | 212 |
| 5 | 1995 | 79.5 |
| 6 | 1996 | 291 |
| 7 | 1997 | 479 |
| 8 | 1998 | 294.7 |
| 9 | 1999 | 285.4 |
| 10 | 2001 | 287.5 |
| 11 | 2002 | 294.9 |
| 12 | 2003 | 296.6 |
| 13 | 2004 | 358.3 |
| 14 | 2005 | 286.3 |

| | | |
|---|---|---|
| 15 | 2006 | 242.8 |
| 16 | 2007 | 392 |
| 17 | 2008 | 526.1 |
| 18 | 2009 | 197.2 |

Rainfall and other related meteorological data availability is core for any projects that require hydrological analysis. However, sufficient availability of such data in the target position is a rarely phenomenon in developing countries. At the proposed outlet point or diversion point, Fidiangoa_1 River has a catchment area of 43.43km2. The nearest meteorological station for the watershed is that of kone and for the command is also fidiangoa_1

Table 3 storm Analysis for kone Rainfall Station

| S.n | Year | Max. RF | Descending order | Rank | Logarithm /Yo/ | (Yo-Ym)2 | (Yo-Ym)3 |
|---|---|---|---|---|---|---|---|
| 1 | 2004 | 328.3 | 410 | 1 | 2.6128 | 0.0074905 | 0.0006483 |
| 2 | 2005 | 340.7 | 401 | 2 | 2.6031 | 0.0059149 | 0.0004549 |
| 3 | 2006 | 335.3 | 361.6 | 3 | 2.5582 | 0.0010235 | 0.0000327 |
| 4 | 2007 | 297.5 | 342.9 | 4 | 2.5352 | 0.0000798 | 0.0000007 |
| 5 | 2008 | 342.9 | 340.7 | 5 | 2.5324 | 0.0000377 | 0.0000002 |
| 6 | 2010 | 311.3 | 335.3 | 6 | 2.5254 | 0.0000006 | 0 |
| 7 | 2011 | 410 | 328.3 | 7 | 2.5163 | 0.0000993 | -0.000001 |
| 8 | 2012 | 258.4 | 311.3 | 8 | 2.4932 | 0.0010928 | -0.000036 |
| 9 | 2013 | 401 | 297.5 | 9 | 2.4735 | 0.0027825 | -0.000146 |
| 10 | 2014 | 361.6 | 258.4 | 10 | 2.4123 | 0.0129831 | -0.001479 |
| SUM | | 3387 | | | 25.2624 | 0.0315 | -0.0005 |
| MEAN | | 338.7 | | | 2.5262 | 0.0032 | -0.0001 |
| Standard deviation | | 45.3713566 | | | 0.0592 | | |
| Skewness coefficient | | 0.0102775 | | | -0.353 | | |

Table 4 Test for goodness to fit using D-index

| Rank | XI | Normal | Log Pearson Type III | Log Normal | Pearson Type III | Gumbel EVI | Gumbel |
|---|---|---|---|---|---|---|---|
| | | XI -'XI' | XI -'XI' | XI -'XI' | XI -'XI' | XI -'XI' | XI -'XI' |
| 1 | 410 | 10.712 | 409 | 7.056 | 10.712 | 8.564 | 407.649 |
| 2 | 401 | 21.087 | 400 | 20.829 | 21.087 | 25.902 | 399.394 |
| 3 | 361.6 | 4.515 | 360.6 | 3.142 | 4.515 | 2.839 | 360.456 |
| 4 | 342.9 | 11.603 | 341.9 | 9.344 | 11.603 | 3.473 | 342.106 |
| 5 | 340.7 | 3.169 | 339.7 | 0.474 | 3.169 | 4.708 | 340.199 |
| 6 | 335.3 | 1.761 | 334.3 | 4.545 | 1.761 | 8.611 | 335.062 |
| Sum | | 52.847 | 2185.5 | 45.391 | 52.847 | 54.097 | 2184.867 |
| Sum/Mean | | 0.156 | 6.453 | 0.134 | 0.156 | 0.16 | 6.451 |
| Point Rainfall | | 431.9 | 432.87 | 432.87 | 423.15 | 456.32 | 491.74 |
| Design Point Rainfall = | | | 491.738 | | | | |

## 3. Design Flood Determination

Peak flood analysis by SCS method

Time of concentration (Tc) has been calculated by taking the stream profile of the longest streamline and dividing it in to different elevation ranges. Kirpich formula is adopted for computation.

Table 5 Determination of Time of Concentration

| Partial length in m | Elevation in m | Elevation Difference in m | Slope of river, Decimal | Total Tc, in hr. |
|---|---|---|---|---|
| 0 | 2540 | 0 | 0 | 0 |
| 8111 | 2480 | 3127 | 42.85 | 0.44 |
| 7297.7 | 2460 | 3125 | 42.82 | 0.13 |
| 2533.96 | 2409 | 3013 | 118.9 | 0 |
| 17942.66 | | 9265 | 68.19 | 0.56 |
| Total Tc, in hr. | | | | 0.56 |

Table 6 Determination of incremental rainfall

| Time(hr.) | Design Rainfall | Rainfall Profile | | Area to Point Ratio % | Areal Rainfall (mm) | Incremental Rainfall (mm) | Descending order | Rank |
|---|---|---|---|---|---|---|---|---|
| 0.5 | 491.738 | 30 | 147.5 | 68.1 | 100.5 | 100.46 | 100.46 | 1 |
| 1 | | 45 | 221.3 | 72.84 | 161.2 | 60.72 | 60.72 | 2 |
| 1.5 | | 54 | 265.5 | 75.95 | 201.7 | 40.49 | 40.49 | 3 |
| 2 | | 59 | 290.1 | 79.05 | 229.3 | 27.67 | 27.67 | 4 |
| 2.5 | | 65 | 319.6 | 80.92 | 258.6 | 29.3 | 29.3 | 5 |
| 3 | | 67 | 329.5 | 82.79 | 272.8 | 14.12 | 14.12 | 6 |

- **Direct Run off Analysis**

Input data: Curve number at antecedent moisture condition III =86.63

Catchment Area, A = 43.43Km2

Direct run-off, $Q = \frac{(p-0.2*S)^2}{(p+0.8*S)}$ Where, I = Rearranged cumulative run-off depth (mm

P= rainfall

$$S = \left(\frac{25400}{CN}\right) - 254$$ S = Maximum run off potential difference,

Peak run-off for incremental; $Q_p = 0.21 * \frac{(A*Q)}{T_p}$ Where, A=Catchment area (Km2)

Tp=Time to peak (hr)

Q = Incremental run-off (mm)

Table 7 Direct Runoff analysis

| D(hr.) | C.R(mm) | I.R (mm) | A. R(mm) | I. R(mm) | R(m3/s) | UH | | | Remark |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | B | P | E | |
| 0.50 | 14.12 | 14.12 | 8.17 | 8.17 | 72.37 | 0.00 | 0.59 | 1.53 | H1 |
| 1.00 | 43.42 | 29.30 | 35.97 | 27.80 | 246.14 | 0.50 | 1.09 | 2.03 | H2 |
| 1.50 | 83.91 | 40.49 | 76.00 | 40.04 | 354.53 | 1.00 | 1.59 | 2.53 | H3 |
| 2.00 | 184.38 | 100.46 | 176.17 | 100.17 | 887.01 | 1.50 | 2.09 | 3.03 | H4 |
| 2.50 | 245.10 | 60.72 | 236.83 | 60.66 | 537.12 | 2.00 | 2.59 | 3.53 | H5 |
| 3.00 | 272.76 | 27.67 | 264.48 | 27.65 | 244.83 | 2.50 | 3.09 | 4.03 | H6 |

CR= cumulative runoff, IR= incremental runoff, AR= accumulative runoff, R= runoff, B= begin, P= peak and E= end

Table 8 Hydrograph components for each incremental runoff and the base flow

| HYDROGRAPH TIME | H1 | H2 | H3 | H4 | H5 | H6 | HT |
|---|---|---|---|---|---|---|---|
| 0 | 0 | | | | | | 0 |
| 0.5 | 61.48 | 0 | | | | | 61.48 |
| 0.59 | 72.37 | 37.03 | | | | | 109.4 |
| 1 | 103.99 | 209.12 | 0 | | | | 313.1 |
| 1.09 | 110.79 | 246.14 | 53.33 | | | | 410.27 |
| 1.5 | 142.41 | 353.7 | 301.19 | 0 | | | 797.31 |
| 1.53 | 144.72 | 361.54 | 319.27 | 45.21 | | | 870.74 |
| 1.59 | 149.33 | 377.22 | 355.08 | 135.64 | | | 1017.3 |
| 2 | 180.84 | 484.4 | 509.44 | 753.58 | 0 | | 1928.3 |
| 2.03 | 183.15 | 492.24 | 520.74 | 798.79 | 27.38 | | 2022.3 |
| 2.09 | 187.76 | 507.92 | 543.33 | 888.39 | 82.14 | | 2209.5 |
| 2.5 | 219.27 | 615.09 | 697.69 | 1274.6 | 456.32 | 0 | 3262.9 |
| 2.53 | 221.57 | 622.94 | 708.98 | 1302.86 | 483.7 | 12.48 | 3352.5 |
| 2.59 | 226.18 | 638.62 | 731.57 | 1359.38 | 537.96 | 37.44 | 3531.1 |
| 3.03 | 260 | 753.63 | 897.23 | 1773.84 | 788.94 | 220.48 | 4694.1 |
| 3.09 | 264.61 | 769.32 | 919.82 | 1830.36 | 823.16 | 245.21 | 4852.7 |
| 3.53 | 298.42 | 884.33 | 1085.48 | 2244.83 | 1074.14 | 359.61 | 5946.8 |

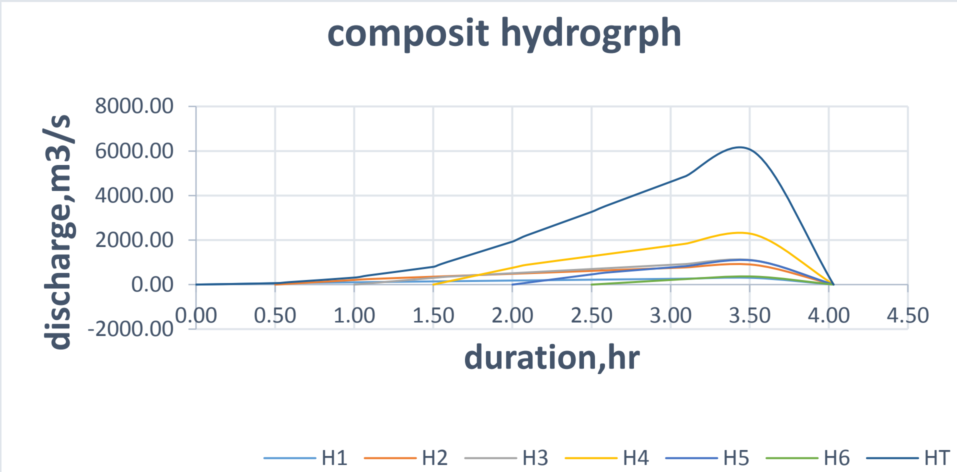



Table 9 River discharge computation at different stages of flow, (downstream)

| Elevation | Depth | Wet area | T. perimeter | T. length | W. perimeter | Hydraulic. R (m) | V (m/s.) | Q (m^3/s.) | Tank |
|---|---|---|---|---|---|---|---|---|---|
| 3015.432 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | |
| 3015.732 | 0.3 | 0.608 | 8.158 | 4.051 | 4.107 | 0.15 | 0.99 | 0.6 | |
| 3016.032 | 0.6 | 2.43 | 16.316 | 8.101 | 8.215 | 0.3 | 1.57 | 3.81 | |
| 3016.332 | 0.9 | 5.34 | 22.765 | 11.294 | 11.471 | 0.47 | 2.12 | 11.34 | |
| 3016.632 | 1.2 | 9.141 | 27.481 | 13.617 | 13.864 | 0.66 | 2.68 | 24.49 | |
| 3115.06 | 99.63 | 1462.61 | 1573.8 | 772.5 | 801.3 | 58.1 | 152.17 | 5946.8 | Design flood |
| 3016.932 | 1.5 | 13.571 | 32.194 | 15.93 | 16.264 | 0.83 | 3.13 | 42.54 | |
| 3017.23 | 1.8 | 18.7 | 36.91 | 18.25 | 18.659 | 1 | 3.54 | 66.23 | |
| 3017.532 | 2.1 | 24.521 | 41.62 | 20.566 | 21.054 | 1.16 | 3.92 | 96 | |
| 3017.832 | 2.4 | 31.649 | 55.339 | 27.402 | 27.937 | 1.13 | 3.84 | 121.64 | flood mark |

**APPENDIXES II**

**Field data collection by interview and group discussion**

Field Data Collection Format

A). Project Description

Name of the scheme-------------------------------------

Kebele ----------------------------------------------------

Year of Construction ------------------------------------

Constructed By-----------------------------------------

Irrigation Area: Planned ---------------------------tual------------------

B). Questionnaires

1. What is the potential of water?

a) Occurred water shortage,

b) There is no water shortage

2. There is no beneficiary from the irrigation system? 1) Yes 2) No

3. If the answer is in question 2 yes why?

4. Do you participate during the construction of the weir diversion structure? 1) Yes 2) No

5. If the answer is question 4 No what is the reason?

6. You know the idea of the project constructed? 1) Yes 2) No

7. The answer in question 6 is No; there is no gated orientation about the project? 1) Yes 2) No

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

9. If the answer in question 8 is yes at which stage of the intervention process you have participated?

a) Planning

b) Construction

c) Post construction

d) Other phases

10. Do you have experts (DA) supervising of the schemes? 1) Yes 2) No

11. Supply of improved seeds is available for farming system? 1) Yes 2) No

12. Do you have get the selective crop production system? 1) Yes

2) No

13. Utilization of pesticides is available? 1) Yes 2) No

14. Do you know cooperative? 1) Yes 2) No

15. If the answer is yes in question 14 when condition is used?

a) At the administration of the scheme

b) At the maintenance of the scheme

c) Other phase

16. Do you get credit? 1) Yes 2) No

17. In the administration proses water user association has properly established the system? 1) Yes 2) No

18. The problem of storage facility occurred in the weir diversion schemes?

1) Yes 2) No

19. Market problems are the main issue after the crop production? 1) Yes 2) No

20. Do you gate additional income from an irrigation system? 1) Yes 2) No

21. In question 19 the answer is No what is the reason?

a) The production is pore

b) Not use the system properly

c) Other problems

22. Participation of the people during the construction phase was?

a) Very good b) good c) poor

C). Group discussion

1. At the construction of irrigation schemes skilled experts, skilled artisans and contractors are available?

2. The construction of the scheme is completely constructed?

3. The beginning of the scheme construction the community is agreeing?

4. After construction what the person is take the responsibility of the scheme administration?

**APPENDIXES III**

**Fiadiangoa_1 Hydraulic design of headwork structure**

The given data below comes from different angels of estimations. So, the discharge value estimate by SCS method and via, the help of the ArcGIS10.1 software to delineate the water shades area of the studied schemes. The other data like High flood level before construction, FSL of the canal and Average bed level was by the reading value GPS, from the schemes. Design flood discharge= 121.64m3/se

High flood level before construction= 3017.71m,

FSL of canal = 3016.68m

Average bed level = 3015.43m

Table 10 Weir dimensioning of fiadiangoa_1 irrigation schemes

| S.no | Formula | Unit | Value | Remark |
|---|---|---|---|---|
| 1 | Lacey's regime width ,L =4.75*√Q | M | 52.4 | |
| 2 | overflow rate over the weir ,q = $\frac{Q}{L}$ | M3/s | 2.32191 | |
| 3 | Normal scour depth(R) = $1.35 * \left(\frac{q^2}{f}\right)^{1/3}$ | M | 2.36717 | |
| 4 | Regime velocity ,V= $\frac{q}{R}$ | M/s | 0.9808 | |
| 5 | Velocity head, ha = $\frac{v^2}{2g}$ | M | 0.049 | |
| 6 | Energy level He = $\left(\frac{q}{1.705}\right)^{2/3}$ | M | 1.23 | |
| | Hd = He-ha | M | 1.191 | |
| | D/S TEL= weir crest level + ha | M | 3017.76 | |
| | U/S TEL = D/S TEL + afflux | M | 3018.76 | |
| | U/S HFL = H/S TEL – ha | M | 3018.71 | |
| | D/S HFL =HFL- Retrogression | M | 3017.21 | |
| | Crest level of weir= U/S TEL- He | M | 3017.53 | |
| | Pond level= FSL of canal + Head loss | M | 3017.18 | |
| | Height of the gate (shutter) =pond level - crest level | M | -0.346 | shatter |
| 7 | Level of U/S pile= u/s HFL-1.5R | M | 3014.2 | |
| | Depth of U/S pile below bed level= Average bed level | | 1.3 | |

| | | | | |
|---|---|---|---|---|
| | - Level of U/S pile | | | |
| | Level of d/s pile= d/s HFL (after retrogression)-2R | m | 3012.47 | |
| | Depth of d/s pile bellow bed level= Average bed level - Level of d/s pile | m | 2.95635 | |
| | Seepage head, Hs= pond level - d/s bed level | m | 1.75 | |
| | Height of crest above bed level= crest level – bed level | m | 2.09602 | |

The depth of downstream pile is always greater than the depth of the upstream pile because of for decreasing or minimizing of the piping problem

**Design of Weir Wall**

The weir wall is proposed to be trapezoidal cross-section with u/s face vertical and d/s face with slope 1:1

**Top Width**

The top width of weir wall (B') is given as the following: - Where, B'= Top width of weir wall and is generally, 1.5 to 1.8

(i) B = 1.1819786m/ (√2.24-1) =2.38m

H=depth of water over the weir wall at the time of maximum flood

G=Specific gravity of weir material (2.24). Range 2-2.4

Depth of water over crest= U/S HFL- crest level, d =3018.71-3017.53=1.1819786m

Top width (a) I a= d/√G =1.1819786m/ (2.24) ^2 =0.78974

II a= s+1=0+1=1 if need to shatter

III a=3d/2G= (1.1819786m*3)/ (2*2.4) =0.7915

Therefore, take Maximum a is minimum Top Width =0.7915 let as say 0.8m

**Bottom Width**

The bottom width should be sufficient so that the maximum compressive stress with in allowable limit &tension does not develop.

(i) $B = \frac{H + Heihetofweir}{\sqrt{G-1}}$ B= $\frac{1.1819786 + 1.3}{\sqrt{2.24-1}}$ =4.997m.

(ii) No flow condition

This occurs when the u/s water level at the pond level and there is no tail water on the downstream.

The overturning moment (Mo) about toe the weir

$MO = \frac{\gamma_W Hs^\wedge 3}{6}$ Where Hs = H + s = 2.096021431 +0= 2.096021431m

=9.81*(2.096021431) ^2/6= 15.05583732 KN. Hs =seepage head (Height of weir+ shutter)

$\gamma$w = Weight of the water (9.81KN/m2)

The resting moment is due to the weight of the weir for a vertical up stream face of the weir.

Mr= $\frac{\gamma HsG}{12}$ (B2 + aB – a2) $= \frac{9.81*2.096*2.1}{12}$ (B2 + 1.7B – 1.72)

=3.598 (B2 + 1.7B – 1.72)

Equating the overturning and the resisting moment, we get, B=1.42m

3.598(B2 + 1.7B – 1.72) =5.53

(iii) High flood condition

During high flood the overturning moment is from difference between upstream and downstream water pressure diagrams

Mo = ( $\frac{\gamma h H^2}{2}$ ) $\Longrightarrow$( $\frac{9.81*2.096^2*1.182}{2}$) $= 25.47\mathrm{KN.m} \ldots \ldots \ldots \ldots \ldots$ (3 )

Mr $= \frac{\gamma H(G-1)}{12}$ (B2 +a2) $= \frac{9.81*2.096(2.24-1)}{6}$=4.23

= 1.47(B2 +a2) ………………………………………… (4)

Equating equation (3) and (4), B=2.8m, B = max [2.86, 2.85, 2.86] $\Longrightarrow$ Adopt B = 2.86m, since the bottom width of the weir is selected during high flood condition ≈3m

Impervious Floor

Seepage head, (Hs) =2.096m

By Bligh’s theory, the total creep length (L) is given by; L=CHs

Where, C=Bligh's Creep coefficient taken as (5-10) for gravel foundation, Let us take C=10, ⇒L=10*2.096=20.96m

⇒Length of downstream impervious floor, L=2d1+Lu+B+L d +2d2

For no shutter…….. L d = 2.21*C$\sqrt{Hs/10}$

Hs = 2.21*C

For shutter Ld. =2.21*10$\sqrt{\frac{Hs}{13}}$ =2.21*10$\sqrt{\frac{2.096}{13}}$, Ld. = 8.87398m

⇒Length of upstream impervious floor, Lu

Lu=L- (Ld +B+2d1+2d2) =20.96-(8.87398 +3+2*1.3+2*2.96) =0.63m, Lu= 2m

Therefore, total length of impervious floor, will be =Lu+B+Ld =0.63+8.87398+3=12.52m

⇒Total creep length changed into= length of impervious floor +2d1+2d2

=12.52+2*1.3+2*2.96 =21.04m

Table 11 Protection Work

| No | Description of the formula | Unit | value | remark |
|---|---|---|---|---|
| 1 | D/S protection work | | | |
| | total length of d/s floor, Lt =18C | M | 12.72 | |
| | Length downstream protection= Lt - Ld | M | 3.85 | |
| | Minimum length d/s concrete block=1.5d2 $\frac{d1}{6}$ | M | 4.44 | |
| | Minimum length d/s lunch apron=2.5d2 | M | 7.4 | |
| | Thickness lunch apron (horizontal= t= $\sqrt{\frac{H_s * q}{13 * 75}}\sqrt{10}$ | M | 1.5 | |
| 2 | Up Stream Protection Work | | | |
| | Minimum length u/s concrete block=1d1 | M | 1.3 | |
| | Length u/s lunch apron=2d1 | M | 5.92 | |
| | Thickness lunch apron | M | 1.5 | |

Thickness of the impervious floor by Bligh's theory

Seepage head (Hs) =2.096m, Creep length (L) = 20.96 m and Specific gravity=2.24

Residual head at point A the toe of weir wall,

$$H = H_S - \left(\frac{H_S}{L}(2 * d1 + Lu + B)\right)$$

$$HA = 2.096 - \frac{2.096}{2.096}(2 * 1.3 + 0.65 + 3) = 1.478$$

The thickness of D/S floor at this point is then obtained by

$$tA = 1.33\left(\frac{HA}{G-1}\right) = 1.33 * \left(\frac{1.47867}{G-1}\right)$$

Provided a thickness of 1.6m for a length of 2

Thickness of D/s Floor after 2m from the function of the weir wall,

$$H_B = H_S - \frac{HS}{L}(2 * 1.3 + 0.63 + 3 + 3) = 1.173\text{m}$$

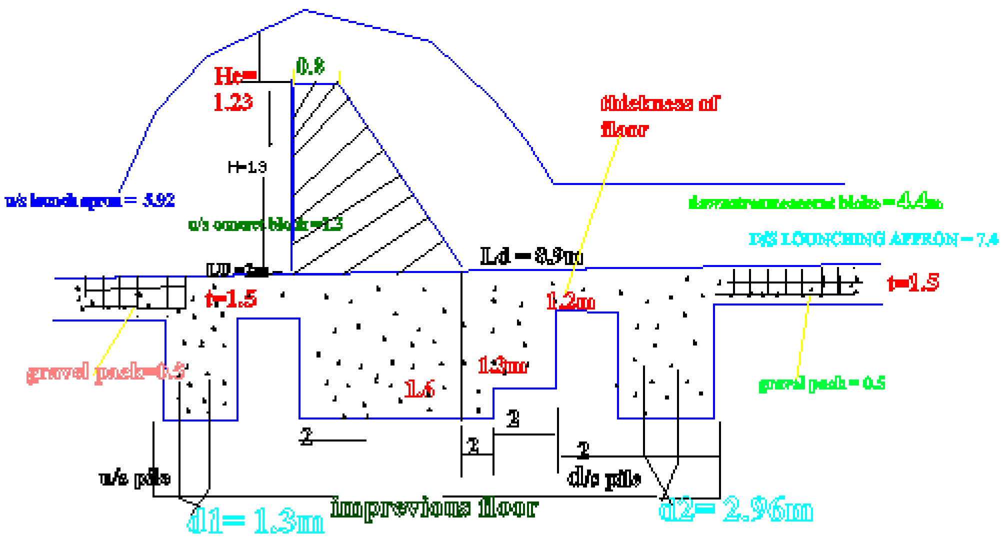
$$t_B = 1.33\left(\frac{H}{G-1}\right) = 1.33 * \left(\frac{1.173}{2.24-1}\right) = 1.3m$$

Also the last length of 2m,

Thickness of D/s Floor, after 4m from the weir wall or toe,

$$HC = HS \quad HC = HS - \frac{HS}{L}(2 * 1.3 + 0.63 + 3 + 3 + 4) = 1.1\text{m}$$

$$tC = 1.33\left(\frac{H}{G-1}\right) = 1.33 * \left(\frac{1.1}{2.24-1}\right) = 1.17491m$$




**Check by Khosla, s Theory**

a) Depth of Downstream pile

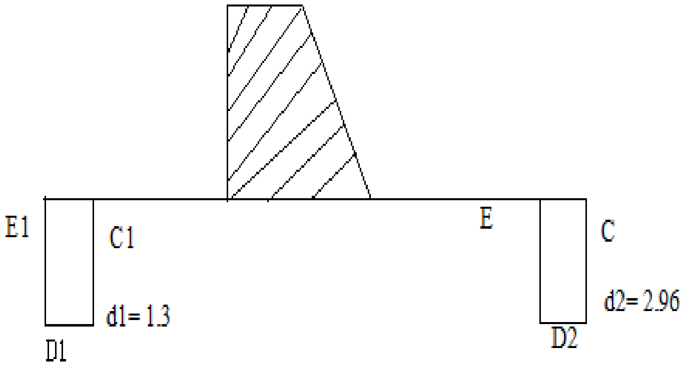
Check the thickness of impervious floor by khosslas theory:

Exit gradient (GE): Check against piping failure

The total length of impervious floor b=12.5m, d2=2.956m, $H_S$ =1.95m

$\alpha = b / d2 = 4.22887$ $\quad \lambda = \dfrac{1+\sqrt{1+\alpha^2}}{2} = 2.67275$

$G_E = \dfrac{H_S * 1}{d_2 * \Pi * \sqrt{\lambda}} = \dfrac{2.096*1}{2.956*\pi*\sqrt{2.67275}} = 0.138112088 \leq 0.1666$ , saf




Up lift pressure

| s.no | pile | ΦE | ΦD | FC1 | FD1 | Thickness correction (Fc1) | Correction for mutual interference | corrected $(\phi E1)$ |
|---|---|---|---|---|---|---|---|---|
| 1 | D/s pile | 41.90% | 28.45% | 58.10% | 71.55% | 5.34% | -1.08% | 37.70% |
| 2 | U/s pile | 28.20% | 71.78% | 19.60% | 80.38% | 10.14% | 0.52% | 82.40% |

Table 12 Check against uplift pressure

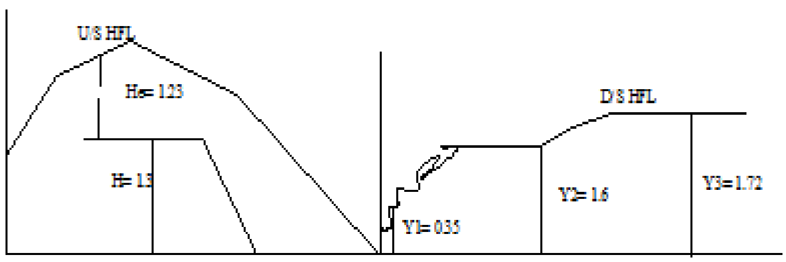
| S.no | Description | Value | recommended value | Remark |
|---|---|---|---|---|
| 1 | Pressure at the toe A, ϕA =E1+((ϕc1-ϕE1)/b)*t | 69.44% | | |
| | Residual head (HA), $H_A = \phi_A * H_S$ | 1.455m | | |

| | | | | |
|---|---|---|---|---|
| | Thickness of the floor = $\frac{h}{G-1}$ | 1.17371 | $m$ $<1.6m$ | ok |
| 2 | Pressure at the toe B (3m from toe), ϕB =E1+((ϕc1-ϕE1)/b)*t $H_B = \phi_B * H_S$ | 46.78% | | |
| | Residual head (HB), | 0.98m | | |
| | Thickness of the floor = $\frac{h}{G-1}$ | 0.79068 | $m < 1.3313069m$ | ok |
| 3 | Pressure at the toe C (3m from toe), ϕC =E1+((ϕc1-E1)/b)*t $H_C = \phi_C * H_S$ | 55.11% | | |
| | Residual head (HC), | 1.2m | | |
| | Thickness of the floor = $\frac{h}{G-1}$ | 0.93161 | $m < 1.17491138$ $m$ | ok |

Therefore, we can conclude that thickness of floor is safe by Khoslas theory.

**Energy Dissipation**

The energy tends to dissipate through a hydraulic jump d/s of the weir.




To determine the water depth of well know Bernoulli's equation is used consider 0-0&1-1

H+ He=y1+q2/2*g+ HL, neglect the HL

H= 1.3m, He=1.23m, q= 2.32m2/s

$1.3+1.23 = y1+\frac{q^2}{2*g*y1^2}$ , 2.53= y1+5.3824^2/2*9.81*y12

2.53y12=y13+0.274 by trial &error y1=0.35 m

$y2 = \frac{y1}{2}(-1+\sqrt{1+8F^2})$ where $f = \frac{q}{\sqrt{gy1^3}} = 3.577$

$y2 = \frac{0.35}{2}(-1+\sqrt{1+8*3.577^2}) = 1.6m$

Critical depth yc is expressed by using formula, yc = $\sqrt[3]{q^2}$/g = $\sqrt[3]{2.23^2}$/9.81 = 0.893m

The head loss dissipated energy As result of jump p =HL = $\frac{(y2-y1)}{4y1y2}$ = ( $\frac{1.6-0.35}{4*0.35*1.6}$ ) = 0.558

The length of jump, Lj= 5(y2-y1) = (1.6-0.35) = 6.25m

y3 =d/s HFL –bed level = 3017.71-3017.71-3015.99 =1.72m.

As y3>y2 the jump occurs on weir face, and there is no need of design stilling basin.

**Stability Analysis of fiadiangoa_1 Weir structures**

Self-weight Unit weight of water and masonry is taken to be 9.81 and 24 KN/m^3 respectively.

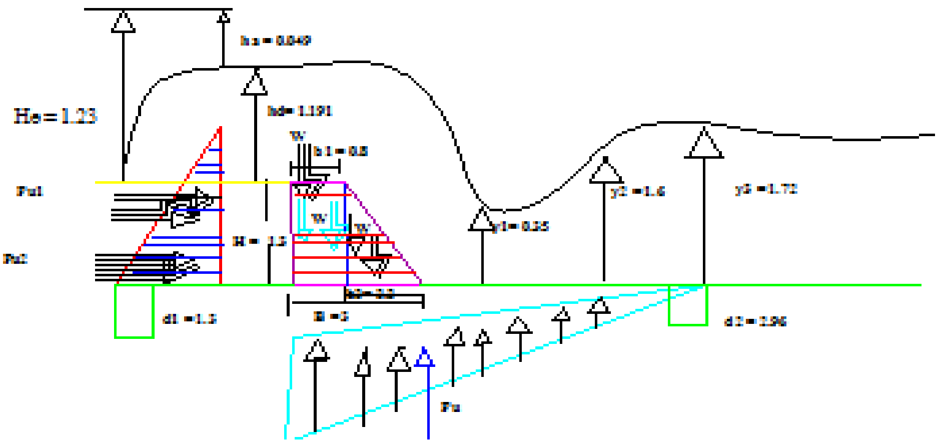



Table 13 Stability analysis of dynamic case

| No | Item | Lever arm(m) | | Moments(KN-m) at toe | | |
|---|---|---|---|---|---|---|
| | | Vertical | Horizontal | | Overturning | Restoring |
| 1 | Pu1 | | -9.220127 | 2.13033 | -19.642 | |
| 2 | Pu2 | | -8.45 | 1.73333 | -14.647 | |
| 3 | Pu | -31.025 | | 2 | -62.051 | |
| 4 | Pd | | 1.75 | 0.11667 | | 0.20417 |
| 5 | Ww | 6.54926 | | 2.6 | | 17.0281 |
| 6 | W1 | 47.84 | | 2.6 | | 124.384 |
| 7 | W2 | 155.48 | | 1.46667 | | 228.037 |
| SUM | | 178.844 | -15.92013 | | -96.339 | 369.654 |

$\sum V = 178.025 KN$ , $\sum H = -15.9201265 KN$

$\sum M_R = 369.654 KN.m$ , $\sum M_O = -96.339 KN.m$

Safety factors

Overturning stability, $S_o = \frac{\sum M_R}{\sum M_O} = \frac{369.654}{96.339} = 3.83 > 1.5 safe.$

Sliding safety factor, $S_s = \frac{\sum H}{\sum V} = \frac{15.92}{178.025} = 0.089 < 0.75$ Safe

Check for tension, x= $\frac{\sum M}{\sum V} = \frac{273.315}{178.025} = -0.04$ and for no tension $e < B/6$

$e = \left|\frac{B}{2} - X\right| = \left|\frac{3}{2} - 1.54\right| = 0.977$

e=-0.04 $< \frac{B}{6} = 0.5$ No tension, ok so, we can conclude that the structure is safe.

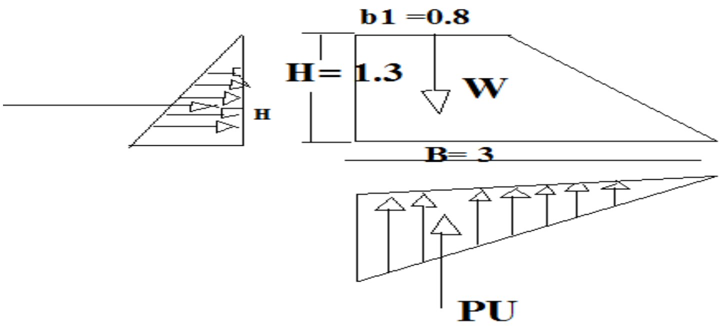



Table 14 Forces and moments acting on weir at static case.

| no | Item | Forces(KN) | | Lever arm(m) | Moments(KN-m) | |
|---|---|---|---|---|---|---|
| 1 | $P_H = 0.5 * \gamma_w * 1.3^2$ | | 8.45 | 0.433 | 3.662 | |
| 2 | $P_u = 0.5 * \gamma_w * 1.3 * 3 -$ | 19.5 | | 2 | 39 | |
| 3 | $W_2 = 0.5 * \gamma_m * 1.3 * 1.3$ | 23.92 | | 2.35 | | 56.212 |
| 4 | $W_1 = \gamma_m * 1.3 * 2.2$ | 32.89 | | 1.4666 | | 48.2387 |

∑V=76.31 ∑H=8.45

∑Mo=42.66166667 ∑Mr=104.4507

M=147

Safety factors

$$S_o = \frac{\sum M_R}{\sum M_O} = 3.05 > 1.5$$

2.448349 Safe

Overturning stability,

Sliding stability, $S_s = \frac{\sum H}{\sum V} = 0.2 < 0.75$ 0.110733 ok

Check for tension, $x = \frac{\sum M}{\sum V} =$ 1.927825

, $e = \left|\frac{B}{2} - x\right| = \left|\frac{3}{2} - 1.93\right| = 0.43, \frac{B}{6} = \frac{3}{6} = 0.5$

Hence, e=0.43 $< \frac{B}{6} = 0.5$ Ok! No tension. Thus, the structure is safe and stable in static condition

**Design of Under Sluice**

Qu = under sluice discharge = 20%*121.64m^3/s=24.328m3/s

- ❖ Water way length (assume 2m). (Basak, 1999)

q=Q/L =24.328/2

=12.164m3/sec

Scoured depth for the sluice section I

$R = 1.35\left(\frac{q^2}{f}\right)^{\frac{1}{3}}$, for f=1

$$R = 1.35\left(\frac{12.164^2}{1}\right)^{\frac{1}{3}} = 7.14m$$

RL of bottom of scour depth on u/s side=U/S HFL-1.5R=3018.81m -1.5*7.14

=3008.19m.

Therefore, the depth of the u/s pile, $d_1 =$3015.99-3008.19m =7.8m.

RL of bottom of scour pile on d/s side= D/S HFL-2R=3017.71m -2*7.14= 3003.43m.

Therefore, the depth of the d/s pile, $d_2$ =3015.99-3003.43=12.56m.

**Impervious Floor**

Min. length of d/s impervious floor, $L_2 = 3.87\sqrt{\frac{H_s}{10}}$

Where H=Hs =2.15m, C=10 (for boulder foundation Dr.K.A.Arora, 2002)

$L_2 = 3.87 * 10\sqrt{\frac{2.096}{10}}$ = 17.71 m ≅ 18m minimum Length of u/s impervious floor,

$L_1 = L - (L_2 + B + 2d_1 + 2d_2) = 21.04 - (18 + 3 + 2 * 7.8 + 2 * 12.56) = -40.68m$

Therefore, take nominal value of 2m for u/s length

**Protection Work**

Total length of d/s impervious floor and protection work

$$L_2 + L_3 = 27C\sqrt{\left(\frac{H_s}{10}\right) * \left(\frac{q}{75}\right)} = 27 * 10 * \sqrt{\frac{2.096}{10} * \left(\frac{12.164}{75}\right)} = 49.78m$$

Length of the d/s protection work, $L_3 = (L_2 + L_3) - L_2 = 49.78 - 18 = 31.78m.$

This length is both inverted filter and launching apron.

Length of the u/s protection work, $L_4 = \frac{L_3}{2} = \frac{31.78}{2} = 15.89m.$

Note; using broad crested weir formula, $Q_S = C_d LH^{\frac{3}{2}}$

Where H=weir height + He=1.5+1.23=2.73m.

L=2m and Cd=1.7

$Qs = 1.7 * 2 * (2.73)^{\frac{3}{2}} = 15.336m^3 / \sec$ Discharge through the under sluice.

And $Qw = C_d LH^{\frac{3}{2}} = 1.7 * 16 * 2.73^{3/2} = 122.69m^3 / \sec$

Discharge through the proper weir with length, L=16m.

Therefore, the total discharges,

$Qs = Qw + Qs = 122.69 + 15.336 = 138.027m^3 / \sec > Q = 121..64m^3 / \sec$ Ok

**Design of Downstream Retaining Wall**

Table 15 Forces and moments acting on Downstream Wing Wall

| No | Item | Forces (KN | | Lever arm(m) | Moments at O(KN-m | |
|---|---|---|---|---|---|---|
| | | Vertical | Horizontal | | Overturn | restoring |
| 1 | w1 = b *H *24 | 24 | | 0.25 | | 6 |
| 2 | w2 = 0,5 * a * H * 24 | 21.6 | | 0.797 | | 17.215 |
| 3 | w3 = B * D * 24 | 20.16 | | 1 | | 20.16 |
| 4 | w4= 0,5 * a * H *soil unit wt | 17.658 | | 1.103 | | 19.476 |
| 5 | Psilt = 0.5* ka *H2*satur. unit wt of soil | | 13.08 | 0.66 | 8.6328 | |
| 6 | ph= 0.5*1.6^2*yw | | 12.5568 | 0.528 | | 6.6299 |
| 7 | pu= 0.5*H^2*yw | 19.62 | | 0.462 | 9.06444 | |
| Total / Summation / | | 103.038 | 25.6368 | 4.8 | 17.6972 | 69.481 |

1. Safety Factor against Overturning

$$\frac{\sum M_+}{\sum M_-} = \quad 3.926$$

2. Determination of eccentricity, e

Net moment, Mn = M+ - M- (KN - m) = 51.78

$$x = \frac{M_n}{\sum F_v} = \quad 0.50$$

$$e = \left| \frac{B}{2} - x \right| = \quad 0.20$$

e<B / 6 (0.23)

3. Factor of safety against Sliding

$$\frac{\mu \sum F_v}{\sum F_H} = \quad 2.61$$

4. Determination of the contact pressure on the foundation

$$P1/P2 = \frac{\sum F_v}{B}\left(1 \pm \frac{6e}{B}\right)$$

P1 = 135.8696192 KN/m2,

P2 =11.32752367 KN/m2

$$F_b = \frac{q_{na}}{P_{max}} \quad =4.415998246$$

Upstream Retaining Wall

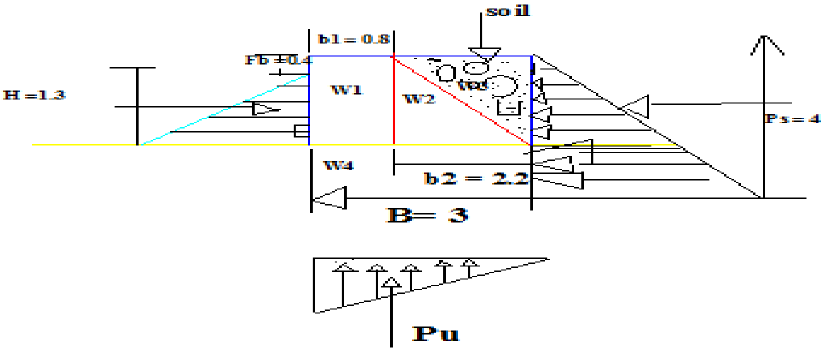



Table 16 Forces and moments acting on upstream wing wall

| no | Item | Forces (KN | | Lever arm(m) | Moments at O(KN-m | |
|---|---|---|---|---|---|---|
| | | V | H | | O | R |
| 1 | w1 = b  *H *24 | 48 | | 0.25 | | 12 |
| 2 | w2 = 0,5 * a * H * 24 | 110.4 | | 1.259 | | 138.99 |
| 3 | w3 = B * D * 24 | 40.32 | | 1.4 | | 56.448 |
| 4 | w4= 0,5 * a * H *soil unit wt | 90.25 | | 2.041 | | 184.20 |
| 5 | Psilt = 0.5* ka *H2*satur. unit wt of soil | | -52 | 1.32 | -69.062 | |
| 6 | ph= 0.5*3.2^2*yw | | 50 | 1.056 | | 53.039 |
| 7 | pu= 0.5*H^2*yw | -78.48 | | 0.924 | -72.516 | |
| Total / Summation / | | 210.5 | 2.1 | 8.25 | 141.58 | 444.68 |

1. Safety Factor Against Overturning

$$\frac{\sum M_+}{\sum M_-} = \quad 3.141$$

2. Determination of eccenticity,e

Net moment, M = M+ - M- (KN - m) = 303.11

$$e = \left|\frac{B}{2} - x\right| = \quad 1.44$$

$$x = \frac{M_n}{\sum F_v} = \quad 0.04$$

B / 6 = 0.47

1. Factor of safety against Sliding

$$\frac{\mu \sum F_v}{\sum F_H} = 65.38$$

2. Determination of the contact pressure on the foundation

$$P1/P2 = \frac{\sum F_v}{B}\left(1 \pm \frac{6e}{B}\right)$$

P1 = 81.61893 KN/m2

P2 = 68.732499 KN/m2

$$F_b = \frac{q_{na}}{P_{\max}} = 7.3512.$$