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JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIROMENTAL ENGINEERING

HYDROLOGY AND HYDRAULIC ENGINEERING CHAIR

ASSESSMENT OF DESIGN PRACTICES AND PERFORMANCE OF DIVERSION WEIR IN SMALL SCALE IRRIGATION (A CASE STUDIES IN OFFIYA IRRIGATION PROJECT)

By: - Tarekegn Kassa Kebede

A research Submitted to School of Graduate Studies of Jimma University, Jimma Institute of Technology, and Faculty of Civil and Environmental Engineering, Hydrology and Hydraulic Engineering Chair in Partial Fulfillment of the Requirements for The Degree of Master of Science in Hydraulic Engineering.

September, 2021

Jimma, Ethiopia

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Main Advisor: Dr. Zeinu Ahmed

Co-Advisor: Mr. Deme Betele

September, 2021

Jimma, Ethiopia

DECLARATION

I declare that this research is my original work and has not been submitted for requirement of degree in any other university. I have acknowledged and quoted all materials in this proposal which is not my works with appropriate referencing.

Mr. Tarekegn Kassa Kebede ____________ ________________

Name Signature Date

APPROVAL SHEET

We certify that the thesis entitled "Assessment of Design Practices and Performance of Diversion Weir in Small Scale Irrigation (A Case Studies in Offiya Irrigation Project)" is the work of Tarekegn kassa and we here by recommend for the examination by Jimma Institute of Technology in partial fulfillment of the requirements for degree of Masters of Science in Hydraulic Engineering.

Dr. Zeinu Ahmed (PhD, Asst. Professor) ____________________ ____________________

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(Co-Advisor) Signature Date

As a member of Board of Examiners of the MSc. Thesis open Defense Examination, We certify that we have read, evaluated the Thesis prepared by Tarekegn kassa and examined the candidate. We recommended that the Thesis could be accepted as fulfilling the Thesis requirements for the Degree of Masters of Science in Hydraulic Engineering.

Dr. __________________________ ____________________ ____________________

External Examiner Signature Date

Mr. ______________________________ ________________ __________________

Internal Examiner Signature Date

Mr. _________________________ ____________________ ____________________

Chair holder Signature Date

Mr. ____________________________ _________________ ____________________

Chair Person Signature Date

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ABSTRACT

Many irrigation scames have been design and built in ethiopia in recent year; about 90% of small scale irrigation projects are under performance. Offiya small scale irrigation project is one of non-functional after it constructed due to destruction of different head work structure components. This study aimed to assess the design practice and performance of Offiya diversion weir through technical performance indicator of hydraulic and structural analysis. The main input data used were collected from Ethiopian metrological services agency, minister of water irrigation and energy office department of GIS, Kaffa zone water mine and energy department, Chena woreda water mine and energy department and direct field survey. The tools that used in this study were Tape, Leveling, GPS Garmin 72, GIS 10.4.1, and Excel spread sheet, DEM, Bentley flow master, HEC-HMS and Geostudio2018. Peak rainfall from Chena metrological station analyzed for feature 50 years return by using Gumbel extreme value distribution method was 140.5846mm. The analysis resulted the river peak design flood of 133.08m3/s and the discharge capacity of the weir over the crest was 80.77988m3/s where about 60.7% of design flood computed by HEC HMS.The weir dimensions top and bottom wi dth, cutoffs, creep length, seepage head and apron thickness were determined or Offiya weirs . The stability analysis of weir body and its appurtenance structures was analyzed and compa red with standard safety factor. The overturning and sliding stability factor for weir body were 2.4 and 1.43 this shows the weir was safe against overturning but not sliding. The upstream retaining wall and wing wall was safe against overturning and sliding but not safe for overstress and bearing capacity of foundation as analysis result. The downstream retaining wall was safe for stability against overturning, but not for sliding, overstress and bearing capacity. The quantity of seepage flux for water at pond level and high flood condition were 1.3479 x 10^-5 m3/s/m and 1.2131 x 10^-5 m3/s/m respectively. The safe exit gradient of silt soil weir foundation was 0.17 whereas the computed was 0.225 which indicate the foundation soil not safe. The thickness of impervious floor for static and dynamic case were than the provided thickness of impervious floor these show the under performance the weir structure to the design period. The other underperformance indicator w as the absence of river training, energy dissipater and up and downstream protection work.

Key words*: - Discharge Capacity, Diversion weir, Exit- gradient, Performance, Seepage.*

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ACRONYMS

ADSWE	Amahara design and supervision work enterprise
AMC	Antecedent moisture condition
CN	Curve number
DEM	Digital elevation model
D/S	down stream
GIS	Geographical information system
FSL	Full supply level
GPS	Global position system
HEC-HMS	Hydrological engineering center –hydrologic model system.
HFL	High flood level
MOA	Ministry of agriculture
MS	Micro soft
NABU	Nature and biodiversity conservation union
SCS	Soil conservation serves
SNNPR	South nation nationality people region
TEL	Total energy level
U/S	Up stream

1. INTRODUCTION

1.1 Background

Ethiopia has a potential of land and water resource to develop irrigation project. About 30 to 70 Mha of land is cultivable. According to Awulachewu (2010), report among this area 15Mha of land is under cultivation. From this cultivated land 4 to 5 percent is irrigated; this shows that the trend of irrigation practice is very low in Ethiopia.

Ministry of water and energy development sector categorize the irrigation scheme based on the size of irrigate land area as small scale irrigation that cover less than 200ha, medium scale irrigation that cover 200-3000ha and large scale irrigation project that cover more than 3000ha(MOA, 2018).

Irrigation is essential to overcome deficiencies in food and stable agricultural production through the year; especially in arid and semi-arid area. It is manly vital in area where the amount and timing of rain fall is not adequate to meet the moisture contents of crops. The development of irrigation and agricultural water management holds significant potential to improve productivity and reduce vulnerability to climatic volatility in any country. Although Ethiopia has abundant rainfall and water resources, its agricultural system does not yet fully benefit from the technologies of water management and irrigation (Awulachewu, 2010).

The majority of population of Ethiopia is dependent on rain fed agricultural production for its livelihood. However, estimated crop production is not close to fulfill the food requirements of the country. One of the best alternatives to consider for reliable and sustainable food security development is expanding irrigation development on various scales (whether small, medium or large) and options diversion, storage, gravity, pumped (Robel, 2005).

Due to small development of water resource potential for irrigation the peoples of Ethiopia have been exposed to food self -sufficient production. Development of small scale irrigation by low diversion weir is important because of their cost effective and reliable agricultural technologies, short construction period and providing income generation. It is one of the best alternatives for reliable and sustainable food sufficient production grant like for our country where the financial is constraint for the development of large scale irrigation projects and other industries(Hora, 2016).

Many irrigation schemes were implemented by the Governments and NGO's for the purpose of food security and food self-sufficiency. However, some of the schemes are performing successfully and some of them are failed to serve the purpose for which they are intended(Ertiro, 2017).

Proper hydrologic, hydraulic and structural analyses are very important which contributes to the good performance and sustainability of the small scale irrigation schemes(Ertiro, 2017).
Diversion head work structure is the structure which constructed on the head of canal or penstock for the purpose of irrigation, water supply or hydropower(Tadesse, 2016).
Headwork structures are engineering facilities built across rivers or canals to store water and/or divert it from its original course. Among these, diversion weir structures are extensively used in irrigation projects to divert water to a canal from either a canal or a natural river by raising the water level upstream(Hora, 2016).
Weirs are one of the important hydraulic structures which are considered as low-level dams constructed across a river to raise the river sufficiently and to divert the flow in full, or in part, into a supplying canal or conduit for the purposes of irrigation, power generation, and flood control, domestic and industrial uses. Diversion structures usually provide a small storage capacity. Weirs provided with or without gates are also used to divert flash floods to the irrigated areas or for ground water recharging purposes. They are also sometimes used as flow measuring structures(Khassaf, 2009).

1.2 Statement of Problem

Many schemes have been designed and built in Ethiopia in recent years; about 90% of Small scale irrigation projects are under performance(Awulachewu, 2010). Most of small and medium scale irrigation projects are failed due to various reasons such as design problem, construction problem, operation and management problem(Ertiro, 2017). The major causes of 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(Desalegn, 2017).
In SNNPR especially Kaffa zone, most of medium and small scale irrigation projects namely Offiya, Gesh, Sheka, Bekko, Wo'I and Worwaro are partially or entirely non-functional project. There are different reason raised roughly for the failure of the irrigation project in Kaffa zone, these are luck of skilled man power, shortage of guide line for design, lack of accurate data needed for design such as meteorological, and hydrological and lack of standardized natural construction material etc. Offiya small scale irrigation project is non-functional after it constructed due to destruction of different head work structure components such as weir protection structures, apron, left and right River training works and the water not diverted to head canal. These problem causes under performance of the Offiya diversion weir.

This study was aim to assess the design practice and performances of small scale diversion weir constructed in Kaffa zone which have defect and functionality problem. The main cause for the failure of weir most of time was poor practice of design. Under or over estimation of hydraulic flood, poor estimation seepage flow under foundation and poor construction material etc. these lends to the failure of weir and make poor performance.

1.3 Objective of Study

1.3.1 Main Objective

The general objective this study is to assess the design practice and performance of Offiya small scale irrigation diversion weir.

1.3.2 Specific Objective

In order to attain the main object of the study, the following specific objectives are set out for major indictor of the study.

i) To evaluate the hydraulic performance of the weir to pass the peak discharge of the ri ver throughout the design period.

ii) To examine the structural stability of the constructed weir and appurtenance structures according to Ethiopian guide line safety value.

iii) To analyze the seepage flow under the foundation of weir using GOESTUDIO2012/1 8 software.

1.4 Research Question

i) How the diversion weir hydraulically preform the river high flood flow?

ii) What is the stability of the weir body and appurtenance structures due to overturning, sliding and stress?

iii) What is the amount of seepage flow discharge under the weir foundation?

1.5 Significance of Study

The main issues that arise from the design consideration of the various elements of the system, as well as the information discrepancy between existing design standards and structure efficiency. There is the problem of non-functionality of the diversion weir in Kaffa zone; this problem is reducing the irrigation practice trends of community; so that performance problem of weir was significance issue and it needs valuable study on the design practice of small scale irrigation diversion weir. While few study are conducted on large, medium and small scale irrigation in a few area of Ethiopia but no one study the performance of constructed diversion weir of small scale irrigation in Kaffa zone. This study has high value significance to the zonal irrigation development problem to reduce the non-functionality rate of the irrigation project.

The significance of the study was to increase the knowledge of problem on the designing of the small scale irrigation diversion weir. Also it uses to brief the design procedure and assumption condition to the designers and concerned professionals participate on designing small scale irrigation diversion weir.

1.6 Scope of Study

The study was geographically limited to SNNPR Kaffa zone, Chenna woreda, Offiya kebele for assess design practice and performance of Offiya small scale irrigation project diversion weir using different design aspects, SCS curve method, Bentley flow master; HEC-HMS and GEOSTUDIO software

1.7 Limitation of the Study

The limitation of this study was mainly shown on the process of primary and secondary data collection. The river was ungagged due to this only rainfall data was used for design analysis; so there was limitation to correlate the real flow with computed. Geological data surveying under weir foundation and river bed subsoil determination have limitation to know all area of the weir site. Determination of the upstream weir impervious floor and cut-off pile condition was difficult due to high accumulation of granular material and back water. The other problem was getting all the design report document Offiya small scale Irrigation project to compare the design procedures and results.

2. LITERATURE REVIEW

2.1 Diversion Weir

Diversion head works are structures constructed across a river or head of a canal to facilitate a regulated and continuous diversion of water into the off-taking canal. Diversion supports the water against its upper face and increase the head of water level to divert the direction of flow when the river have naturally low water head in its stream line(Desalegn, 2017).

The diversion weirs differ from barrages by the mode of raising the water level to the off taking canals.In case of diversion weirs water the canal's full supply level requirement is met by raising the height of the diversion structures itself whereas barrages utilize gates(Afera, 2004).

The function of a diversion head work is as follows(Fikru, 2015):

a. It raises the water level on its upstream side.

b. It regulates the supply of water in to the canal.

c. It controls the entry of silts in to the canal.

d. It creates temporary storage up stream of the weir

The weir body, river training work, under sluice, off-taking canal, fish ladder, upstream and downstream apron, upstream and downstream protection work and cut-off pile are all part of the diversion head work(Tadesse, 2016).

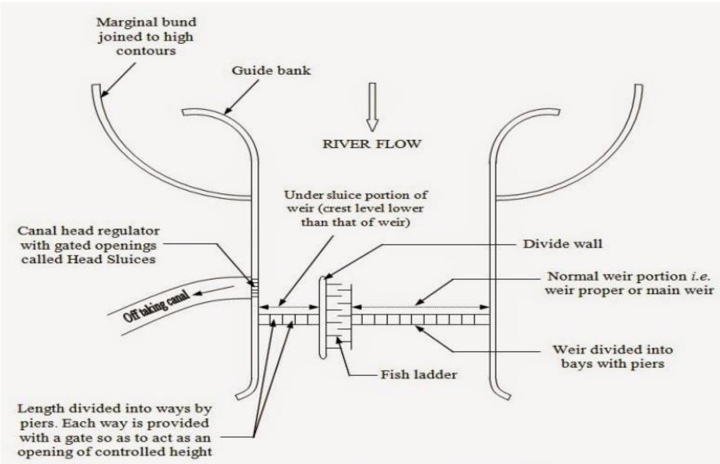


Figure 2. 1 Typical layout of diversion head work

Normally the water level of any perennial river is such that it cannot be diverted to the irrigation canal. The bed level of the canal may be higher than the existing water level of the river. In such cases weir is constructed across the river to raise the water level.

Surplus water passes over the crest of the weir. Adjustable shutters are provided on the crest t o raise the water levelto some required height. When the water level on the upstream side of the weir is required to be raised to different levels at different time, barrage is constructed.

Barrage is an arrangement of adjustable gates or shutters at different tiers over the weir(Tadesse, 2016).

River training works are required near the weir site in order to ensure a smooth and an axial flow of water and thus to prevent the river from outflanking the works due to a change in its course. The river training works required on a canal head-work are guide banks, marginal bunds, spurs or groynes (Fikru, 2015).

Fish ladder is provided just by the side of the divide wall for the free movement of fishes. Rivers are important sources of fishes. The tendency of fish is to move from upstream to downstream in winters and from downstream to upstream in monsoons. This movement is essential for their survival. Due to construction of weir or barrage, this movement gets obstructed, and is determined to the fishes. In the fish ladder, the fable walls are constructed in a zigzag manner so that the velocity of flow within the ladder does not exceed 3 m/s. The width, length, and height of the fish ladder depend on the nature of the river and type of weir or barrage(Tadesse, 2016).

Under sluices /scouring sluices are openings provided at the base of the weir or barrage. These openings are provided with adjustable gates. Normally, the gates are kept closed. The suspended silt goes on depositing in front of the canal head regulator. When the silt deposition becomes appreciable the gates are opened and the deposited silt is loosened with an agitator mounting on a boat. The muddy water flows towards the downstream through the scouring sluices so the gates closed. But, at the period of flood, the gates are kept opened(Fikru, 2015)

The divide wall is a long wall constructed at right angles in the weir or barrage; it may be constructed with stone masonry or cement concrete. On the upstream side, the wall is extended just to cover the canal head regulator and on the downstream side it is extended up to the launching apron. To form a still water pocket in front of the canal head so that the suspended silt can be settled down which then later be cleaned through the scouring sluices from time to time. It controls the eddy current or cross current in front of the canal head. It provides a straight approach in front of the canal head. It resists the overturning effect on the weir or barrage caused by the pressure of the impounding water(Tadesse, 2016).

The canal head regulator is a structure built at the head of the off-taking canal to control the flow of water into the canal, prevent silt from entering the canal, and prevent river floods from entering the canal. The regulator has a gate to regulate the flow of water into the canal, which is aligned 90^0 to 110^0 meters from the weir proper (Garg, 2011).

The impervious floor is designed in all cases to reduce the surface flow action that causes scouring due to unbalanced pressure in the hydraulic jump trough. Generally speaking, except very few sites, end sills are not seen at the constructed structures. These could have played significant role in controlling receding jumps and hence reducing erosive power of the flowing water(Robel, 2005).

The thickness of aprons the upstream apron, sloping apron and the downstream apron are calculated taking the maximum unbalanced head between the subsurface uplift force and the surface flow for different case of flow ,high anticipated flood flow; pond level flood flow and static water condition where the gates are closed with water at pond level upstream(Hora, 2016)

Reduce the exit gradient, i.e. increase the creep length, to reduce the piping phenomenon. Increase the impervious floor length and provide upstream and downstream cutoffs to increase the creep length. Cutoffs are vertical impervious piles installed upstream, downstream, or in the middle of an impervious floor to protect seepage through the weir's under bed and reduce uplift force by increasing seepage creep length (Garg, 2011).

To protect the riverbed from erosion, a protection structure and launching apron are provided at the upstream and downstream ends of the impervious floor(Garg, 2011).

2.2 Design Practice of Weir

There are two elements to the design of a modern diversion weir or barrage (Afera, 2004).

i) Hydraulic analysis

ii) Structural analysis

2.2.1 Hydraulic Analysis

Hydraulic design entails determining the flow conditions upstream and downstream of the weir at various flow rates and assuming the necessary data to size different parts of the flow structure accordingly, ensuring that the structure serves the intended purpose. Subsurface and surface flow conditions are used to assess the size of the weir and its apparatus structure on a permeable foundation.The subsurface and surface flow conditions of the river are used to assess the size of the weir and its apparatus structure on a permeable foundation (Afera, 2004)

The hydraulic analysis of the weir entails determining the following parameters, which are used to calculate the dimensions of the head work structures: Waterway length, water level upstream and downstream, energy level upstream and downstream The depth of scouring, the amount of affluent, and the shape of the weir are all factors to consider. The design discharge is regarded as an inflow upstream of the structures that can safely pass downstream of the weir without causing any damage to the head work apparatus (Desalegn, 2017).

Hydraulic flow is usually divided into two categories. Surface flow analysis and seepage or sub-surface flow analysis are two types of flow analysis.

Analyze the seepage flow when a weir is built on a pervious foundation, it may be built on either an impervious solid rock foundation or a pervious foundation. It is prone to water seepage underneath it. There are three main factors in any seepage problem: The type of flow, the soil media, and the boundary condition are all factors to consider. Seepage is a major issue that has an impact on the structures. When large quantities of water exist in an unsuitable foundation, it causes significant damage. As a result, seepage beneath weir foundations is an important topic in the safety of weir structures(Khassaf, 2009).

Under diversion head work, seepage flow is the flow of water across porous media due to hydraulic head gradient in accordance with the law of continuity. Water flow from the region of high energy level to the region of low energy level under the beneath foundation and around any hydraulic structure on permeable foundation (Hora, 2016).

The data required for structural and economical safe weir are discussed below(Asawa, 2005)

i) The water way length of river at weir site

ii) Stage discharge relationship including HF and the corresponding discharge.

iii) Characteristics' of sediment and river bed material

The estimation of the shape and height of the weir, clear waterway or length of weir crest, discharge and head over the weir, and flood and energy level, afflux and scour depth are all part of hydraulic design.(Ertiro, 2017).

Waterway Length: The weir crest length should be sufficient to safely pass the design flood without causing damage to the weir's apparatus structure or requiring outflanking from the bank (Hora, 2016).

The waterway and afflux are correlated. With increase of afflux waterway decreases and vice versa, hence a limit placed on maximum afflux shall limit the minimum waterway. A weir with a long crest gives a small linear discharge and hence the required energy dissipation per meter of the crest is smaller than what is needed for a shorter crest length(Afera, 2004)

The waterway's width was usually kept between 1.2 and 1.4 lacey perimeters; some engineers preferred to keep the waterway narrower despite the costlier works (Novak, 2007)

The amount of the afflux determines the top level of guide banks and marginal bunds, as well as the length of the marginal bunds. The afflux also has an impact on the dynamic action downstream of the weir, as well as the location and parameters of the hydraulic jump.

Although a greater afflux narrows the waterway, it increases the discharge per unit length of weir. As a result, the depth of the scour increases, raising the cost of protection works. Due to possible outflanking, a higher afflux increases the risk of river training structures failing. At the same time, the discharge intensity as a result of scour will increase, necessitating an increase in the length of loose protections upstream and downstream, as well as the depths of pile lines at both ends, all of which will be expensive. As a result, it is often preferable to keep the afflux to a safe value of 1 to 1.2 meters, most commonly 1 meter. However, in a steep reach with a rocky bed, a higher afflux value is allowed (Garg, 2011).

Pond level: The pond level is the water level that must be kept in the under sluices pocket (i.e., upstream of the canal head regulator) in order for the canal to maintain full supply level when full supply discharge is fed into it. The full supply level at the canal's head is determined by the canal's longitudinal section. The pond level is maintained about 1.0 to 1.2 meters above the canal's full supply level to ensure that enough working head is available even though the canal's head reach has silted up or if the canal needs to be fed excess water. If the pond level is limited in certain circumstances, the full supply level is determined by subtracting the working head from the pond level(Asawa, 2005).

Scouring: During high flood flows, the river bed scours, and a large scour hole may grow gradually to the adjacent solid apron, causing undermining of the weir structure.

2.2.2 Structural Analysis

The structural analysis of the head work structure general consists of weir proper body and River training work in both static and hydrodynamics condition.

Diversion head work stability analysis is to keep compressive stresses under control while preventing the growth of tension stresses in the concrete. The ultimate tension strength of unreinforced concrete or possibly stone masonry is just one-tenth of the ultimate compressive strength. As a result, allowing any tension stresses is considered unwise. The "middle third rule," which maintains the resultant of all forces in the middle third of the structure, is used to achieve this. Overturning will not be a problem for the typical diversion weirs for small scale irrigation projects if the reinforced concrete apron is connected to the weir, which is necessary to resist apron uplift. Forces acting on the structure and possible moments resulting from these forces are included in this analysis, which is done around the external toe of the critical section of a weir as shown above, but internal toe for wing walls (MOA, 2018).

2.3 Cause of Weir Failure

Failure of hydraulic systems such as a weir or a barrage is caused by a combination of subsurface and surface flow at the site, as well as poor construction quality. Pipes, uplift force, suction caused by standing waves, and scouring on both upstream and downstream of the buildings are examples of such causes. (MOA, 2018). When the hydraulic gradient or exit gradient is greater than the critical value of the soil, the surface soil at the d/s end boils first and is washed away by percolating water. Seepage water forms a channel in the form of a pipe as the process of removing or washing out soil continues. This is known as piping, and it can lead to foundation failure. Similarly, percolating water exerts an uplift force on the floor from the bottom, and if the weight of the floor is insufficient to resist this uplift force, the floor may crack or burst (MOA, 2018).

2.3.1 Cause of Subsurface Flow

Uplift pressure and piping are the two main causes of such failure(MOA, 2018).

1) Piping or undermining

This happens when water from the upstream side percolates through the foundation's bottom and appears at the weir or barrage floor's downstream end. By scouring at the point of emergence, the force of this percolating water eliminates soil particles. As the soil particles are steadily removed, a depression gradually forms at the bottom of the foundation, extending backwards towards the upstream. As a result of this erosion, a channel or a pipe forms beneath the weir's floor, causing it to fail, and a hollow-like pipe formation forms beneath the foundation, causing the weir or barrage to fail by subsiding. This is referred to as pipe failure or undermining(MOA, 2018).

2) Uplift pressure

When percolating water exerts upward pressure on the foundation of the weir or barrage, this phenomenon happens. It may fail by rapture if this uplift is not counterbalanced by the structures self-weight. Its distribution is greatest upstream and diminishes as it moves downstream(MOA, 2018).

2.3.2 Cause of Surface Flow

1) By hydraulic jump

Hydraulic jump occurs when water flows at a high velocity over the crust of the weir or over the gates of the barrage. On the downstream side of the hydraulic jump, a suction pressure or negative pressure acts in the direction of uplift pressure. If the impervious floor is not thick enough, the structure would fail due to rapture (MOA, 2018).

2) By scouring during floods

The barrage's gates are kept open, and the water flows at a high rate. The water may also flow at a very high rate over the weir's crest. In all cases, scouring may occur on both the downstream and upstream sides of the structure. Due to scouring on both the downstream and upstream sides, shearing jeopardizes the structure's stability (MOA, 2018).

2.3.3 Failure Due to Silt

When a weir is built across a river, it causes progressive retrogression on the downstream side and aggravation on the upstream side. Because the initial flow area calculated for the approach velocity in the upstream side encloses between the U/S high flood level and pond level, upstream aggradations have the propensity to increase the approach velocity in the upstream side of the weir (MOA, 2018).

Downstream retrogression, on the other hand, results in a decrease in downstream river phases, which must be taken into account during the design process. Increased exit gradients are caused by lower river water levels due to retrogression on the downstream side, putting the structure's safety in jeopardy. As a rule of thumb, when computing the design floor and exit gradient, assume a retrogression of 0.3-0.5m. Retrogression would not be inferred if the downstream river course is solid rock(MOA, 2018).

As a result, silt deposition could impair the soundness of the diversion headwork's operation by clogging gates (under the sluice gate for the weir and the gate for the barrage) and other waterways, diminishing its productivity or causing it to collapse. As a result, the processes listed below are proposed (MOA, 2018):

1) On Rivers with high sediment concentrations, such as those with poor catchments on the upper reach of a river or seasonal rivers, an excluder in the river or an extractor along the canal may be constructed, depending on the magnitude and type of sediment.
2) Because suspended silt is usually carried by the flood on the lower reach, an extractor in the canal would be appropriate. The decision is made based on the current state of the silt, and the designer chooses between the two mechanisms.

2.3.4 Failure Due to Seismic Load

Failure due to an earthquake is very likely in seismic zones, especially on any elevated structure. As a result, an appropriate ground acceleration coefficient can be used in the design process based on the delineated seismic zones in our country(MOA, 2018).

2.3.5 Failure Due to Man-Made Activities

The failure of a diversion headwork system can also be caused by man-made actions. Such offenses as the use of substandard construction activity and materials for the purpose of deception, as well as the lack of competent construction labor, including deliberate interference such as steel looting, cement and other materials and deliberate attack on the structure due to unhealthy attitude towards it can also cause danger to the structure. This necessitates close monitoring of the construction activities as well as raising awareness among not only the project beneficiaries but also the surrounding communities(MOA, 2018).

2.4 Remedies for Failure of Diversion Headwork Structure

According to (MOA, 2018) Remedies to avoid failure due to piping and uplift pressure is to decrease the hydraulic gradient: this can be made by increasing the path of percolating or seeping flow through:

Increasing the path of percolation or creep length of seepage water by installing sheet piles upstream, downstream, or in the middle of the impervious layer to lessen the hydraulic gradient or lengthen the impervious layer itself;

1) Provision of energy dissipater blocks such as friction blocks and impact blocks, i.e. select appropriate type of stilling basin;
2) Provision of inverted filter with concrete blocks on the top so that the percolating water does not wash out the soil particles;

Remedies to avoid failure due to scour (MOA, 2018):

1) provide a launching apron with sufficient length
2) provide a pile or curtain with a greater depth than the scour level

Remedies to avoid failure due to man-made problem are(MOA, 2018):

1) allow enough time for research and investigation (hydrological study, geotechnical investigation) to fully comprehend the project's hydrologic and geological conditions
2) supplement analytical design with model simulation to increase the design's confidence

2.5 Previous Studies.

The assessment of design practice and performance carried out in western Oromia, Ethiopia on six selected diversion weir and analysis the design method, parameter and assumption that make change the weir dimensions and cause low performance(Hora, 2016).

Seife Tadesse used hydrology and hydraulic analysis, structural analysis, and assessment of weir foundation condition to evaluate the cause of Tana Beles diversion weir failure (Tadesse, 2016).

Appraisal of Current River Diversion Structure Design Practices (The Case of Amhara Region) (Afera, 2004). The goal of this study was to identify and assess the current knowledge gap in river diversion structure design and operation. The analysis is based on the problems with 33 river diversion systems. The investigation and analysis of the problems allowed for the identification of a knowledge gap in the scheme design. As a result, a number of issues are discovered to be important in contributing to the structure's current problems. Many factors can be blamed for the current design practice's shortcomings in terms of site and structure selection, hydrology, hydraulics, and sediment consideration (Afera, 2004). Due to a lack of options for designing different types of structures for different site conditions, site and structure selection can be problematic. Problems with hydraulic computation and sediment consideration arise from the inability to obtain locally calibrated research results and guidelines. Due to a lack of hydrologic and sediment data or regionalized formulas, empirical formulas and processes intended for areas with no comparable features are used, resulting in issues with insufficient hydrologic and sediment consideration. It's impossible to come up with a universal standard for the design and operation of diversion structures due to differences in topography, catchment, and river morphologic characteristics(Afera, 2004).

The first thesis reviewed for this research is performance assessment of Fentale diversion headHeadwork (Fikru, 2015).

Appraisal of Design Practice And Failure of River Diversion For Irrigation Schemes: a Case of Wadla Woreda North Wollo (Desalegn, 2017).

The aim of this study was to evaluate river diversion structure failures and design practices for small-scale irrigation schemes (Desalegn, 2017).

Seepage Analysis Underneath Diyala Weir Foundation (Khassaf, 2009) The GEO-SLOPE, SEEP/W finite element package was used to analyze seepage flow under the foundation of the Diyala weir in this study. The problem was solved using a two-dimensional quadrilateral finite element model. Water seepage is one of the most serious issues that have an impact on hydraulic structures (Khassaf, 2009). Such engineering issues plague the Diyala weir structure. It was used as a case study, and it was drawn and tested using a numerical model against piping and uplift pressure. (Khassaf, 2009).

According to (Al Siaede, 2019), Practical Geotechnical Analysis of in situ Stress Variations and Hydraulic Stability of Small Weirs Using SEEP/W and SIGMA/W Simulation Geotechnical soil problems underneath foundation of hydraulic structures occur due to engineering soil properties, geological setting and hydraulic properties of the projects. Two finite element programs of Geoslope 2012 software, SIGMA/W and SEEP/W, were used for analysis of in situ stresses, load deformation behavior, seepage quantity and vertical gradient below Teeb weir foundation, to compute factors of safety against seepage uplift(Al Siaede, 2019).

3. METHODS AND MATERIALS

3.1 Description of the Study Area

3.1.1 Location

The study area is found in SNNPR, Kaffa zone, Chenna woreda, in south western part of Ethiopia. Geographically found $7^{0}11$"N and $35^{0}35$"E and 1325m.a.m.s.l. altitude.

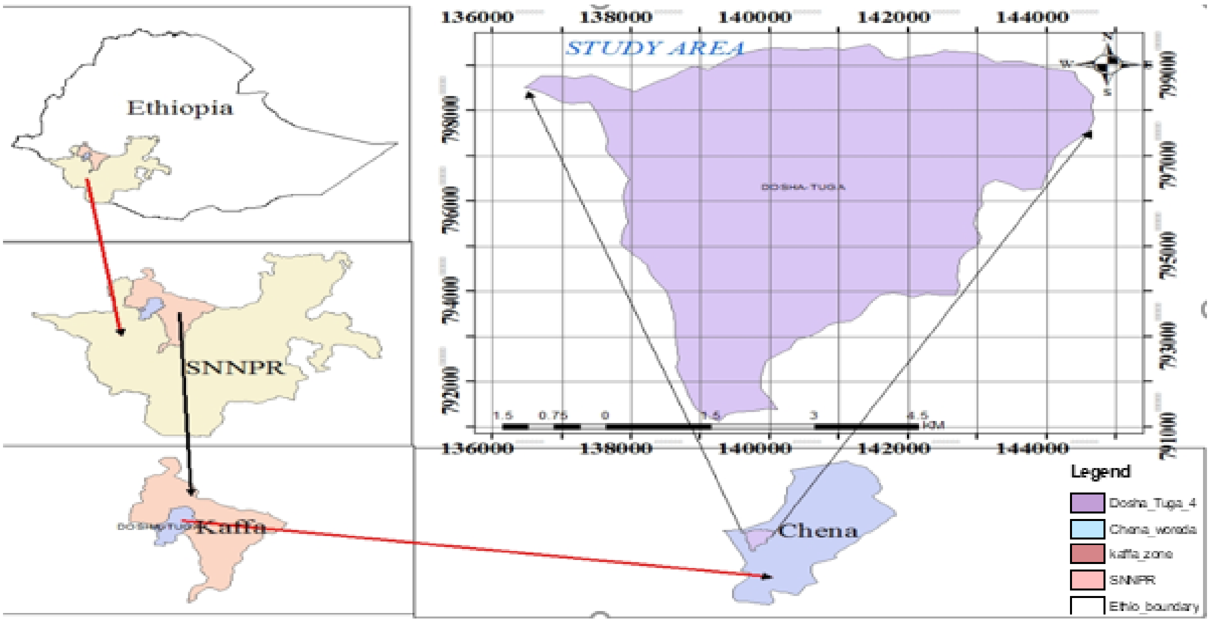


Figure 3.1:-Study area map

3.1.2 Climate

According to NABU (2017), stated that the climate is characterized by bimodal rainfall pattern, with the main rainy season between June and September, and a short rain period from February to April. Kaffa receives its rain fall from the southwest monsoon, which reaches its maximum intensity during July, August and September.

The mean annual temperature of the study area was 22.5^{0}c. Monthly maximum temperature range from 27.4^{0}c in February to 24.9^{0}c in July; the minimum temperature ranges from 11.5^{0}c in May to 9^{0}c in January(Tsedeke, 2017).

Shisho Inde, Chena, Bita Genet and Shewa Bench are the nearest rain gage station around the study area. Mean monthly rain fall of Chena and Shisho-Inde were 144.115mm and 125.5183mm respectively and Over 85% of total annual rainfall, with mean monthly values in the range of 114.45-185.2564 occurs in the 9 months long rainy season.

3.1.3 Water Source of Project.

Offiya irrigation project head work was constructed on the Offiya River that drains to Meni River which is the largest and main tributary to Meni River. The sub- tributaries of Offiya River are Dosha and Kosha stream they join each other and form big Offiya River.

3.1.4 Socio-Economic Situation

According to CSA (2007), the total population of Chena woreda is 161,292. Out of these population 79,514 (49.6%) are male and the remaining 81,778 (50.4%) are female. The population cover of urban is 16.3% and 83.7% of population live in rural area. According to documented data of woreda water office report Dosha-Tuga kebele total population was estimated in 2004 E.C was 7,284. Out of these populations 4,010 are male and the rest 3274 are female. The total households in the area are 943 houses (Tsedeke, 2017)

The study area communities' dependent on agricultural production and livestock. The cash crop production in the area is sorghum, maize, teff, haricot bean, rice, coffee etc.

The study area kebele livestock population was estimated as following table according to kebele agricultural office report.

Table 3. 1:- population of livestock in Dosha Tuga kebele (Tsedeke, 2017)

s/n	Type of animal	Number
1	Cattle	3781
2	Sheep	807
3	Goats	1146
4	Donkey	8
5	Horse	22
6	Mule	159

Source: design report of Offiya irrigation project

3.2 Material

The tools that would be used in this study were Tape, Leveling, GPSGarmin72, GIS10.4.1, Excel spread sheet, DEM 30*30, hydrologic soil group map, LULC, HEC-HMS, Geostudio2018/2012 and Bentley flow-master. The references used for these study are: different authors' books, Journals, Modules, Guide lines lecture note and other published and non-published document related to the study. The data has been required to accomplish this study were: metrological data, Physiographic data, Topographic data, Hydrological data, Geological data of weir foundation and Catchments characteristic data or LU/LC. These data were collected and sampled by using different technical aspects of data collection methods.

3.2.1 Data Source

The data source for this study were primary and secondary data sources. Primary data collecting process include field topographic surveying, river physiographic survey, Observing foundation materials and their characteristics', Observing watershed characteristics and taping headwork structures. The secondary data collecting process including collecting meteorological data from metrological agency, history of weir from KZWMED, land use land cover data using ArcGIS and design drawing of the project. The Design aspect and recommendations were stated using reference, Guide line, and literature of review from previous study.

Table 3. 2:- Data type and source

S/N	Data type	Data source	Remark
1	Rainfall	EMSA	Monthly
3	Geological data	Site	Site observation by geologist
4	LULC map	Satellite	African map 2020
5	Soil map	MOWE and GIS dep’t	Baro basin
7	Water shed characteristic’s	MOWE and GIS dep’t	DEM 30*30
8	Weir dimension	Site	Direct surveying
9	Physiographic data	Site	Direct surveying

3.3 Data Entry and Consistence Test

The data has been collected from primary and secondary source were entered by using different computer program such as MS excel, GIS, AUTOCAD,HEC-HMS, GEOSTUDIO, flow master, MS word and other that were used for data correlate and data analysis tools. The rainfall data from Chena and Shisho Inde station were used for constituency test. Which are nearer to the study area, but they may have consistence problem due to data missing due to lag of recording, maintains of recording instrument and other different reasons. Mass curve approaches were used to check for consistency in the rain gauge's rainfall record.

The strong linear correlation coefficient was between 0.6 and 1 Making accumulative precipitation against time was used to plot the mass curve(Hora, 2016).

3.4 Description of Offiya Diversion Head Work

The main component of Offiya small scale irrigation diversion head work were weir proper body, up and downstream impervious floor, right and left retaining wall, under sluice, canal head regulator, and downstream sheet pile and divide wall. The natural water way at weir upstream site was 36m. The constructed weir dimension was 3m bottom, 0.8m top width and 16.8m clear water way. The length of upstream and downstream impervious floor were 3m and 13m respectively with 0.3m thickness. The height of upstream weir over river bed was 3.75m and the downstream retaining wall was 3.65m the length of up and down stream retaining wall were 2.7m and 13m respectively. The width of under sluice was 1m with 1.5m heighted steel sheet gate maximum opening height was 1.5m. The head canal regulator was circular in shape opening to left side of river bank with 0.5m diameter opening. Offiya diversion weir pile sheet was provided at downstream end of impervious floor which was constructed from stone masonry with 3m depth and 0.3m thickness and at upstream 1m with the same thickness with floor structure with 0.3m. Divide wall was provided from upstream hell of weir down 2m length with 1m thickness and 3.75m height.

3.5 Geological Condition of Weir Foundation.

To characterize the foundation condition of the headwork site, both surface and sub-surface geological observation have been performed. From visual identification and observation of the site, the entire portion of the active streambed bed, both up- and down-stream of the axis, is covered with coarse alluvial sediments at the thin upper portion. At the bed of the river the most top part of the bed is covered with bolder material deposition and at the depth observed at eroded area of under downstream floor fine grain alluvial sediment (silt) deposition was observed at depth with reddish and yellow colored. On the other hand, the left bank is mainly made up of rock outcrops, which are classified as acidic volcanic rock, named as rhyolite/ignimbrite. This rock unit at the bank is covered with variably thick (relatively thick at top of the bank, and gets thin towards its depth) which is derived from both from slope processes and weathering of the rock.

Figure 3. 2:- Left side river bank material.

The right side bank totally covered by the fine grained flood plain deposits, especially its top portions. This gentle flood plain land extends to some limited width and joins relatively sloppy ground, which is covered with different soil type that derived from weathering of underlying rhyolite rock unit. The alluvial flood plain deposit is fine grained, dominated with silt and clay and it is alluvial in origin that it is transported and deposited by the stream during flood times. It is long time deposit of suspended materials of the flood. It is firm, moist, impermeable soil. From visual examination, it is fine grained soil dominated with silt and has dark brown color. Some coble are shown dispersed at the depth.

Figure 3. 3:- Right side river bank material.

3.6 Catchment Delineation and Analysis

3.6.1 Catchment Area Delineation

The study area was delineated from 30m*30m DEM by using ARC GIS software. Therefore the sub-watershed area of Offiya River at Offiya small scale irrigation diversion weir site was 28.42km^2.

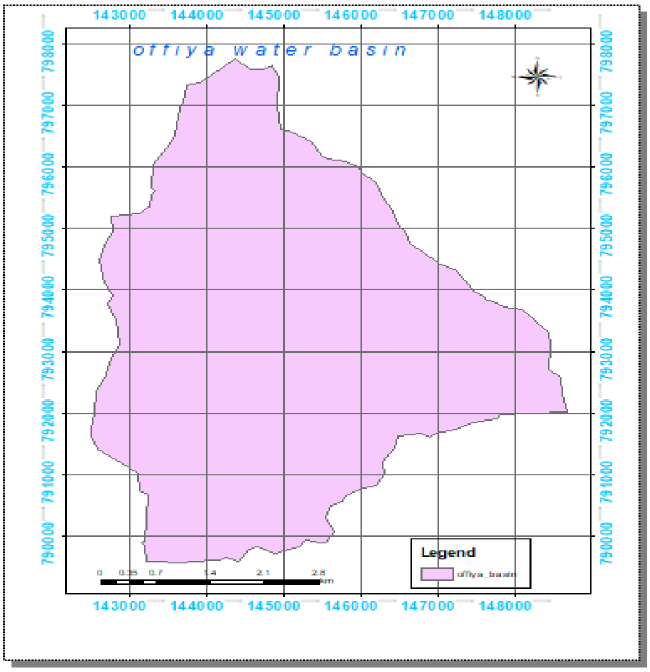


Figure 3. 4:-Sub- watershed area of study catchments.

3.6.2 Characteristics of Watershed

Offiya River watershed covers very wide range of altitude between about 2134m.a.m.s.l. at upper stream part of the watershed and drops 1328m.a.m.s.l. at the project intake point downstream. The watershed can be dominantly classified into five land cover groups:

Tree/forest, Grass Land, Agricultural Land, Scrub/Shrub and Village or Built Area. Cultivated watershed widely covered with domesticated crops such as sorghum, maize and fruits. The non-cultivated area is covered with grass, forest, shrubs/bush and houses. Vegetation area cover with few big and scattered trees and grasses. The soil group of offiya watershed was grouped into three soil group which were soil group B, soil group C and soil group D.

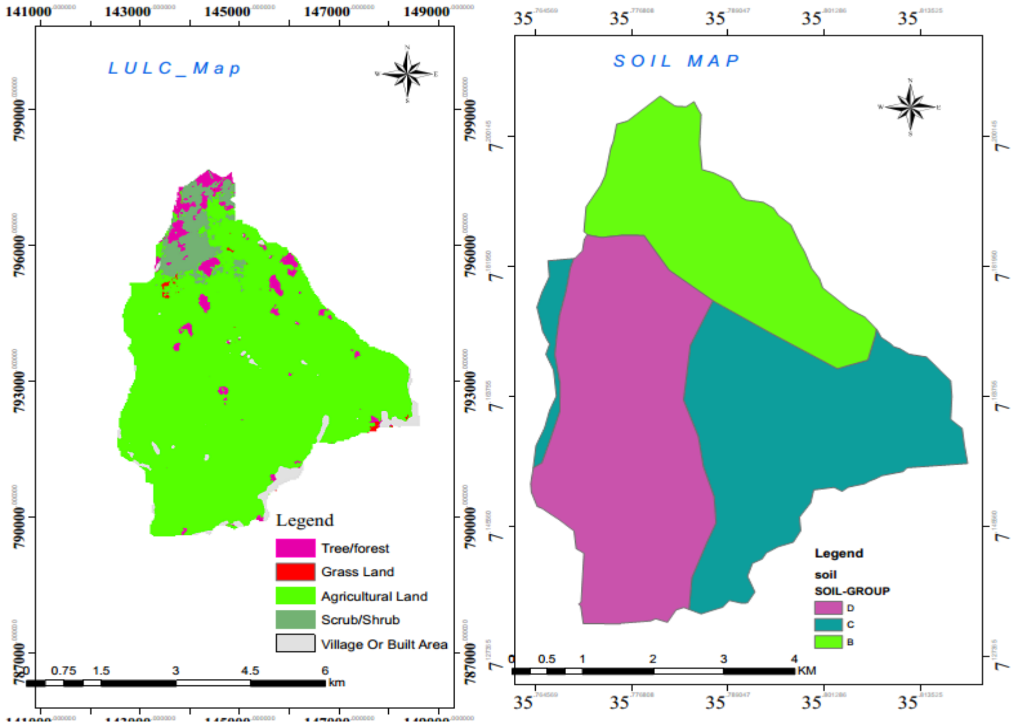


Figure 3. 5:-LULC map 2020 and soil map.

3.6.3 Slope Analysis

The land form of Offiya catchment is dominated by undulating terrain and rolling plain with the slope range 5-15% followed by hill plains or almost slope ranging from 15-21% slope located on the left side of the catchment. As it can be seen from at site level & the map, slops ranging above 21% is covering about 4.5742% of the catchment. Some part of the catchment is also gently slope with slope range of 0-5 % but the area coverage is 2.5km^2. The slope class map below indicates the spatial distribution of landform.

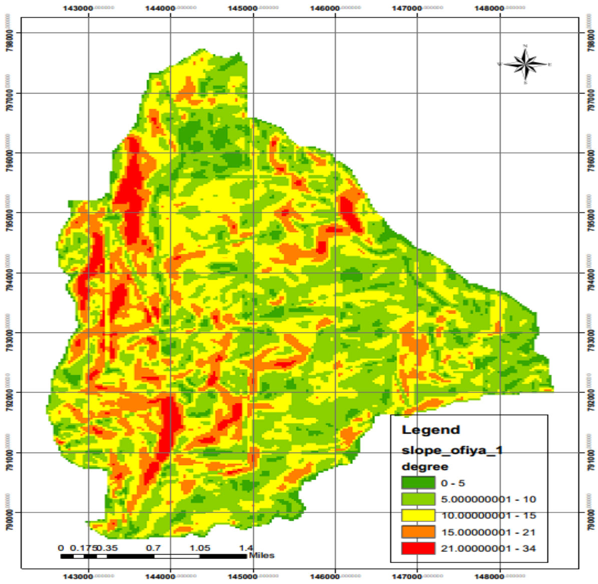


Figure 3. 6:-Slope variation of the study area

Table 3. 3:-Areal percentage of slope variation

Slope	Land form	area km2	areal percentage
0-5%	Gently sloping,	2.5	8.796622
5-10%	undulating plain	10.72	37.71992
10-15%	Rolling plain	9.7	34.13089
15-21%	Hilly plains	4.2	14.77833
21-34%	Steep hilly, very steep slopes, ridges & mountains	1.3	4.574243

The Offiya watershed with comprises of mountainous, undulating to rolling landscape conditions, forest clearance for expansion of agricultural land for cultivation supplemented with monoculture farming practices and sadden heavy and intensive rainfall occurrence enhanced mainly the cultivated land for venerability of soil erosion and forest deterioration risks.

3.6.4 Catchment Land Use Land Cover and Soil

Land use land cover was the main factor for increasing and decreasing of the hydrology of the river; the natural futures (land cover) and human made futures (land use) play an important role in the runoff process (Hora, 2016).

Therefore the land use/land cover of the study area where analyzed by using ARC GIS from African map 2020 that taken from satellite.

Table 3. 4:-Land use land cover classification of the catchment.

S/N	LULC Type	Area Km2	Areal %
1	Tree/forest	1.07	3.764954
2	Grass Land	0.08	0.281492
3	Agricultural Land	24.87	87.5088
4	Scrub/Shrub	1.6	5.629838
5	Village Or Built Area	0.8	2.814919
Total Area		28.42	100

Generally the watershed of Offiya divided into five land use/cover, which express the tree/forest, grass land or grazing, agricultural/crop & village/built area. According to the table below 87.3% of the watershed of the study area was poorly cultivated agricultural land this show that most of rain fall covert to direct runoff rather than forming base flow. The area about 5.6% was covered by scrubs/shrub which was open area most of the feature are short desert tree scattered on the space. The dense tree area was about 3.76% of the total catchment natural mountains tree were densely shown at these area of the catchment. The forth wide area was covered by village area building these area was usually considered as impervious area which cause for increase of surface flood. The area of catchment about 0.28% was covered by grass land this area was considered as having high infiltration rate.

3.6.5 Drainage Pattern of Offiya Catchment

In ARC GIS terrain preparation DEM 30*30 resolution method was used to identify the surfa ce drainage pattern. Once preprocessed, the DEM and its derivatives can be used for efficient watershed delineation and stream network (Hora, 2016).

By taking the weir site as outlet point the streamline of Offiya River was extracted from stream order. The longest stream line for calculating of time of concentration was analyzed from process of arc hydrology.

After the watershed delineation the area of the watershed and determination of longest stream of the catchment to the outlet of drainage area, the average slope of main water course or longest stream and time of concentration of each segment of stream course with uniform slope were computed and by sum upping the time of concentration at weir site was obtained.

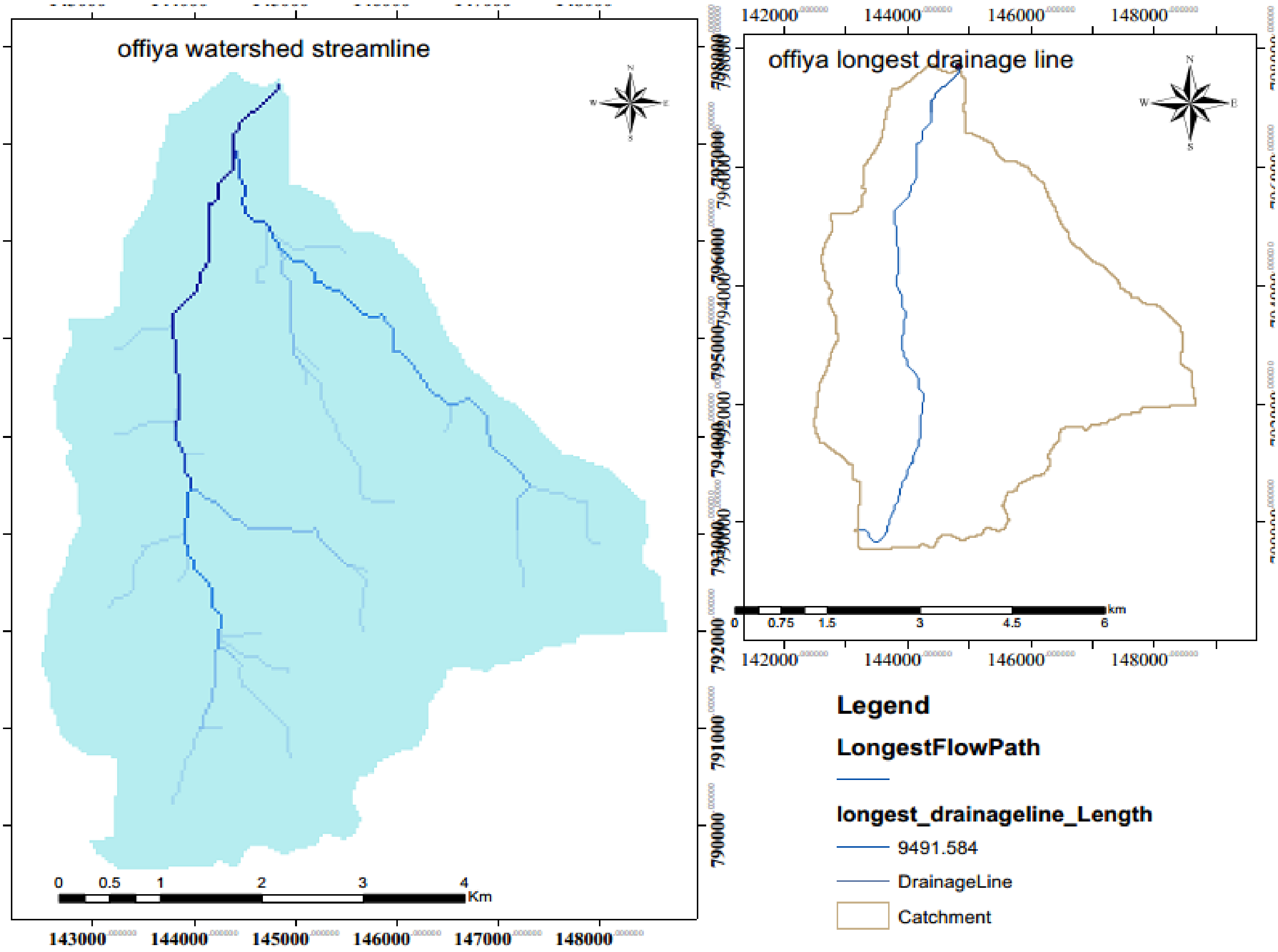


Figure 3. 7:-Stream line and longest stream of the catchment.

3.7 Time of concentration

Time of concentration (T_C) has been calculated by taking the stream profile of the longest streamline and dividing it in to different elevation. Kirpich formula is adopted for computation(ADSWE, 2010).

$$\mathbf{T_c} = \sum\left\{\mathbf{0.948} * \left(\frac{\mathbf{L_1}^3}{\mathbf{H_1}}\right)^{\mathbf{0.385}} + \left(\frac{\mathbf{L_2}^3}{\mathbf{H_{12}}}\right)^{\mathbf{0.385}} \ldots\ldots\ldots\ldots. + \left(\frac{\mathbf{L_n}^3}{\mathbf{H_{1n}}}\right)^{\mathbf{0.385}}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.9a$$

Time of peak runoff on the basis of large number of small rural watershed SCS found that(Subramanya, 2008):

$$\mathbf{T_p} = \frac{\mathbf{t_r}}{\mathbf{2}} + \mathbf{0.6T_c} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.9b$$

Tb = 2.67Tp…... *3.9c*

Where t_r is duration of the effective rain fall

Tb is base time of effective rain fall

Table 3. 5:-Time of concentration of the for Offiya weir site.

OID	LENGTH	Z_MIN	Z_MAX	SLOPE	Tc
1	1589.447	1433.769	1541.838	0.067992	0.266853
2	617.141	2134.707	2203	0.110661	0.106774
3	885.1018	2031.406	2134.707	0.116711	0.138088
4	842.9957	1929.846	2031.406	0.120474	0.131386
5	620.1334	1837.464	1929.846	0.148972	0.095582
6	999.087	1729.797	1837.464	0.107765	0.156315
7	1145.838	1630.629	1729.797	0.086546	0.189013
8	1264.932	1537.545	1630.629	0.073588	0.21711
9	1526.908	1341	1433.769	0.060756	0.270186
Total	9491.584				1.571307

The time of concentration is around 1.57hours. Therefore, the time increment was $T_c/6=1.57/6=0.26$hour. Hence, the time increment can be taken as 0.3hours. The peak time of the rainfall runoff according to equation (3.9a) was 1.092hr and the base time of rainfall runoff was based on equation (3.9b) was 2.91564hr. Therefore according to these result the time of peak rain fall runoff at weir site was fast.

3.8 Soil Curve Number

In the SCS-CN method of estimating runoff from rainfall the curve number plays an important role. This variable is influence by land use land cover, treatment of land i.e., management of cultivated agricultural lands, hydrologic conditions, the effect of cover type and treatment on infiltration, runoff and hydrologic soil group (Subramanya, 2008).

The runoff Curve Number (CN) is developed through field experiments by measuring runoff from different soil at multiple sites. To create hydrologic soil groups, the antecedent moisture condition and the physical characteristics of the watershed are correlated.

The soil of any watershed can be classified into the following four hydrologic groups:

I. Group – A
II. Group – B
III. Group – C
IV. Group – D

Table 3. 6:-Soil group and its descriptions (Subramanya, 2008)

Soil Group	Descriptions
Group –A	A low runoff potential with high infiltration rate from such soils the runoff expectations are low infiltration rate 8-12mm/hr. transmissions rate is high.
Group –B	Moderately low runoff potential soil group, with moderate rate of water transmission. Soil textures vary from fine to moderately course and Infiltration rate 4-8mm/hr
Group –C	Moderately high runoff potential with low infiltration rates, with moderately fine to moderately course. With slow rate of water transmission final rate of infiltration 1-4mm/hr
Group –D	Vary slow infiltration rate when the thoroughly wet clay soils from such Groups the final infiltration rate for soil 0-1mm/hr varies. Low rate of Transmission.

Source: Engineering hydrology books

Curve number: Curve numbers from the Soil Conservation Service are dimensionless numbers that indicate a basin's runoff potential. The curve number method was created using 24-hour rainfall-runoff data. It is based on the properties of the catchment:

a) Land use and Land cover
b) Hydrologic soil group.
c) Antecedent moisture conditions
d) Ground surface condition.

Antecedent Soil Moisture Condition:-The volume and rate of runoff are both known to be influenced by antecedent soil moisture. SCS established three antecedent soil moisture conditions, labeled I, II, and III, after recognizing that it is a major factor. The following is the soil condition for each condition:

Condition I: Soils are dry but not to wilting point; satisfactory cultivation has taken place

Condition II: Average conditions

Condition III: Heavy rainfall, or light rainfall and low temperatures have occurred within the last five days; saturated soil(Subramanya, 2008).

Estimation of hydrologic soil cover complex number was made from the top map and during field study to the watershed and the estimated wet antecedent moisture condition III Based on most of daily rain fall amount >28mm for last previous five days for dormant season and >53mm for growing season

Table 3. 7 :-AMC for determining the value of CN

AMC type	Total rain fall in previous 5 days	
	Dormant season	Growing season
I	Less than 13mm	Less than 36mm
II	13 to 28mm	36 to53mm
III	More than 28mm	More than 53mm

Source: (Subramanya, 2008)

Land use land cover: - the variation of CN under AMC-II called CN-II, for various land use land cover conditions(Subramanya, 2008). The study area river catchment is categorized as cultivated and uncultivated rural area. The value of runoff curve number annexed on appendix table 1

The conversion of CNII to other two AMC conditions can be made through the of correlation equation (Subramanya, 2008)

According to the above table (3.7), description the study area rain fall was more than 28mm during dormant season and 53mm for growing season; therefore the moisture condition was AMC III whereas the equation for AMC III was given as following equation.

For AMC-III $\mathbf{CNIII} = \frac{\mathbf{CNII}}{\mathbf{0.427+0.00573CNII}}$ ……………………………………………..*3.10a*

The potential of maximum retention (S) depends up on the land use land cover condition attendance moisture contents in the catchment just prior to the commence of the rainfall event for convenience in practical application the soil conservation services (SCS) of USA has been expressed S(in mm) in terms dimensionless parameter CN as follow (Subramanya, 2008)

$\mathbf{S} = \frac{\mathbf{25400}}{\mathbf{CN}} - \mathbf{254}$ …………………………………………………………………*3.10b*

Based on soil map using Arc GIS the type of soil in the catchment grouped in to three soil groups these are soil group “C” which cover 36.74% of the catchment area soil group “D” which cover 38.97% of the catchment and soil group “B” also cover 24.28% of the study area catchment. After determination of LULC of each soil type area covered the value of soil curve number (CN II) was assumed by using curve number in appendix B table 1 provided. Then after for AMCIII the soil curve number (CN III) form weighted soil curve number (CNII) was calculated as equation (3.10a) and tabulated in appendix (B) table (2).

3.9 Offiya River Sub Catchment Characteristics for HEC-HMS Soft Ware

By using digital elevation model (DEM) of the Offiya catchment as input, the terrain preprocessing with series step performed in Arc Hydrology to derive the drainage networks. These step consist of computing the fill sinks, flow direction, flow accumulation, stream definition, stream segmentations, watershed delineation, watershed polygon processing, drainage line processing and final point delineation was applied to delineate sub-watershed polygons used for back ground of the HEC-HMS modeling tools and to identify characteristic's and length of reach's and sub catchment area and its weighted curve number and initial abstraction, time of concentration and other necessary parameters used as input of HEC-HMS modeling.

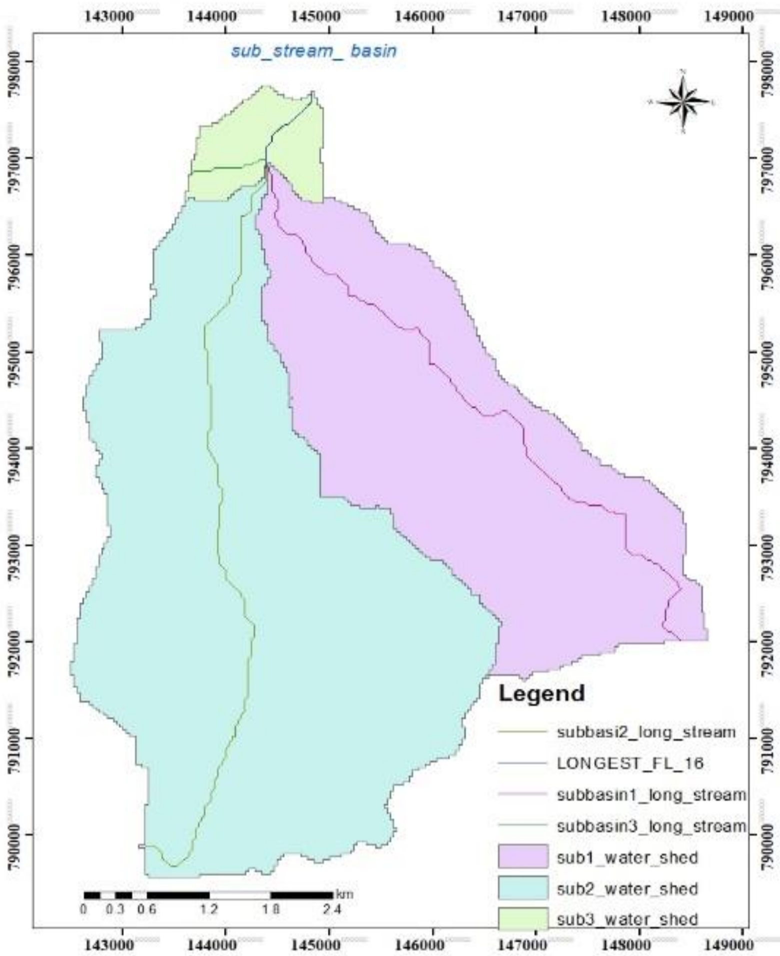


Figure 3. 8:-The longest stream of each sub catchments

The length sub catchment longest stream to each outlet, time of concentration, area and lag time to peak flood at outlet point of each sub catchment were by using Arc Hydrology terrain process calculated and tabulated on appendix (B) table(3).

Table 3. 8:- Sub catchment properties for HEC-HMS Modeling

Sub_ Catchment	Soil Group	% Soil Group	CNI	Product	Weighted (CNII)	CNIII	S	Ia= 0.2 *S	%Of Impervious
1	C	51.55	90	4640.0	87.786	94.39	15.09	3.0	3.243
	B	45.84	85	3896.4					
	D	2.60	93	242.16					
2	C	29.98	90	2698.5	91.611	96.24	9.931	2.0	2.75
	B	6.11	85	519.53					
	D	63.90	93	5943.0					
3	B	100	77	7700	77	88.68	32.39	6.5	0.425

By using ARC GIS the above table parameter were calculated from Ethiopian soil map and African LULC 2020 map. The percentage of village and built area were taken as a percent of impervious sub catchment used input data of HEC-HMS. The storage of the sub catchment were calculated by using equation *3.10b* given in section three.

3.10 Rain Fall Data Analysis

Rainfall is the main source of water for river flood discharge in the area of high rainfall region. There are four nearer station to the study area. From those two most nearest station according Thiessen polygon using GIS are selected for Consistence test to calculate the design rainfall for estimation of river hydrology.

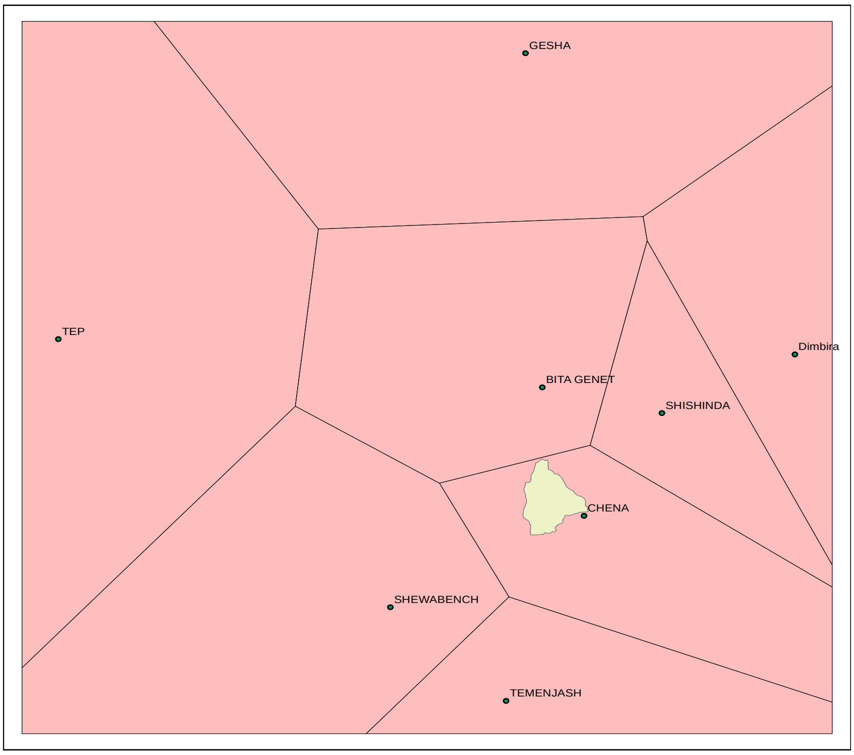


Figure 3. 9:- Thiessen polygon for nearest rainfall station

Table 3. 9:- List of rainfall gauge for study

S.N	Name of station	Latitude (x)	Longitude (y)	Altitude (Mekuria)	Data range	Remark
1	Chena	35.81667	7.15	2203	1985-2018	In study area
2	Shishinda	35.88333	7.25	2000	1985-2019	Near the study area

3.10.1 Filling Missing Rainfall Data

The rainfall data missed from Chena station was about 11.9% whereas the Shishinda missed rainfall data was 10.47% from 35 year rainfall records.

The missed rainfall data for Chena and Shishinda station are filled using Xlstat 2019 software which integrated with Microsoft excel 2016.

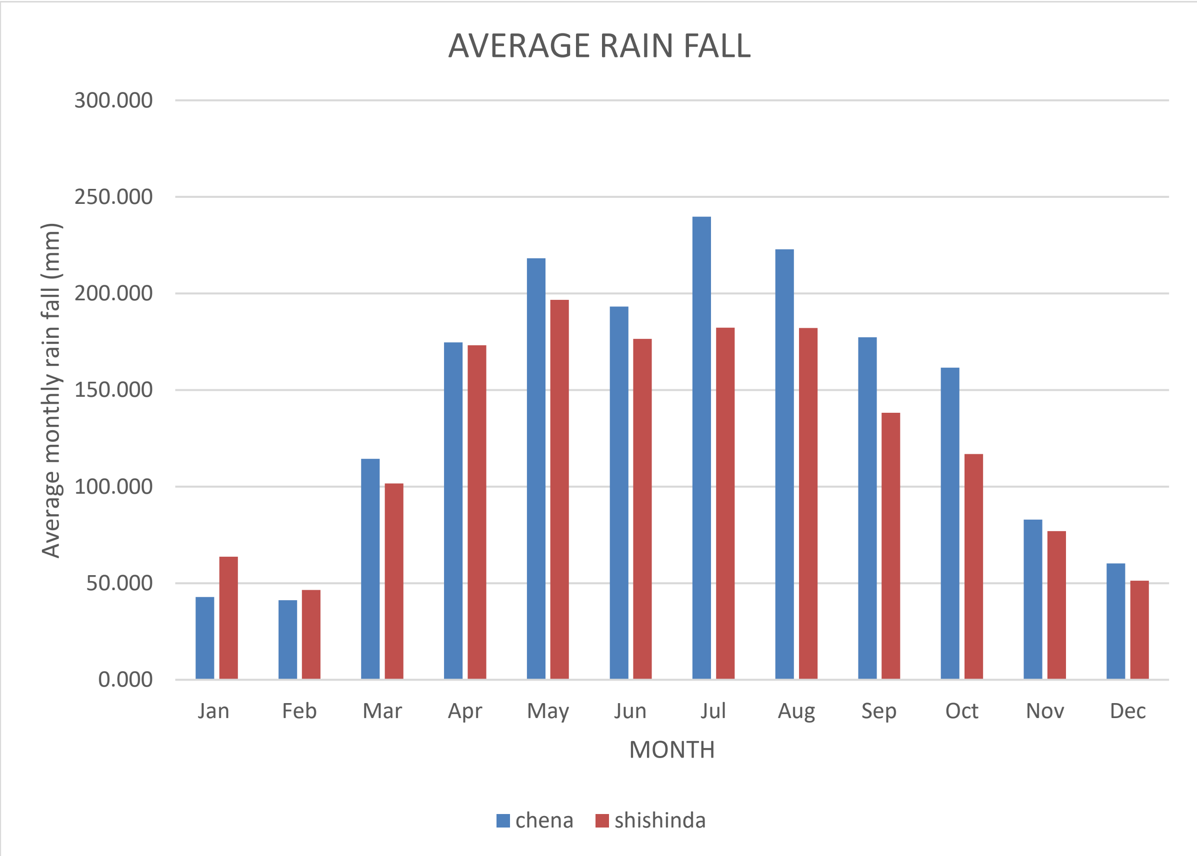


Figure 3. 10:- Average monthly rainfall

3.10.2 Consistency testing of rainfall data

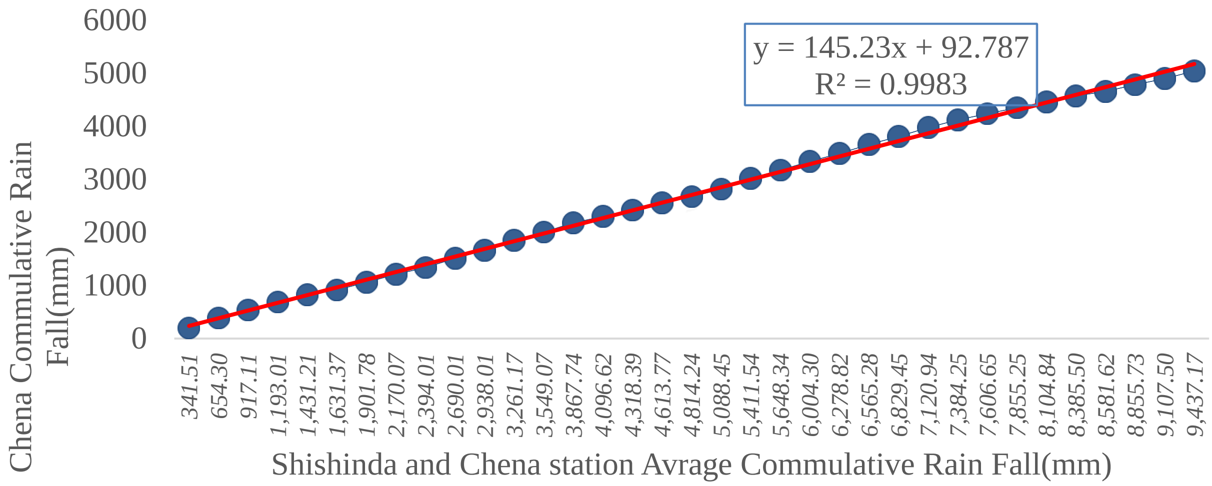


Figure 3. 11:-Mass curves of Chena rainfall stations

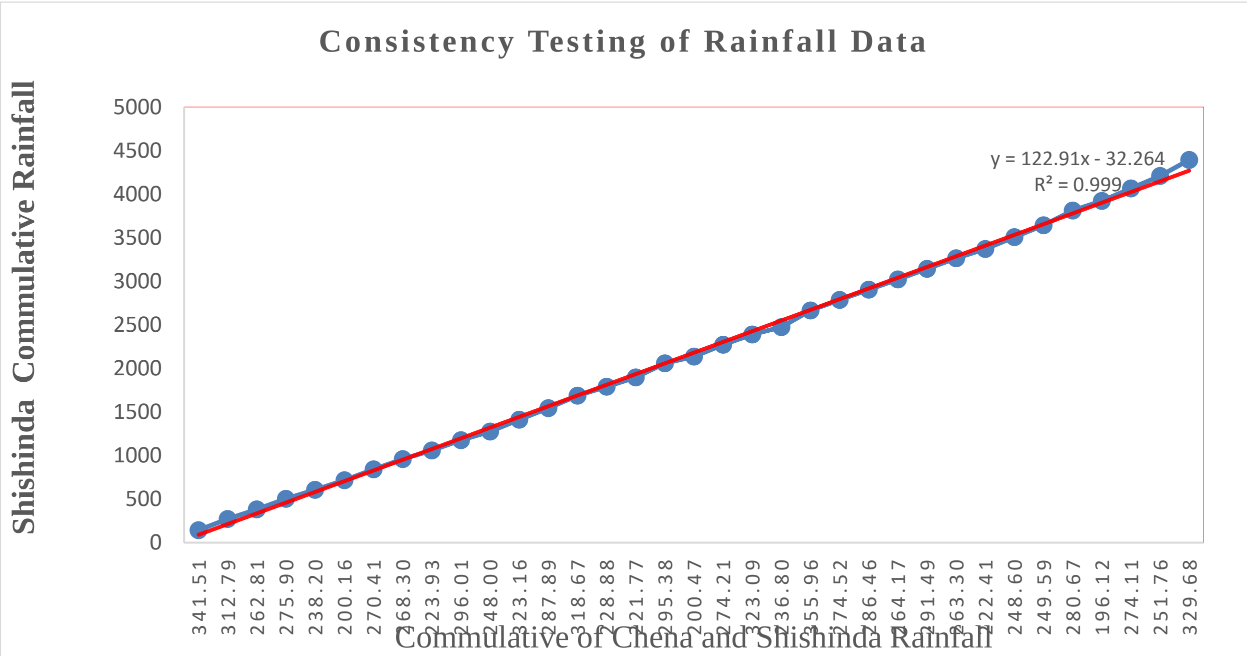


Figure 3. 12:- Mass curves of Shishinda rainfall stations

The mass curve shows that there are a good direct correlation between the averages cumulative of two nearest station with individual cumulative rain fall. As shown in Figure 3.11 and 3.12, the mass curve analysis revealed a solid direct link between cumulative rainfall records at Chena and Shishinda rain gauge station with average cumulative rain fall of two station. This shows that both the rainfall data at the Chena and Shishinda stations were reliable. As a result, there was no significant change in the slope in their separate plots, and the correlation coefficient of the two stations indicated. Where Chena station was more nearest station to the study area; therefore Chena rain gage station rain fall data was selected to analyze the design rain fall of this study.

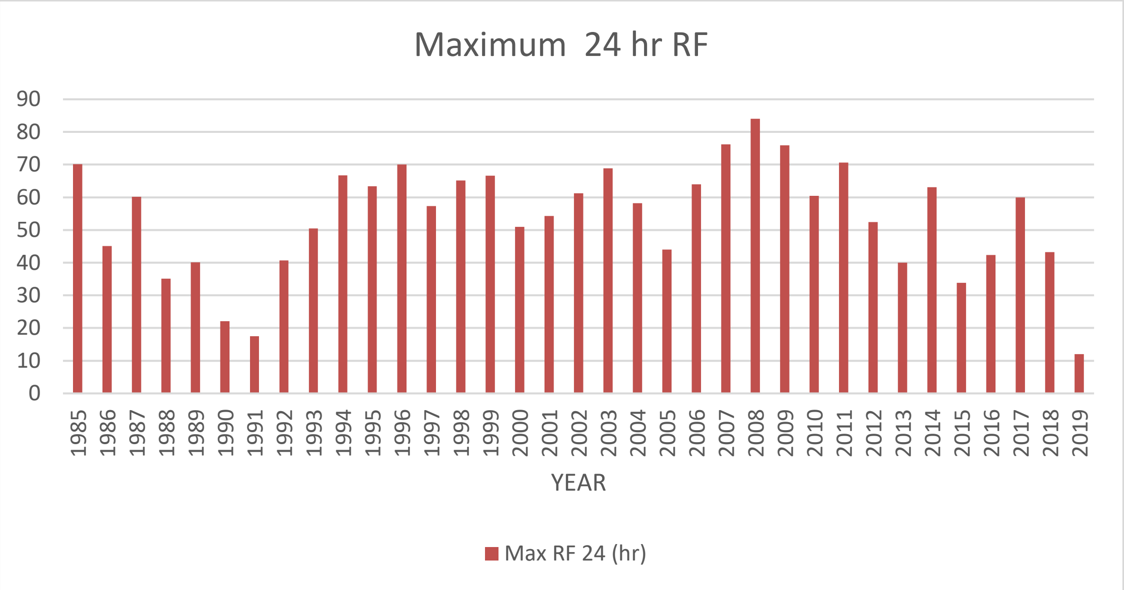


Figure 3. 13:- Daily maximum rainfall within each year of Chena station

The annual maximum rainfall data series of thirty five (35) years from Chena station were used to predict the possible design flood magnitude of future 10, 25, 50, and 100 years. The analysis result were tabulated as table 4.3 shown; using normal, Gumbel's extreme value, Log normal and Log-person type III distribution method. The annual maximum 24 hours rainfall data used to estimate the design rainfall was tabulated in appendix (A) table (1)

3.10.3 Maximum Rain Fall Frequency Analysis

The rainfall data has been analyzed by using different statistical methods such as Gumbel extreme value distribution method, Log-normal distribution method and Log-person type III distribution method to determine peak rain fall (ADSWE, 2010)

The one with maximum output was taken to analyze the peak flood of the river.

1) Gumbel extreme value distribution method

It is a widely used probability distribution function for extreme values in hydrologic and metrological research for peak flood, maximum rainfall, maximum wind speeds, and other applications. Gumbel (1941) introduced this extreme value distribution, which is also known as the Gumbel distribution and has the following equation(Subramanya, 2008).

$$\mathbf{X_T = \bar{X} + K * \sigma_{n-1}} \quad \text{...3.12.3a}$$

$$\mathbf{K = \frac{(y_T - \bar{y}_n)}{s_n}} \quad \text{...3.12.3b}$$

$$\mathbf{y_T = -\left[\ln.\ln\frac{T}{T-1}\right]} \quad \text{...3.12.3c}$$

$$\sigma_{n-1} = \sqrt{\frac{\sum(x-\bar{x})^2}{N-1}} \quad \text{……3.12.3d}$$

Where: X_T is annual maximum value of varieties for return period T, $\bar{X}$ is mean x value, $\sigma_{(n-1)}$ is standard deviation of sample size N, K is Frequency factor. Y_T is reduced varieties, $\ddot{y}_n$ is reduced mean and s_n = reduced standard deviation are given

2) Log-person type III distribution

This distribution is widely used in the United States for government-sponsored projects. The data on rainfall is first converted into logarithmic form (base ten) and then analyzed using this method. If X is a random rain fall series variety, then Z is a series of Z varieties, where Z is obtained for any recurrence interval T. Where K_Z is the frequency factor which is function of recurrence interval T and Cs (Subramanya, 2008)

$$z = \log x \quad \text{……3.12.3e}$$

$$Z_T = \bar{Z} + K * \sigma_{z-1} \quad \text{……3.12.3f}$$

$$\sigma_{z-1} = \sqrt{\frac{\sum(z-\bar{z})^2}{N-1}} \quad \text{……3.12.3g}$$

$$C_s = \frac{N\sum(Z-\bar{Z})^3}{(N-1)(N-Z)(\sigma_z)^3} \quad \text{……3.12.3h}$$

3) Log normal distribution

Long normal distribution method is special type of log-person type III distribution method with Cs is zero(Subramanya, 2008).

Table 3. 10:- Summer of Design Rain Fall Analysis Result.

Station name	Method of analysis	Expected design rain fall			
		10	25	50	100
Chena	Normal Distribution	81.69266	101.0758	114.6552	127.6423
	Gumbel’s Extreme Value Distribution	88.96181	118.5984	140.5846	162.4084
	Log Normal	86.09807	104.7762	118.9464	133.2919
	Log- Person Type III Distribution	74.65745	77.47116	78.55612	79.13507

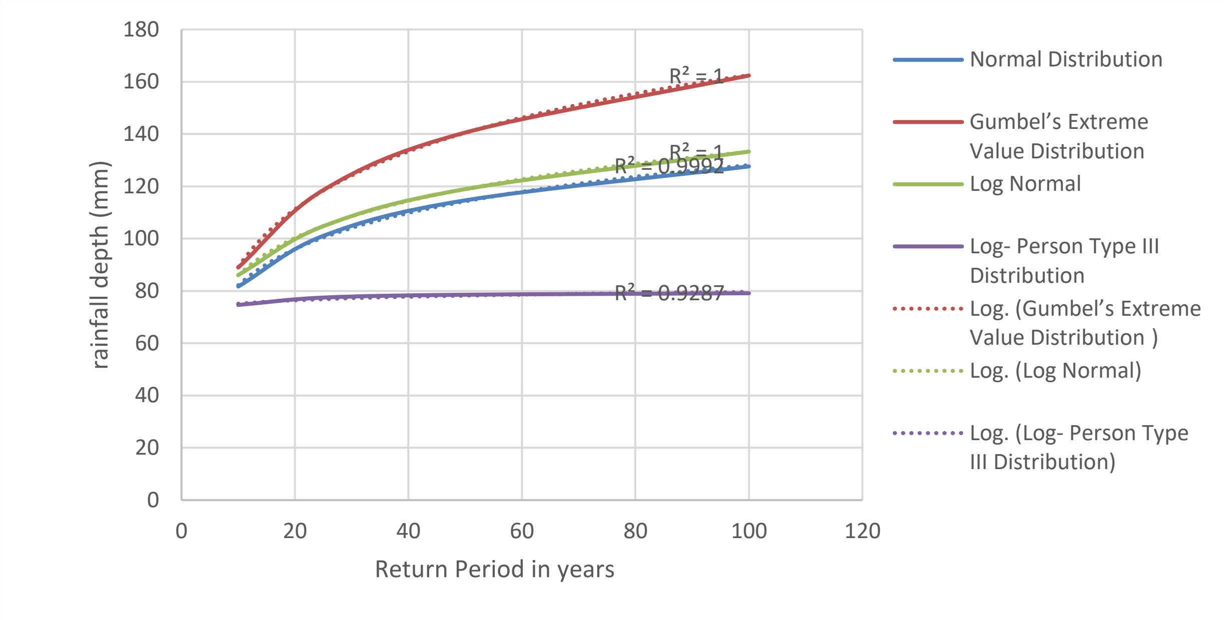


Figure 3. 14:-fitting test of distribution method

According to (Asawa, 2005), the return period for pick up weir was recommended 50-100 years. According to these recommendation the return period for small diversion weir was selected 50 year; therefore 50 year return period design rain fall was selected for this study.

Table 3. 11:- 50 Years Expected Rain Fall.

Station name	**Method of analysis**	**50 year return period design rain fall (mm)**
Chena	Normal Distribution	114.6552
	Gumbel's Extreme Value Distribution	140.5846
	Log Normal	118.9464
	Log- Person Type III Distribution	78.55612

3.12.4 Goodness of Fit Test

The fitted probability distribution for the available sample data. The fitting of the probability distribution can be evaluated with three most commonly used GOF tests. (Kolmogorov Smir ov, Anderson Darling, Chi Squared) using Easy Fit Excel add in or moment diagram as graph ical approach(MOA, 2018).

In all three tests a parameter or statistic unique to each method is calculated for the required distribution types and these distributions are ranked based on their parameter values using easy fit test 5.5 excel software.

Table 3. 12:-Good fit test easy fit result

Distribution methods	Kolmogorov Smirnov Test		Anderson-Darling Test		Chi-Squared Test	
	Statics	Rank	Statics	Rank	Statics	Rank
Normal Distribution	0.07765	3	0.22779	3	2.0985	1
Gumbel's Extreme Value Distribution	0.0751	2	0.21075	1	3.7924	3
Log Normal	0.0877	4	0.2928	4	2.6923	2
Log- Person Type III Distribution	0.07005	1	0.2626	2	3.7924	4

According to above table (3.12), and graph figure (3.14) the most reliable distribution method result for this study was Gumbel's extreme value distribution. According to above table (3.11) the Gumbel's Extreme Value Distribution analysis method result was 140.5846mm for the 50 year return period design rainfall which used to calculate areal design rainfall for hydrological analysis of Offiya small scale irrigation diversion weir.

3.13 Design Arial Rainfall

The point rainfall data can't be taken as design rainfall data because as the area of catchment gets larger and larger, coincidence of all hydrological incidences generally becomes less and less. This may be taken care of by introducing an aerial reduction factor (ARF). ARF is estimated by the equation:

Arial design rainfall = Design point rainfall*ARF……………………………………….*3.13a*

Where: ARF- Area reduction factor

$$ARF = 1 - 0.044 * A^{0.275} \quad \text{……………………………………………} \mathit{3.13b}$$

A=is the area of the Catchment.

Arial reduction factor is calculated by equation (3.13b) is 0.9; Hence the design rainfall over the catchment area by using equation (3.13a) was calculated was 126.5262mm.

As a result, the 50-year, 24-hour rainfall (126.5262mm) will be converted to incremental rainfall using the alternate block approach as shown in appendix (A) table (6). Therefore incremental depth of precipitation will be arranged sequentially using alternate block method, which used for HEC-HMS for peak flood estimation was tabulated in appendix (A) table (6).

3.14 Hydrological Analysis

The problem of computation design flood for structure was carried out using following methods for ungagged catchments (Subramanya, 2008)

i) Rational Approach

ii) Empirical Equation

iii) SCS Curve Number method

The SCS curve method unit hydrograph analysis, Empirical Equation of peak flood mark analysis method using Bentley flow master software and HEC-HMS software would be used in determination of peak direct discharge of the river in this study and the maximum analysis result of them was used to design analysis of the offiya diversion weir.

3.14.1 SCS Curve Number Method

The SCS technique, developed in 1969 by the United States' Soil Conservation Service (SCS), is a practical conceptual method for estimating direct runoff depth from storm rainfall. It is only dependent on one parameter, CN. It is now a well-established approach that is widely used in the United States and other countries(Subramanya, 2008).

Direct runoff the catchment is given by

$$\mathbf{Q_d} = \frac{(\mathbf{P}-\mathbf{I_a})^2}{\mathbf{P}-\mathbf{I_a}+\mathbf{S}} \qquad \textit{3.14.1a}$$

Where Q_d is daily runoff, p is daily rainfall, S is retention parameter and Ia is initial abstraction. The relationship between Ia and S was developed from experimental data is given as Ia = 0.2S; Therefore, Equation 3.14.1a is given as Equation 3.14.1b below.

$$\mathbf{Q_d} = \frac{(\mathbf{P}-\mathbf{0.2S})^2}{\mathbf{P}-\mathbf{0.8S}} \qquad \textit{3.14.1b}$$

Peak discharge:-the SCS unit hydrograph is a popular method in watershed development activities, especially in small watersheds(Subramanya, 2008)

$$\mathbf{Q_P} = \frac{(\mathbf{0.208} * \mathbf{Q}_{Icr} * \mathbf{A})}{T_p} \qquad \textit{3.14.1c}$$

3.14.2 Peak Flood Mark Analysis Method

During field investigation; the peak flood mark was defined based on information of the elders living around river. The flood during high rain season reached up to maximum elevation 1327.99m above mean sea-level that was measured at downstream of weir. Bentley flow master software by applying manning's equation the peak flood was computed.

To collect data from all the previous flood marks the river slope S is computed from knowledge of two flood marks at distance L apart along river and the area of river cross-section can be surveyed corresponding to the river stage of the flood marks. A suitable roughness coefficient “n” was selected (MOA, 2018).

$$Q = \frac{1}{n} * A * R^{\frac{2}{3}} * S^{\frac{1}{2}} \qquad \text{3.14.2a}$$

Where n is manning roughness coefficient, A is cross-sectional area, S is average longitudinal slope.

3.14.3 HECS-HMS Software

The United States Army Corps of Engineers; Hydrologic Engineering Center–Hydrologic Modeling System (HEC-HMS) is a watershed-scale open access hydrologic model developed by the Hydrologic Engineering Center (HEC). HEC-HMS, like many physically-based hydrologic models, simulates most important hydrologic processes in great detail, including runoff transformation, open channel routing, meteorological data processing, rainfall-runoff modeling, and parameterization estimate (Abdul Kareem, 2018).

SCS curve number method was used to estimate the magnitude of peak flood in HEC-HMS. The maximum rainfall data was used for the hydrologic analysis to determine the maximum design discharge and checking the consistency of the structures constructed for the design period. But before estimating the peak flood the model has to be optimized (Tadesse, 2016).

The parameters used for modeling of HEC-HMS were peak rain fall, soil curve number, time of concentration, length of reach, areas of sub-catchment and other parameter of ungagged river catchment such as initial abstraction, Muskingum and x value.

3.15 Hydraulic Analysis

The hydraulic analysis includes surface and subsurface hydraulic analysis to determining the weir shape and height, as well as its clear waterway, discharge over the weir, head over the weir, circumference, food and energy levels, and afflux, and scour depth and seepage flow characteristic’s; to design a weir, all external forces acting on weir were calculated (Ertiro, 2017).

Water flow length or water way is calculated from Lacey’s perimeter formula.

$$\mathbf{L = 4.75\sqrt{Q_d}} \qquad \text{3.8.3a}$$

Where; L is length of water way in meter and Q_d is design discharge in m^3/s

Scouring depth of the flow (R)

$$\mathbf{R = 1.35\left(\frac{q^2}{f}\right)^{1/3}} \qquad \text{3.8.3b}$$

Where; q is the discharge per unit width of the river, *f* is the silt factor.

Table 3. 13:-Silt Factor of River Material

Type of Reach	Mean value of "f"
Coarse gravel	4.75
Coarse bajri and sand	2.75
Heavy sand	2.0
Fine bajri and sand	1.75
Coarse sand	1.5
Medium sand	1.25
Standard silt	1.0

Source: - National guide line for small scale irrigation project.

3.16 Structural Analysis

All external forces acting on a weir are caused by the flow of water in the river on which it was built. The structural stability of the weir body and retaining wall of the weir was evaluated against overturning, sliding stress, and uplift pressure or pore water pressure after the dimension of the head work structure is known. The following components make up a normal weir force system

1. Water pressure that remains constant
2. The weir's and water wedges' weight
3. At the base, there is a sliding force.
4. Silent force

For both static and dynamic conditions, a stability analysis of the guide wall and weir body was performed(Ertiro, 2017).

i) overturning

When the overturning moment exceeds the resisting moment, overturning failure happens. As a result, overturning failure the weir is normally preceded by tension or crushing failure. If no tension at any point in the weir body or retaining wall it meet the maximum compressive stress does not exceed the permissible limit, a weir may be considered safe against overturning.

The factor of safety against overturning is one of the measures for stability of the structure.

$$\mathbf{F_O = \frac{\sum M_r}{\sum M_O} \geq 1.5} \quad \text{..........} \quad 3.16a$$

Where; F_o is factor of safety against overturning, M_r is the resisting moment and M_o is disturbing or overturning moment(Garg, 2011)

ii) sliding

When the weir and retaining wall lied’s over its base or when part of the weir above the horizontal plane slides over that plane, the sliding failure happens. The weir should be engineered so that the sliding forces do not surpass the resisting force at any horizontal section or at the base to prevent this failure(Ertiro, 2017)

The safety factor against the sliding is given as

$$\mathbf{F_s} = \frac{\mu \sum V}{\sum H} \geq \mathbf{1.5} \quad \text{......} 3.16b$$

Where, μ is sliding coefficient usually varies from 0.6 to0.75(Garg, 2011)

V is vertical force on the structure

H is horizontal force on the structure

iii) Over-stress;

$$e = \left|\frac{e}{2} - \mathbf{X_{avr}}\right| \leq \frac{B}{6} \quad \text{......} 3.16c$$

where; e is Eccentricity developed X_{avr};=(∑M)/(∑V),B is base width of weir or retaining wall(Ertiro, 2017).

3.17 Seepage Analysis

The theory used to determine the hydraulic gradient of the seepage under the weir foundation, the creep length of the seepage is determined by Bligh's creep, and Lane's weighted creep theory and Khosla's theory.

Khoslas's theory is chosen to determine creep length in order to check the exit gradient at the end of downstream apron.

The exit gradient for given floor length, b with vertical cut of depth d is given as

$$\mathbf{G_E} = \frac{H}{d} * \frac{1}{\pi\sqrt{\lambda}} \quad \text{......} 3.17a$$

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2} \quad \text{......} 3.17b$$

$$\boldsymbol{\alpha} = \frac{b}{d} \quad \text{......} 3.17c$$

The thickness of the impervious floor was determined by uplift pressures, which can be calculated using the method suggested by Khosla's for various stages of flow, the thickness of the floor in the jump through should be explored. However, for other portions of the downstream floor, the maximum uplift would occur when the water level on the upstream side was at pond level and there was no tail-water (or flow) on the downstream side (Asawa, 2005)

Table 3. 14:- The safe exit gradient based on soil type.

Soil type	Safe exit gradient
Shingle	¼ to 1/5 (0.25 to0.2)
Coarse sand	1/5 to 1/6 (0.2 to0.17)
Fine sand	1/6 10 1/7 (0.17 to 0.14)

Source: (Garg, 2011)

The critical exit gradient (i_i) of the weir foundation was calculated as the following equation (Khassaf, 2009)

$$icr = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} \quad \text{......} 3.17d$$

The factor of safety against uplift pressure due to piping of water through weir foundation was given as follow (Khassaf, 2009)

$$F_S = \frac{i_{cr}}{GE} \quad \text{......} 3.17e$$

Where the value of Fs is equal or greater than 3 the weir floor was safe against uplift pressure due to piping of water under foundation of weir (MOA, 2018).

The Geostudio2012/2018 version software SEEP/W finite element package has been used to calculate the seepage flow discharge and pore-water pressure distribution under weir foundation soil media for this study.

SEEP/W model is a finite element package that can be used to model the fluid flow and water pressure distribution within porous materials such as soil and rock. Its comprehensive formulation makes it possible to analyze both simple and highly complex seepage problems. The inclusion of unsaturated flow in groundwater modeling is important for obtaining physically realistic analysis results. In soils, the hydraulic permeability and the water content, or water stored, changes as a function of pore-water pressure. The discretization of this model into a finite element mesh is calculated as quadrilateral regions and drawn in the problem domain. Inside each region, any number of finite elements can automatically be generated (Khassaf, 2009).

The steps of this model were used for analyzing seepage discharge, pore water pressure head and total head were identifying input data, identifying the region and bounder condition, computing the analysis result and interpreting the result by contouring table and graphic output methods. The parameter used as input for steady state SEEP/W analyses were water content and conductivity of foundation soil in saturated or unsaturated condition and boundary condition of upstream and downstream water head.

3.18 Study Design

The study has been conducted by using both descriptive and experimental study design. This study was used to know the design practice and performance of constructed diversion weir and recommend the design procedures based on national and international guide line values of design assumption.

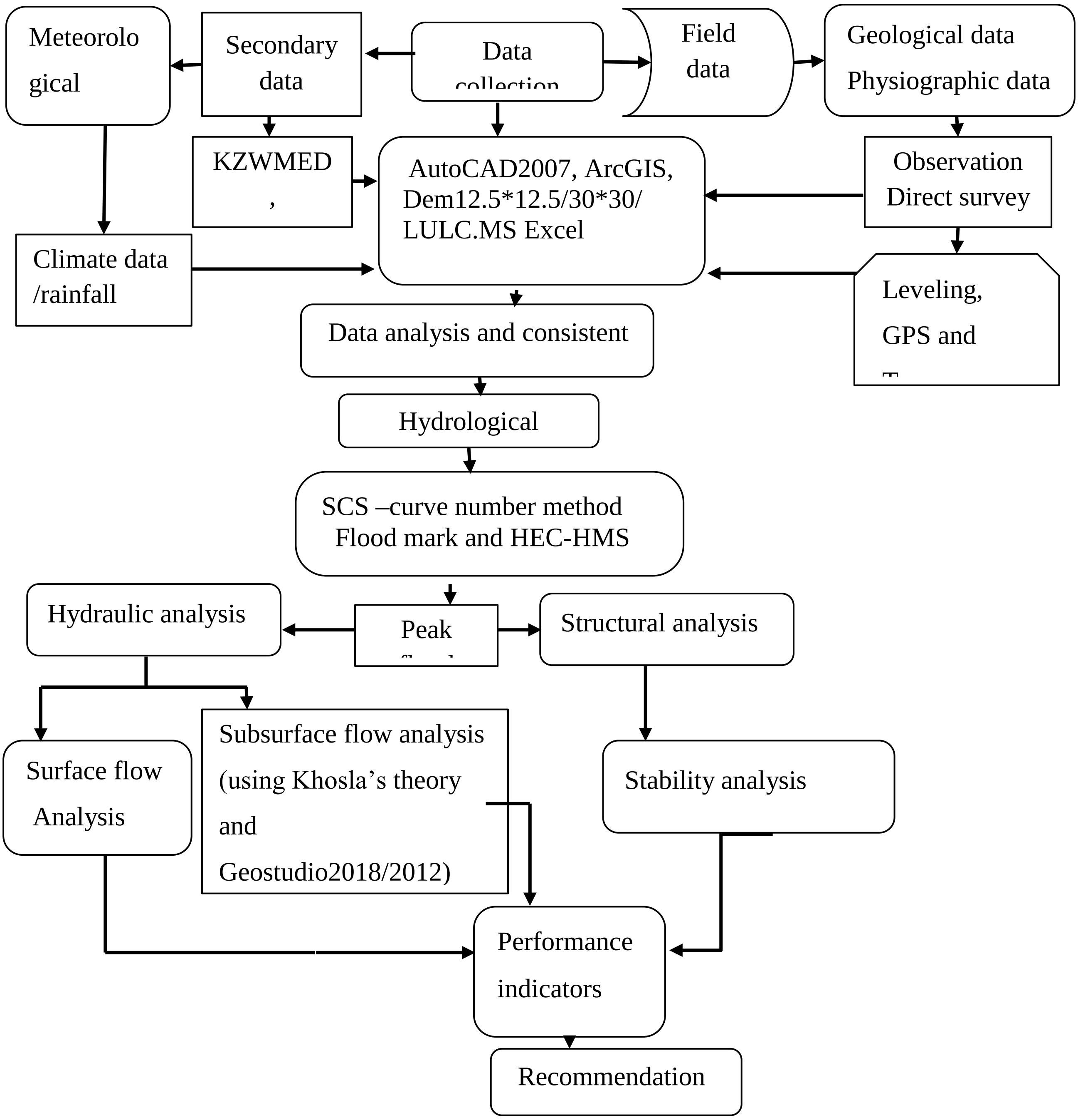


Figure 3. 15:- Study design flow chart

4. RESULTS AND DISCUSSIONS

4.1 Hydrological Analysis

In the design of hydraulic structures the peak flood that can be expected with an assigned frequency, at a given location in the stream, is a primary importance to adequately proportion the structure to accommodate its effect. Estimation of peak run off has paramount importance for design of hydraulic structures and for flood for casting (Garg, 2011).

For this particular case study for the estimation of peak run of SCS curve number method, peak flood mark method and HECHMS software were applied.

4.1.1 Base flow of Offiya River

According to Kaffa zone water and mine energy department report the base flow measured at Jan 12, 2011 G.C by using float were 180l/sec; where the dry season of the study area were between November up to February.

4.1.2 Peak Flood Analysis Using SCS Curve Number Method.

The peak discharge is computed by developing hydrograph using the time conditions and the computed runoff depth (Q) based on the maximum daily rainfall (P) of a given return period. Hydrograph development depends on the catchment area. If the catchment area is less than 10km2; we can approximate it with single triangular hydrograph analysis otherwise has to be computed with multiple triangular hydrographs analysis based on rainfall profile(MOA, 2018).

According to Bansode (2014) SCS CN provides an empirical relationship forestimating initial abstraction and runoff as a function of soil type and land use. Curve Number (CN) was an in dex developed by the Natural Resource Conservation Service (NRCS), to represent the potential for storm water runoff within a drainage area. This method provided only the depth of runoff generated by a given rainfall from the catchment. However, for the design of hydraulic structures it was not the depth of runoff but the peak flow rate was required. Thus the depth of runoff generated from the given rainfall is converted to runoff hydrograph using synthetic unit hydrograph technique. In this technique it is assumed that a rainfall of a certain recurrence interval generates discharge of the same recurrence interval. Rainfall of 50 years return period was first estimated and this rainfall was finally transformed to discharge hydrograph.

According to soil data analyzed above the Offiya watershed catchment; the weighted antecedent moisture condition III of CNIII was 92.2. The Maximum potential difference between rainfall and runoff starting at the storm begin was computed by equation 3.14.1b was 21.48807mm.

According (MOA, 2018), the areal rainfall based on time incremental up to 6D was calculated as the following table. Rain fall profile in the table below where read from hourly rain fall distribution chart.

Table 4. 1:-Design Rainfall arrangement

Duration(Hr)	Design rainfall(mm)	Rainfall Profile%	Areal to point ratio%	Cumulative Areal rain fall (mm)	Incremental Rainfall(mm)	descending order	rearranged rain fall (mm)
0-0.3	140.5846	20	90	25.305228	25.305228	25.305	2.5305
0.3-0.6	140.5846	24	90	30.366273	5.0610456	16.448	5.061
0.6-0.9	140.5846	37	90	46.814671	16.4483982	8.856	16.448
0.9-1.2	140.5846	44	90	55.671501	8.8568298	5.061	25.305
1.2-1.5	140.5846	47	90	59.467285	3.7957842	3.795	8.856
1.5-1.8	140.5846	49	90	61.997808	2.5305228	2.530	3.795
1.8-24	140.5846	100	90	126.52614			

Table 4. 2:- Computation of the Peak for each incremental runoff.

rearranged order No.	rearranged incremental R.F (mm)	cumulative rain fall (mm)	time of incremental hydrograph		
			time of beginning (hr)	time of peak (hr)	time of end (hr)
6	2.5305228	2.5305228	0	1.092	2.91564
4	5.0610456	7.5915684	0.3	1.392	3.21564
2	16.448398	24.0399666	0.6	1.692	3.51564
1	25.305228	49.3451946	0.9	1.992	3.81564
3	8.8568298	58.2020244	1.2	2.292	4.11564
5	3.7957842	61.9978086	1.5	2.592	4.41564

Maximum potential retention (S) between rainfall (P) and direct runoff (Q); Whereas the direct runoff (Q) was computed from cumulative rain fall by using equation 3.14.1b also peak runoff in m^3/s by using 3.14.1c were tabulated in the following table below.

Table 4. 3:- Direct runoff corresponding to incremental rainfall

Cumulative Rain Fall (mm)	Runoff(mm)	Incremental Runoff (mm)	Time Of Incremental Hydrograph			Peak Runoff Qp
			Time of Beginning (Hr)	Time of Peak (Hr)	Time of End (Hr)	
2.530523	0.1561651	0.1561651	0	1.092	2.91564	0.8535
7.591568	0.4428286	0.2866635	0.3	1.392	3.21564	1.22907
24.03997	9.4810438	9.0382152	0.6	1.692	3.51564	31.8805
49.34519	30.546473	21.065429	0.9	1.992	3.81564	63.1138
58.20202	38.592305	8.0458316	1.2	2.292	4.11564	20.9508
61.99781	42.096094	3.5037896	1.5	2.592	4.41564	8.06764

This method provides only the depth of runoff generated by a given rainfall from the catchment. However, for the design of hydraulic structures it is not the depth of runoff but the peak flow rate is required. Thus the depth of runoff generated from the given rainfall was converted to runoff hydrograph using synthetic unit hydrograph technique.

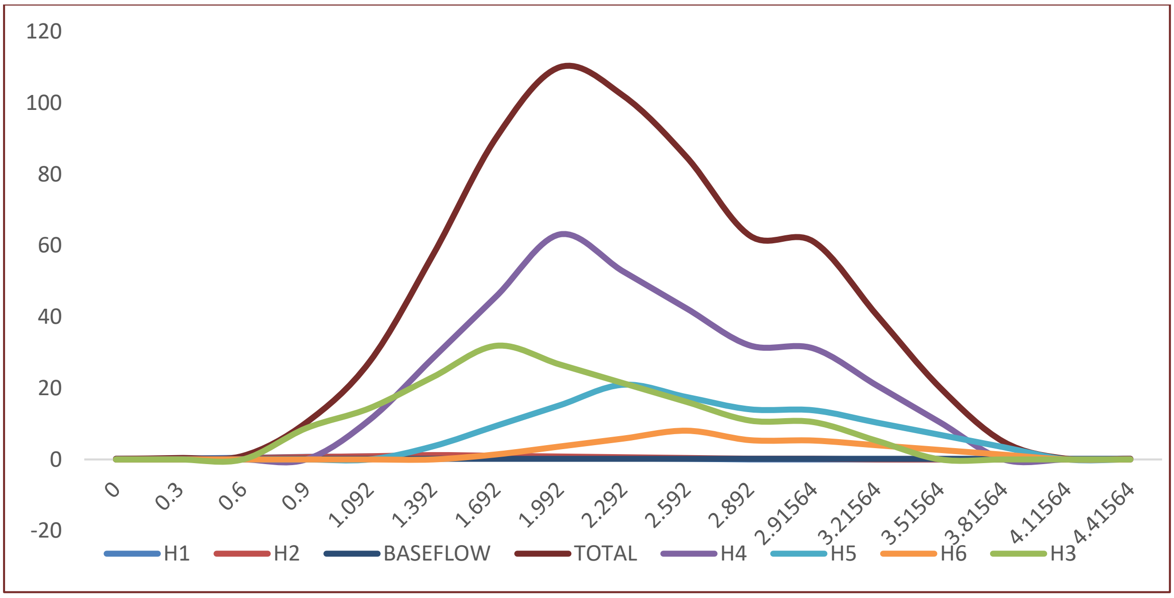


Figure 4. 1:- Runoff hydrograph from synthetic unit hydrograph.

Table 4. 4:- Synthesis of Complex Hydrograph

QP	0.854	1.229	31.881	63.114	20.951	8.068	base flow	TOTAL
incremental time(Hr)	H1	H2	H3	H4	H5	H6		
0.000	0.000						0.180	0.180
0.300	0.234	0.000					0.180	0.414
0.600	0.469	0.338	0.000				0.180	0.987
0.900	0.703	0.675	8.758	0.000			0.180	10.317
1.092	0.854	0.891	14.364	11.097	0.000		0.180	27.386
1.392	0.713	1.229	23.122	28.436	3.684	0.000	0.180	57.364
1.692	0.573	1.027	31.881	45.775	9.439	1.418	0.180	90.293
1.992	0.432	0.825	26.636	63.114	15.195	3.635	0.180	110.017
2.292	0.292	0.623	21.391	52.731	20.951	5.851	0.180	102.019
2.592	0.151	0.420	16.147	42.349	17.504	8.068	0.180	84.819
2.892	0.011	0.218	10.902	31.966	14.058	5.413	0.180	62.749
2.916	0.000	0.202	10.489	31.148	13.786	5.309	0.180	61.114
3.216		0.000	5.245	20.765	10.340	3.982	0.180	40.511
3.516			0.000	10.383	6.893	2.654	0.180	20.110
3.816				0.000	3.447	1.327	0.180	4.954
4.116					0.000	0.000	0.180	0.180
4.416						0.000	0.180	0.180

As shown in the above table analysis, the peak discharge is 110.0167m^3/s, which occurs at time of 1.992hr from the start of the first rainfall.

4.1.3 Peak Flood Analysis by Using Flood Mark Method.

Based on the on the information of elders living around the site and by observing the earlier flood mark around the river bank the cross-sectional and longitudinal survey was made at the downstream of the weir was tabulated at the following table and graph.

Table 4. 5:- Longitudinal profile of downstream bed river level.

Change	0	3	3	3	3	3	3	3	3
distance	0	3	6	9	12	15	18	21	24
elevation	1326	1325.99	1325.85	1325.8	1325.63	1325.55	1325.35	1325.11	1325.19
Elevation difference	-	0.01	0.14	0.05	0.17	0.08	0.2	0.24	-0.08
slope (m/m)	-	0.003	0.046	0.016	0.056	0.026	0.066	0.08	-0.026

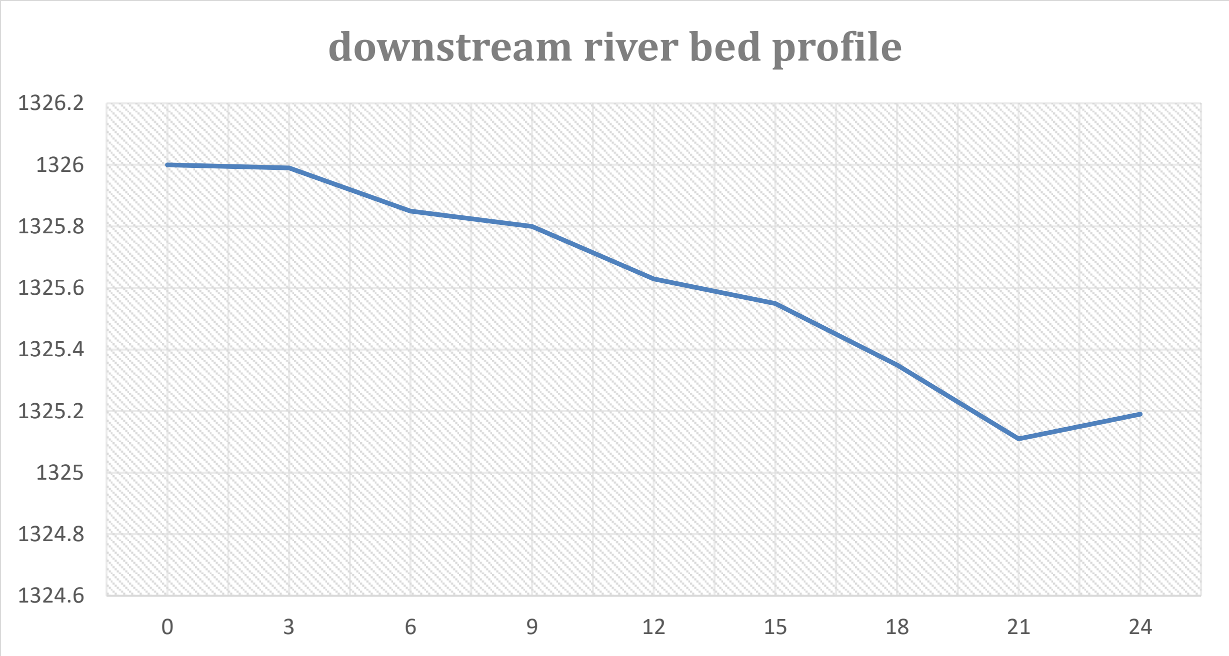


Figure 4. 2:- Downstream river bed profile

River bed slope is one of parameter to calculate the peak flood of the river. A river slope is usually expressed in m/m that was vertical drop per longitudinal distance of the reach. Based on the above table and graphical description Offiya river bed have 0.034 average river bed slope.

The high flood mark was marked based on the information of the elders near the site was observed and the leveling of cross-section of the river bank was surveyed by using leveling instrument about 37m width as shown on appendix (C) table (1) and the graph shown below.

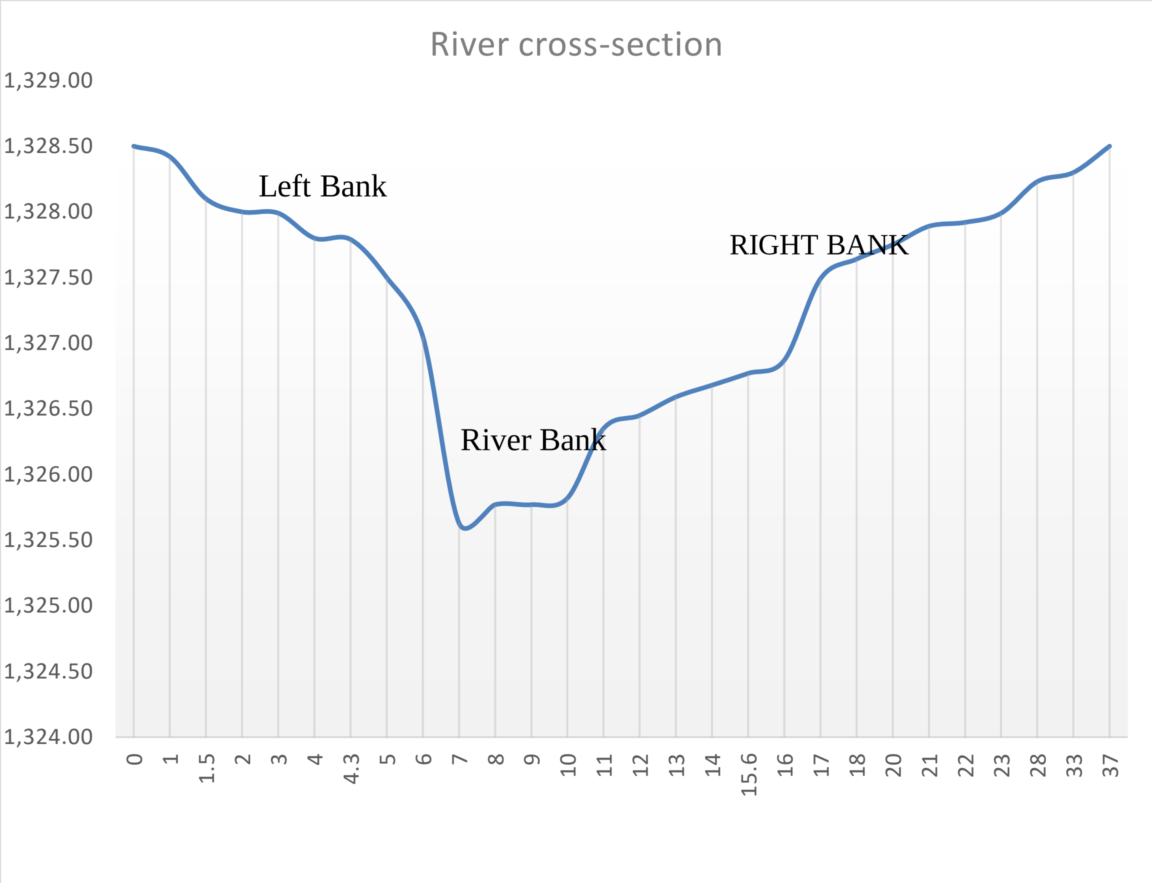


Figure 4. 3:-River bank cross-section

The roughness coefficient of the river bed was taken based on the river bed material from standard table. The river bed material of the Offiya at weir site and most part of the river channel was covered by massive rock and weathered basalt as observed during the field study time. Therefore the value of roughness coefficient corresponding to characteristics of river bed and bank material was 0.03 up to 0.04 as observed during the study period; therefore peak flood of the river was computed by using flow Bentley flow master version10 software with in 0.246m elevation different.

Table 4. 6:- Peak flood mark analysis

Water Surface Elevation (m)	Discharge (m^3/s)	Velocity (m/s)	Flow Area (m^2)	Wetted Perimeter (m)	Top Width (m)
1,325.63	0	0	0	0	0
1,325.87	0.148	0.85	0.2	2.25	2.2
1,326.10	1.444	1.86	0.8	3.11	2.92
1,326.34	3.874	2.5	1.6	3.98	3.64
1,326.57	7.223	2.7	2.7	6.27	5.82
1,326.81	12.95	2.91	4.4	9.7	9.19
1,327.05	24.116	3.57	6.8	10.89	10.27
1,327.28	38.634	4.16	9.3	11.92	11.18
1,327.52	56.023	4.66	12	13.08	12.23
1,327.75	73.265	4.81	15.2	16.55	15.64
1,327.99	96.176	4.96	19.4	20.95	20
1,328.23	125.943	5.03	25	27.61	26.61
1,328.46	166.627	5.11	32.6	36.83	35.77

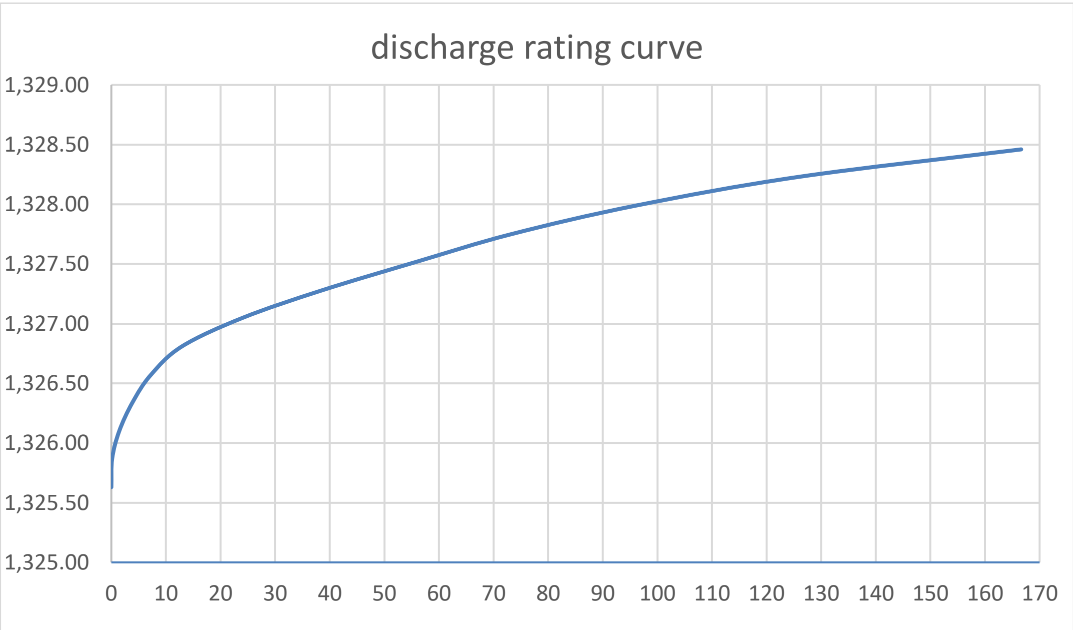


Figure 4. 4:-Stage discharge curve at downstream Diversion Site

The maximum elevation of the point marked by the elder dwellers was 1327.99m; according to above table the peak flow was 96.176m3/s by using the Bentley flow master. The extreme depth of the flow at the downstream of weir was 1.36m as shown figure below.

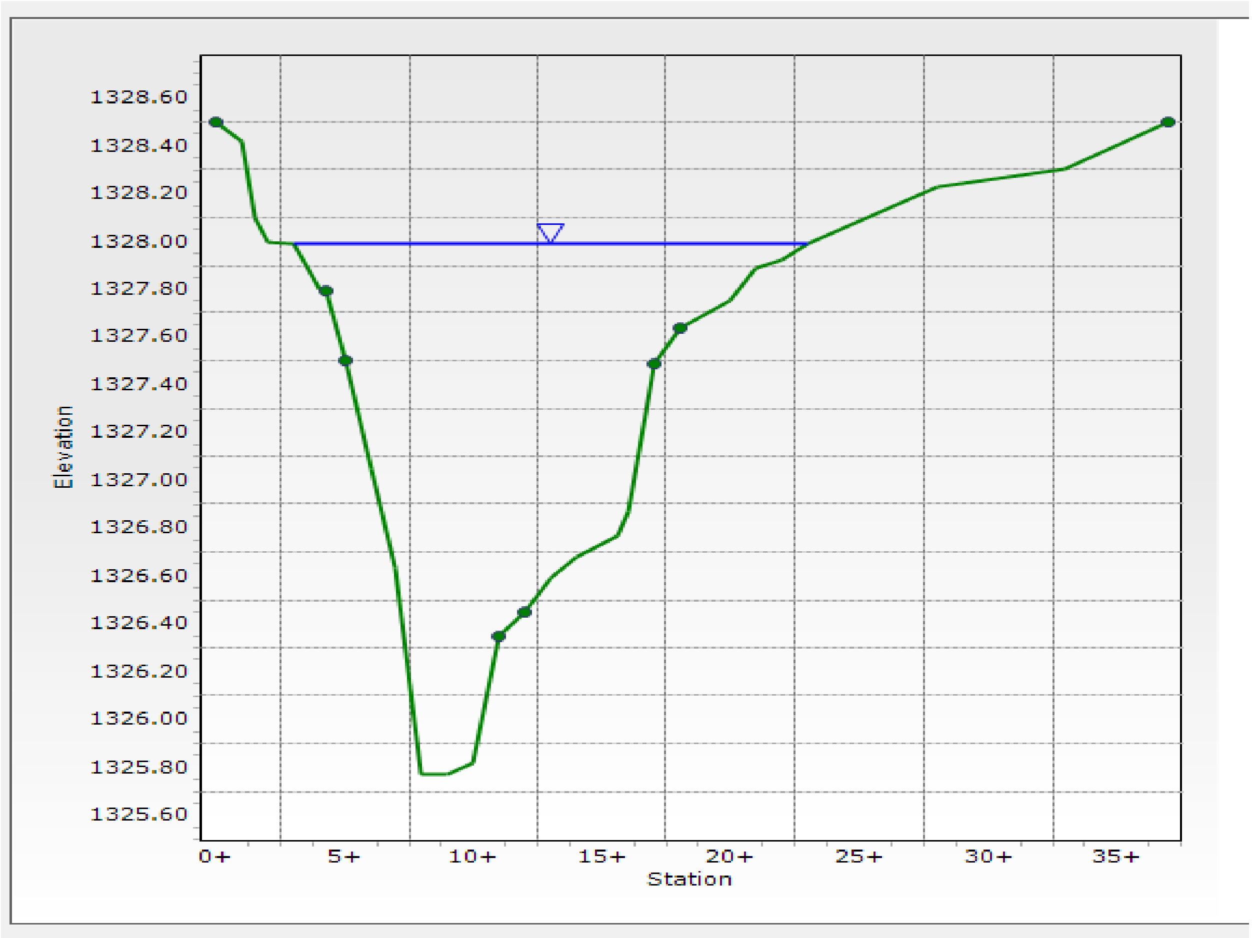


Figure 4. 5:-maximum flow depth by peak flood mark analysis.

4.1.4 Peak Flood Analysis by Using HEC-HMS

One of the methods used in this study to determine the peak flood is the Hydrologic Engineering Center-hydrologic Modeling System (HEC-HMS).

SCS curve number loss method, SCS unit hydrograph transform method and Muskingum routing methods were used to optimize model parameters. Each of these hydrologic parameters was automatically calibrated using the optimization manager function included in the HEC HMC model. The fundamental parameters for sub catchment in SCS unit hydrograph transform method were time of concentration (Tc) and Lag time (la) and SCS curve number loss method were initial abstraction, weighted soil curve number and % impervious are tabulated in following table 4.7. The SCS suggests that the UH lag time may be related to time of concentration, tc for ungagged catchment is given as; t_{lag}=0.6tc.

Table 4. 7:- Input parameter value for HEC-HMS

sub Basin	area(sqkm)	Loss			transformation method	
		initial abs	CN	%IMPERVIOUS	Tc	lag time
1	10.3742	3	87.786	3.243	1.186	0.7116
2	16.92865	2	96.24	2.75	1.24	0.744
3	1.124	6.5	88.68	0.425	0.22	0.132

The reach routing method was Muskingum routing method; the parameter required for these methods were Muskingum k and x value. Muskingum k value was the time interval of the flood to travel from upper reach junction to downstream reach outlet junction which was calculated from steam length between two outlet point and slope of stream bed. The Muskingum method is a common lumped flow routing technique. In this model, a calibration for two parameters, X and K, was required. X is a dimensionless weight, which is a constant coefficient that varies between 0 and 0.5, where X is a factor representing the relative influence of flow on storage levels (Hamdan, 2021).

According to SCS, recommendation the value of Muskingum x value was for any river have average value from 0.2 to 0.4.

In case of this study there was one reach at the junction one to outlet junction. Therefore Muskingum x value were assumed 0.2 for reach one as shown on HEC-HMS layout. The meteorological model was computed by using specified hyetograph incremental rain fall data in mm were prepared in appendix (A) table (6).

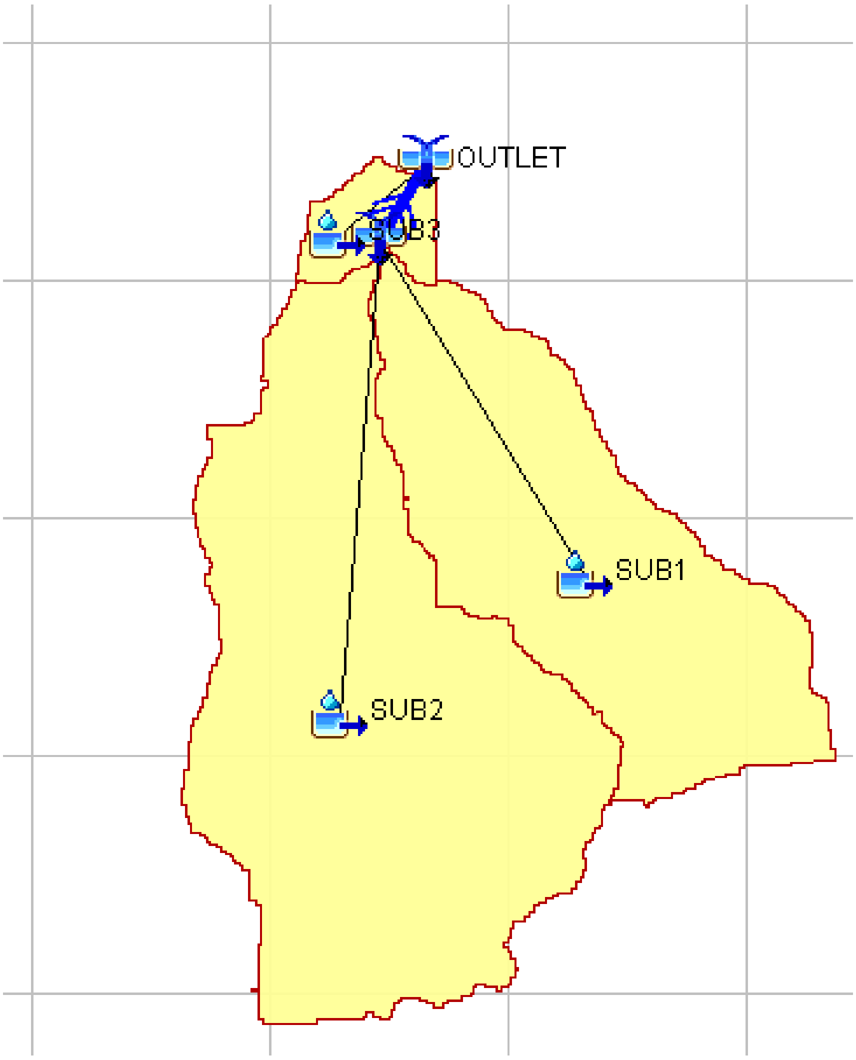


Figure 4. 6:-Layout of HEC-HMS

After creating basin model, meteorology model and control specification for each sub basin simulation runs are created in EC-HMS to compute the output with the initial parameter estimates. Simulation runs produce a graph to visually compare observed hydrograph with the computed or simulated hydrograph and several tables such as summary results table such peak discharge, total discharge values, total precipitation, loss, direct runoff, average absolute residuals, and total residuals can be seen and time series results table can be seen in each time step (Tadesse, 2016).

In case of this study there is no river gage data to compare the simulation of the HEC-HMS model. The data used to simulate this model was maximum rain fall data for 50 year return period incremental depth of precipitation will be arranged sequentially using alternate block method.

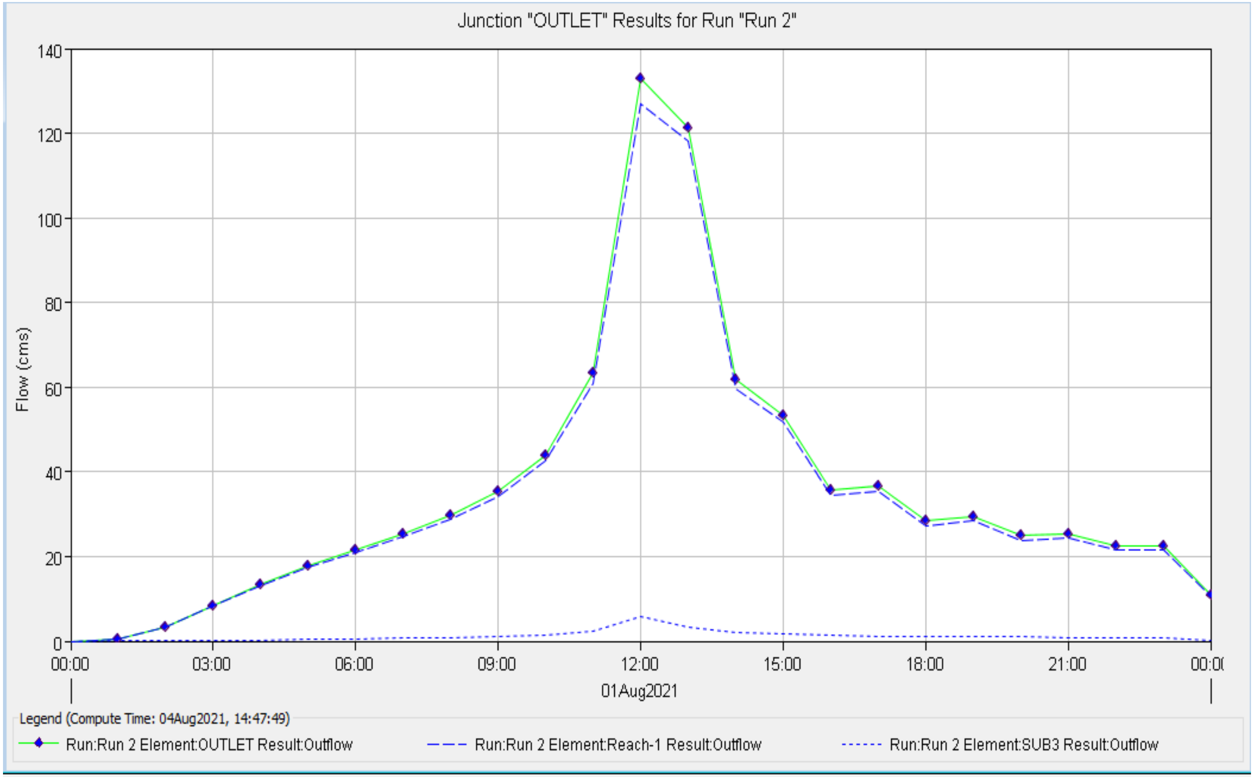


Figure 4. 7:-HEC-HMS simulation hydrograph

Table 4. 8:- Summary result table

Project: OFIYA_HEC_HMS   Simulation Run: Run 2

Junction: OUTLET

Start of Run:	01Aug2021, 00:00	Basin Model:	Basin 1
End of Run:	02Aug2021, 00:00	Meteorologic Model:	Met 1
Compute Time:	04Aug2021, 14:47:49	Control Specifications:	Control 1

Volume Units: ○ MM ◉ 1000 M3

Computed Results

Peak Discharge:	132.9 (M3/S)	Date/Time of Peak Discharge:	01Aug2021, 12:00
Volume:	3105.1 (1000 M3)		

Table 4. 9:- Out let discharge with in time step

Project: OFIYA_HEC_HMS Simulation Run: Run 2
Junction: OUTLET

Start of Run: 01Aug2021, 00:00 Basin Model: Basin 1
End of Run: 02Aug2021, 00:00 Meteorologic Model: Met 1
Compute Time: 04Aug2021, 14:47:49 Control Specifications: Control 1

Date	Time	Inflow from... (M3/S)	Inflow from... (M3/S)	Outflow (M3/S)
01Aug2021	00:00	0.0	0.0	0.0
01Aug2021	01:00	0.4	0.0	0.4
01Aug2021	02:00	3.2	0.0	3.2
01Aug2021	03:00	8.3	0.0	8.4
01Aug2021	04:00	13.2	0.2	13.4
01Aug2021	05:00	17.3	0.3	17.7
01Aug2021	06:00	21.0	0.5	21.5
01Aug2021	07:00	24.7	0.6	25.3
01Aug2021	08:00	28.8	0.8	29.7
01Aug2021	09:00	34.2	1.1	35.3
01Aug2021	10:00	42.5	1.5	43.9
01Aug2021	11:00	61.0	2.4	63.3
01Aug2021	12:00	127.0	5.8	132.9
01Aug2021	13:00	118.0	3.3	121.3
01Aug2021	14:00	59.7	2.1	61.8
01Aug2021	15:00	51.9	1.6	53.5
01Aug2021	16:00	34.5	1.4	35.8
01Aug2021	17:00	35.4	1.2	36.6
01Aug2021	18:00	27.3	1.1	28.4
01Aug2021	19:00	28.4	1.0	29.5
01Aug2021	20:00	23.9	1.0	24.9
01Aug2021	21:00	24.4	0.9	25.3
01Aug2021	22:00	21.6	0.9	22.4
01Aug2021	23:00	21.7	0.8	22.5
02Aug2021	00:00	10.7	0.2	10.9

The HEC-HMS model analysis result at the weir site or at outlet junction was 132.9m3/s and the base flow of Offiya river was 180l/sec which was the taken from kaffa zone water mine and energy department; therefore the total discharge of the river was 133.08m^3/s.

Table 4. 10:- Summary of peak flood analysis

s/n	Method	Q m^3/s	Remark
1	SCS curve number	110.017	
2	Peak flood mark analysis	96.18	
3	HEC-HMS Modeling	133.08	MAX

The result shown above table for SCS curve unit hydrograph analysis, peak flood mark analysis and HEC-HMS modeling were slightly related there is no high variation. Therefore using the maximum value of the analysis is compatible for design analysis of Offiya weir structure.

4.2 Hydraulic Analysis

4.2.1 Adequacy of Water Way

The weir crest length (P) should be adequate to pass the design flood safely. The minimum stable width of an alluvial channel was usually determined from the Lacey's wetted perimeter (P). Lacey calculated a series of regime flow equations. One of these describes the regime perimeter of a river in alluvial material in terms of its dominant flow (Ertiro, 2017).

$L = 4.75\sqrt{Q} = 4.75\sqrt{133.08} = 54.79\text{m}$

Constructed weir crest length = 16.8m

Thickness of dived wall =1m

Width of under sluice=1m

Total width of water way provided=20.8m

For most of cases the calculated waterway was wider than actual regime of the river. According to MOA, 2018 the reduction in the width of the water way was inevitable. As the result of loose factor which was the ratio of the actual linear water way provided to water way to be calculated is being adopted. The recommended of looseness factor was been 0.5 to 1 (MOA, 2018).

The width of the constructed weir was 16.8m where 69.34% reduction taken from the required maximum width it was not recommended to reduce more than 50% for allowance of silt factor. So the width of weir was not adequate to pass the high flood of the river.

As per MOA for most common type of diversion weir in small scale irrigation project in Ethiopia the energy level over the crest of the weir was recommended up to 2m head would corresponds to a maximum discharge intensity of 58.8m^3/s/m over the weir crest (MOA, 2018).

Therefore the maximum allowable discharge over weir crest was computed by using weir discharge formula for broad crested weir was 80.77988m^3/s where about 60.7% of design flood computed by HEC-HMS 133.08m3/s. The energy head level for the design flood of Offiya River was 2.79m which was greater than 2m; therefore the water way of Offiya small scale irrigation weir was not adequate to pass the design flood with return period of 50 year according to recommendation of minister of agriculture guide line for small scale diversion weir.

4.2.2. Water Depth over Constructed Weir

During the field observation the upstream floor level of the constructed weir was 1328m.a.s.l. the height of weir was 1.5m the crest level of weir was 1329.5m.a.s.l and the top width of weir was 0.8m. The computation of depth of water was calculated by try and error with velocity head equation given as (Garg, 2011).

$$v_a = \frac{Q}{A} = \frac{Q}{(H_d+p)*L} = \frac{133.08}{(H_d+1.5)*16.8}, \; v_a = \frac{7.92}{(H_d+1.5)}$$

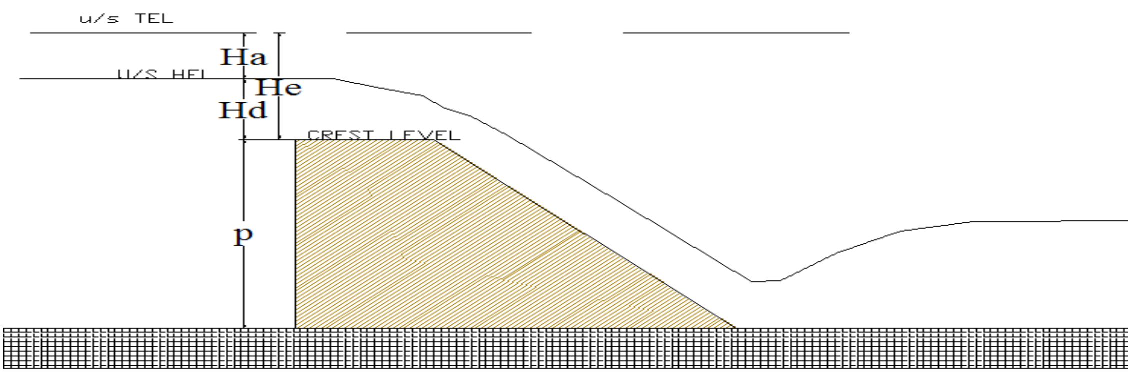


Figure 4. 8:- Weir profile

$$H_e = Hd + Ha = Hd + \frac{V_a^2}{2g}; where\ Ha = \frac{V_a^2}{2g}$$

$$Hd = H_e - \frac{V_a^2}{2g}; \; \frac{V_a^2}{2g} = H_e - Hd; \; ☞\frac{V_a^2}{2g} = 2.79 - Hd$$

$$hence; \; \frac{V_a^2}{2g} = 2.79 - Hd ------------------eq\ 4.2.2a$$

Substitute the value of V_a into Eq-4.4.2a

$$\frac{\left[\frac{7.92}{(H_d+1.5)}\right]^2}{2g} = 2.79 - Hd; \qquad \frac{(7.92)^2}{(Hd+1.5)^2} * \frac{1}{2*9.81} = 2.79 - Hd$$

$$(Hd+1.5)^2 * (2.79 - Hd) - 3.2 = 0 -----------------eq\ 4.2.2b$$

The values of H_d can be obtained from equation 4.2.2b by trial and error method the depth of water over weir crest was 2.58m. The extreme depth over the weir measured from retaining wall top to weir crest was 2.25m which was 87.2% the designed water depth over the weir to pass the peak flood of the study. This indicate the underperformance of weir crest and upstream retaining wall constructed for Offiya diversions weir.

4.2.3 Hydraulic Jump Computation

Hydraulic jump is a stream of water moving with a high velocity and low depth (i.e. Supercritical flow) strikes other stream of water moving with a low velocity and high depth (i.e. subcritical flow), sudden rise in the surface of the former takes place. The type of jump where characterized based on the value of Froude number and the provision of protection structure was deseeded based on the characteristics of jump (MOA, 2018).

Table 4. 11:-Classification of jump based on Froude number

Types of Jump	Froude number	Jump Characteristics	Energy dissipation, %
Strong jump	$F_r > 9$	Rough jump, lots of energy dissipation	5
Steady jump	$4.5 < F_r < 9$	Considerably energy losses	20
Oscillating jump	$2.5 < F_r < 4.5$	Unstable oscillating jump; production of large waves of irregular period	20-40
Weak jump	$1.7 < F_r < 2.5$	Little energy loss	45-70
Undular jump	$1.0 < F_r < 1.7$	Free-surface undulations d/s of the jump; negligible energy loss	70-85

Source minister of agriculture guide line for SSI diversion structures

Calculation of hydraulic jump stated from continuity equation where the energy level before jump equal to after jump as follow.

$$p + He = Y1 + \frac{V^2}{2g}; \quad 1.5 + 2.79 = Y1 + \frac{V^2}{2g}$$

$$4.29 = Y1 + \frac{V^2}{2g} ----------------------------- eq\ 4.2.3a$$

Where, p = height of Weir

He= total Head over Weir Crest

V_1= velocity at section 1

$$V_1 = \frac{Q}{L * Y_1} = \frac{133.9}{16.8 * Y_1};$$

$$V_1 = \frac{7.97}{Y_1} ------------------------------- eq\ 4.2.3b$$

Substituting eq 4.4.3a into eq 4.4.3b

$$4.29 = Y1 + \frac{\left(\frac{7.97}{Y_1}\right)^2}{2g} = Y_1 + \frac{7.97^2}{{Y_1}^2 2 * 9.81}$$

$$4.29 = Y_1 + \frac{3.2}{{Y_1}^2}$$

$Y_1^3 = 4.29Y_1^2 - 3.198$ -- $eq\ 4.2.3c$

The values of Y1 can be obtained from $eq\ 4.2.3b$ by trial and error method. Hence, the value of Y_1=0.98m; Critical depth (Y_C) must be greater than y1

$Y_c = (q^2/g)^{1/3} = \left[\frac{7.97^2}{9.81}\right]^{1/3} = 1.86m > 0.98$m −−−−−−ok

V1=7.97/0.98= 8.1m/s.

The downstream apron was flat, the conjugate depth of Y_1 is calculated by

$Y_2 = Y_1/2*(-1+\sqrt{8F^2+1})$ where $F = \frac{V}{\sqrt{gY1}}$ called Froude number

$F = \frac{8.1}{\sqrt{9.81*0.98}} = 2.605$

$Y_2 = 0.98/2*(-1+\sqrt{8*2.605^2+1})$ = 3.15m

Based on the above (table 4.11) description the characteristics of jump was unstable oscillating jump; production of large waves of irregular period and the type of Oscillating jump. Where about 20-40% of the energy needs to reduce or dissipate to avoid scoring of the downstream river bed material; but there was no provided energy dissipater at down floor of the weir this case the high scoring of the downstream river and constructed floor.

The downstream depth of water from rating curve of downstream water way was 2.73m which is less than the conjugate depth. In this case the jump occur downstream away from impervious floor so it needed to design and construct energy dissipating structure to ensure the jump to occur within protected river bed. But in case of this study there is no energy dissipating structure constructed also there is no downstream launching apron to minimize the scouring occurring due to high jump.

4.2.4. Evaluation of Constructed Weir Components

1) River Training Work

River training work was used to protect the outflow of water due increasing of water depth after construction of weir and used to narrowing down the natural river bank (Tadesse, 2016). In case of this study the average natural river bank width measured at upstream of the weir is 36m while the width between at end of upstream right and left wing wall was 28m but there is no constructed river training work at Offiya diversion weir due to this river water was out flanking during high rain season at the side of off-taking canal.

2) Up and Down Stream Cut-off Structure

The cut off structure is structure used to reduce uplift pressure and prevent or reduce flow of water seepage underneath of foundation of the weir by increasing flow path or creep length of seepage. Usually constructed from RCC concrete at upstream end and downstream end of impervious floor; sometimes provided at middle of impervious floor of weir (MOA, 2018).

In case of Offiya head work the upstream and downstream of cut off structure were constructed from bolder stone material was constructed up to depth of 1m and 3m respectively. As design drawing of Offiya upstream cut off depth is 1m and downstream cut off depth is 3m. The constructed cut off depth at downstream was from bolder material which observed during field study and there was 1m constructed upstream cut-off reported by site supervisor.

Figure 4. 9:- Improperly constructed downstream cut-off pile

To fix bottom levels of cut-offs, the scour depth, R is multiplied by a factor of safety ranging from 1.25 to 1.5 and 1.75 to 2.0 for upstream cut-off and downstream cut-off depths respectively. By using U/S HFL-1.25R for upstream and D/S HFL-1.75R for downstream pile level (MOA, 2018).

Based on geological description of in section (3) the weir site bed material was fine alluvial deposition at the bed of the river so the mean value the silt factor was 1 based on above table 3.14. By considering the foundation of the material is uniform for upstream pile and downstream pile.

Table 4. 12:- Required pile and provided pile

s/n	Design discharge (m3/s)	Silt factor (f)	Scouring depth (R)	required pile depth	Provided pile depth
Upstream pile	133.08	1	5.36	2.62	1
Downstream pile	133.08	1	5.36	6.65	3

Based on the physical observation the weir foundation was highly loose material deposition was visual at the exposed river bed of up and down stream of the weir axis. But the construction of the weir does not consider the actual foundation condition of the weir site. The under design and construction of cut-off pile structure for upstream and downstream end of impervious floor was the basic cause for the failure of Offiya head work as shown figure below.

Figure 4. 10:- The effect of improper construction of pile structure.

3) Up and Down Stream Impervious Floor.

Impervious floor or apron is provided to protect the main body of the weir from the scouring effect and ultimate failure. The total floor length of impervious floor includes the downstream basin/floor length, glacis, weir crest, and upstream floor. The impervious floor in conjunction with the downstream cutoff should result in safe exit gradient. The purpose of impervious apron in the weir structure is to resist uplift pressures and to dissipate the incoming energy over the weir. The length of the apron is determined considering the hydraulic jump length and the available exit gradient at the end of the apron and the apron varies in thickness from maximum at the start of the hydraulic jump stilling basin/apron to a minimum at the end is recommended. In general impervious apron of sufficient length must be provided on the U/S and D/S side of the weir. Similarly the thickness of the apron especially of the D/S apron should be sufficient to resist the hydraulic jump as well as the uplift pressure (Garg, 2011)

Table 4. 13:-Required and provided length of apron.

Type of weir	Foundation	Bligh's creep coefficient	Seepage head(m)	CH(m)	L_d(m)	Weir base width(m)	Cut-off(m)		LU(m)
							UP	DS	
Broad crested	Coarse sand	18	1.5	27	15.40	3	1	3	3.6

The constructed downstream impervious floor length was 84.37% of the design impervious floor as shown in table above the constructed upstream impervious floor was 83.33% of the designed in table 4.13. The downstream impervious floor of weir was destructed and moved by flooding due to this reason the downstream river bed was highly scoured. The constructed impervious floor thickness was 30cm with 60cm-100cm spaced center to center steel bar. The gravel material used for construction of the impervious floor was not standard with its size as observed on site on destructed floor slab. The floor of Offiya was constructed from mix of bolder material with improper material ratio as shown on the following image taken during field observation from destructed floor.

Figure 4. 11:- Effect of improper construction of impervious floor of Offiya weir.

The main cause of the destruction of the Offiya diversion of weir impervious floor was improper use of material during construction of the weir floor and under design of the length of impervious floor. The aggregate used for concrete work was bolder material as shown on the above image.

4) Up and Down Stream Protection Work.

The up and downstream impervious floor were protected by protection structure constructed from stone or concrete block at end to protect the scouring of river bed due to jump of flood; however in case of Offiya diversion weir there is no protection structure for upstream and downstream end of impervious floor. These were increase the failure of the weir river bed impervious floor due to scouring.

5) Weir Body Dimension

The weir dimension includes the bottom and top width which was stable under condition of maximum stress. The top width of broad crested with vertical upstream face was designed for the following three cases and taken the maximum value (Garg, 2011).

a) on the consideration of no tension criteria

$a = \frac{d}{\sqrt{G}}$ Where d is the depth water on top of weir.

G is specific gravity of material where weir constructed (For masonry G = 2.24).

$$a = \frac{2.58}{\sqrt{2.24}} = 1.72m$$

b) On the consideration of no sliding criteria

$a = \mu \frac{d}{G}$; Where μ is coefficient of friction usually taken 3/2

$a = \frac{3}{2} * \frac{2.58}{2.24} = 1.72m$

C) for crest shutter condition criteria

a= s+1; in case of Offiya weir there is no shutter over crest of the weir body so the top width is 1m

The top width of the weir for design flood was 1.72m the constructed top width of the weir was 0.8m which was 46.516% of the designed weir top width.

The bottom width (B) of the weir wall may be obtained by providing suitable side slopes for u/s vertical faced weir. The bottom width of the weir should not be less than (Hd+h)/ (G-1) for no flow, for high flow and for high flow when weir is submerged condition (Garg, 2011).

B ≥ (Hd+h)/ (G-1) = (2.58+1.5)/2.25-1) = 3.264m

The constructed weir bottom width measured at site during field study was 3m which was 91.9% the recommended design width.

6) Scouring sluice gate

Under sluice of Offiya diversion weir constructed with 1m width which was clogged by river silt and gravel material.

Scouring sluice gate basically intended to avoid sediment from the upper side of the weir body to downstream and flushing without causing damage to the downstream. The accumulation of high silt, gravel and bolder material shown by clogging the way of under sluice gate. This show the improper design of scouring sluice gate.

According to (MOA), small scale irrigation design guide line the size of under scouring sluice gate should be designed and constructed to flushing off 5 to 20% of design flood discharge during expected design floods so as to reduce flood height over the structure and hence wing wall height, i.e. it should be designed to ensure sufficient scouring capacity to dispose of the above range of the peak flood. This value shall at least be greater than 2 times the intake capacity(MOA, 2018).

The discharge of scouring sluice gate was calculated by using discharge formula of box type under sluice for high flooding case i.e. the head above the orifice is the sum of weir height and depth of water over crest minus the half of depth of scouring sluice opening.

Table 4. 14:- Required and provided capacity of under sluice

Peak design discharge of the study area (m3/s)	Recommend Discharge of scouring sluice gate (m3/s)	Discharge Capacity off existing scouring sluice gate (m3/s)	Remark
133.08	26.62	7.28	Less the recommended value

According to (MOA), the velocity of flow through each sluice channel should be greater than critical velocity, Vc, so as to enable easy flushing of sediment through the openings, i.e. flow within the scouring sluice should be in supercritical condition to remove sediment deposited in-front of intake, but that through the intake should give a critical flow condition so as to allow flow to the canal(MOA, 2018).

The actual velocity through under sluice and critical velocity of river material was compared as the following table.

Table 4. 15:-Velocity through under sluice

Velocity of under sluice (m/s) $V = C_d \, x \sqrt{2gh}$	Critical velocity for bolder material with average grain size 50.1mm (m/s) $V_c = \sqrt{20 * d_m}$	Remark
4.8497	1.001	$V > V_c$
	Critical velocity for high flood case dc =(q2/g)^(1/3); v_c=q/dc	
	1.105726	$V>V_c$

The velocity through scouring sluice gate was greater than the river material critical velocity accumulated at the weir upstream side and the critical velocity of flow under sluice gate during high flood condition. The accumulation of the river bolder material was occurred due to the closing of the gate throughout the year the river bed material was increasing time to time and total closing the under sluice gate.

Figure 4. 12:-Effect of underperformance design of Offiya under sluice

7) Energy dissipation

The high energy loss that occurs in a hydraulic jump has led to its adoption as a part of high energy dissipater system below a hydraulic structure. Three types of energy dissipaters have been commonly used: stilling basins, flip buckets, and roller buckets (Tadesse, 2016).

Each dissipaters has certain advantages and disadvantages and may be selected for a particular project depending upon the site characteristic and type weir. In case Offiya diversion weir there is no energy dissipater constructed or provided during the design and construction time. But about 20-40% of energy needs to dissipate also the jump is out of the impervious floor this indicate the need of any obstruction to dissipate the energy for protecting tail river bed.

4.3 Structural Analysis of the Constructed Weir and Appurtenance Structure

Structural analysis of this study including stability analysis of weir proper body and appurtenance structure such as retaining wall and wing wall. In this study the stability of weir body checked for all mode of failure in static case or pond water level condition. Mode of failures are Overturning, Sliding and Stress also checked for bearing capacity of the foundation.

The force acting on the weir body in static case are uplift pressure, water pressure at U/s side, self-weight of weir and silt active pressure. The other force such as wave, wind and earth quake are not considered in case of this study because the weir height was to short and the probability to occur the earth quake was no because the weir site zone was free from earth quake.

4.3.1 Stability Analysis of Weir Body

1) Stability against overturning

Overturning failure occurred when the overturning moment exceeded the resisting moment. The failure of overturning was usually preceded by the tension failure or crushing failure. When the weir was considered as safe against overturning if the criterion of no tension at any point in the weir body was satisfied and also no maximum compressive stress does not exceed the allowable limit. The weir body to be safe the resting moment exceeded 150% than overturning moment (Tadesse, 2016).

In case of this study the stabilizing moment on the weir body was greater than the disturbing moment the factor of safety against overturning for weir section in static condition was 2.4 (>1.5). Therefore there was no overturning failure.

2) Stability against Sliding

The sliding failure occurred when the weir and their appurtenance structure sliding over their base. Sliding failure caused when the resting force or sum of horizontal or destabilizing force acting on weir or structure exceeded the sum of vertical force or resting force acting on the structure. Where the structure said safe against to sliding the ratio of sum of horizontal force acting on the structure to sum of vertical force multiplied by sliding coefficient was greater than 1.5; then the structure was safe(Tadesse, 2016).

The Offiya weir was checked for stability against the sliding and the result of analysis safety factor of sliding in case of static was 1.43 which was less than the recommended value of sliding so there is sliding problem of the weir based on constructed dimension in static condition.

3) Stability against tension

For no tension on the base of the head work structure, for critical section, the resultant of vertical force, should act at the middle third part of the critical section. This implies that the eccentricity (e) should be less than or equal to one-sixth (1/6) of the base width (B) of the weir at the critical section (MOA, 2018).

Whereas the Offiya constructed weir have the base width of 3m then the 1/6 of base was 0.5 eccentricity of the vertical force acting on the weir toe was 0.2116 which was less than the recommended value. So there is no tension at the toe of the weir in static case of the weir.

Based on the above description the weir was well perform the failure due to overturning, sliding and Stability against tension as analysis shown on appendix (D) table (1).

4) **Factor against contact pressure**

The foundation condition Offiya diversion weir site was most dominated by silt soil at the depth and with outcropped bolder material mixture clay soil at thin top layer.

The contact pressure (Stress) on the foundation at the toe or heel of the weir body should be less than the allowable bearing capacity of the foundation material. According to MOA the allowable bearing pressure was 80KN/m2 for silt foundation. Maximum and minimum Compressive stress developed at toe of the weir does not exceed the allowable compressive stress, otherwise reinforcement need to be provided in concrete at base of the walls(MOA, 2018).

However the maximum and minimum compressive stress as analysis of Offiya weir on appendix (D) table (1) were 20.63KN/m^2 and 8.36 KN/m^2 respectively; so the weir is safe against contact pressure of the foundation soil.

4.3.2 Stability Analysis of Apparatuses Structure

The stability analysis of apparatuses structure includes up and downstream retain wall and wing wall was checked for empty condition. During the site observation the wall constructed for Offiya diversion weir were up and down stream retain wall and upstream wing wall. In this diversion weir project there is no downstream wing wall. Therefore stability of up and down stream retaining wall and upstream wing wall was analyzed as table on appendix (D). Based on the site observation the maximum depth of back fill soil up to 2m depth on inclined surface of retain and wing wall.

Figure 4. 13:- Offiya retaining wall

Table 4. 16:- Summer of stability factor of safety analysis of wall structure.

Type of structure	Sliding> 1.5	Overturning >1.5	Overstress< B/6	B/6	P_{max}	P_{min}
U/S Retaining Wall	4.99	4.62	0.926	0.5	239.33	-71.49
U/S Wing Wall	1.89	2.86	0.5196	0.42	108.67	-11.95
D/S Retain Wall	1.44	2.37	0.8	0.5	216.35	-68.25

As table indicated upstream retaining and wing wall were stable against sliding and overturning whereas downstream retain wall was stable for stability against overturning and sliding, but not stable stability against overstress. The result of analysis of was greater than the recommended safe value or B/6. Upstream retaining and wing wall were not stable due over stress/tension more than B/6 at constructed bottom width. The maximum compressive stress pressure due to contact of foundation were exceeded for upstream retaining and wing wall.

The divide wall was not constructed properly generally it is not serve properly the intended purpose.

4.4 Seepage Analysis

Seepage analysis was carried out by using Khosla's theory and Geo studio 2018 software for two condition of the water at upstream of the weir. These two condition were water at pond level (water at crest of weir) and water at high flood level. The water level at pond the tail water depth were considered as zero where the water level at high flood level the tail water depth becomes 2.73m as described as description of back water discharge curve figure 4.4.

Table 4. 17:-Input data for Offiya weir seepage analysis

s/n	Description of data	Value	Remark
1	Weir crest level in meter	1329.5	Measured
2	U/S High flood water level(m)	1332.08	Calculated
3	U/S and D/S bed level (m)	1328	Measured
4	Thickness of floor(m)	0.3	Measured
5	D/S high flood level (m)	1330.73	Calculated
6	U/s cut-off depth	1	From site report
7	D/s cut-off depth (m)	3	Measured at scoured area
8	Length of impervious floor (m)	19	Measured
8	Unit weight of soil under weir floor foundation (KN/m2)	18	For saturated silt soil
9	Permeability of silt soil under Offiya weir (m/s)	0.000035	From design guide line
10	Volumetric water content of saturated silt soil	0.5	For saturated silt soil
11	Impervious soil strata under weir.	10	Geologist recommendation.

4.4.1 Seepage Pressure Analysis's Using Khosla's Theory

Table 4. 18:-Table Summary of corrected pressures at various key points

Upstream Pile Nr-1		Downstream Pile Nr-2	
$\phi E1=$	100%	$\phi E=$	36.71%
$\phi D1=$	86.0%	$\phi D=$	23.9%
$\phi C1=$	82.93%	$\phi C=$	0.00%

The head of water over the floor during pond level and high flood condition were 4.08m and 1.5m respectively so the thickness of impervious floor required in high flood condition and pond level condition at upstream pile inner corner and downstream pile inner corner were calculated as following table.

Table 4. 19:-Under weir pressure by Khosla's theory

Height of water		Height of sub soil H.G line above the datum d/s pile line			
		u/s pile line		u/s pile line	
		ϕC1=		ϕE=	
		82.9%		36.71%	
		Residual pressure	Thickness of floor	Residual pressure	Thickness of floor
At high flood condition	4.08	3.34	2.69	1.49	1.2
At pond level	1.5	1.24	1	0.55	0.44
Provided thickness			0.3		0.3

4.4.2 Seepage Analysis for Constructed Weir.

The study problem of water seepage underneath of Offiya head work weir structure; the quantity of seepage, water pressure head and water head were computed by using GEO Studio (SEEP/W) steady state model.

The constructed weir under foundation seepage water, pore water pressure, and exit gradient was computed for 1m upstream cut of depth and 2m down cut of depth water at weir crest level and high flood level.

The four quadrilateral and three triangular nodal elements were used to idealize the vertical cross section of permeable soil underneath of Offiya small scale irrigation diversion weir with 346 nodes and 305 elements for approximate global element size of one meters.

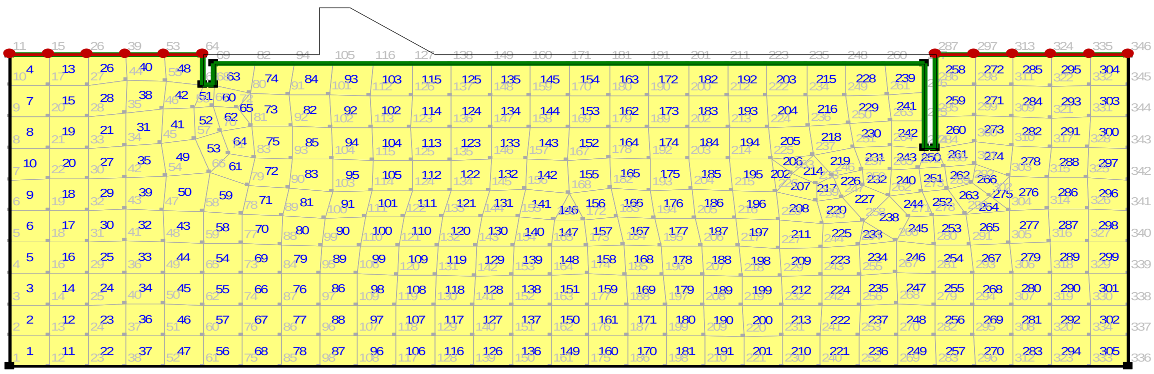


Figure 4. 14:- Finite element mesh for Offiya Weir resting on pervious soil foundation

When the u/s water level at pond level condition and the tail water depth at considered as at bed level then the boundary condition for upstream water head was 1.5 and for downstream water head was zero. The upstream water high flood level calculated was 1232.08 m above mean sea level and downstream and tail water level was 1230.73m above mean sea level. So the upstream water head boundary condition 4.08m and tail water head boundary condition 2.73m were used in high flood condition for Offiya diversion weir underneath seepage flow analysis.

The value of quantity of seepage flow flux (q) underneath of weir foundation was 1.3479x 10^-5 m3/s/m for water at pond level as shown figure below when the u/s water level is at high flood level value of quantity of seepage flow flux (q) underneath of weir was 1.2131 x 10^-5 m3/s/m as shown figure on appendix (E)

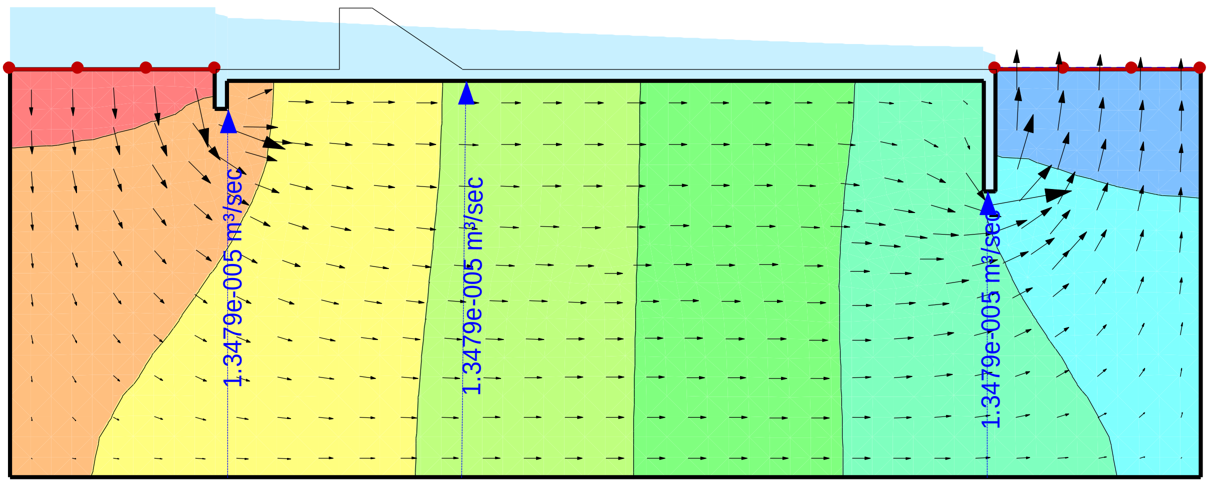


Figure 4. 15:- Seepage of water underneath Offiya Weir Foundation

The seepage water pressure head and water flow line of weir for pond water level with 1m U/S pile and 3m D/s pile sheet were illustrated as following figure.

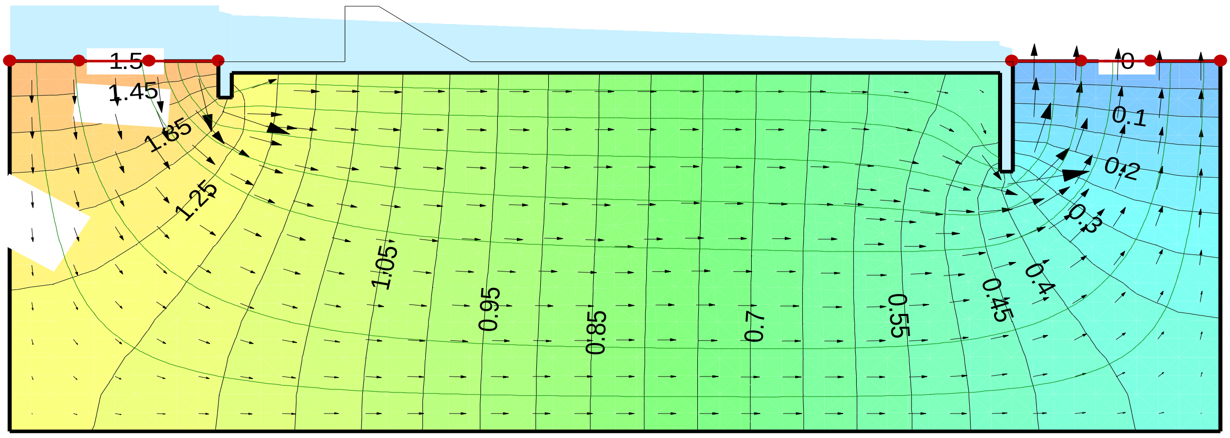


Figure 4. 16:- Seepage water head of Offiya weir

To insure the stability of floor against uplift pressure the necessary thickness of downstream floor is provided at different point along longitudinal section is provided. For checking the thickness of downstream impervious floor the equation was given (Khassaf, 2009).

t = 2/3* (uplift pressure)

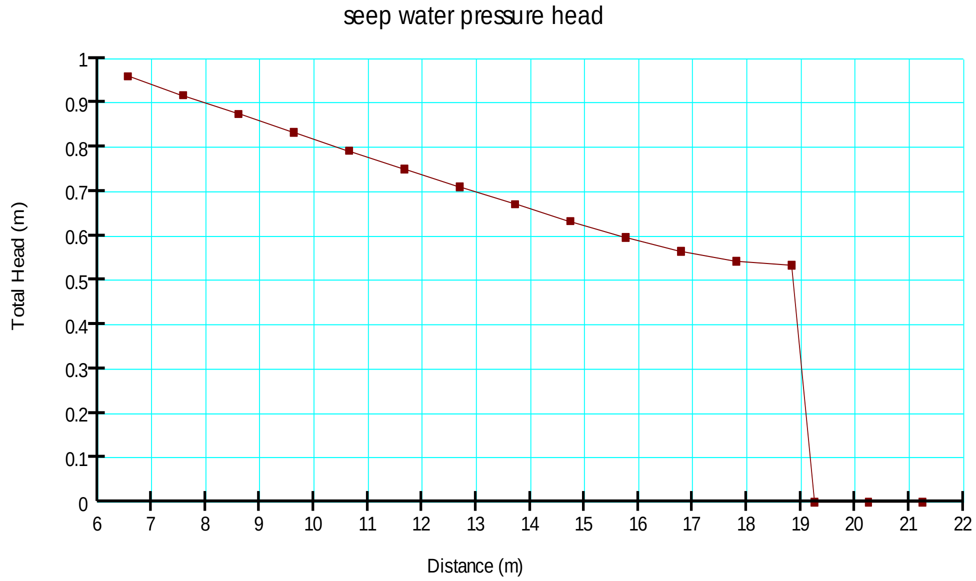


Figure 4. 17:- Uplift pressure distribution of constructed weir downstream floor in static case.

The required thickness of downstream impervious floor from uplift water pressure analysis tabulated as following table.

Table 4. 20:-Required thickness for downstream floor of offiya Weir

Distance from upstream floor end to downstream(m)	Water Pressure Head at pond level (m)	Required thickness of d/s floor(m)	Water Pressure Head for high flood condition (m)	Required thickness of floor (m)	Provided thickness of floor(m)
6	1	0.667	3.62	2.41	0.3
6.55	0.96	0.64	3.55	2.37	0.3
7.57	0.92	0.6	3.51	2.345	0.3
8.60	0.87	0.58	3.47	2.32	0.3
9.6	0.83	0.55	3.44	2.295	0.3
10.6	0.792	0.53	3.40	2.27	0.3
11.6	0.75	0.5	3.37	2.246	0.3
12.7	0.71	0.47	3.33	2.22	0.3
13.7	0.67	0.44	3.3	2.2	0.3
14.7	0.63	0.42	3.26	2.17	0.3
15.7	0.597	0.39	3.23	2.15	0.3
16.7	0.56	0.37	3.21	2.14	0.3
17.8	0.54	0.36	3.21	2.14	0.3
18.8	0.53	0.35	3.59	2.39	0.3

The required thickness of floor shown above table indicate the thickness of downstream floor for Offiya at the point distance indicated on table column (1). The required thickness at the point was greater than the provided thickness in both pond and high flood condition for Offiya diversion weir downstream floor. The analysis shown that the floor provided for Offiya diversion weir is not protected against uplift pressure due to thickness of impervious floor provided. In order to check hydraulic gradient of the weir floor when the maximum total water depth at the upstream of the weir was 4.08m, the length of impervious floor was 19m the depth of downstream pile sheet was 3m. The exit gradient of the weir was 0.225 based on equation 3.17a. The critical gradient of the foundation soil was 0.834 as the equation 3.17d which stated methodology section. The factor of safety against uplift pressure due to piping of water through foundation of weir was 3.7. A factor of safety (Fs) of 3.7 is considered adequate for th e safe performance of the offiya weir structure against piping because the factor of safety (Fs) is more than 3.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The Offiya river hydrological analysis's at the weir site was carried out by using SCS unit hydrograph, peak flood mark by using Bentley flow master and HEC-HMS software. The maximum value of from these analysis's method was taken for design performance analysis. Peak discharge of the river was 133.08m^3/s as result of HEC-HMS software whereas the capacity of constructed weir is 80.77988m^3/s which was less than the peak discharge of the river. The crest length of the constructed weir was short which was highly reduced over the recommended value of reducing factor. The capacity of the weir to pass the discharge at extreme depth over weir crest was under performance to pass the high flood of the river for 50 year design period. The upstream wing wall and retaining wall height were under design for the constructed weir crest length so there is out flanking of water during river high flow period or rainy season. As seen from flied observation there is no river training work at the upstream wing wall end due to this the water is flowing to the framing land during high rainy season. Also the absent of downstream wing wall was cause the scouring of the downstream right and left river bank.

As evaluation of under sluice performance there is high velocity to excluding silt and bolder material from upstream of the weir but due to improper management problem the weir was field with the silt at the upstream also the under sluice and of taking opening were clogging by boulder.

The base width of the Offiya weir was structurally stable and safe against overturning, overstress capacity and bearing capacity of foundation while it was not stable against sliding. The constructed upstream retaining wall and wing wall were physical stable against overturning and sliding but not stable against overstress and contact pressure capacity. The constructed downstream retaining wall was stable for over turning but not for other stability condition.

Determination of scour depth by Lacey was 5.36 and the cut-off pile for upstream and downstream were 2.62m and 6.65m respectively the provided cut-off depth were shorter for upstream and downstream than the calculated also the downstream impervious floor was more shorter than design length these also cause the scouring of downstream river bed as shown on field. Also the absence of downstream protection structure more expose the river bed for scouring and destruction of downstream impervious floor. The other cause for destruction of downstream floor was improper use of construction material as observed on the field study.

The overturning and sliding stability factor for weir body were 2.4 and 1.43 this shows the weir was safe against overturning but not sliding. The upstream retaining wall and wing wall was safe against overturning and sliding but not safe for overstress and bearing capacity of foundation as analysis result. The downstream retaining wall was safe for stability against overturning, but not for sliding, overstress and bearing capacity.

The provided thickness of floor is not adequate to protect the uplift of pressure due to seepage pressure but the exit gradient of the foundation soil was in the recommended limit.

The seepage flow underneath of Offiya weir foundation for static and dynamic condition were 1.3479 x 10^-5 m3/s/m and 1.2131 x 10^-5 m3/s/m respectively. The exit gradient of the weir foundation was 0.225 whereas the recommended maximum safe exit gradient was 0.17 for fine grained foundation soil; therefore the foundation not safe for saturated silt soil beneath of the weir.

5.2 Recommendations

The weir structure should have designed using its own dataset. However, data from Bonga meteorological station, which is found at very far distance from the weir site, was used for this design. Rational method was wrongly used in the design process of the structure. We observed that this is unacceptable state-of-the-art. Thus, Usage of data from the reasonably nearby meteorological station for peak rainfall analysis and selecting the best method for hydrological analysis rather than using rational method is recommendable to get best and more accurate flow condition for such ungagged catchment. Furthermore;

1. Provision of river training work mainly at right bank of the river is desirable to protect flowing of water out of the bank and also increasing the height of upstream retaining wall and wing wall so as to safely control the surplus discharge from weir.
2. Providing downstream wing wall based on downstream water level is also observed to be essential.
3. The under sluice gate should regularly be operated following the consistent operating time so as to allow the water to pass through under sluice during the rainy season to avoid the accumulation of silt at upstream of the weir axis.
4. Geological investigation of an area should be done with great care to get the actual output which may not be the same geological feature at the thinnest top layer and at the depth.

5. Provisions of adequate thickness of the down and upstream impervious floor for restoring the uplift pressure and avoiding the scouring of river bed are desirable.
6. Increasing the depth of upstream and downstream cut-off pile to reduce the underneath seepage flow more safely is essential.
7. The provision of intermediate pile was more adequate in design practice to reduce the uplift pressure and seepage of water under downstream impervious floor.
8. The proper using of construction material with design aspect and checking the quality of material during construction was more advantageous to avoided non functionality of weir.
9. Providing downstream protection work with appropriate thickness and length to avoid the scouring of downstream river bed is required.

REFERENCE

ABDULKAREEM, J. etal. 2018. Review of studies on hydrological modelling in Malaysia. Modeling Earth Systems and Environment, 4**,** 1577-1605.

ADSWE, 2010. Gobu Spate Irrigation project hydrology report. Bahir Dar,Ehiopia: ANRS Water Resource Development BUREAU

AFERA, A. 2004. Appraisal of Current Design Prcatices for River Diversion Structures.

AL SIAEDE, R. 2019. A Practical Geotechnical Analysis of in situ Stress Variations and Hydraulic Stability of Small Weirs Using SEEP/W and SIGMA/W Simulation. Geology and Geotechnics, Missan University, .

ASAWA G. 2005. Irrigation and Water Resources Engineering New Delhi-110002, New Age International (p) Ltd.

AWULACHEWU, S. 2010. Irrigation potential in Ethiopia. Constraints and Opportunities for Enhancing the System; International Water Management Institute: Addis Ababa, Ethiopia.

DESALEGN, K. 2017. Appraisal of Design Practice and Failure of River Diversion for Irrigation Schemes: A Case of Wadla Woreda North Wollo, Ethiopia.

ERTIRO 2017. Evaluation of Failures and Design Practices of River Diversion Structures for Irrigation: A Revisit of Two SSI Schemes in Ethiopia. International journal of Earth science and engineering.

FIKRU. 2015. Performance Assesment of diversion headwork for irrigation case study on fentale irrigation based integrated development project. RESEARCH, Addis Abeba.

GARG, S. 2011. Irrigation engineering and hydraulic structure,, NewDelhi Khanna.

HORA, M. 2016. Assessment of Design Practices and Performance of Diversion Weir in Small Scale Irrigation (Case Study From Projects in Western Oromia, Ethiopia)

KHASSAF, S. *etal.* Seepage analysis underneath Diyala weir foundation. Proceedings of the Thirteen International Water Technology Conference, IWTC, Hurghada, Egypt, 2009. Citeseer, 12-15.

MEKURIA. 20219. Investigation of The Sustainability of Urban Water Supply System in Bonga Town, South Western Ethiopia. JIMMA

MEKURIA, Z. 2019. Investigation of The Sustainability of Urban Water Supply System in Bonga Town, South Western Ethiopia. JIMMA

MOA 2018. National Guidelines for Small Scale Irrigation Development In Ehiopia. In: AGRICULTURE, M. O. (ed.) 1ST ed. A.A.Ethiopia: Agricultural Growth program.

NOVAK, P., MOFFAT ALB, NALLURI C, 2007. Hydraulic Structures, London and Newyork, Taylor and Francis Group.

ROBEL, L. 2005. Assesment of Design Practice and Perfomance of small-scale irrigation structure in south region MSC, Arba Minch

SUBRAMANYA 2008. Egineering Hydrology New DELHI, TataMcGraw-hiill publishing Company.

TADESSE. 2016. Investigation on the causes of failure of Tana Beles Weir. Addis Ababa University.

TSEDEKE, M. 2017. Final Study and Detail Engineering Design Report for Offiya SSI project. Hawassa, Ethiopia: SNNPRS Irrigation Development and Schemes Administration Agency.

APPENDICES

Appendix (A)

Table 1:- Annual maximum 24 hr rainfall for Normal and Gumbel's distribution method

YEAR	Max RF 24 (hr)	YEAR	Max RF 24 (hr)	YEAR	Max RF 24 (hr)
1985	70.1	1997	57.3	2009	75.9
1986	45.1	1998	65.2	2010	60.5
1987	60.2	1999	66.6	2011	70.6
1988	35.1	2000	51	2012	52.4
1989	40.1	2001	54.3	2013	40
1990	22.1	2002	61.2	2014	63.1
1991	17.5	2003	68.9	2015	33.8
1992	40.7	2004	58.2	2016	42.4
1993	50.5	2005	44	2017	60
1994	66.7	2006	64	2018	43.2
1995	63.4	2007	76.2	2019	12.056
1996	70	2008	84		

Mean	53.89589
Standard Deviation	16.8849
Minimum	12.056
Maximum	84
Count	35

Table 2:- Maximum rainfall for different return period by normal distribution method

Normal distribution method				
T	P	W	KT	XT
10	0.1	2.302585	1.646251	81.69266
25	0.04	3.218876	2.794209	101.0758
50	0.02	3.912023	3.598444	114.6552
100	0.01	4.60517	4.367596	127.6423

Table 3:- Maximum rainfall for different return period by Gumbel's distribution method

Gumbel's distribution method			
T	YT	KT	XT(mm)

10	2.250367	2.076763	88.96181
25	3.198534	3.831978	118.5984
50	3.901939	5.134096	140.5846
100	4.600149	6.4266	162.4084

Table 4:- LOG NORMAL and log- person type III distribution

YEAR	Max RF 24 (hr)	Z=LOGX	YEAR	Max RF 24 (hr)	Z=LOGX	YEAR	Max RF 24 (hr)	Z=LOGX
1985	84	1.924279	1997	63.1	1.800029	2009	44	1.643453
1986	76.2	1.881955	1998	61.2	1.786751	2010	43.2	1.635484
1987	75.9	1.880242	1999	60.5	1.781755	2011	42.4	1.627366
1988	70.6	1.848805	2000	60.2	1.779596	2012	40.7	1.609594
1989	70.1	1.845718	2001	60	1.778151	2013	40.1	1.603144
1990	70	1.845098	2002	58.2	1.764923	2014	40	1.60206
1991	68.9	1.838219	2003	57.3	1.758155	2015	35.1	1.545307
1992	66.7	1.824126	2004	54.3	1.7348	2016	33.8	1.528917
1993	66.6	1.823474	2005	52.4	1.719331	2017	22.1	1.344392
1994	65.2	1.814248	2006	51	1.70757	2018	17.5	1.243038
1995	64	1.80618	2007	50.5	1.703291	2019	12.056	1.081203
1996	63.4	1.802089	2008	45.1	1.654177			

Mean	1.701912
Standard Deviation	0.181811
Skewness	-1.81416
Range	0.843076
Minimum	1.081203
Maximum	1.924279
Count	35

Table 5:- Maximum rainfall for different return period by LOG NORMAL and log- person type III distribution method

T	KT value for		Calculated ZT value		XT=antilog(ZT) mm	
	Log-p type III	Log	Log-p type III	Log normal	Log-p type	Log normal

		normal			III	
10	0.9414246	1.282	1.873073	1.934993	74.65745	86.09807
25	1.0297962	1.751	1.88914	2.020263	77.47116	104.7762
50	1.0630174	2.054	1.89518	2.075351	78.55612	118.9464
100	1.0805572	2.326	1.898369	2.124804	79.13507	133.2919

Table 6:- Incremental depth of precipitation

time (Hr)	Hourly p=M*sqrt(T). (mm)	Incremental Depth (mm)	Time Interval(Hr)	Precipitation using Alternate Method	cumulative
1	25.82705	25.82705	0-1	2.664008	2.664008
2	36.52497	10.69792	1-2	2.785191	5.449199
3	44.73377	8.208799	2-3	2.924578	8.373777
4	51.6541	6.920338	3-4	3.087238	11.46102
5	57.75104	6.09694	4-5	3.280466	14.74148
6	63.2631	5.512055	5-6	3.51522	18.2567
7	68.33196	5.068858	6-7	3.808892	22.06559
8	73.04994	4.717978	7-8	4.191154	26.25675
9	77.48116	4.431222	8-9	4.717978	30.97472
10	81.67231	4.191154	9-10	5.512055	36.48678
11	85.65864	3.986331	10-11	6.920338	43.40712
12	89.46753	3.808892	11-12	10.69792	54.10503
13	93.12076	3.653228	12-13	25.82705	79.93208
14	96.63598	3.51522	13-14	8.208799	88.14088
15	100.0277	3.391762	14-15	6.09694	94.23782
16	103.3082	3.280466	15-16	5.068858	99.30668
17	106.4877	3.179455	16-17	4.431222	103.7379
18	109.5749	3.087238	17-18	3.986331	107.7242
19	112.5775	3.002608	18-19	3.653228	111.3775
20	115.5021	2.924578	19-20	3.391762	114.7692
21	118.3544	2.852333	20-21	3.179455	117.9487
22	121.1396	2.785191	21-22	3.002608	120.9513
23	123.8622	2.722578	22-23	2.852333	123.8036
24	126.5262	2.664008	23-24	2.722578	126.5262

Appendix (B)

Table 1:- Runoff CN-II for hydrologic soil cove complex under AMC-II condition

Land Use	Cover		Hydrologic soil group			
	Treatment or practice	Hydrologic condition	A	B	C	D
Cultivated	Straight row		76	86	90	93
Cultivated	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
Cultivated	Contoured &	Poor	66	74	80	82
	Terraced	Good	62	71	77	81
Cultivated	Bunded	Poor	67	75	81	83
		Good	59	69	76	79
Cultivated	Paddy		95	95	95	95
Orchards	With understory cover		39	53	67	71
	Without understory cover		41	55	69	73
Forest	Dense		26	40	58	61
	Open		28	44	60	64
	Scrub		33	47	64	67
Pasture	Poor		68	79	86	89
	Fair		49	69	79	84
	Good		39	61	74	80
Wasteland			71	80	85	88
Roads (dirt)			73	83	88	90
Hard surface areas			77	86	91	93

Source: - Engineering hydrology book (Subramanya)

Table 2:- Calculated Runoff CN-III for hydrologic soil cove complex under AMC-III condition

LULC	SOIL GROUP C (36.74188%)				WEIGHTED	AMCIII
	AREA	%	CNII	PRODUCT	CNII	CNIII
Tree/forest	0.089	0.326412	60	0.1958471		
Grass Land	0.0291	0.106726	79	0.0843133		
Agricultural Land	9.2763	34.02129	85	28.9181		92.2249024
Scrub/Shrub	0	0	0	0		
Village Or Built	0.6237	2.287451	90	20.58706	83.5116616	

Area				
TOTAL	10.0181	36.74188		31.25697

LULC	SOIL GROUP D (38.97445%)			
	AREA	%	CNII	PRODUCT
Tree/forest	0.163	0.58013	64	0.3712832
Grass Land	0.0326	0.116026	89	0.1032631
Agricultural Land	10.4128	37.06001	89	32.98341
Scrub/Shrub	0.1978	0.703987	86	0.6054288
Village Or Built Area	0.1445	0.514287	92	0.473144
TOTAL	10.9507	38.97444		34.53653

LULC	SOIL GROUP B (24.28368%)			
	AREA	%	CNII	PRODUCT
Tree/forest	0.8119	2.988483	44	1.314933
Grass Land	0.0067	0.024662	79	0.019482
Agricultural Land	4.3744	16.10151	77	12.39816
Scrub/Shrub	1.3858	5.100923	77	3.927711
Village Or Built Area	0.0185	0.068096	85	0.0578816
TOTAL	6.5973	24.28367		17.71817

Table 3:- Sub catchment parameter for HEC-HMS Modeling

	sub catchement1						
OID	Length	Z_Min	Z_Max	slope	Tc	area (KM^2)	Lag time

1	1755	1958.0	2135.99	0.101432	0.24	10.3742	0.7116
2	687	1385.0	1455	0.101	0.119		
3	714	1455.0	1493.0	0.001	0.158		
4	1247	1493.0	1663	0.001	0.169		
5	2085	1663.0	1884	0.106	0.277		
6	1162	1883.99	1958.0	0.06	0.215		
total	7648.9875				1.186		
sub catchement2							
OID	Length	Z_Min	Z_Max	slope	Tc	area (KM^2)	Lag time
1	2201	1947.99	2202.99	0.115831	0.279	16.92865	0.744
2	890	1390.0	1467.00	0.086	0.155		
3	1989	1467.0	1593.0	0.063	0.325		
4	1929	1593.00	1757.99	0.08	0.28		
5	1505	1757.9	1947.99	0.12	0.201		
total	8515.04				1.24		
sub catchement3							
OID	Length	Z_Min	Z_Max	slope	Tc	area (KM^2)	Lag time
1	1706	1339.0	1543.00	0.119610	0.22	1.124026	0.132

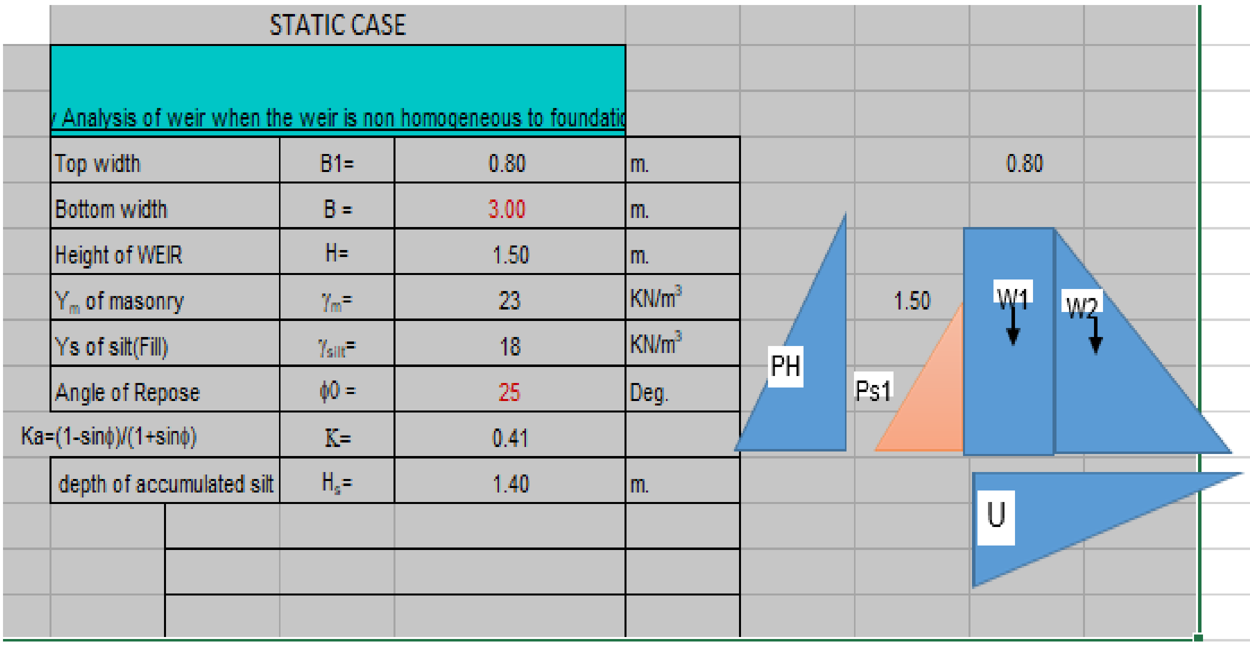
Appendix (C)

Table 1:- River cross-section elevation difference left to right bank

chainage	Elevation(m)	remark
0.00	1,328.50	
1.00	1,328.42	
1.50	1,328.10	
2.00	1,328.00	
3.00	1,327.99	max.F.L
4.00	1,327.80	
4.30	1,327.79	left River bank edge
5.00	1,327.50	
6.00	1,327.05	left River bed edge
7.00	1,325.63	river center
8.00	1,325.77	
9.00	1,325.77	

10.00	1,325.82	
11.00	1,326.35	right bank
12.00	1,326.45	
13.00	1,326.59	
14.00	1,326.68	
15.60	1,326.77	
16.00	1,326.87	
17.00	1,327.49	
18.00	1,327.64	
20.00	1,327.75	
21.00	1,327.89	
22.00	1,327.92	
23.00	1,327.99	max.F.L
28.00	1,328.23	
33.00	1,328.30	
37.00	1,328.50	

Appendix (D)


STATIC CASE

/ Analysis of weir when the weir is non homogeneous to foundatic

Top width	B1=	0.80	m.
Bottom width	B =	3.00	m.
Height of WEIR	H=	1.50	m.
Y_m of masonry	γ_m=	23	KN/m^3
Ys of silt(Fill)	γ_{silt}=	18	KN/m^3
Angle of Repose	$\phi 0$ =	25	Deg.
Ka=(1-sinϕ)/(1+sinϕ)	K=	0.41	
depth of accumulated silt	H_s=	1.40	m.

Figure 1: - force acting on weir in static case.

Table 1:-stability analysis of weir body in static case

Item Description	Forces(KN.m) Vertical +ve	Vertical -ve	Horizontal +ve	Horizontal -ve	Lever arm (m.)	Moment about toe(KN.m) Resisting	Overturning
Bering capacity of Foundation material gravel material		80	KN/m²				
1. Vertical force							
1.1. Self weight(W1)	27.60				2.6	71.76	
1.2. Self Weight(W2)	37.95				1.47	55.66	
1.3 Uplift pressure(U)		-22.0725			2.00		-44.145
2.Horizontal force							
upstream water pressure				-11.03625	0.50		-5.518125
1.3. Silt preasure(Ps1)				-7.16	0.47	0.00	-3.3410273
TOTAL	**65.55**	**-22.07**	**0.00**	**-18.20**		**127.42**	**-53.00**

I) Overturning Stability		$F_o=\Sigma M_r/\Sigma M_o$		2.40	Safe	1.50	
II) Sliding Stability		$\frac{\eta \sum F_v}{\sum H_r} > 1.5$		1.43	Not Safe	1.5	
III) Overstressing Stability			ΣM=Mr+Mo	74.42			
$e=\left\|\frac{\sum M}{\sum F_v}-\frac{L}{2}\right\|<\frac{L}{6}$			ΣM/ΣFv	1.7115944			
			L/2	1.50			
			e=	0.2116	safe	0.50	
IV) Bearing Capacity		P=ΣV/B(1+6e/B)					
Max compression stress at the toe		80	Pmax =	20.63	Safe	80	KN/m2
Tension develop at the heel		80	P =	8.36	Safe	80	KN/m2

Stability Analysis of U/s Retaining wall			
Top width	B1=	0.50	m.
Bottom width	B =	3.00	m.
Height of wall	H=	3.75	m.
FoundatioN Thickness	t=	1.20	m.
γ_m of masonry	γ_m=	23	KN/m³
Ys of silt(Fill)	γ_{silt}=	18	KN/m³
γ_{co} of masonry	γ_{RCC}=	25	KN/m³
Angle of Repose	ϕ1 =	25	Deg.
Ka=(1-sinϕ)/(1+sinϕ)	K=	0.41	
depth of accumulated silt	H_s=	1.40	m.
aximum depth of back fill s	hs=	2.00	m.

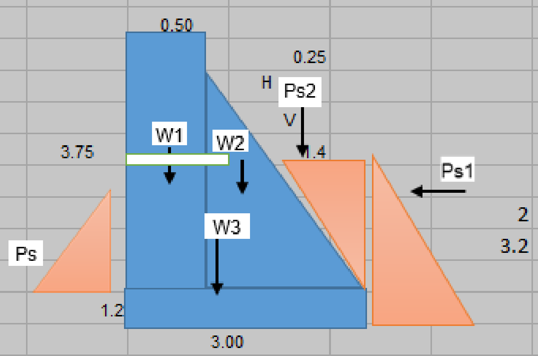


Figure 2:- force acting on U/S retaining wall

Table 2:-stability analysis of U/S Retaining wall in empty condition.

Bering capacity of Foundation material gravel material		80	KN/m^2				
Item Description	**Forces(KN.m)**				**Lever arm**	**Moment about toe(KN.m)**	
	Vertical		Horizontal		(m.)	Resisting	Overturning
	+ve	-ve	+ve	-ve			
1. Vertical force							
1.1. Self weight(W1)	43.13				0.25	10.78	
1.2. Self Weight(W2)	100.63				1.33	134.17	
1.3 Self weight(w3)	82.80				1.50		
1.3. weight of soil (Ps2)	25.20				1.43	36.12	
2. Horizontal force							
2.2. Active preasure (Ps)			7.159344		0.47	3.3410273	
2.3. Active preasure (Ps1)				-37.403921	1.07		-39.90
TOTAL	**251.75**	**0.00**	**7.16**	**-37.40**		**184.41**	**-39.90**

I)	Overturning Stability	$F_o=\Sigma M_r/\Sigma M_o$		4.62	Safe	1.50	
II)	Sliding Stability	$F_{ss}=\mu\Sigma V/\Sigma H$		4.99	Safe	1.5	
III)	Overstressing Stability		ΣM=Mr+Mo	144.51			
	$e=\left\lvert\frac{\sum M}{\sum F_v}-\frac{L}{2}\right\rvert<\frac{L}{6}$		ΣM/ΣFv	0.5740275			
			L/2	1.50			
			e=	0.9260	Not safe	0.50	
IV)	Bearing Capacity	P=ΣV/B(1+6e/B)					
	Max compression stress at the toe	80	Pmax =	239.33	Unsafe	80	KN/m2
	Tension develop at the heel	80	P =	-71.49	Safe	80	KN/m2

Stability Analysis of U/s wing wall			
Top width	B1=	0.50	m.
Bottom width	B =	2.50	m.
Height of wall	H=	2.700	m.
FoundatioN Thickness	t=	1.20	m.
Y_m of masonry	γ_m=	23	KN/m^2
Ys of dry compucted back fill	γ_{silt}=	18.5	KN/m^2
Y_{co} of masonry	γ_{RCC}=	25	
Angle of Repose	ϕ1 =	25	Deg.
Ka=(1-sinϕ)/(1+sinϕ)	K=	0.41	
bt =D/2-D		0.60	m
bh =10-15cm		0.15	m

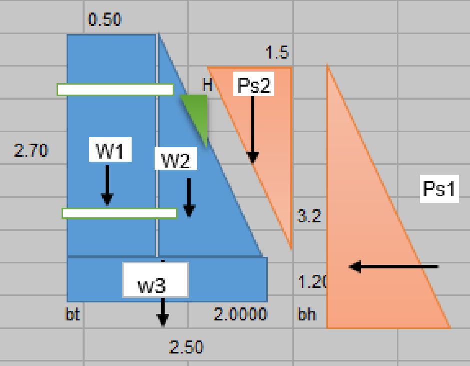


Figure:3 - The force acting on U/S wing wall

Table 3:- Stability analysis of u/s wing wall

Item Description	Forces(KN) Vertical +ve	Vertical -ve	Horizontal +ve	Horizontal -ve	Lever arm (m.)	Moment about toe(KN.m) Resisting	Overturning
Bering capacity of Foundation material gravel material		80	KN/m^2				
1. Vertical force							
1.1. Self weight(W1)	31.05				0.25	7.76	
1.2. Self Weight(W2)	62.10				1.17	72.45	
1.3. Silt preasure(Ps2)	27.75				2.00	55.50	
2. Horizontal force							
2.2. Active preasure (Ps1)				-38.442919	1.23		-47.41
TOTAL	**120.90**	**0.00**	**0.00**	**-38.44**		**135.71**	**-47.41**
I) Overturning Stability		$F_o=\Sigma M_r/\Sigma M_o$		2.86	Safe	1.50	
II) Sliding Stability		$F_{ss}=\mu\Sigma H/\Sigma V$		1.89	Safe	1.5	
IV) Overstressing Stability			$\Sigma M=Mr+Mo$	88.30			
			$\Sigma M/\Sigma Fv$	0.7303521			
			L/2	1.25			
			e=	0.5196	Not safe	0.42	
V) Bearing Capacity		$P=\Sigma V/B(1+6e/B)$					
Max compression stress at the toe		=	Pmax=	108.67	Unsafe	80	
Tension develop at the heel		=	Pmin=	-11.95	Safe	80	

Stability Analysis of D/S Retaining wall			
Top width	B1=	0.50	m.
Bottom width	B =	3.00	m.
Height of wall	H=	3.650	m.
FoundatioN Thickness	t=	1.20	m.
Y_m of masonry	γ_m=	23	KN/m^2
Ys of dry compucted bacl	γ_{silt}=	18.5	KN/m^2
Y_{co} of masonry	γ_{RCC}=	25	
Angle of Repose	$\phi 1$ =	25	Deg.
Ka=(1-sinϕ)/(1+sinϕ)	K=	0.41	
bt =D/2-D		0.60	m
bh =10-15cm		0.15	m

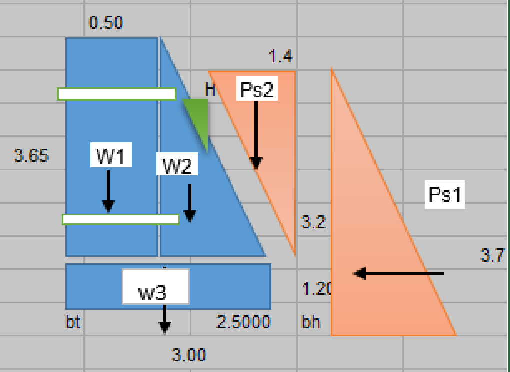


Figure 4:- Force acting D/S retaining wall

Table 4:-Stability analysis of D/S Retaining wall in empty condition.

Bering capacity of foundation material gravel material			80	KN/m^2				
	Item Description	Forces(KN)				Lever arm	Moment about toe(KN.m)	
		Vertical		Horizontal		(m.)	Resisting	Overturning
		+ve	-ve	+ve	-ve			
1. Vertical force								
1.1. Self weight(W1)		41.98				0.25	10.49	
1.2. Self Weight(W2)		104.94				1.33	139.92	
3. weight of soil preasure(Ps2		38.20				1.93	73.86	
2. Horizontal force								
2.2. Active preasure (Ps1)					-76.885837	1.23		-94.83
TOTAL		**185.12**	**0.00**	**0.00**	**-76.89**		**224.27**	**-94.83**
I)	Overturning Stability		$F_o=\Sigma M_r/\Sigma M_o$		2.37	Safe	1.50	
II)	Sliding Stability		$F_{ss}=\mu\Sigma H/\Sigma V$		1.44	Not Safe	1.5	
IV)	Overstressing Stability			ΣM=Mr+Mo	129.44			
				ΣM/ΣFv	0.6992557			
				L/2	1.50			
				e=	0.8007	Not safe	0.50	
V)	Bearing Capacity		P=ΣV/B(1+6e/B)					
Max compression stress at the toe			=	Pmax=	216.35	Unsafe	80	
Tension develop at the heel			=	Pmin=	-68.25	Safe	80	

Table 4 Internal angle of friction (Ø) of Soil

SN	Soil Type	Angle of internal friction, ϕ
	Gravel	40^0-55^0
	Sand-Gravel	35^0-50^0
	Sand-Loose	28^0-34^0
	Sand-Dense	34^0-45^0
	Silt, silty sand- Loose	20^0-22^0
	Silt, silty sand- Dense	25^0-30^0

Note: For small structures a conservative value of ϕ=25^0 is commonly used

Table 5:- Particle Size Distribution and Corresponding Permeability Coefficient

Classification	Clay	Silty Clay	Silty sand	Fine sand	Medium sand	Course sand	Gravel

d (mm)	0~0.01	0.01~0.05	0.05~0.10	0.10~0.25	0.25~0.50	0.50~1.0	1.0~5.0
K (cm/s)	0.000003	0.00045	0.0035	0.015	0.085	0.35	3.0

Note: d is average or 50% of grain size distribution, mm.

Appendix (E)

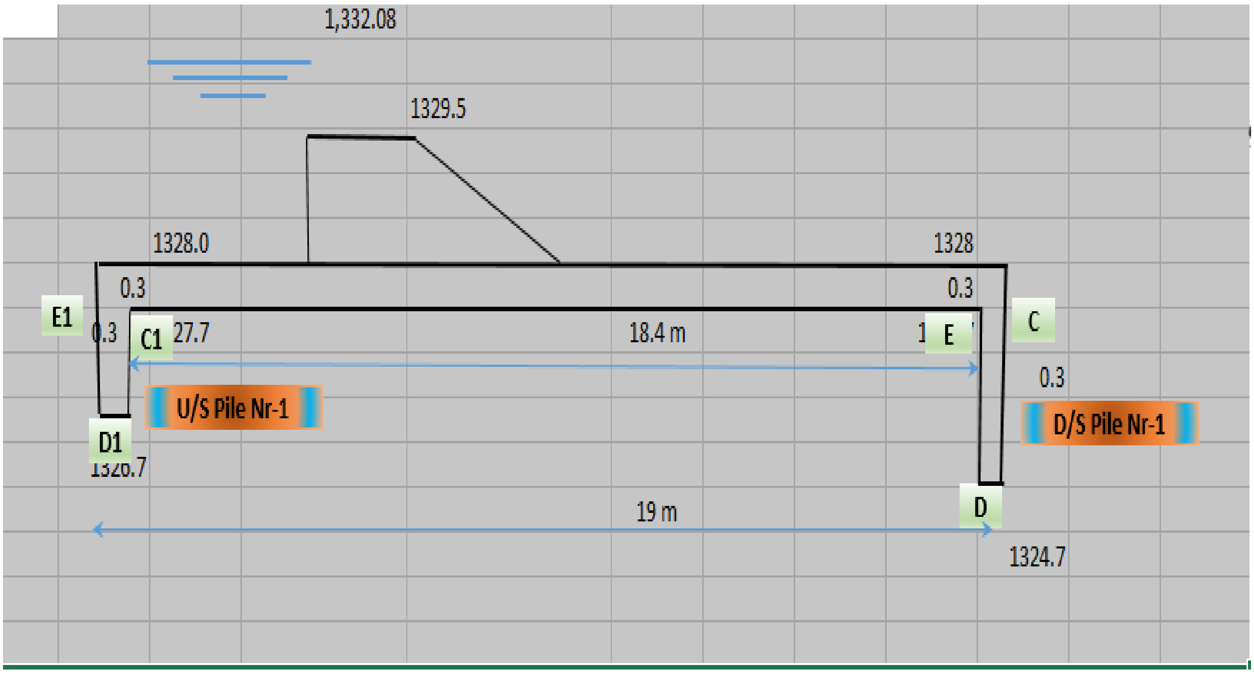


Figure 1:-Weir profile for khosla's theory

(1) For Upstream Pile Line No. (1)		
Total length of the floor = b =	19 m	
Depth of u/s pile line =d=	1.0	
α=b/d=	19.00	
1/α=	0.053	
$\lambda = \frac{1}{2}\left[1+\sqrt{1+\alpha^2}\right]$	10.031	
From the curve Plate, ϕ_{c1} =	79%	
From same curve, ϕ_{d1} =	86%	
These values of ϕ_{c1} & ϕ_{d1} must be corrected for three corrections as follow		
Corrections for ϕc1		
(a) Correction at C1 for Mutual Inteiference of Piles. ϕc1 is affected by D/S pile Nr-2.		
$\text{Correction} = 19\sqrt{\frac{D}{b}}\left(\frac{d+D}{b}\right)$		
Where, D = Depth of pile Nr-2=	3.0	
d=Depthofpile Nr-1 =	1.0	
b' = Distance b/n two piles =	18.4 m	
b = Total floor length =	19 m	
Thus, Correction	1.62%	
Note: Since the point C1 is in the rear of direction of flow, the correction is +ve.		
:. Correction due to pile interference on C1=	1.62%	
(b) Correction at C1 due to thickness of floor.		
Pressure calculated from curve is at C1',but we want pressure at C1. Pressure at C1 shall be		
more than at C1' as direction of flow is from C1 to C1' as shown; & hence, the correction will be +ve &		
Thus, Correction	1.86%	(+ ve)
(c) Correction due to slope at C1 is nil, as this point is neither situated at the start nor at the end of a slope.		
:. Corrected ϕc1 = 79% + 1.62% + 1.86% =	82.93%	
Hence, corrected ϕc1 =	82.93%	Ans
and ϕD1 =	86%	

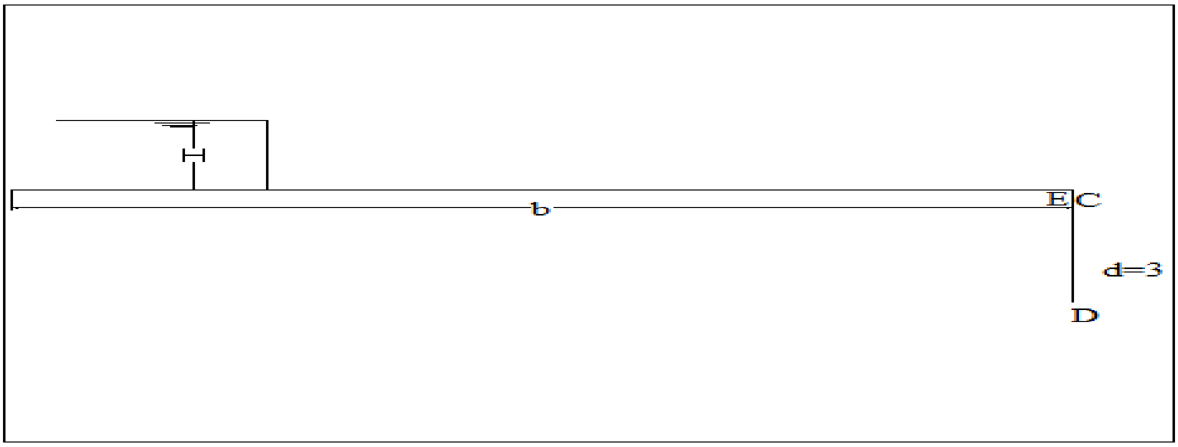


Figure 2:- D/S pile profile.

(3) Downstream Pile Line		
D/S pile depth	3 m	
b = Total floor length =	19 m	
α=b/d=	6 m	
$\lambda = \frac{1}{2}\left[1 + \sqrt{1 + \alpha^2}\right]$	3.706 m	
ϕD =	23.90%	
ϕE =	34.70%	
Corrections for α_{E3} =		
(a) Correction due to piles. Point E is affected by pile Nr-1, & since Eis in the forward direction of flow from pile Nr-2, this correction is negative & Its amount is given by:		
$\text{Correction} = 19\sqrt{\frac{D}{b'}}\left(\frac{d+D}{b}\right)$		
here, D=Depth of pile Nr-1, effect of w/h is consider	1.0 m	
d= Depth of pile Nr-2, effect on which is	3.0	
b' = Distance between piles =	18.4 m	
b = Total floor length =	19 m	
Thus, correction =	-0.93%	
(b) Correction due to floor thickness		
It can be stated easily that pressure at E shall be		
less than at E' & since pressure observed from	-1.08%	0
(c) Correction due to slope at E3 is nil, as the point E3 is neither situated at start nor at end of any slope		
Hence, corrected ϕE3 =	36.71%	Ans

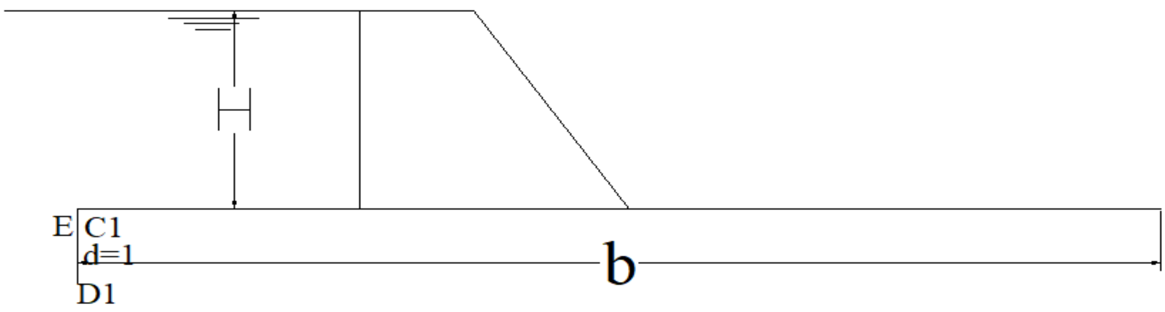


Figure 3:- U/s profile for khosla's theory

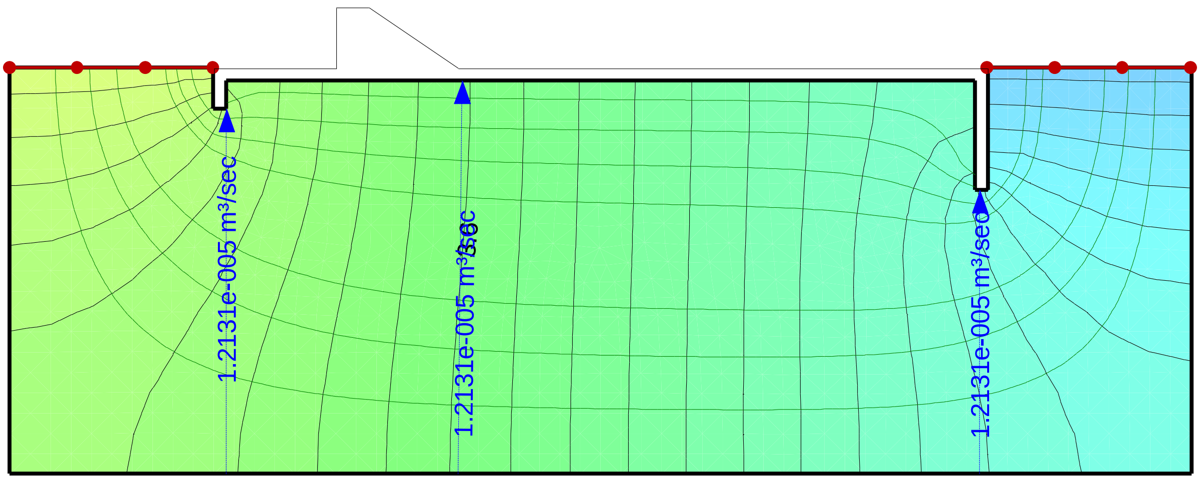


Figure 3:- Seepage analysis for constructed weir at high flood water condition

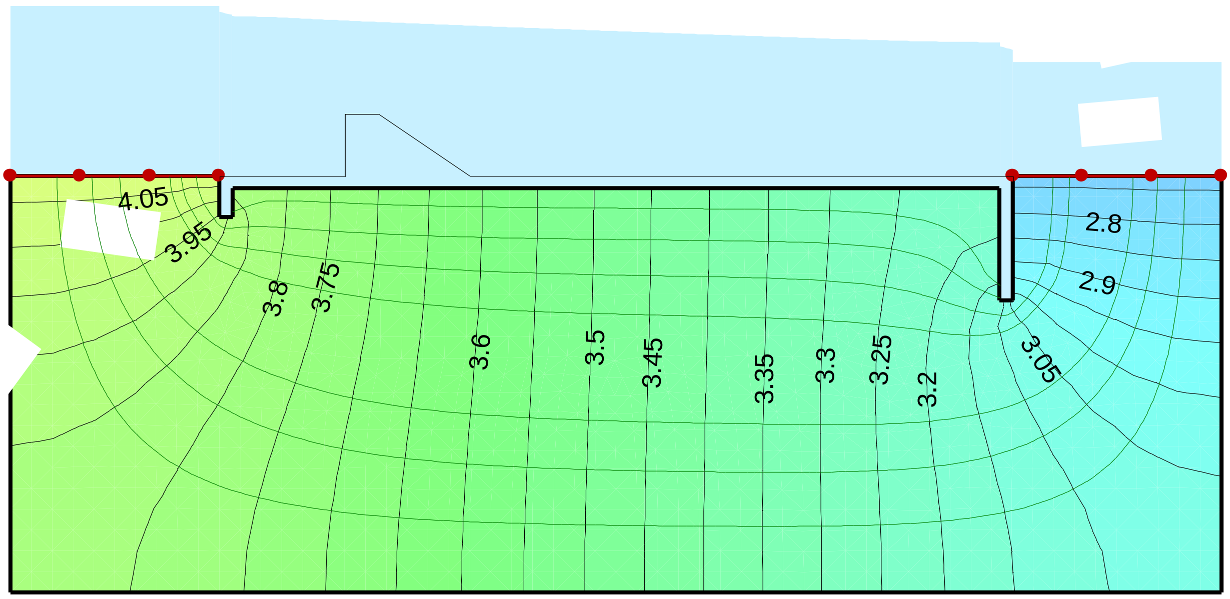


Figure 4:- Uplift pressure head distribution on D/S floor water at high flood level.

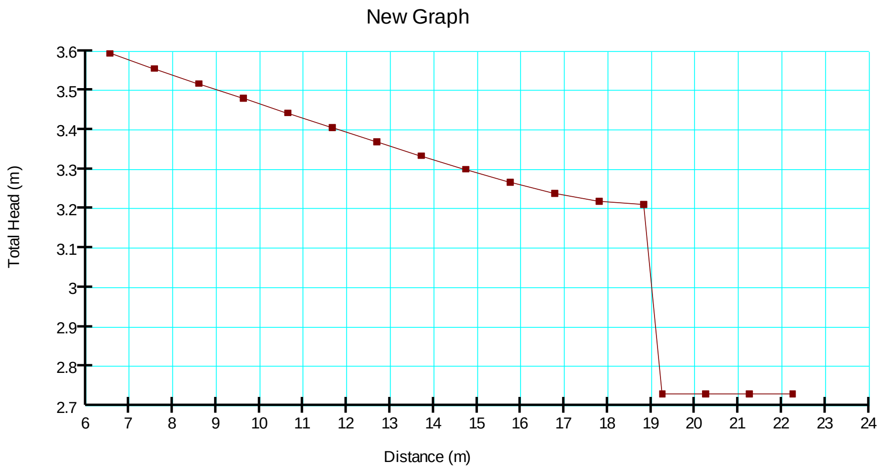


Figure 5:- Uplift pressure head distribution on D/S floor water at high flood level.