Investigation on the causes of failure of Tana Beles Weir

Master of Science Thesis

By

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Declaration

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, a required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Seife Tadesse

Signature………………………

Abstract

An investigation on Tana Beles diversion head-work is carried out for this research work and attempt is made to understand the failure problems encountered. This serve for different purposes such as for construction of new diversion structure (either as replacement for existing structures, or as an entirely new structure), rehabilitation of existing structures (from minor repairs to complete re-engineering either to maintain existing function, or to meet new requirements).

Hydrology and hydraulic analysis, structural analysis and evaluation of the foundation condition of the weir has been made to investigate the causes for the failure of the weir.

The hydrology analyses is made by Hec-hms and the result shows that the peak flood used for the design of the weir is smaller than the result obtained in this investigation, but the water way of the weir built can accommodate the estimated peak flood. Models used for the hydrology analysis include Global Mapper, Arc-GIS, Hec-Geohms and Hec-hms. On the other side, hydraulic design is evaluated and the evaluation exposes the absence of concrete floor, absence of energy dissipater and protection works on the hydraulic design of the weir and clogging of under sluice as a cause for the failure of the weir. The structural analysis involves checking stability analysis of the weir against overturning, sliding and shear. Surveying data upstream and downstream of the weir is collected and foundation condition were investigated by comparing the collected surveying data with the design bed level.in addition to this the applied foundation treatment is evaluated by comparing it with standards and found that the main structure was built on weak foundation which activates the weir failure. This investigation shows that the main cause of Tana Beles weir failure is due to the weak foundation condition of the weir and absence of the necessary concrete floor and energy dissipater on its design. Finally, the remedial design for the weir is recommended.

Key words: Analysis, Concrete Floor, Diversion, Downstream, Energy Dissipater, Gates, Head work, Regulator, Scour, Seepage, Silt, Stability, Upstream, Under-sluice, Weir.

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List of Symbols

A = Area of opening

B = Total length of the floor

B =Width of the weir base.

C_d=Discharge coefficient

D =depth of downstream cut off below downstream floor level

e = eccentricity

f = Silt factor for foundation material

g = Gravity, 9.8 m/s

G =Density of construction material for apron

GE = Safe exit gradient

H = Maximum static head (pond level -downstream floor level)

E_f = specific energy

R = scour depth

D=depth of pile no.2

d=depth of pile no.1

b'=distance b/n two piles

1 Introduction

1.1 General

Ethiopia has considerable development potentials and its land and water resources are adequate relative to its population (Awulachew, 2010). However, crop production is not enough to fulfill the food requirements of the country and most of the agricultural production is based on rain fed farming (Lambiso, 2005). Variable rainfall and frequent drought conditions have adverse effects on rain fed dependent farming, which has negative impacts on Ethiopia's economy. It is reported that due to floods, droughts and lack of water infrastructures Ethiopia is at risk of chronic food shortage . One of the best alternatives for achieving food security is expanding irrigation on various scales through river diversion, constructing micro-dams and water harvesting structures etc. (Awulachew, 2010)

As the demand for sugar is increasing at alarming rate as a result of improving living standard, increasing global demand and its multipurpose application, encroach upon these lowland plains for sugar production is inevitable. It is in this regard that the Government of Ethiopia set a program to develop sugar industry that can satisfy the increasing demand and generate additional wealth to help boost the economy. The plan demands preparation of biophysically suitable sites for irrigation sugarcane plantation that is also favorable in their infrastructure development status. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Irrigation and Drainage Report, 2013)

The Ethiopian Sugar Corporation with an objective of raising the sugar production of the country planned to develop about 250,000 hectare of land for cane production in GTP I and II. One of the potential sites selected for cane production is Tana Beles Integrated Sugar Development Project. The project was initially started by Amhara National Regional State and later transferred to the Sugar Corporation. The productivity of Sugar cane at Tana Beles Sugar Development project fully depends on the hydraulic functioning of weirs and main canal structures. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Irrigation and Drainage Report, 2013)

Weirs are man-made barriers constructed across water courses for the benefit of mankind and have been playing a vital role in the development of any country by meeting the water demand for domestic use, irrigation, power generation, flood protection etc. they 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, flood control, domestic and industrial uses.

Construction of weirs has been a long-established practice with the oldest known weir, Mnjikaning Fish Weirs built by the first nations people well before recorded history, dating to about 3,300 BCE during the Archaic period in North America, according to carbon dating done on some of the wooden remnants. (wikipedia, n.d.)

Functions of diversion headwork implemented for irrigation are to raise the water level on its upstream side, regulate the supply of water in to canals, control entry of silt in to canals, creates a small pond (not reservoir) on its upstream and provides some pondage, help in controlling the fluctuation of water level in river during different seasons. (Garg, 2005)

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

A number of river diversion structures have been designed and constructed in the previous years. However, while some schemes are performing successfully, it has been observed in various reports that some of the schemes have failed to serve the purpose for which they are intended (Lambiso, 2005). Therefore it is important to deal with the causes of failure of diversion head works implemented for irrigation, hydropower and any other.

1.2 Statement of the problem

Tana Beles Integrated Sugar Development Project is located near Fendka Town, capital of Jawi Woreda, found in the western periphery of ANRS (Amhara National Regional State), 145 km from Bahir Dar. The project covers about 57,614 ha of land situated at Upper Ayma and Upper Beles mainly emphasized on 38,334 ha of land placed at Right side of Beles River. The project overall includes a total area of about 95,948 ha of land found in Jawi woreda of Amhara and Dangur woreda of Benishangul National Regional States. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

The diversion work for Tana Beles Integrated sugar Development Project is located at Beles River at about 28 km from Jawi / Fendka. Geographically the weir is located at 247847.76 Easting and 1289961.39 Northing. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

The Head Work is heavily damaged along the line of intersection between the weir and the divide wall. In addition to this, a crack on the surface of the weir is observed and the plastering is detached on some part of the weir.

The above problems would cause total failure of the head work structures, which eventually leads to failure on the irrigation scheme, downstream flooding and other related problems.

Therefore, studying the investigation for the cause of failure of the weir will be advantageous in taking the required measures so as to make the weir operational without further damage and to design other weirs with better care on the expected causes of failure.

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

1.3 Research Question

The following questions are the main factors that are dealt in this research work.

- What are the reasons for the failure of the weir?
- Is there any hydrologic, hydraulic and geotechnical problem which may cause the failure of the weir?
- Is there any structural problem such as failure due to uplift, overturning, shear and sliding?
- Is there any problem on the constructed diversion weir which may lead to failure?

This study will go deeper into scientific analysis to answer the above mentioned questions.

1.4 Objectives

1.4.1 General Objectives

The primary objective of this research work is

- To identify the major causes of the failure of the Tana Beles Irrigation Development Project diversion weir
- To recommend remedial design of the Weir.

1.4.2 Specific Objectives

- To estimate the peak flow and compare it with the design discharge of the weir
- To check the capacity of the weir by using the estimated peak flood
- To check the actual site condition of the headwork
- To evaluate the hydraulic design of the weir
- To evaluate the foundation condition of the weir
- To check stability analysis
- To recommend remedial design

1.5 Significance of the research

In Ethiopia recently lots of Irrigation projects were under construction and lot of design works were done for the future expansion for food security of the increasing population. The country has experienced many cases of failure of diversion head works and below capacity for many decades but there is no any information for the causes of their failures (Awulachew, 2010).
The following are some of the uses of this research work

- ✓ To identify the major causes of the failure of Tana Beles diversion headwork.
- ✓ This research work aims to provide some information to parties engaged in maintenance and improvement of the existing Tana Beles head work, so as to ensure that mistakes are avoided and opportunities are not missed.
- ✓ Investigation of the failure cause of diversion head works implemented for irrigation can also serve for different purposes such as for construction of new diversion structures, either

as a replacement for an existing structure, or as an entirely new structure, rehabilitation of existing structures, from minor repairs to complete re-engineering, either to maintain existing function, or to meet new requirements, decommissioning of a structure.

1.6 Organization of the thesis

The thesis is organized in to five chapters. Chapter one deals with introduction that covers the General background, the problem statement, objectives of the research, the research question, Significance of the research, and organization of the study. Chapter two deals with Literature review. Chapter three deals with the methods and materials used. Chapter four covers Hydrology analysis. Chapter five deals with evaluation of the foundation condition of the weir. Chapter six deals with hydraulic analysis. Chapter seven deals with structural analysis. Chapter eight deals with remedial design of the weir. Chapter nine deals with conclusion and recommendations.

2 Literature Review

2.1 Head work

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

Storage head work is a barrier constructed across the river valley to form the storage reservoir. The water is supplied to the canal from the reservoir through the canal head regulator. This serve as multipurpose functions like hydroelectric power generation, fishery, flood control, etc. (Garg, 2005).

Diversion headwork provides an obstruction across a river, so that the level of the water is raised and water is diverted to the channel at required level. The flow of water in the canal is controlled by the canal head regulator. This increased water level helps the flow of water by gravity and the increasing the commanded area and reducing the water fluctuation in the river (Garg, 2005).

Diversion headwork serves the following purposes: (Garg, 2005)

- To rise the water level at the head of the canal
- To control the intake of water into the canal.
- To control the entry of silt in to the canal and to control deposit of silt at the head of the canal.
- To store water for small period of time so that water is available throughout the year
- To control the fluctuation of water level in the river during different season

2.2 Components of Head works:

The essential components of head works (s.k.sharma, 2002) are:

- Weir
- Under sluices
- Canal head regulator
- Divide wall or groyne
- Piers and abutments
- Protection works
- River training works
- Fish ladder
- Silt excluder/silt prevention device

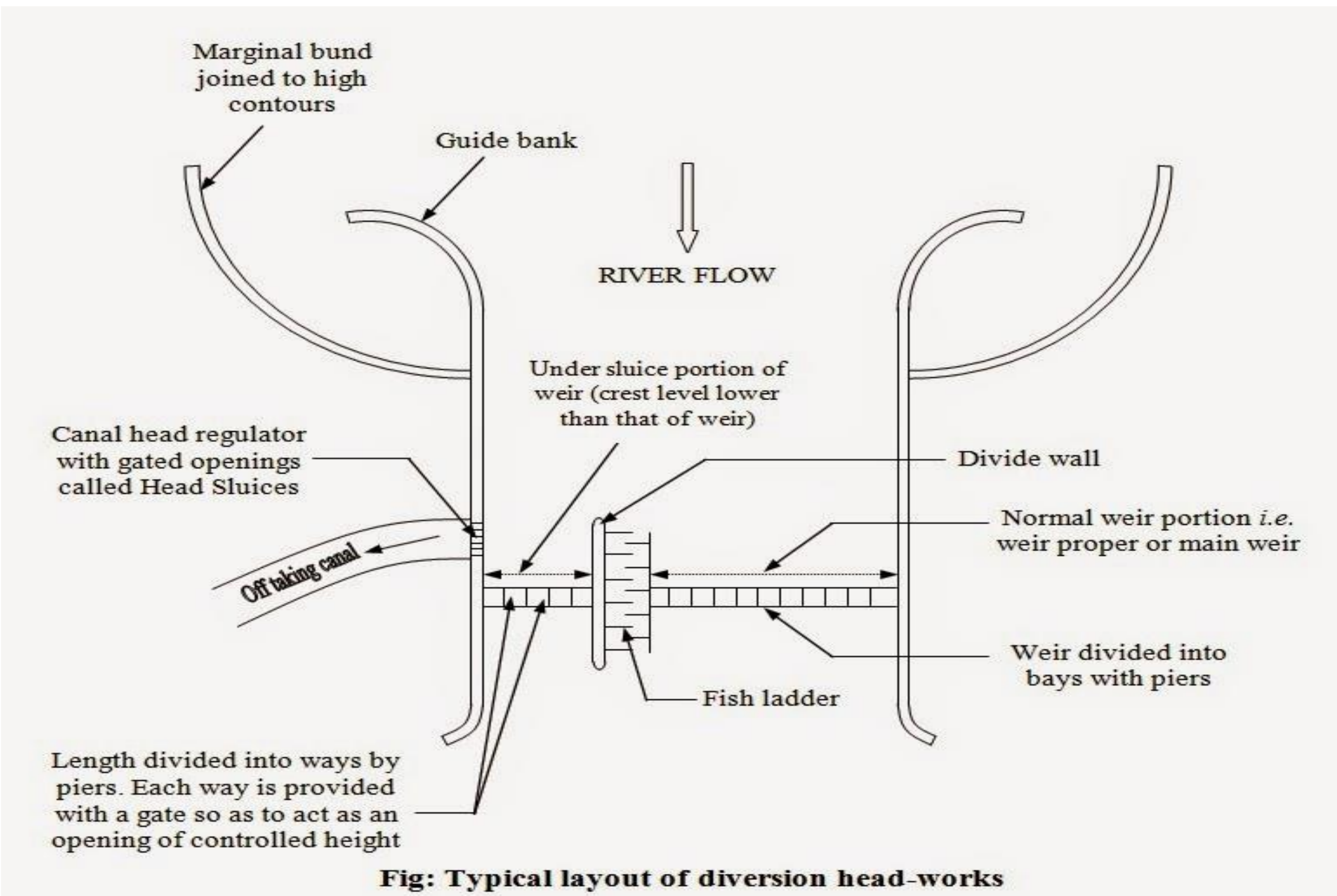


Figure 1: Typical Layout of Diversion Headwork

Below is definition about the essential component of diversion head work by (Mohanty, 2012). 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 to raise the water level to 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.

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.

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.

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.

A structure which is constructed at the head of the canal to regulate flow of water is known as canal head regulator. It consists of a number of piers which divide the total width of the canal into a number of spans which are known as bays. The pier consists of tiers on which the adjustable gates are placed. The gates are operated from the top by suitable mechanical device. A platform is provided on the top of the pier for the facility of operating the gates. Again some piers are constructed on the downstream side of the canal head to support the roadway.

Functions of canal head regulator are (Mohanty, 2012).

- It regulates the supply of water entering the canal,
- It controls entry of silt in the canal,
- It prevents the river flood from entering the canal.

Entry of silt in to canal which takes off from headwork's can be reduced by constructed certain special works called silt control works. These works may be classified in to two types, silt excluders and silt ejectors (Mohanty, 2012).

Silt excluders are those works which are constructed on the bed of the river upstream of the head regulator. The clearer water enters the head regulator and silted water enters the silt excluder. In this type of works the silt is therefore removed from the water before it enters the canal.

Silt ejectors also called silt extractors are those devices which extract the silt from the canal water after the silted water has travelled a certain distance in the off taking canal. These works are therefore constructed on the bed of the canal and little distance downstream from the head regulator.

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 headwork are guide banks, marginal bunds, spurs Or groynes.

2.3 Weir failure

Weir failure is a bad phenomenon that leads the constructed weir useless with respect to its main purpose. The failure type can be exposed to the surface or may not be exposed depending on the type of failure. There are different types of failure and Here, Some types of failure are listed below.

- Damage of floor
- Deterioration of floor due to standing waves
- Scour on upstream and downstream of the weir
- Others

2.3.1 Causes of weir failure

Common causes of failure of weirs (Jamal, March 2017) include:

- Excessive and progressive downstream erosion, both from within the stream and through lateral erosion of the banks
- Erosion of inadequately protected abutments
- Hydraulic removal of fines and other support material from downstream protection (gabions and aprons) resulting in erosion of the apron protection
- Deterioration of the cutoff and subsequent loss of containment
- Additional aspects specific to concrete, rock fill or steel structures

In addition to the above causes, the main causes of weir failure and there remedy is presented below (Jamal, March 2017)

2.3.1.1 Piping

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

Since there is always a differential head between upstream and downstream, water is constantly moving from upstream to downstream under the base of weir. However, if the hydraulic gradient becomes big, greater than the critical value, then at the point of existence of water at the downstream end, it begins to dislodge the soil particles and carry them away. In due course, when this erosion continues, a sort of pipe or channel is formed within the floor through which more particles are transported downstream which can bring about failure of weir.

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

Remedies:

- Decrease Hydraulic gradient i.e. increase path of percolation by providing sufficient length of impervious floor
- Providing curtains or piles both at upstream and downstream

2.3.1.2 Rupture of floor due to uplift

If the weight of the floor is insufficient to resist the uplift pressure, the floor may burst. This bursting of the floor reduces the effective length of the impervious floor, which will increase exit gradient, and can cause failure of the weir.

Remedies:

- Providing impervious floor of sufficient length of appropriate thickness.
- Pile at upstream to reduce uplift pressure downstream

2.3.1.3 Rupture of floor due to suction caused by standing waves

Hydraulic jump formed at the downstream of water

Remedies:

- Additional thickness
- Floor thickness in one concrete mass

2.3.1.4 Scour on the upstream and downstream of the weir

Occurs due to contraction of natural water way.

Remedies:

- Piles at greater depth than scour level
- Launching aprons

In addition to the above causes, there are also failures that will be caused due to the flow condition and differ with respect to the flow type's. Correspondingly, it is known that the flow types have an effect on the type of weir failure. Surface and subsurface flow types can cause their own type of failure as described below.

2.3.1.5 Failure due to subsurface flow

a. Failure by piping or undermining

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

b. Failure by direct uplift

The percolating water exerts an upward pressure on the foundation of the weir or barrage. If this uplift pressure is not counterbalanced by the self-weight of the structure, it may fail by rapture.

2.3.1.6 Failure by surface flow

a. By hydraulic jump

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

b. By scouring

During floods, the gates of the barrage are kept open and the water flows with high velocity. The water may also flow with very high velocity over the crest of the weir. Both the cases can result in scouring effect on the downstream and on the upstream side of the structure. Due to scouring of the soil on both sides of the structure, its stability gets endangered by shearing.

2.4 Previous studies on investigation of weir failure

2.4.1 Studies in Ethiopia

A number of irrigation schemes have been designed and constructed here in Ethiopia in the previous years. However, while some schemes are performing successfully, it has been observed in various reports that some of the schemes have failed to serve the purpose for which they are intended. With the ground that reviewing studies made on failed weir will help to choose the best methodology to be used later in investigating causes of failure of Tana Beles weir, the following studies are presented below after a simple review on their methodology used, result adopted and recommendation, if any given by the author.

2.4.1.1 Performance Assessment of Fentalle head work, Ethiopia

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

Fantale Irrigation Based Integrated Development Project is located on Awash River at about 50km south west of Metehara town with a total command area of 27,000 hectares. The total water way (four bays of the under sluice each 2.7m wide, three piers each 1.5m thick, one divide wall 2m thick, width of un gated weir 128m) is 145.3m. Provided depth under sluice upstream cutoff and downstream cutoff is 3.8m and 4.5m respectively. Depth of weir upstream cutoff and downstream cutoff is 4.5m and 5.5m respectively. The under sluice sill level is 1173.1m (lowest river bed level at the weir axis).

It has problems in regulating the supply of water in to the canal, in controlling entry of silt in to the canal, and in controlling the fluctuation of water level in the river during different seasons. In addition problem of sedimentation in and around the intake works is prevalent.

Photo 1: Fentale Head Work

This research work is aimed to identify the major causes of underperformance of the diversion headwork components of Fantale Irrigation through performance assessment.

The first parameter that is assessed for this study is the design flood i.e. passage of floods, including hazard floods. Discharge through the sluice way and canal head regulator is calculated by submerged orifice flow equation. The quantity of seepage, pressure head and exit gradient were calculated using both Geo-Seep/w and khoslas Method of independent variable. Mathematical solutions of the flow nets for the simple standard profiles have been used for determining the percentage pressures at various key points. For checking the adequacy of the upstream and downstream cut off depth Lacey's equation is used.

The forces and moments acting on the corresponding structure are calculated and the structure is checked for its stability against overturning, shear and sliding.

As per the studies, there is no extreme flooding occurred after the structure begins operation. The flood discharge capacity of the head work is greater than the designed discharge capacity. The result also implies that there is no extreme flooding occurred after Fantale diversion structure begins operation. The existing flow of water in to the main canal is 11.5m3 /s but the amount of water required to be diverted to the main canal is 18m3 /s. This implies that there is a shortage of water by an amount 6.5 m3/s to meet the demand of the command area. The above result is the consequence of malfunctioning of the gates. To achieve the required 18m3/s discharge the two gates should have to be opened by an amount 0.78m or one gate should have to be opened by an amount 1.565m. But gates provided both on the under sluice and canal head regulator are not functioning because of incomplete construction procedure and failure.

On the other side, The designed under sluice sill level that is the lowest river bed level at the weir axis was 1173.10m but the existing under sluice level is raised to a level of 1174 i.e. about 0.9m depth sediment deposited at the back of head regulator.

By using value of discharge intensity per unit width that is 4.75 for under sluice and 1.7 for the weir portion. The calculated values of the cutoff depth are 3.76m and 2.95m for the under sluice and weir portion respectively. There is not likely to be any appreciable scour on the downstream of the under sluice bays and weir portion, no extra protection is therefore, provided on the upstream and downstream side.

There is no lifting up of the structures heel and the structure is not susceptible to any tension on the base. The diversion structure is safe against shear and sliding both for worst condition and existing situation on the site at the time of investigation.

Henok Fikru finally recommends: Complete control on the river discharge is required with proper regulation area in front of the headwork. This can be achieved by providing a barrage than the weir, which is actually provided on the site. The gates should have to be repaired or changed to mechanical operating system and trash racks should have to be provided for a good performance of the head work. The design should consider transporting capacity of the design flow to prevent the weir and especially sluice gates from damage.

2.4.2 Studies outside of Ethiopia

Reviewing study related to weir failure investigation made by foreign author is good and this will help to know professional foreign engineers procedure and methodologies adopted in investigating those failure causes. Out of those weirs, a study made on Singapore weir failure is discussed below (Sharma, 2017)

2.4.2.1 Studies on Failure of Downstream Block Protection of Singapore Weir

The Singapore weir was constructed across the Tapi River in 1995 between Singapore village and Rander, Surat (Gujarat). It was designed for a design discharge of 50-year return period of 24,000 m^3/s.

The weir was constructed by Surat Municipal Corporation (SMC) for two purposes, namely

a) To provide a standing pool of water over infiltration wells of Varachha water works and
b) To provide a surface and sub-surface barrier to prevent tidal water from entering into infiltration well areas.

The weir is divided into three spans,

I. Un gated weir first span from Ch. 40 to 310 m,
II. Un gated weir second span from Ch. 310 to 522 m, and
III. Gated under sluice third span from Ch. 522 to 620 m.

Ever since its construction, recurring damages were observed in the first span (Ch. 40–310 m) of the weir. Further, significant damages were reported during recent floods in the years 2013 and 2014, wherein substantial scouring and settlement of the concrete blocks were observed, which eventually posed a threat to the main structure of the weir.

The severe nature of settlement was reported in the first span (Ch.40–310 m) of the cement concrete blocks, and failure of toe wall resulted in gully or channel formation.in addition to this, there is significant scouring downstream and the heavy concrete blocks were dislodged from their mean position on account of enormous disturbance (velocity) in the flow.

The investigation was made to ascertain the causes of failure of the first span of the weir as it serves as the only source of freshwater for citizens of Surat city. The study investigates the likely causes of the failure/disturbance of downstream block protection of the first span (Ch. 40–310 m) of the weir through detailed hydraulic analyses. The detailed hydraulic analyses for existing weir portion for the first span (Ch. 40–310 m) have been carried out at different discharge conditions and investigated the likely causes of damage to the downstream concrete block and launching apron.

From the detailed hydraulic jump analyses, it is observed that for discharges greater than 4000 m^3/s, the hydraulic jump formation takes place within the concrete floor due to sufficiency of tail water depth. However, at lower discharges (<4000 m^3/s), the available tail water depth is not enough for the formation of the jump within the concrete floor. as discharge reduces, the hydraulic jump formation does not take place at the toe of weir and gets shifted further downstream, i.e., for discharges less than 4000 m^3/s, it is found that the jump gets shifted from concrete floor to the block protection and further downstream in the river bed. Due to this, the energy dissipation does not take place on the concrete floor, and thus, there is scouring of the soil underneath the concrete blocks

Further, the toe wall was found broken due to flow concentration in the gully or channel .Also the loose stone protection exhibited substantial settlement due to scouring of soil under the protection work. If the foregoing condition persists for a longer duration, it would result in severe degradation immediately downstream of the weir. It is also found that 60% of time the discharge was observed to be less than 4000 m3/s during the flood of 2013, wherein significant damage was reported to the weir. Thus, the analysis of hydraulic jumps on existing geometry of weir for Ch. 40–310 m indicated the persistence of repelled hydraulic jump condition at lower discharges (deficient tail water condition) which was the major reason for dislodging and settlement of downstream concrete block and loose stone protection of the weir

The following key conclusions can be summarized on the basis of study undertaken in the paper:

i. The analysis of hydraulic jump for the first span of Singapore weir (Ch. 40–310 m) revealed that, for lower discharges, i.e. less than 4000 m^3/s there was shifting of hydraulic jump in the downstream of concrete floor. At discharge 2000 m^3/s, there is a significant shifting of jump, even beyond the loose stone protection. The shifting of hydraulic jump for lower discharges is due to deficient tail water condition in the river under these discharges. The formation of hydraulic jump on launching apron/concrete block at lower discharges is the prime reason for failure of protection works.

ii. The observation of previous flow data for the year 2013 validates the findings in the present investigation. The observed data statistics revealed that 60% of time the discharge was observed to be less than 4000 m^3/s (which is the critical limit for the formation of hydraulic jump within the concrete floor).Hence, the persistence of low discharges over the weir for a longer time would be the possible cause of maximum damage of downstream concrete blocks and protection works.

As it was illustrated above the thesis work by Henok Fikru did not incorporate hydrologic, hydraulic and structural design analysis of the head work which will further help in reviewing the respective designs.

On the other side the procedure used for identifying the causes of failure of Singapore Weir is more specific to hydraulic jump analysis and energy dissipation.

Finally I decided to separate and select the best methodology part from the two and use it in my own thesis. Accordingly, evaluation and analysis of Geotechnical, hydrologic, hydraulic and structural design of the weir is a must to be incorporated in my methodology and further site investigation will also help me to check the site condition in detail.

3 Method and Material

3.1 Study Area

Tana Beles Integrated Sugar Development Project is located near Fendika Town, capital of Jawi Woreda, found in the western periphery of Amhara National Regional State (ANRS), 145 km from Bahir Dar. The project covers about 57,614 ha of land situated at Upper Ayma and Upper Beles mainly emphasized on 38,334 ha of land placed at Right side of Beles River (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Irrigation and Drainage Report, 2013).

The project overall includes a total area of about 95,948 ha of land found in Jawi woreda of Amhara and Dangur woreda of Benishangul National Regional States. The study area has low to medium relief differences with an altitude range of 806 to 1242 meters above sea level. The Upper Beles (Right side) irrigation command area has an altitude ranged from 962 to 1,242 m.a.s.l, which is characterized by flat topography (plain land),whereas the Upper Ayma irrigation command area is between 806 to 1154 m.a.s.l, mainly characterized by undulating topography (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Irrigation and Drainage Report, 2013).

The Project is proposed to grow sugarcane in a total of about 50,000 ha for the two sugar factories to be established at Upper Beles and Upper Ayma sites. To grow sugar cane in a total of 50000 ha, 60 m3/s of water is proposed to be diverted from Upper Beles River in to the main canal. For diverting 60 m3/s of water, a diversion head works has been constructed on the Upper Beles River at a distance of about 28 km from Fendika town. The diversion head work comprises of a weir, under sluices and a head regulator. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

The project under this study is Sugar Cane development schemes by diverting the available water in Beles catchment and additional release of water through the Beles hydropower turbine which will transfer more than 77 m^3/s of water from the Lake Tana to Beles head water system. It is large-scale irrigation project designed and constructed by Amhara Design Supervision Works Enterprise and Amhara Water Works Construction Enterprise respectively.

3.2 Description of TanaBeles Diversion Head-work

The diversion weir for Tana Beles Integrated sugar Development Project is located at Beles River at about 28 km from Jawi / Fendika. Geographically the weir is located at 247847.76 Easting and 1289961.39 Northing (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011) as shown on the figure below.

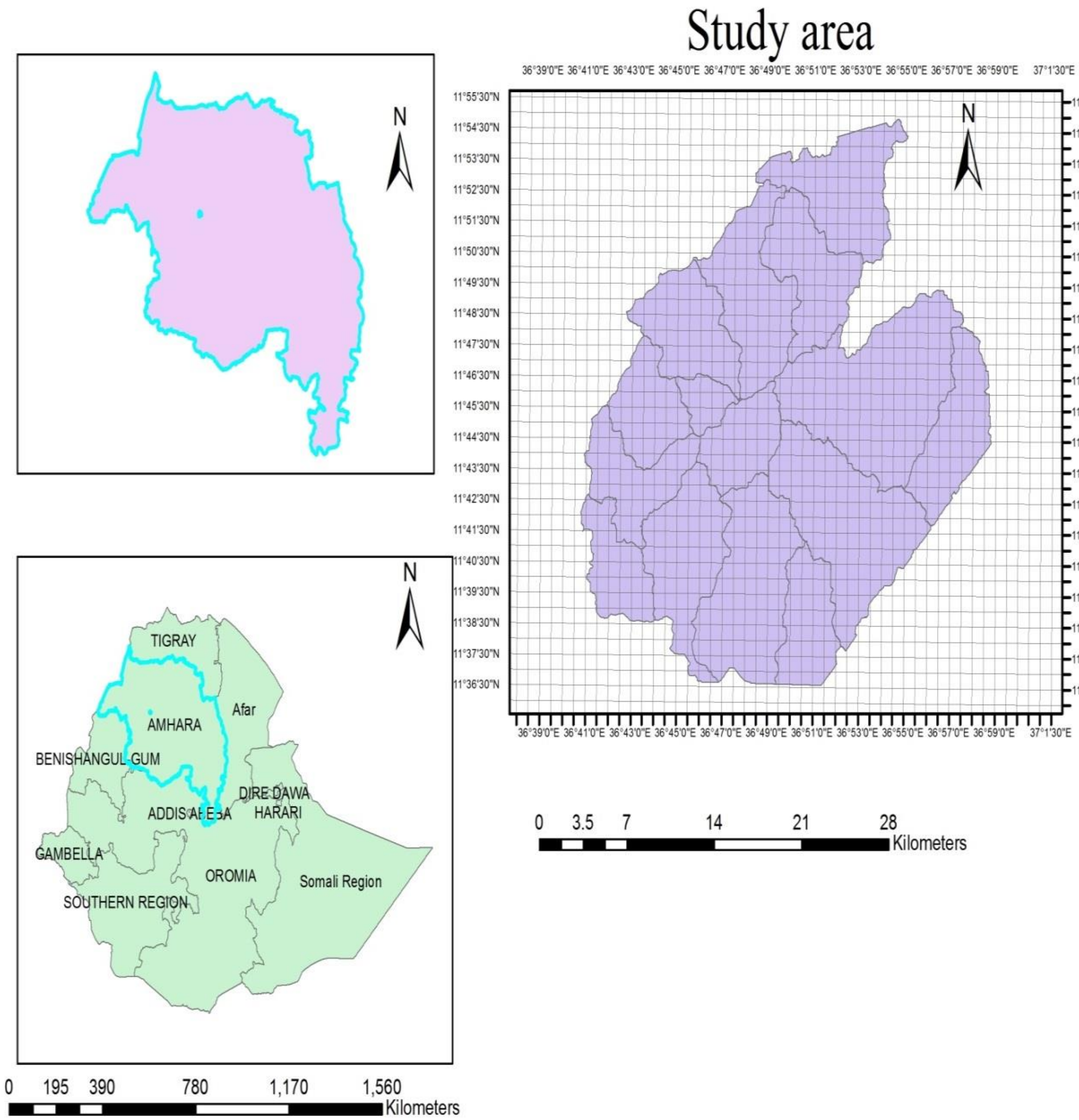


Figure 2 Geographical Location of TanaBeles Weir

The main features of Tana Beles Irrigation diversion head work are weir, under sluices, divide wall, and canal head regulator. The total water way (three bays of the under sluice each 3.3 m wide, two piers each 1m thick, one divide wall 1.3m thick, width of ungated weir 30m) is 39.9m.

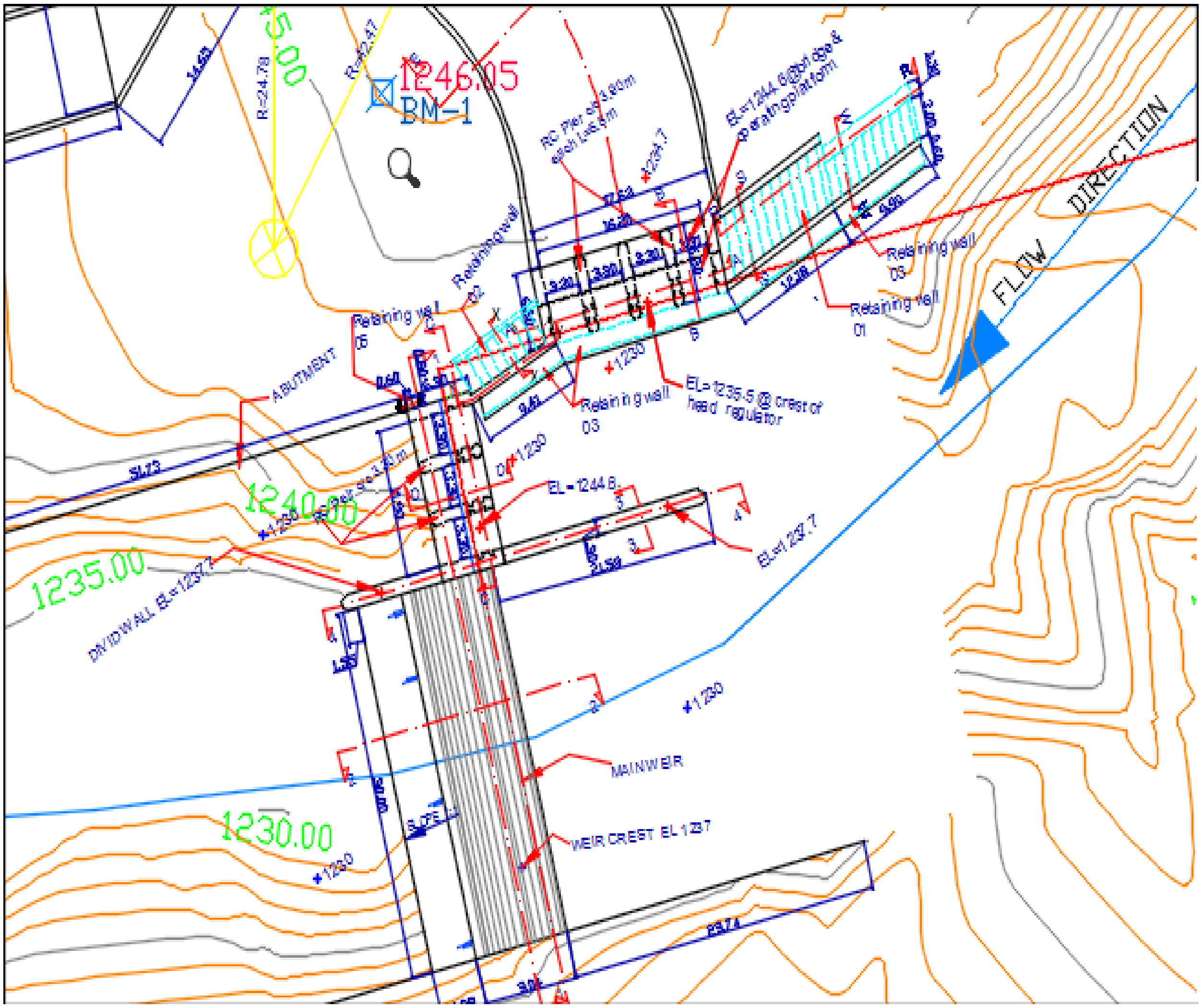


Figure 3: Plan View of Tana Beles Headwork

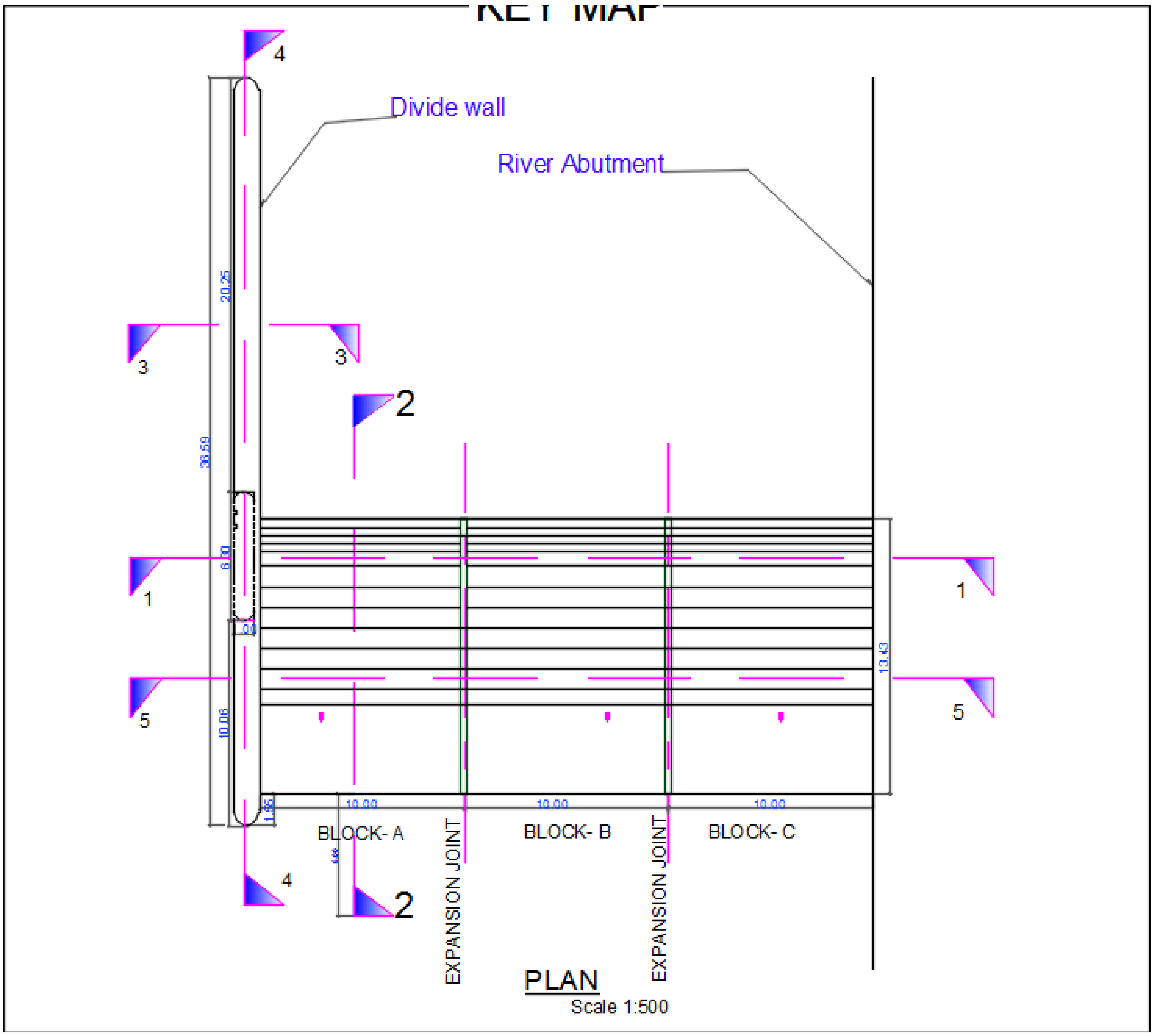


Figure 4: Plan View of Tana Beles Weir

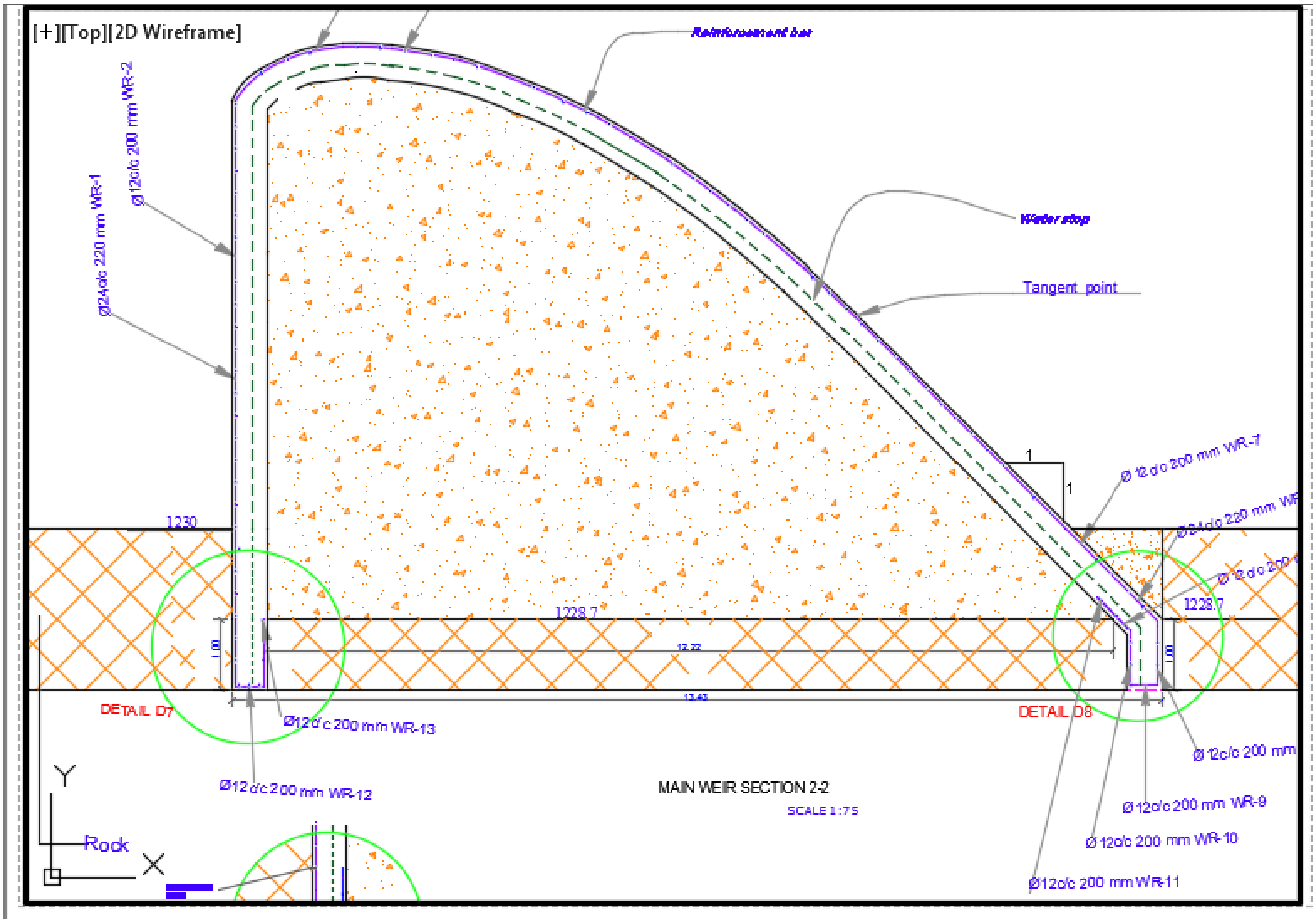


Figure 5: Sectional View of Tana Beles Weir

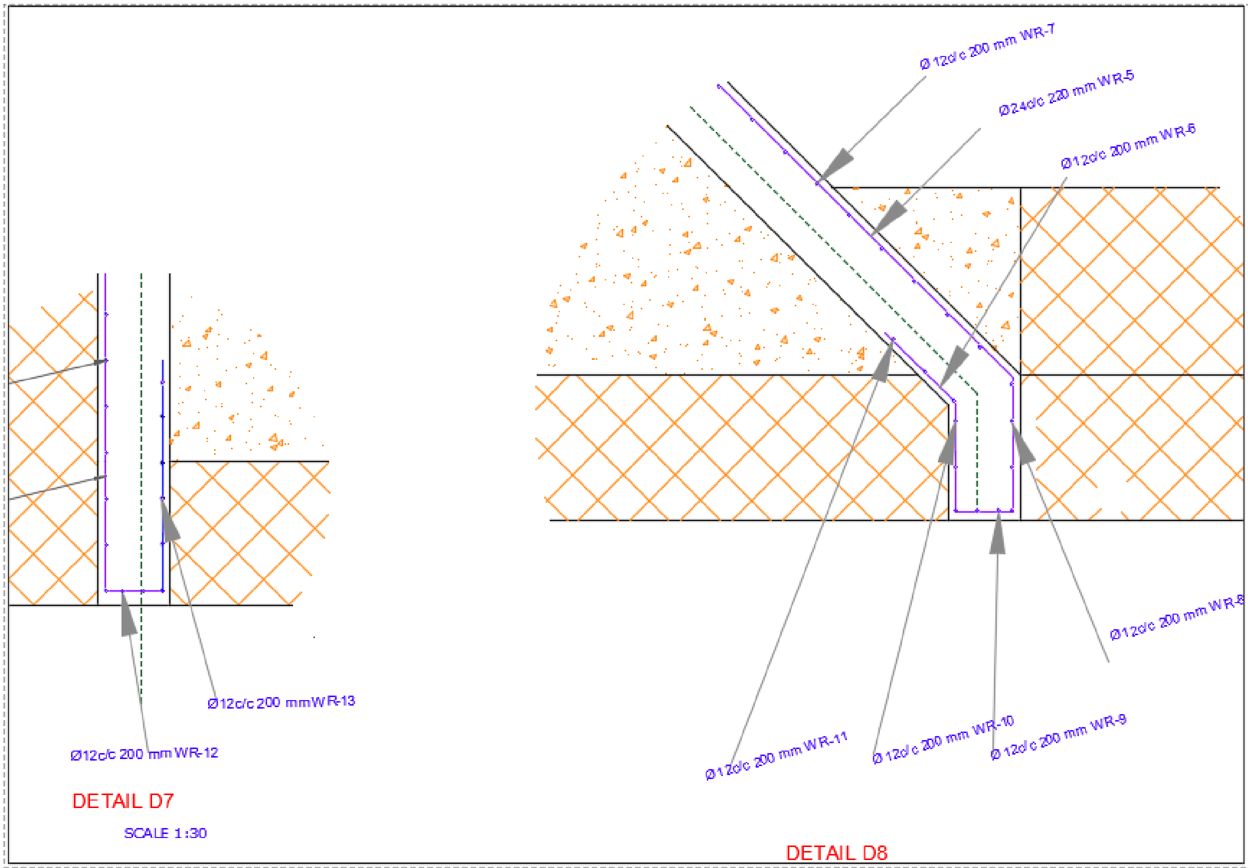


Figure 6: Sectional View of the foundation of Tana Beles Weir

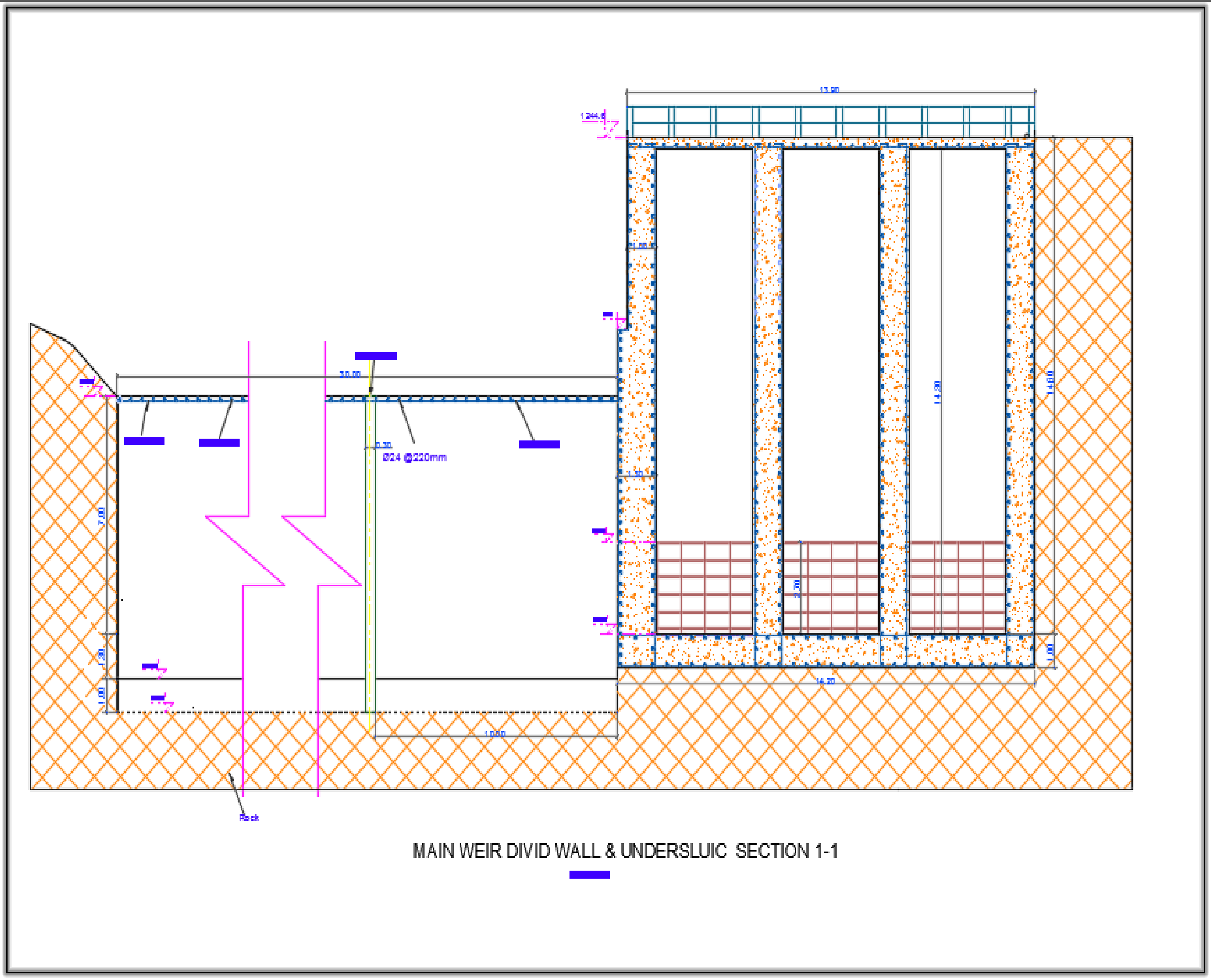


Figure 7: main weir divide wall and under sluice section

The total length of under sluice and weir part is 43.2m. Provided depth under sluice upstream cutoff and downstream cutoff is 3.8m and 4.5m respectively.

Weir:

- Length of the weir 30 m
- Crest level of the weir 1235.80 mamsl
- Type of weir Ogee Type
- Slope of upstream face vertical
- Slope of the downstream face 1:1
- Design (1 in 100) flood discharge 1033 m^3/sec
- High flood level in the river 1236.57mamsl
- Affluxed HFL/ Upstream HFL 1241.27mamsl
- Bed Level of the river 1228.80 mamsl

Under- Sluice Bays:

- Clear waterway (3 bays of 3.3 m each) 9.9 m
- RCC piers 2nos each 1 m thick 2.0 m
- over all water way 11.9 m
- Crest level of the under sluice 1228.80
- Thickness of the Divide wall 1.3 m
- Total length of weir and under sluices 43.2 m
- Gates: 3.3x2.7 m electrically operated vertical lift gates 3 in number.

Photo 2: Tana Beles diversion head work (Dec 2017)

From visual observation it has been noted that:

- The damage of weir is heavy along the line of intersection between the weir and the divide wall
- Crack propagation and expansion tendency along the expansion line is observed
- The plastering is detached
- The rest concrete part is found intact and in good stand.

HEAD REGULATOR:

- Clear Water way (6 bays of 3.3m each) 19.8 m
- RCC piers 4 nos of 1m +1 no of 2.0 m 6.0 m
- Crest level 1234.3 mamsl
- FSL of canal 1235.30 mamsl
- Gates: 3.3x1.5 m electrically operated vertical lift gates 6 nos

Photo 3: Under-sluice and canal head regulator of Tana Beles irrigation development project (Dec, 2017)

3.2.1 Operation methodology

The profile of the ogee (weir crest) is designed for the design head. The design head is generally chosen to give the maximum practical hydraulic efficiency, in keeping with the operational requirements, stability and economy. If the actual head is less than the design head, the pressure on the crest will be positive (i.e. above atmospheric). However, for the actual heads greater than the design head, the pressure on the crest will be negative (i.e. less than the atmospheric pressure) and it may lead to cavitation. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

To avoid any possibility of negative pressures on the crest, the ogee crest has been designed for a design head in the condition, when the under sluices are inoperative and whole of the design flood passes over the weir crest only (rarest of rare case).

The openings of the under sluice 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 will move away. 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.

In case of Tana Beles diversion headwork, out of the three bays of the under sluice two of them were fully closed and cannot be opened freely due to malfunctioning of gate. The rest one was opened by force but cannot be closed. Hence, Efficient flushing of the sediment from the under sluice pocket is not ensured.

3.3 Methodology, Data collection and analysis

The methodologies to attain the objectives are described below:

- ✓ Data collection
- ✓ Data analysis
- ✓ Peak discharge estimation
- ✓ Evaluation of the actual foundation condition
- ✓ Evaluation of the hydraulic design
- ✓ Evaluation of the structural design
- ✓ Recommend remedial design

3.3.1 Data Collection

The consultant on the project site, field measurements and observations are the primary sources of data for the study. In order to achieve the objectives of the study secondary data is also used. These data are obtained from the Ministry of Water, Irrigation and Electricity, Amhara design and supervision work enterprise and Ethiopian Meteorological Agency. In addition to these literatures, different project documents or proposals, project evaluation and completion reports are also refereed.

On first stage previous studies, documents and papers related to diversion headwork have been revised and desk study have been undertaken to identify the key issues. Accordingly, the available relevant data on existing headwork is important to be collected from the concerned department. Field visit surveys have been conducted for gathering out of data for the purpose of describing the nature of existing conditions and to compare existing conditions with the design standard of the diversion headwork. To assess the operation and functionality of constructed diversion head works as well as operations of each component parts of the head work, field visit survey coupled with primary data obtained by interviewing administrative officials at the project site have been conducted. The secondary data's are also collected from Amhara Design and Supervision Works Enterprise design reports.

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

3.3.1.1 Primary data collection

Field observation at Tana Beles Sugar Development project was made to identify where different parameters of the head-work must be taken. Necessary Dimensions of the head work and related data of the project were taken in collaboration with Amhara Design and Supervision Work Enterprise workers. Surveying data upstream and downstream of the weir was also collected with the help of Amhara Design and Supervision Work Enterprise workers.

3.3.1.2 Secondary data collection

Secondary data used for this research were collected from responsible bodies and officials. These data include hydrology data, geological data, GIS data and design report of the headwork components for the project. In general the design report was also revised to get some insight in to the head work components and these help in identifying the sensitive parameters for the problems on the head work.

3.3.2 Data Analysis

Ten or more stations with installation period going as far as 1950 can be identified in and around the basin. It is not always easy to get data for all stations listed in the NMA list of stations. However, the required rainfall data is collected from Ethiopian ministry of Electric, Energy and Irrigation. The weir site is the place where the peak flood could be determined and it is known that the weir site is ungauged.as a method before determining the peak flood at the ungauged station it is necessary calibrate the gauged station, therefore the data analysis incorporate the rainfall analysis at the gauged and ungauged station.

3.3.2.1 Rainfall analysis of main beles gauging station

The following three stations are selected for rainfall runoff analysis of the large gauged watershed based on thiessen polygon and as it is shown on the figure the project area rainfall is represented mostly by mandura, Bahir Dar and Pawe respectively. The Table below provides the list of stations used in the rainfall runoff analysis of main beles at bridge.

Table 1: List of Rainfall station used for main beles

No	Station name	Latitude	Longitude	Altitude	Data used	Remark
1	Bahir Dar	11.36	37.24	1770	1980-2015	
2	Mandura	11.07	36.25	1290	1980-2015	
3	Pawe	11.09	36.23	1200	1980-2015	

The three stations are selected based on thiessen polygon and as it is shown on the figure the project area rainfall is represented mostly by mandura, Bahir Dar and Pawe respectively.

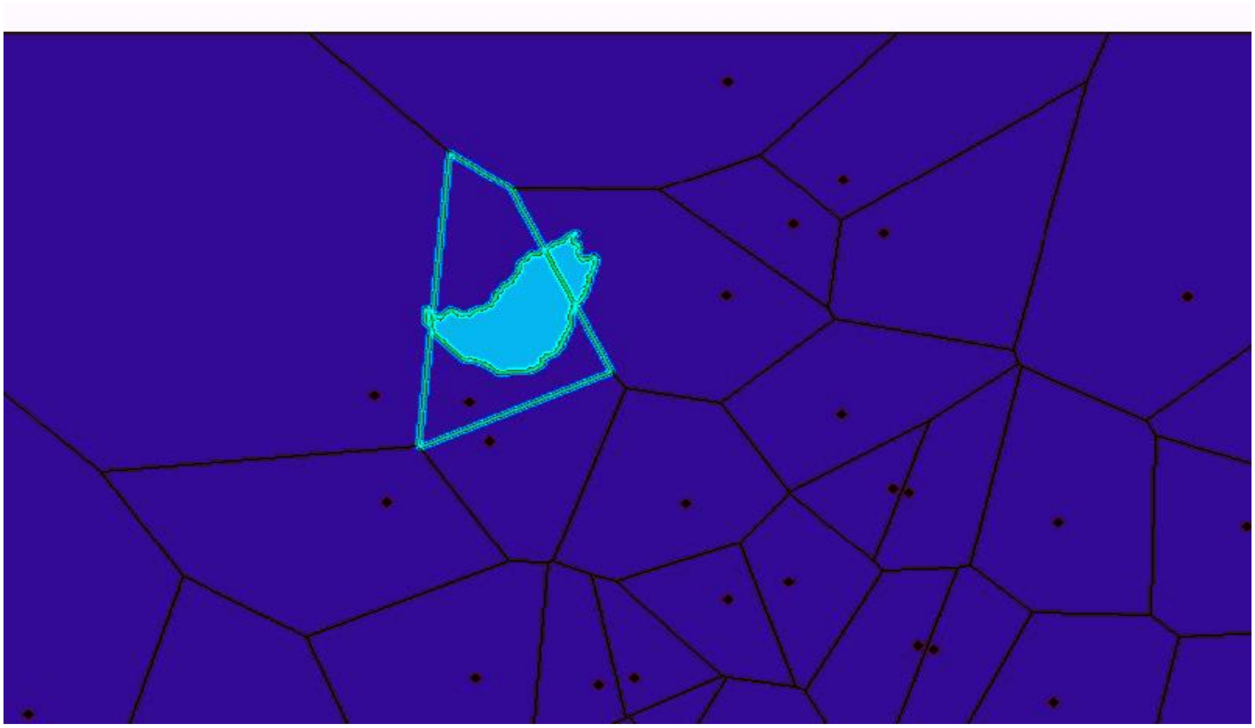


Figure 8: Thiessen Polygon for Main Beles at Bridge

Table 2: Rainfall analysis of main beles guaging station

24 Hr Duration Max. Rain fall(mm)			
Year	24hr duration Max. annual ppt (mm)	**Assending**	**Y-logr**
1985	54.74	85.12	**1.93003**
1986	52.08	81.36	**1.91039**
1987	47.05	67.63	**1.83016**
1988	58.95	64.86	**1.81199**
1989	67.63	62.39	**1.79515**
1990	49.79	58.95	**1.77051**
1991	54.22	54.74	**1.73831**
1992	50.16	54.39	**1.73548**
1993	37.88	54.22	**1.73413**

1994	52.93	53.21	**1.72603**
1995	62.39	52.99	**1.72418**
1996	52.99	52.93	**1.72374**
1997	81.36	52.08	**1.71671**
1998	50.29	50.87	**1.70650**
1999	37.10	50.59	**1.70404**
2000	49.75	50.29	**1.70148**
2001	50.59	50.16	**1.70033**
2002	42.17	50.01	**1.69907**
2003	54.39	49.79	**1.69710**
2004	47.02	49.75	**1.69676**
2005	47.18	47.18	**1.67375**
2006	42.96	47.05	**1.67254**
2007	38.74	47.02	**1.67228**
2008	53.21	46.31	**1.66567**
2009	50.87	42.96	**1.63309**
2010	50.01	42.17	**1.62503**
2011	35.02	38.74	**1.58814**
2012	46.31	37.95	**1.57926**
2013	64.86	37.88	**1.57843**
2014	85.12	37.10	**1.56933**
2015	37.95	35.02	**1.54426**

ave	**51.80**	**51.80**	**1.70**
Stan deva	**11.45**	**11.45**	**0.09**
Cs			0.543
K			0.091

Log pearson Type III distribution							
Yt=	$Y+k_TSy$	W=[ln(1/p2)]^0.5	for 0<p<0.5				
		K_T=Z=w-2.515517+0.802853w+0.010328w^2/(1+1.432788w+0.189269w^2+0.001308w^3)					
Y	1.70						
Sy	0.09		P=1/T	T=Return Period			

T	P	W	Z	k	K_T	Y_T	X_T	I_T
2	0.5	1.177	-0.388	0.091	-0.458	1.66376	46.1067777	1.921
5	0.2	1.794	0.727	0.091	0.674	1.76557	58.28638864	2.429
10	0.1	2.146	1.363	0.091	1.425	1.8331	68.09201056	2.837
25	0.04	2.537	2.070	0.091	2.356	1.91683	82.57158172	3.440
50	0.02	2.797	2.540	0.091	3.032	1.97771	94.99656154	3.958
100	0.01	3.035	2.969	0.091	3.694	2.03722	108.9490969	4.540
500	0.002	3.526	3.856	0.091	5.194	2.17216	148.6484013	6.194
1000	0.001	3.717	4.201	0.091	5.831	2.22939	169.5860359	7.066

Summery of 24hr	
Return period	**Using log-Pearson Distribution**
(years)	**XT=10^YT(mm)**
2	46.11
5	58.29
10	68.09
25	82.57
50	95.00
100	108.95
500	148.65
1000	169.59

Therefore the 100 years, 24hour rainfall (108.95mm) shall be changed to incremental rainfall by using alternate block method as shown below. The incremental depth of precipitation will be ordered sequentially by using alternate method which then finally entered as an input into the Hec-hms for peak flood estimation.

Table 3: incremental depth calculation of gauged station by alternate block method

Time	Hourly p=M*sqrt(T)	Incrimental depth	Time Interval	Pricipitation using Alternate Method	cummulative
hr	mm	mm	hr	mm	mm
0.5	15.73	15.73	0---0.5	**1.14**	1.14
1	22.24	6.51	0---1	1.17	2.31
1.5	27.24	5.00	1---1.5	1.19	3.50
2	31.45	4.21	1---2	1.22	4.72
2.5	35.16	3.71		1.25	5.97
3	38.52	3.36	2---3	1.28	7.25
3.5	41.61	3.09		1.32	8.57
4	44.48	2.87	3---4	1.36	9.93
4.5	47.18	2.70		1.40	11.33
5	49.73	2.55	4---5	1.45	12.78
5.5	52.16	2.43		1.50	14.28
6	54.47	2.32	5---6	1.56	15.84
6.5	56.70	2.22		1.62	17.46
7	58.84	2.14	6---7	1.70	19.16
7.5	60.90	2.07		1.78	20.94
8	62.90	2.00	7---8	1.88	22.82
8.5	64.84	1.94		2.00	24.81
9	66.72	1.88	8---9	2.14	26.95
9.5	68.55	1.83		2.32	29.27
10	70.33	1.78	9---10	2.55	31.82
10.5	72.06	1.74		2.87	34.70
11	73.76	1.70	10---11	3.36	38.05
11.5	75.42	1.66		4.21	42.27
12	77.04	1.62	11---12	6.51	48.78
12.5	78.63	1.59		15.73	64.51
13	80.18	1.56	12---13	5.00	69.50
13.5	81.71	1.53		3.71	73.22
14	83.21	1.50	13---14	3.09	76.30
14.5	84.68	1.47		2.70	79.00
15	86.13	1.45	14---15	2.43	81.43
15.5	87.56	1.42		2.22	83.65

16	88.96	1.40	15---16	2.07	85.72
16.5	90.34	1.38		1.94	87.65
17	91.69	1.36	16---17	1.83	89.48
17.5	93.03	1.34		1.74	91.22
18	94.35	1.32	17---18	1.66	92.88
18.5	95.65	1.30		1.59	94.46
19	96.94	1.28	18---19	1.53	95.99
19.5	98.21	1.27		1.47	97.47
20	99.46	1.25	19---20	1.42	98.89
20.5	100.69	1.24		1.38	100.27
21	101.91	1.22	20---21	1.34	101.61
21.5	103.12	1.21		1.30	102.91
22	104.31	1.19	21---22	1.27	104.18
22.5	105.49	1.18		1.24	105.41
23	106.66	1.17	22---23	1.21	106.62
23.5	107.81	1.15		1.18	107.80
24	108.95	1.14	23---24	1.15	108.95

3.3.2.2 Rainfall Analysis of weir site ungauged station

The Table below provides the list of stations used in the rainfall runoff analysis of Tana Beles weir site.

Table 4: List of Rainfall station used for Tana Beles weir site

No	Station name	Latitude	Longitude	Altitude	Data used	Remark
1	Bahir Dar	11.36	37.24	1770	1980-2015	
2	Mandura	11.07	36.25	1290	1980-2015	

The above two stations are selected for rainfall runoff analysis of the ungauged watershed based on thiessen polygon and as it is shown on the figure the project area rainfall is represented mostly by Bahir Dar and mandura respectively.

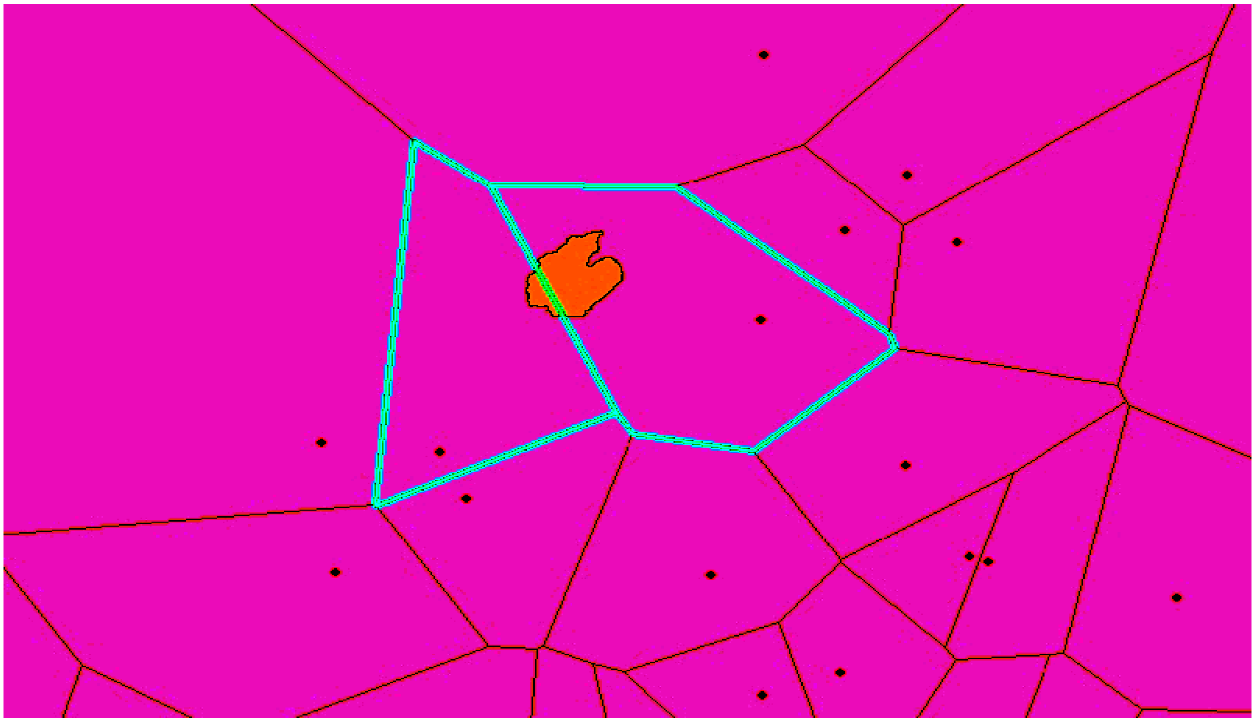


Figure 9: Thiessen Polygon of Tana Beles Weir site

Table 5: Rainfall analysis of weir site unguaged station

Log-pearson type III Distribution			
24 Hr Duration Max. Rain fall(mm)			
Year	24hr duration Max. annual ppt (mm)	**Assending**	**Y-logr**
1985	89.70	119.33	**2.07677**
1986	70.20	110.84	**2.04469**
1987	63.90	107.07	**2.02965**
1988	65.16	91.02	**1.95913**
1989	79.10	90.00	**1.95426**
1990	110.84	89.70	**1.95281**
1991	90.00	88.73	**1.94805**
1992	79.26	88.36	**1.94624**
1993	78.04	88.26	**1.94575**
1994	72.14	87.89	**1.94395**
1995	82.79	84.77	**1.92826**
1996	76.81	84.66	**1.92766**

1997	88.36	84.13	**1.92497**
1998	76.87	82.79	**1.91796**
1999	91.02	80.36	**1.90502**
2000	107.07	79.55	**1.90065**
2001	119.33	79.26	**1.89906**
2002	84.66	79.10	**1.89818**
2003	79.55	78.04	**1.89229**
2004	84.13	76.87	**1.88578**
2005	87.89	76.84	**1.88558**
2006	76.84	76.81	**1.88541**
2007	84.77	75.10	**1.87567**
2008	88.26	72.14	**1.85820**
2009	66.61	70.20	**1.84631**
2010	64.49	67.37	**1.82849**
2011	75.10	66.61	**1.82356**
2012	88.73	65.16	**1.81396**
2013	80.36	64.49	**1.80950**
2014	67.37	63.90	**1.80553**
2015	59.11	59.11	**1.77165**
ave	**81.56**	**81.56**	**1.91**
Stan deva	**13.58**	**13.58**	**0.07**
Cs			0.386
K			0.064

Log pearson Type III distribution						
Yt=	$Y+k_TSy$		W=[ln(1/p2)]^0.5		for 0<p<0.5	
	K_T=Z=w-2.515517+0.802853w+0.010328w^2/(1+1.432788w+0.189269w^2+0.001308w^3)					
Y	1.91					
Sy	0.07		P=1/T	T=Return period		

T	P	W	Z	k	K_T	Y_T	X_T	I_T
2	0.5	1.177	-0.388	0.064	-0.439	1.87532	75.0452	3.127
5	0.2	1.794	0.727	0.064	0.691	1.95419	89.9883	3.750
10	0.1	2.146	1.363	0.064	1.410	2.00434	101.003	4.208
25	0.04	2.537	2.070	0.064	2.276	2.06473	116.074	4.836
50	0.02	2.797	2.540	0.064	2.891	2.10763	128.124	5.339
100	0.01	3.035	2.969	0.064	3.482	2.14888	140.889	5.870
500	0.002	3.526	3.856	0.064	4.792	2.24027	173.888	7.245
1000	0.001	3.717	4.201	0.064	5.337	2.27828	189.793	7.908

Summary of 24hr Rain fall	
Return period	**Using log-Pearson Distribution**
(years)	**XT=10^YT(mm)**
2	75.05
5	89.99
10	101.00
25	116.07
50	128.12
100	140.89
500	173.89
1000	189.79

Therefore the 100years, 24 hour rainfall (140.89mm) shall be changed to incremental rainfall by using alternate block method as shown below. The incremental depth of precipitation will be ordered sequentially by using alternate method which then finally entered as an input into the Hec-hms for peak flood estimation.

Time	Hourly p=M*sqrt(T)	Incremental depth	Time Interval	Precipitation using Alternate Method	cumulative
hr	**mm**	**mm**	**hr**	**mm**	**mm**
0.5	20.34	20.34	0---0.5	**1.48**	1.48
1	28.76	8.42	0---1	1.51	2.98
1.5	35.22	6.46	1---1.5	1.54	4.52
2	40.67	5.45	1---2	1.58	6.10
2.5	45.47	4.80		1.62	7.72
3	49.81	4.34	2---3	1.66	9.38
3.5	53.80	3.99		1.71	11.09
4	57.52	3.71	3---4	1.76	12.84
4.5	61.01	3.49		1.81	14.66
5	64.31	3.30	4---5	1.87	16.53
5.5	67.45	3.14		1.94	18.47
6	70.44	3.00	5---6	2.01	20.48
6.5	73.32	2.88		2.10	22.58

7	76.09	2.77	6---7	2.19	24.77
7.5	78.76	2.67		2.30	27.07
8	81.34	2.58	7---8	2.43	29.50
8.5	83.85	2.50		2.58	32.09
9	86.28	2.43	8---9	2.77	34.86
9.5	88.64	2.36		3.00	37.85
10	90.94	2.30	9---10	3.30	41.15
10.5	93.19	2.25		3.71	44.87
11	95.38	2.19	10---11	4.34	49.21
11.5	97.53	2.14		5.45	54.66
12	99.62	2.10	11---12	8.42	63.08
12.5	101.68	2.05		20.34	83.42
13	103.69	2.01	12---13	6.46	89.88
13.5	105.67	1.98		4.80	94.68
14	107.61	1.94	13---14	3.99	98.67
14.5	109.51	1.90		3.49	102.16
15	111.38	1.87	14---15	3.14	105.30
15.5	113.22	1.84		2.88	108.18
16	115.04	1.81	15---16	2.67	110.85
16.5	116.82	1.78		2.50	113.35
17	118.58	1.76	16---17	2.36	115.71
17.5	120.31	1.73		2.25	117.96
18	122.01	1.71	17---18	2.14	120.10
18.5	123.70	1.68		2.05	122.16
19	125.36	1.66	18---19	1.98	124.13
19.5	127.00	1.64		1.90	126.04
20	128.61	1.62	19---20	1.84	127.88
20.5	130.21	1.60		1.78	129.66
21	131.79	1.58	20---21	1.73	131.39
21.5	133.35	1.56		1.68	133.08
22	134.89	1.54	21---22	1.64	134.72
22.5	136.41	1.52		1.60	136.31
23	137.92	1.51	22---23	1.56	137.87
23.5	139.41	1.49		1.52	139.40
24	140.89	1.48	23---24	1.49	140.89

The rainfall pattern in this part of Abay basin is distinctively uni-modal with peak rainfall located in the July-August months. It appears the southern stations peak is located in August (such as

Pawe), while the peaks of the stations located north of the weir are in July (Bahir dar) (see Table3 and Figure 14). The mean annual rainfall around the irrigation scheme is represented by Pawe station with mean annual rainfall of 1576 mm with the low variability during the wet season (CV less than 0.3).

3.3.2.3 Stream flow analysis of main beles gauging station

The 30 years maximum flow data of main beles gauging station received from ministry of water, electric and energy shall be analyzed by Log Pearson before going to the next step.

Table 6: Flow analysis of main beles guaging station

Log-Pearson type III Distribution			
Max. flow(m3/s)			
Year	Max. annual flow (m3/s)	**Ascending**	**Y-logr**
1985	848.32	275.80	**2.44060**
1986	849.32	355.69	**2.55107**
1987	912.36	390.68	**2.59182**
1988	795.13	416.36	**2.61947**
1989	912.36	419.35	**2.62258**
1990	926.78	428.74	**2.63219**
1991	914.66	429.58	**2.63304**
1992	745.37	438.13	**2.64161**
1993	966.60	445.65	**2.64900**
1994	666.99	537.55	**2.73042**
1995	950.05	560.36	**2.74847**
1996	2,146.50	666.99	**2.82412**
1997	873.90	669.20	**2.82556**
1998	929.90	739.84	**2.86914**
1999	739.84	745.37	**2.87237**
2000	1,192.07	795.13	**2.90044**
2001	897.26	813.99	**2.91062**
2002	537.55	848.32	**2.92856**
2003	918.38	849.32	**2.92907**
2004	669.20	873.90	**2.94146**
2005	813.99	897.26	**2.95292**
2006	429.58	912.36	**2.96017**
2007	419.35	912.36	**2.96017**

2008	438.13	914.66	**2.96126**
2009	416.36	918.38	**2.96302**
2010	428.74	926.78	**2.96698**
2011	275.80	929.90	**2.96843**
2012	445.65	950.05	**2.97775**
2013	355.69	966.60	**2.98525**
2014	390.68	1,192.07	**3.07630**
2015	560.36	2,146.50	**3.33173**
ave	**753.77**	**753.77**	**2.84**
Stan deva	**350.58**	**350.58**	**0.19**
Cs			0.013
K			0.002

Log pearson Type III distribution					
Yt=Y+kTSy	W=[ln(1/p2)]^0.5	for 0<p<0.5	P=1/T,T is return period		
KT=Z=w-2.515517+0.802853w+0.010328w^2/(1+1.432788w+0.189269w^2+0.001308w^3)					
Y	2.84				
Sy	0.19				

T	P	W	Z	k	K_T	Y_T	X_T
2	0.5	1.177410023	-0.38796883	0.002132544	-0.390	2.76428	581.142
5	0.2	1.794122578	0.72696149	0.002132544	0.726	2.97415	942.212
10	0.1	2.145966026	1.36293488	0.002132544	1.365	3.09431	1242.53
25	0.04	2.537272482	2.0701292	0.002132544	2.077	3.2283	1691.62
50	0.02	2.797149623	2.53973159	0.002132544	2.551	3.31751	2077.33
100	0.01	3.034854259	2.9692241	0.002132544	2.986	3.39924	2507.51

Summary of 24hr	
Return period	**Using log-Pearson Distribution**
(years)	**XT=10^YT(m3/s)**
2	581.14
5	942.21
10	1242.53
25	1691.62
50	2077.33
100	2507.51

Therefore the 100years, 24 hour flood (2507.51m^3/s) shall be changed to incremental flood by using alternate block method as shown below. The incremental flood will be ordered sequentially by using alternate method which then finally entered as an input into the Hec-hms for calibration.

4 Hydrologic analysis

In The hydrologic analysis, the peak discharge at the weir will be estimated. The determination of the peak flood is a major task in the hydrology part and Peak discharge estimates are often needed at un gauged sites where no observed flood data are available.the value obtained will be compared with value used for the design purpose and additionally will be used for redesign of the weir.
The peak flood flow is the maximum expected flow at a certain location for a given frequency. According to (Asquith, 1997), peak flood flows depend on the catchment area, the slope of the main channel, the basin shape factor, the hydrologic region, and the return period.

Potential extreme peak discharges are estimates of the highest peak discharges expected to ocur at a certain location and, according to Asquith and Slade (Asquith, 1997), are explained mostly by the area of the corresponding catchment and by the hydrologic region where the catchment is located. Maximum design discharge is the peak river discharge that corresponds to a certain return period. The maximum design discharge Q_{Max} is used in the design to determine the back water curves results from constructing to the weir which enables to predict the highest water level that occurs average once every T years, where T is the selected return period of the discharge.

In this research, The HEC-Hms model is preferred and SCS curve number method is used for peak flood estimation. It is obvious that the data has to be processed in ArcGIS before going into HECHMS model.

By using ArcGIS, two types of watershed is delineated, one for the gauged site and one for the un guaged site. The gauged watershed is delineated by using main beles at bridge gauging station as an outlet and after several processes on Arc-hydro tool and hec- Geohms, the system is optimized in hec-hms by using rainfall records of the nearby town and stream guage records of main beles at bridge. On the other hand the unguaged watershed is delineated by using weir site as an outlet and after passing the same process as that of the gauged watershed on Arc Hydro and Hec-GeoHMS, estimation of peak flood is finally done as the system is already optimized by the gauged site.

On the other hand, the geological cross section of the weir and its treatment is evaluated. Surveying data upstream and downstream of the weir is collected and used in order to compare the existing bed level with the design bed level.

Finally, the peak flood estimated after using all the above hydrologic process is used on the remedial redesign the weir.

The materials used include, but not limited to:

- ✓ Global Mapper to obtain DEM and Shape file of the project area
- ✓ Arc-GIS to obtain hydrological and physical parameters and spatial information and to delineate the catchments of the study area.
- ✓ 30*30 DEM data is used as an input data for ArcGIS software for catchment delineation and estimation of catchment characteristic.
- ✓ HEC-HMS software to develop rainfall runoff model.
- ✓ Hydrological and meteorological data etc.

In Arc GIS Terrain Preprocessing uses DEM to identify the surface drainage pattern. Once preprocessed, the DEM and its derivatives can be used for efficient watershed delineation and stream network generation (Ambaw, 2016). Taking the weir site as an outlet Clipped DEM of the project area is created by using Global Mapper and mosaic of Ethiopia. The clipped DEM is used as Raw DEM for terrain processing.

Using the digital elevation model (DEM) of the basin as input, the terrain preprocessing with a series of steps is performed in HEC-Geo HMS to derive the drainage networks. The steps consist of computing the flow direction, flow accumulation, stream definition, watershed delineation, watershed polygon processing, stream processing, and watershed aggregation. Once these data sets are developed, they are used in later steps for sub basin and stream delineation.

The Geo HMS tool is designed to have the output files from the Arc Hydro terrain preprocessing tools as inputs. These Hydrographic features, which are already executed using Arc Hydro are:

Flow direction grid (Fdr), flow accumulation grid (Fac), stream grid (Str), stream link grid (Lnk), catchment grid (Cat), curve number grid, slope grid.

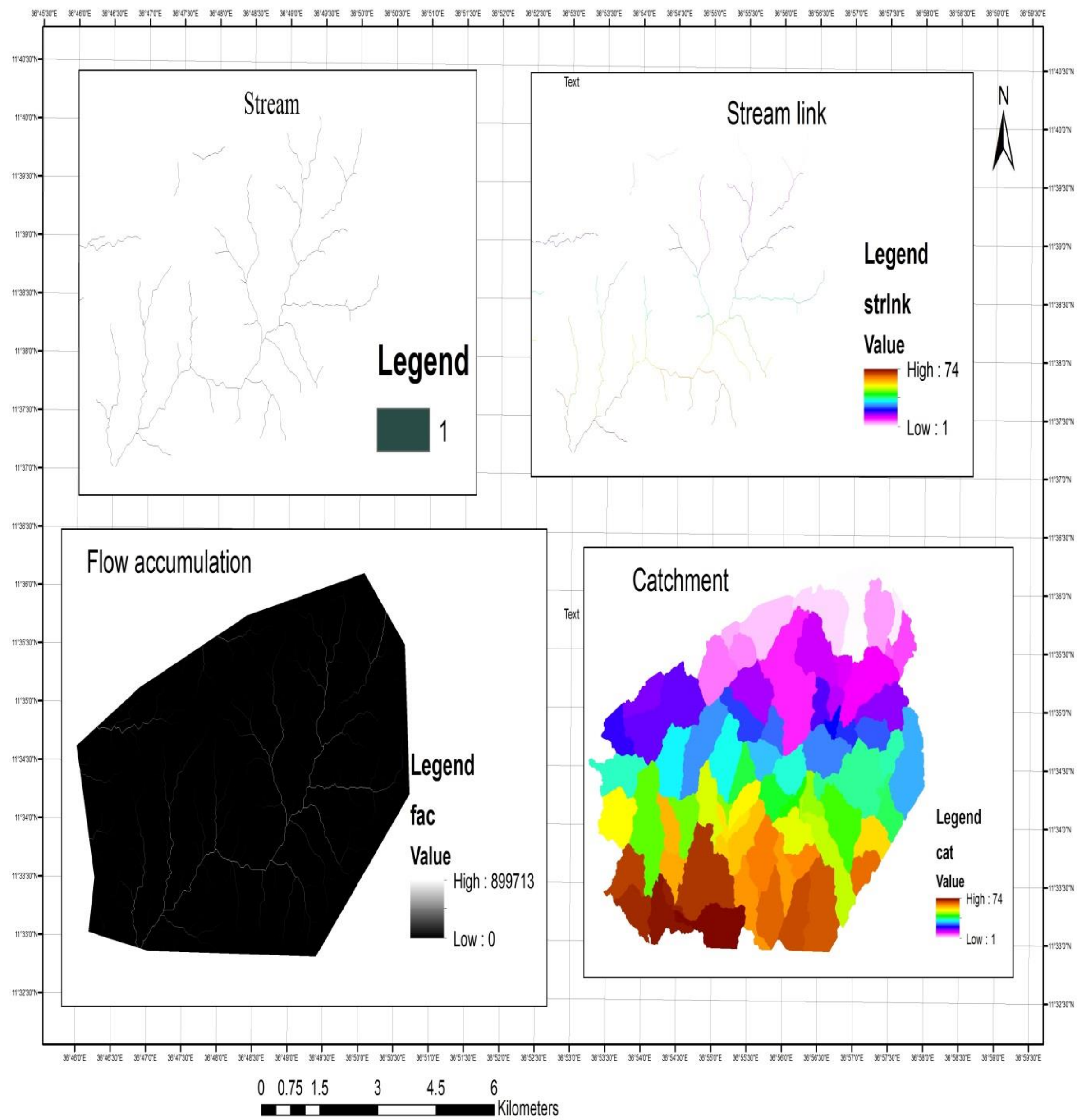


Figure 10: Arc Hydro output 1

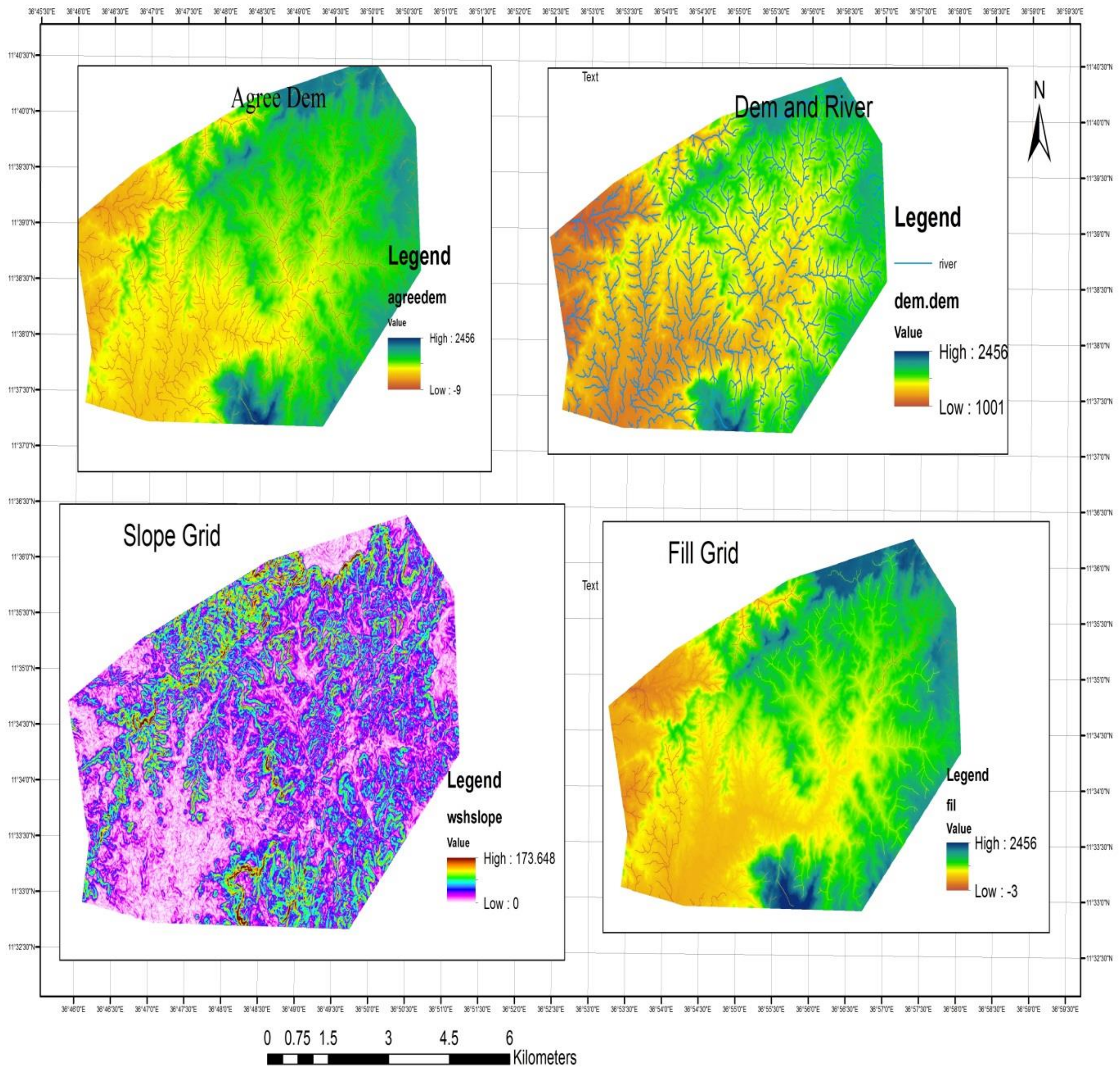


Figure 11: Arc Hydro Output 2

The above output files from Arc Hydro are finally used to automatically create sub basins, longest paths and basin centroids in HEC-Geo HMS. Additionally, parameters such as slope, length and average curve number are assigned to flow lines and basins. In Geo HMS by using spatial analyst tools geographic information is converted into parameters for each of the basins and flow lines.

These parameters are then used to create a HEC-HMS model that can be used within the HEC-HMS.HEC-Geo HMS creates background map file, lumped basin model, a grid-cell parameter file, and a distributed basin model, which can be used by HMS to develop a hydrologic model.

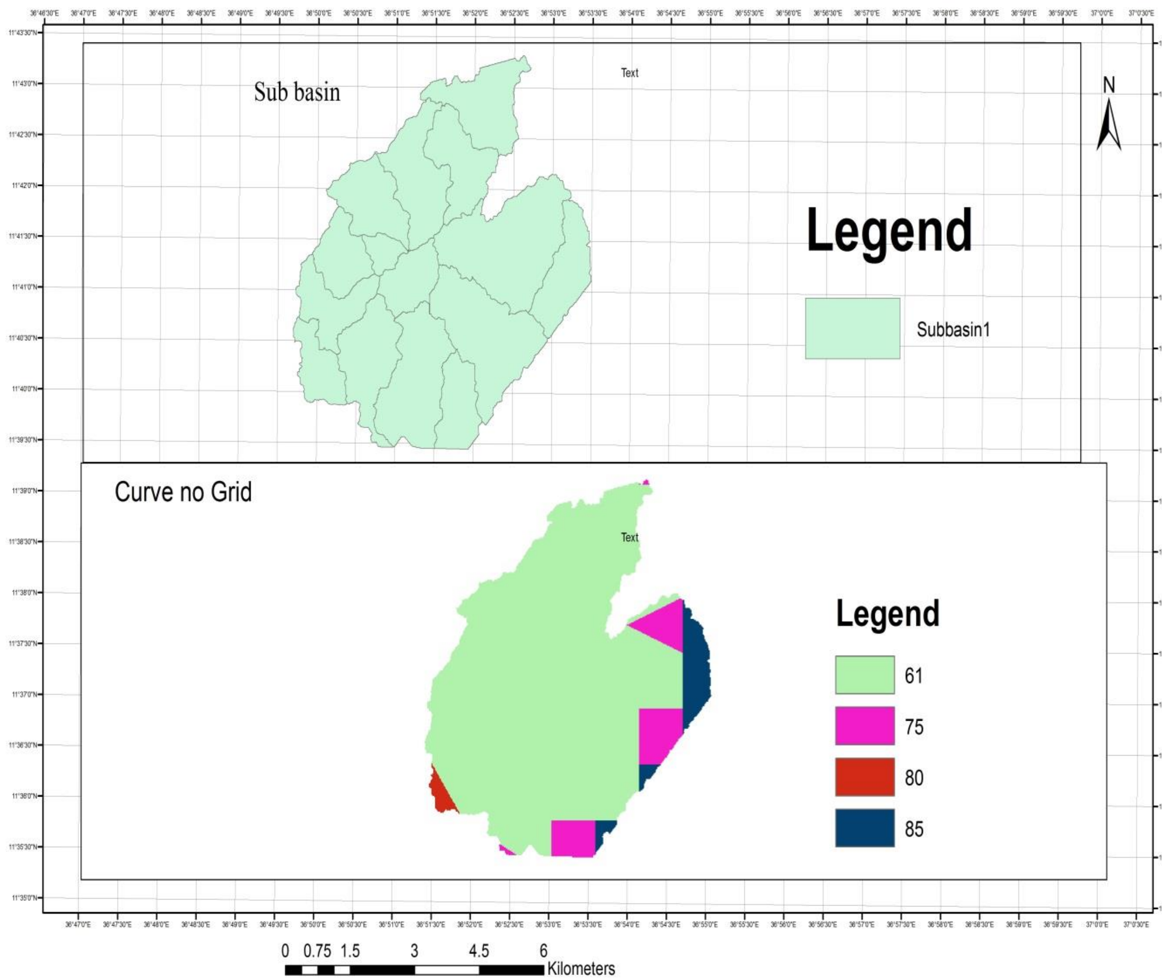


Figure 12: CN Grid and Sub basin

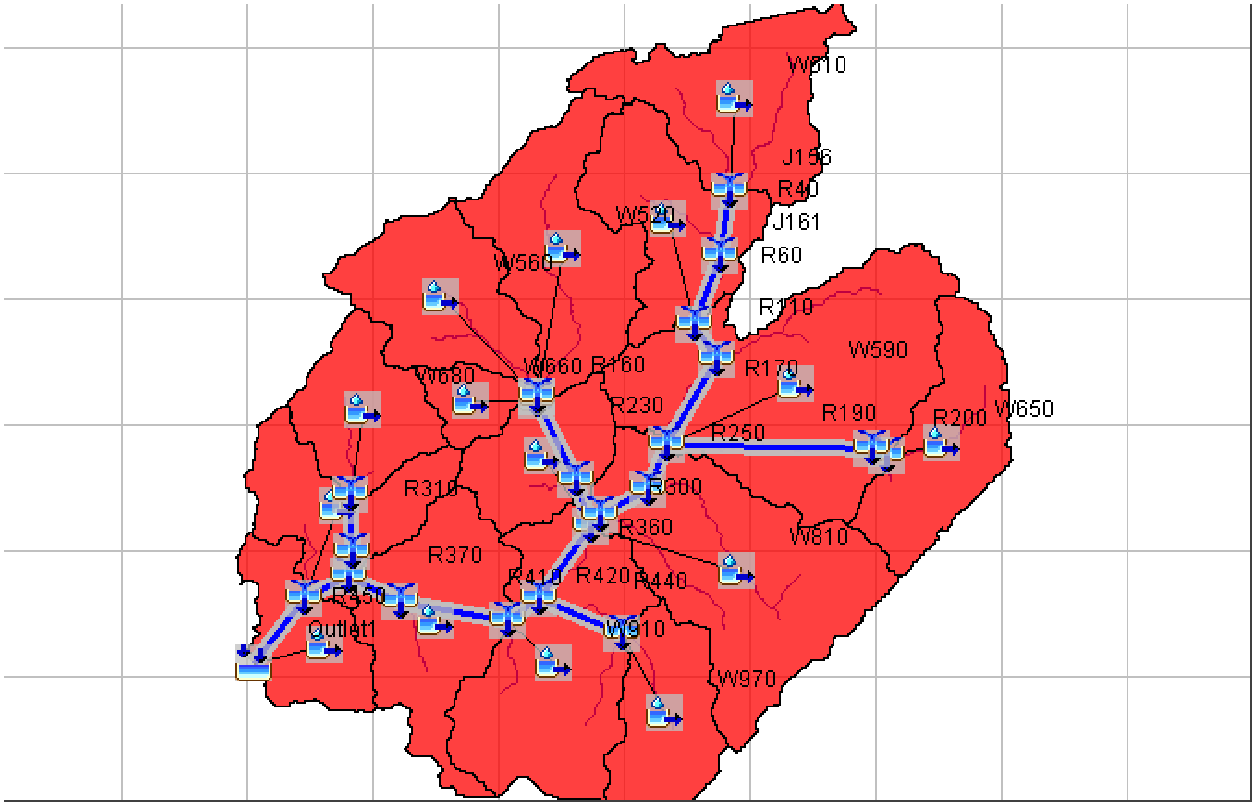


Figure 13: Hec-Geo Hms Output

Maximum design discharge is the peak river discharge that corresponds to a certain return period. The maximum design discharge Qmax is used in the design to predict the highest water level that occurs average once every T years, where T is the selected return period of the discharge.

SCS curve number method is 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 (Yener, 2006) and the result is shown in the output below.

The model result for peak flow is 989.5m^3/s and According to the study by (halcrow, 2000), the turbine release fluctuates between 64.78 and 80m^3/s. Recent information provided by the Tana Beles hydropower Station is that the minimum and maximum release between 77 and 158m^3/s, which appears to be the latest information available for the operation of the hydropower. Adding the maximum value of 158 m^3/s to the simulated result, the total peak flood will be 1147.5m^3/s.

Table 7: Simulation result

After creating basin model, meteorology model and control specifications for each of the sub basins, simulation runs are created in HEC-HMS to compute the output (runoff hydrograph) with the initial parameter estimates. Simulation runs produce a graph to visually compare observed hydrograph with the computed (simulated) hydrograph and several tables such as summary results table (where peak discharges, total discharge volumes, total precipitation, loss, direct runoff, average absolute residuals, and total residuals can be seen), and time series results table (where the results can be seen at each time step) (Choudhari, 2014). According to the obtained results, initial parameter estimates are refined.

After that, the iterative optimization process starts with the creation of optimization trials in HEC-HMS.

Simulated (predicted) flow and the observed flow relationship is the main tool used in hydrology to assess the performance of a hydrologic model. In general, the differences between the simulated and observed flow data are computed using several different mathematical expressions, i.e. goodness-of-fit criteria, to show whether the model yields satisfactory predictions. (Ambaw, 2016)

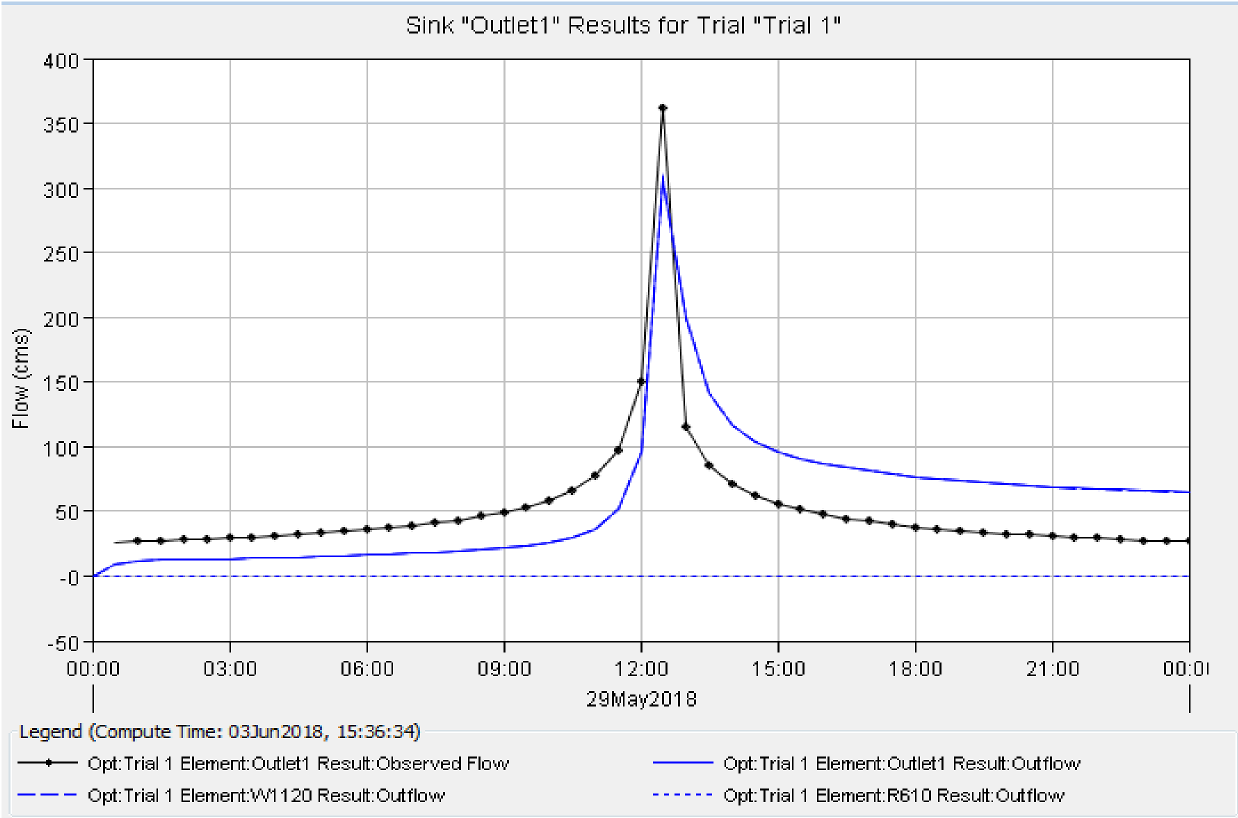
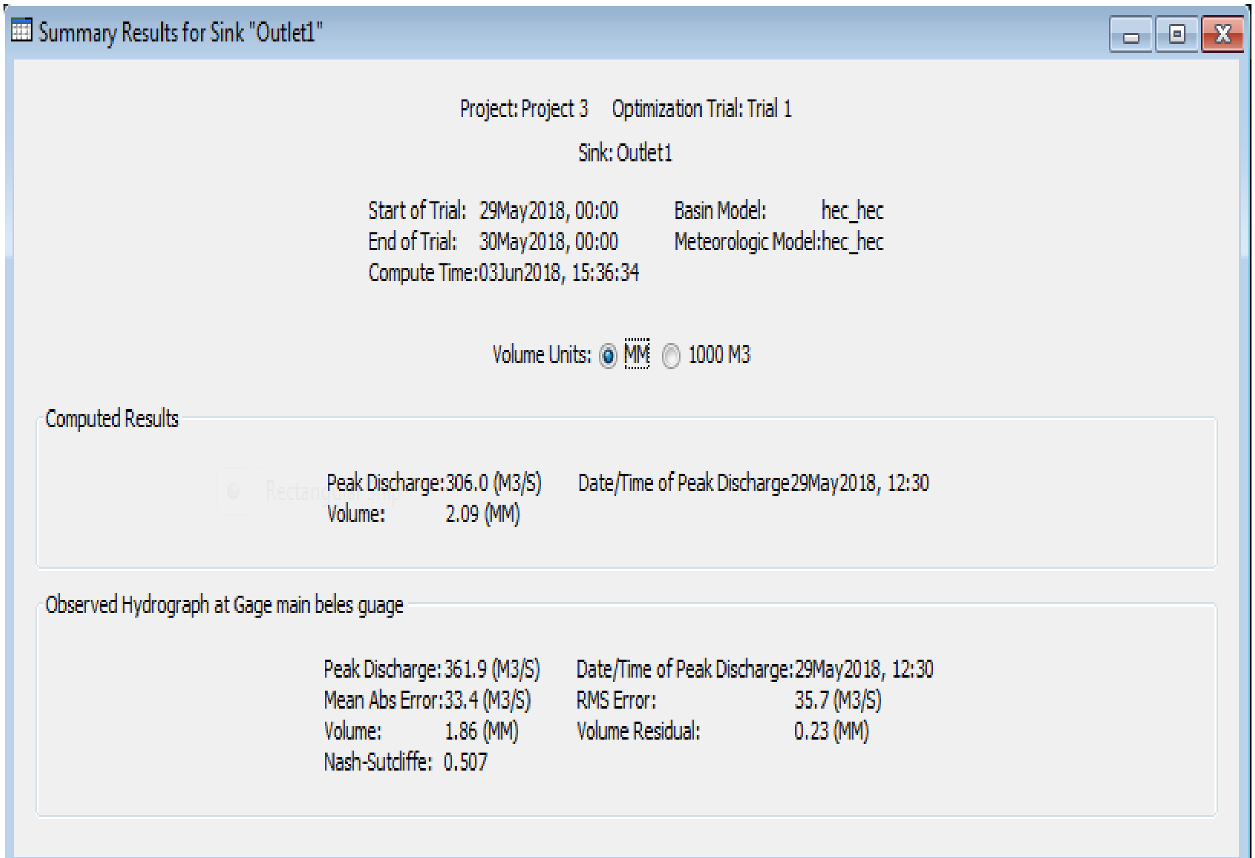


Figure 14: Optimization result graph

Table 8: summary result table



Summary Results for Sink "Outlet1"

Project: Project 3 Optimization Trial: Trial 1

Sink: Outlet1

Start of Trial:	29May2018, 00:00	Basin Model:	hec_hec
End of Trial:	30May2018, 00:00	Meteorologic Model:	hec_hec
Compute Time:	03Jun2018, 15:36:34		

Volume Units: MM 1000 M3

Computed Results

Peak Discharge:	306.0 (M3/S)	Date/Time of Peak Discharge	29May2018, 12:30
Volume:	2.09 (MM)		

Observed Hydrograph at Gage main beles guage

Peak Discharge:	361.9 (M3/S)	Date/Time of Peak Discharge:	29May2018, 12:30
Mean Abs Error:	33.4 (M3/S)	RMS Error:	35.7 (M3/S)
Volume:	1.86 (MM)	Volume Residual:	0.23 (MM)
Nash-Sutcliffe:	0.507		

5 Evaluation of the Foundation condition of the Weir

The material and the method used include the following:

- The geotechnical investigation report of the weir site is evaluated
- The foundation treatment of the weir is evaluated
- The existing actual bed level for the upstream of the weir and downstream of the weir is surveyed
- Comparison of the surveyed bed level with the original bed level during design

5.1 Evaluation of Geological Investigation report of the Weir Area

Foundation of the weir is basaltic rock excavated to the fresh surface. Loose depositions, weathered and fractured materials were all avoided by blasting and jack hammering activities. On the foundation, at the center of the river course unexpected deep cut cavity was encountered during foundation excavation which led for modification of foundation design (geological cross section). The cavity was filled by C35 cyclopean cement concrete before founding the weir structure as shown on the drawing below (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011).

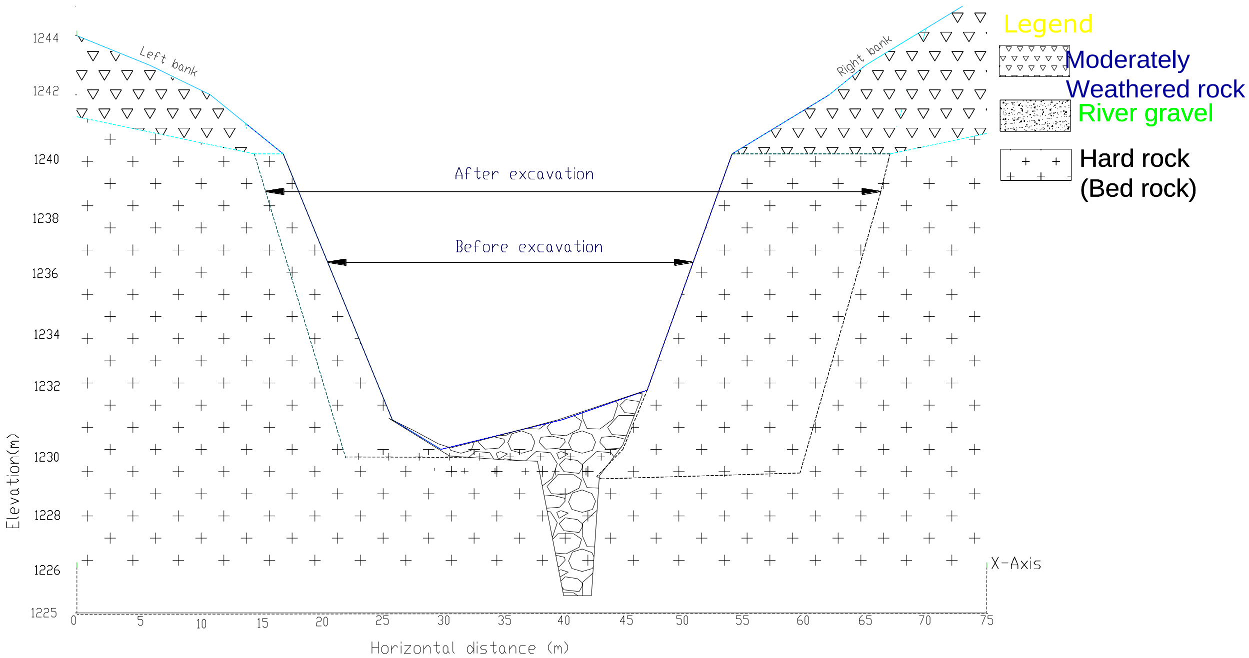


Figure 15 : Geological cross section of the weir axis before and after excavation

Photo 4: Cavity encountered at the center of the weir axis during foundation excavation

The rock downstream of the weir is dark colored fresh vesicular basalt crossed by two sets of joints; one horizontal and the other vertical (at an angle to the weir body). The oblique joint is 10-20cm wide and was filled by friable and loose weathered product of gravel, sand and fine materials to prevent damage of the structure due to sub-surface water. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

On the abutment the top most part is covered by red brown and dark brown color moderately weathered and fractured rock. On this top weathered part there is one major horizontal joint about 3cm wide filled by weathered rock fragments of sand and gravel with few fines. Mostly the unit is vesiculated and amygdaloidal textures of dark, red brown and grey colors. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

The formation upstream of the weir on the same abutment is the same and the joints are less pronounced on the excavated faces. While at the top crown of the weir body where the top weir portion is keyed to the rock is found to have light in color and amygdaloidal texture. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

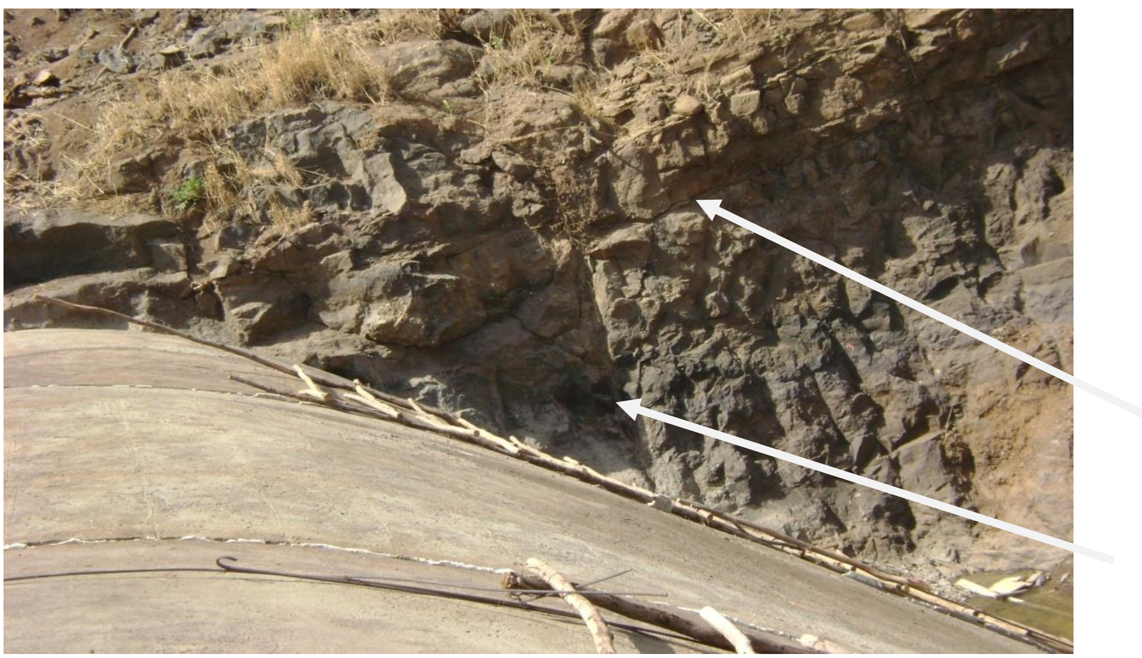


Photo 5: Left abutment d/s of the weir, one vertical joint intersecting in to the structure foundation and one horizontal joint

Upstream side excavation wall of the gate structures, the rock is fresh vesicular and amygdaloidal basalt intercalated with highly phorphiritic basalt. At this short excavation face there is one major horizontal joint 3-5cm wide filled by weathered products. In addition there is one horizontal joint 2cm wide having the same feature. There are also some vertical joints tight to 2mm opening. Both horizontal and vertical joint walls have weathered and stained surfaces which indicate sub-surface circulation during wet seasons. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

On the right abutment side of the under sluice structure, the top part is highly to moderately weathered and fractured unit. But most part of the face is fresh vesicular and amygdaloidal basalt with cavities filled by secondary mineralization. Here two types of rocks are intermingled together (vesicular dark and highly mineralized (rounded and sub-rounded shapes) reddish colored breccia type basalt. The major horizontal joint observed on other faces has continued to this direction. While at the bottom there is one sheared type portion (fracture) not observed on other faces due to excavation depth limitation is found to have sub-surface water seepage throughout the year. This sheared zone is about 25cm thick. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

On the same abutment 25m downstream of the weir axis 3 major horizontal, vertical and oblique joints are observed. Between the top two horizontal joints there is 1m spacing and about 2.2m between 2&3. The first and second joints are 2cm wide filled by weathered product while the third one is having different opening (tight to 3cm wide). The major vertical one is 5-15cm wide filled by loose gravel and sand sized weathered products. The oblique one is oriented SE and at bottom deflects to SW. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

Mostly through the top horizontal joint seepage has been taking place to the river course during wet seasons because there are clear indications of dissolved white mineral precipitations on the excavation walls and same to the middle one but at specific points. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

The top most rock unit is highly weathered and avoided during excavation while the existing top part is dark colored vesicular slightly weathered but fractured naturally and mostly due to the blasting effects. The central face about 2.5m thick is fresh red brown vesicular and amygdaloidal texture. The bottom part is dark and aphanitic type with minor vesicles. (Enterprise, Feasibility and Design Study of Tana Beles Integrated Sugar Development Project Head Work Design Report, 2011)

The rock units have almost similar strength from the field hammer test on lumps of rock pieces (3-5 blows).

At about 40m downstream of the axis there is one NNW-SSE oriented major joint (20-50cm wide) crossing the river course and filled by highly weathered product. On the left abutment the joint is cut by dry stream line. White mineral precipitation is observed from sub-surface water circulation through joints.

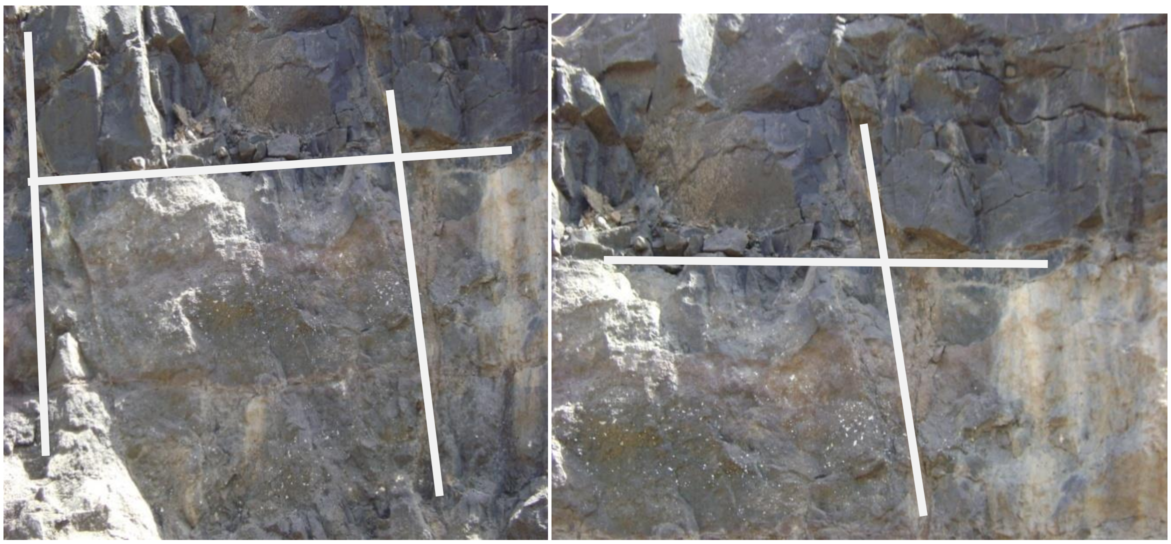


Photo 6: Horizontal and vertical joints on right abutment d/s of the weir axis.

Photo 7: Seepage through sheared rock at right abutment at the side of the under sluice structure

The deep cut cavity encountered at the center of the river course during foundation excavation was filled by C35 cyclopean cement concrete before founding the weir structure. A minimum excavation depth was made only on 3m of the total weir length which implies a poor foundation treatment which may lead to failure.

It was clearly stated on design report that many joints were observed and these joints were filled by weathered rock fragments of sand and gravel with few fines, loose gravel and sand sized weathered products and other materials. These filled portion may create weak point by itself which may lead to seepage and finally to failure.

The top most rock unit is highly weathered and avoided during excavation while the existing top part is dark colored vesicular slightly weathered but fractured naturally and mostly due to the blasting effects. This indicates the type of rock at the weir site is sheared as shown on the figure above, which is exposed to seepage.

5.2 Evaluation of the Foundation Treatment of the weir

The weir was constructed through the necessary working procedure between consultant and contractor in that every of the construction activity progresses by requisition from the contractor and instruction from the consultant. Every step of the contractor activity was checked by the consultant and compressive strength test of the concrete has been executed as reported from the contractor's representative.

Foundation shall be excavated up to competent bedrock and in this excavated area; Roller Compacted Concrete (RCC) has to be placed to create the weir foundation. The weir is not founded on competent rock as shown in the geological cross section of the weir. The geological cross section of the weir also shows an approximate smaller foundation excavation depth prior to this type of weir construction. Over-excavation approximately in limited areas was required due to presence of weathered, fractured, or faulted rock.

Geology played an integral role in determining depth of excavation. To build this type of weir, the foundation shall be well treated to some depth (7-8m) and the treatment should also be up to the downstream section.

The top most rock unit is highly weathered and avoided during excavation while the existing top part is dark colored vesicular slightly weathered but fractured naturally and mostly due to the blasting effects. The joint was filled by friable and loose weathered product of gravel, sand and fine materials to prevent damage of the structure due to sub-surface water. Over-excavation has to be made in zones of weathered rock and foundation should also be treated.

As discussed above the foundation is not treated as per the required standard which results subsurface erosion to prevail.

On the other side, it was known that there is a direct release from the hydropower but scouring was not expected during the design even though it was a must.in this situation Provision of energy dissipater is necessary to prevent scouring.

5.2.1 Surveying of the actual bed level upstream and downstream of the weir

Field visit surveys have been conducted for gathering out of data for the purpose of describing the nature of existing conditions and to compare existing conditions with the design standard of the diversion headwork.

Table 9: bed level surveying data collection for upstream and downstream of the weir

Easting	Northing	Elevation
247986.1	1290025	1236.771
247986.1	1290027	1236.667
247989.2	1290028	1236.569
247987.8	1290030	1236.464
247985.9	1290033	1236.138
247983.1	1290035	1236.314
247985.9	1290047	1236.254
247982.3	1290039	1236.183
247981.6	1290043	1235.649
247982.9	1290052	1235.758
248075.6	1290146	1236.067
248069.5	1290136	1236.052
248020.3	1290068	1235.647
248017.2	1290066	1235.568
247980	1290087	1235.261
247840.4	1289942	1228.044
247840.8	1289940	1228.306
247839.4	1289946	1227.994
247838.9	1289948	1227.996
247838.9	1289948	1227.996
247838.7	1289949	1227.989
247838.7	1289949	1227.989
247838.2	1289951	1227.902
247837.9	1289953	1227.88
247837.4	1289954	1227.821
247837	1289956	1227.789
247836.6	1289958	1227.714
247836	1289960	1227.698
247835.5	1289962	1227.697
247835.1	1289964	1225.597
247829.7	1289967	1225.756
247834.3	1289966	1227.46

5.2.2 Comparison of the actual bed level with the original bed level

For further conclusion about the foundation condition, it is important to check the existing elevation of the rock which may help to understand whether it is weathered or not.

Surveying data downstream of the weir is used to compare the existing original ground level with the river bed level during construction and reported on the chart below.

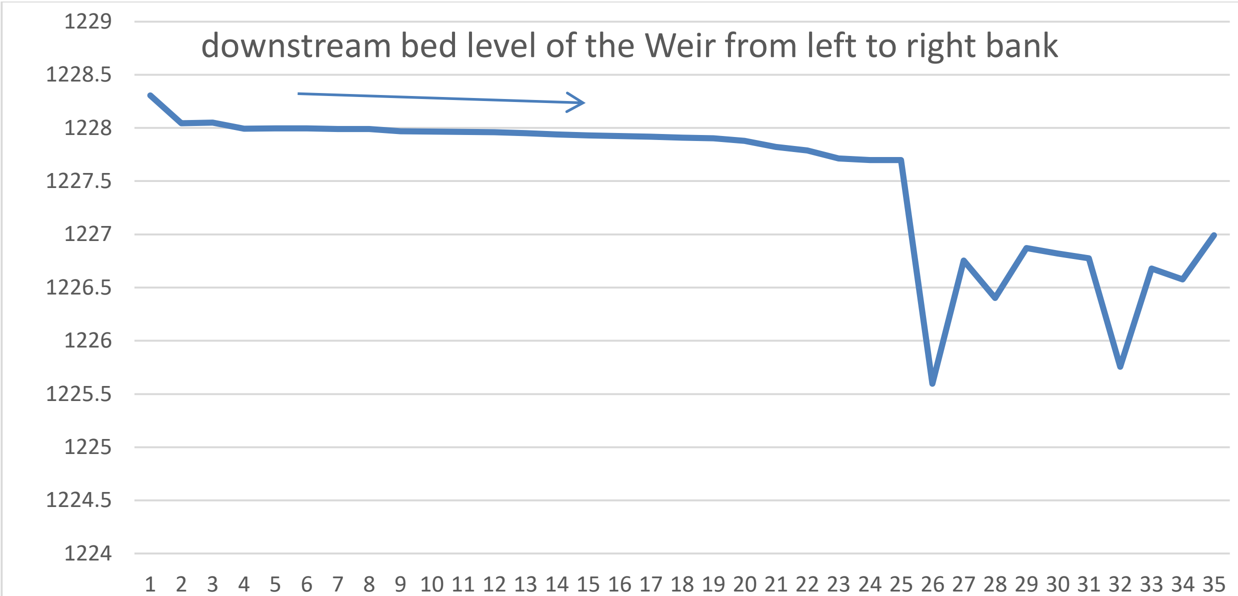


Figure 16: Bed level Cross-Section at the toe of the Weir

The chart clearly shows the existing bed level just downstream of the weir is lowered from the original bed level (1228.8).The bed level of right side of the weir, which is near to major failure area is highly reduced when compared with left side of the weir and the average scour depth is 2m.

The cross-section upstream of the weir is also the same with that of the downstream, which means there is an elevation difference between the left and right side of the channel along the flow direction.

It is also observed that the left side of the channel bed is rock and the right side is filled with alluvial deposit. I can judge from the site condition that the alluvial deposit on the left side is due to long term weathering.

Even though the right side has an alluvial deposit, the bed elevation is still lower than the left side. Due to this the river flow will be more dominant on the right side of the channel than on the left side. Consequently, as the water exert much pressure on that side, crack will prevail and become potential failure cause of the weir.

Surveying data upstream of the weir is collected and used to compare the existing original ground level of the right side of the channel and the left side of the channel along the flow direction.

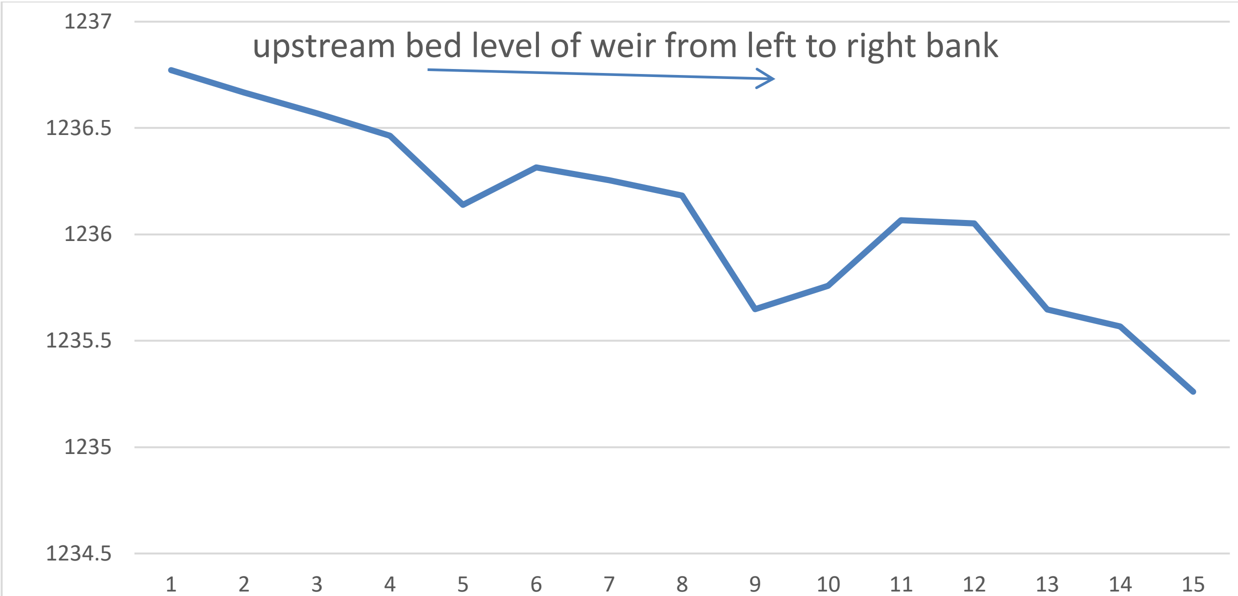


Figure 17: Upstream bed level comparison

The chart clearly shows the existing ground elevation of right side of the channel is lower than the left side. In this situation the water will select to flow on the right side and excessive pressure will be exerted on that side of the weir, which may cause failure of the structure.

6 Hydraulic analysis

In the hydraulic analysis, the Hydraulic design report is evaluated and the main factor which may facilitate the failure of the weir is identified. The method and data used include:

- The hydraulic design report has been evaluated
- Field observation has been made to identify problems
- Factors which facilitate the failure of the weir is identified

6.1 Clogging of under sluice

The under sluices are the gate controlled openings in the weir with crests at low level. They are located on the same side as off taking canal. The usual functions of the under sluices are:

- To preserve a clear and defined river channel approaching the canal head regulator

- To scour silt deposited in front of canal regulator and control silt entry in the canal
- To lower the highest flood level by providing greater discharge per meter length than the weir.

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 will be removed by different techniques. 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.

The width of the under sluice portion is determined on the basis of the following considerations. (Fikru, 2015)

- It should be capable of passing at least double the canal discharge to ensure good scouring capacity
- It should be capable of passing about 10 to 20 percent of the maximum flood discharge at high floods
- It should be wide enough to keep the approach velocities sufficiently lower than critical velocities to ensure maximum settling of suspended silt load. .

In case of Tana Beles diversion headwork, out of the three bays of the under sluice two of them were fully closed and cannot be opened freely due to malfunctioning of gate. The rest one was opened by force but cannot be closed. hence, Efficient flushing of the sediment from the under sluice pocket is not ensured.as a result sediment accumulation will be high around the under sluice which increases its bed level and this cause the concentrated water to flow into weir finding lower bed level. Consequently high water pressure will be exerted on the structure and this will have an effect on the weir.

6.2 Retrogression Negligence

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

In the alluvial rivers due to construction of weir the regime of the river is affected. Initially most of the sediment is dropped in the pond and relatively clear water passes over it. This clear water scours the downstream river bed to make up the deficiency in the silt load and causes a progressive lowering or retrogression of downstream levels. If this retrogression or lowering of bed levels in the early stages is not taken into account it may lead to undermining of the structure. A retrogression of about 1.0 to 2.0 meters at low water levels and 0.3 to 0.5 m at high flood levels has been observed on weirs in India. The actual design practice is to allow a retrogression of 0.5m increasing linearly up to 2.0m at low discharges. In the case of Tana the Beles Weir, it was believed that the strata at the downstream of the weir site is rocky (vesicular basalt), hence retrogression /scour was not expected. No concrete floor is provided both on the upstream and downstream of the weir. On my presence at the site I observed upstream and downstream of right side of the weir which was previously concluded as rock is eroded.

Therefore negligence of retrogression may be taken as the potential cause for Tana Beles weir failurity.

6.3 Absence of Concrete Floor

The head work site is founded on the naturally bedded basaltic rock, which extends upstream and downstream of the weir site. The abutments of the river around the weir, downstream and upstream of the weir axis are basaltic rocks, which has been considered to be quite competent to withstand the forces due to design flood flowing over the weir and it was decided to dispense with the lowering of floor up to EL. 1228 as a design standard and provision of concrete/ RCC floor on upstream and downstream. But taking into account for the foundation, No concrete floor is provided both on the upstream and downstream of the weir.

Physical observation of the weir site exposes part of downstream and upstream of the weir axis is not basaltic rocks and design consideration does not guarantee for the actual site condition. Absence of the concrete floor on the design should be taken as the potential cause for Tana Beles weir failurity.

6.4 Absence of Upstream and downstream cut off

Cut-off is barrier provided below the floor of the weir both at the upstream and the downstream ends. It may be in the form of concrete lungs or steel sheet-piles. The cut-off extends from one end up to the other end (on the other bank). The purpose of providing cut off is two-folds as explained below.

During low-flow periods in rivers, when most of the gates are closed in order to maintain a pond level, the differential pressure head between upstream and downstream may cause uplift of river bed particles. A cut off increases the flow path and reduces the uplift pressure, ensuring stability to the structure. During flood flows or some unnatural flow condition, when there is substantial scour of the downstream riverbed, the cut offs or sheet piles protect the undermining of the structures foundation.

The upstream and downstream cut offs should generally be provided to cater for scours up to 1 R and 1.25R respectively where R is the depth of scour below water level and is given by :

$$R = 0.473\,(Q/f)^{1/3} \quad \text{or } R = 1.35\,(q/f^2)^{1/3}$$

However since the strata at Tana Beles weir site is taken as basalt rock and is considered to be un erodible, no scour is expected to take place during design. On the design, since no concrete floor has been provided, no cut offs at the end of floor is required.

In contrary to this, Physical observation of the weir site exposes part of downstream and upstream of the weir axis is not basaltic rocks and design consideration does not guarantee for the actual site condition. Absence of the cutoff on the design should be taken as the potential cause for Tana Beles weir failurity.

6.5 Absence of Upstream and downstream protection works

The impervious floor of a weir and head regulator is normally protected on the upstream as well as downstream by loose aprons. However as the rock at the weir site is considered quite competent, no scour is expected and hence no additional protection works have been provided on the design of Tana Beles Weir.

6.6 Absence of Energy Dissipater

The high energy loss that occurs in a hydraulic jump has led to its adoption as a part of high energy dissipater system below a hydraulic structure. Three types of energy dissipaters (HAGER, 1992) have been commonly used: stilling basins, flip buckets, and roller buckets. Each dissipater has certain advantages and disadvantages and may be selected for a particular project depending upon the site characteristic. Out of the three energy dissipaters types, flip bucket is commonly used on weir design.

The flip bucket energy dissipater is suitable for sites where the tail water depth is low (which would require a large amount of excavation if a hydraulic jump dissipater were used) and the rock in the downstream area is good and resistant to erosion. It is also called ski-jump dissipater, throws the jet at a sufficient distance away from the structure` where a large scour hole may be produced. Initially, the jet impact causes the channel bottom to scour and erode. The scour hole is then enlarged by a ball-mill motion of the eroded rock pieces in the scour hole. A small amount of the energy of the jet is dissipated by the internal turbulence and the Shearing action of the surrounding air as it travels in the air. However, most of the energy of the jet is dissipated in the plunge pool. (HAGER, 1992)

In the case of Tana Beles weir design, as water flows by high energy down the sloping glacis and this maximum energy is not dissipated by flip bucket due to absence on the design, the energy will scour the downstream floor until the main weir and this may be taken as failure causes of Tana Beles weir.

6.7 Absence of river training work

It is necessary at many instances to narrow down and restrict the course of the river through the weir and it is achieved by the use of the river training works. Proper alignment of guide bunds is essential to ensure satisfactory flow conditions in the vicinity of the weir. In case of wide alluvial banks, the length and curvature at the head of the guide bunds should be kept such that the worst meander loop is kept away from either the canal embankment or the approach embankment. If the alluvial bank is close to the weir, the guide bunds may be connected to it by providing suitable curvature, if necessary. If there is any out-crop of hard strata on the banks, it is advisable to tie the guide bunds to such control points.

Because, no standard river training work adopted for Tana Beles weir design, flow conditions in the vicinity of the weir is not satisfactory and right side of the weir is mostly affected by the flow as compared to the left which apparently leads to its failure.

7 Structural analysis

In general, structural analysis includes stability analysis of the weir body. In this analysis, the weir has been checked for stability against all modes of failure.

7.1 Forces acting on the weir

A weir was considered to be subjected to the following main forces:

- Self-weight of the weir
- Water pressure
- Sediment / silt pressure
- Uplift pressure
- Wave pressure
- Wind pressure
- Earth quake pressure

As the height of the weir is not much, the wave pressure and wind pressures are ignored. Since the project area is in earth quake free zone, no earth quake pressure is considered.

Once a section of the weir has been designed, it has to be analyzed and checked, whether it satisfies the safety requirements. Gravity method (or two dimensional methods) has been used for the analysis of the weir. In this method of analysis the weir is considered as a two dimensional structure. A unit length of the weir is considered for the analysis. The weir is assumed to consist of a series of vertical cantilevers of unit length and fixed at the base. These cantilevers are assumed to be independent of one another. The loads acting on the cantilevers are transferred to the foundation through the cantilever action. The stability of these cantilevers is checked against all possible modes of failure.

7.2 Stability against Uplift

The weir is laid on durable and fresh rock unit after removing the weathered and jointed rock and the weir body is anchored in to foundation rock, so that weir and foundation behave as one unit. No uplift pressure is, therefore, assumed to be created.

Physical observation of the actual site condition exposes the stability analysis against uplift have to be checked. As the actual foundation condition shows that provision of impervious floor with cut off is necessary to prevent failure due to uplift and piping and the structure has to be checked for its stability.

But from physical observation and evaluation of the foundation condition of the weir site, the structure is subjected to uplift.

7.3 Stability against Overturning

Overturning failure occurs when the overturning moment exceeds the resisting moment. Thus, failure of the weir by overturning is usually preceded by tension failure or crushing failure. Therefore, a weir may be considered safe against overturning if the criterion of no tension at any point in the weir body is satisfied and also the maximum compressive stress does not exceed the allowable limit. To be on the safer side the resisting moment should exceed the overturning moment at least by 100%.

The stabilizing moment is greater than destabilizing moment and Factor of safety against overturning for weir section is 1.75 (>1.5). There is no lifting up of the structures heel and the structure is not susceptible to any tension on the base.

7.4 Stability against shear and Sliding

The sliding failure occurs when the weir slides over its base or when part of the weir lying above the horizontal plane slides over that plane. To avoid this failure at any horizontal section or at the base, the weir should be designed so that the sliding forces do not exceed the resisting force. The weir structure is checked for stability against shear and sliding and the result (FS=1.6>1) manifested that the diversion structure is safe against shear and sliding.

7.5 Over all stability of the weir

The following procedure is used for checking the stability of the weir.

(i) All the forces , vertical and horizontal, acting on the weir are determined

(ii) The algebraic sum of all the horizontal forces(ΣH) and vertical forces(ΣV) is determined

(iii) The moments of all the forces components about the downstream edge or toe is determined. The algebraic sum (ΣMr) of resisting moments and algebraic sum (ΣMh) of overturning moments is determined. Also the net moment (ΣM) about the toe is computed. Thus,

$\Sigma M = \Sigma Mr - \Sigma Mh$

(iv) Determine the distance x of the point where the resultant R strikes the base.

$X = \Sigma M/\Sigma V$

Determine the eccentricity e =0.5B-x, ensure that the eccentricity is with in the middle one third of the base width.

(v) Determine the Factor of safety against overturning ($\Sigma Mr/\Sigma Mh$)

(vi) Determine the factor of safety against sliding ($\mu\Sigma V/\Sigma H$)

The overall stability analysis involves checking for the design margin for eccentricity, sliding, overturning and bearing capacity.

The eccentricity of loads worked out is found to be within one third of the base implying thereby that the pressure at the base shall always be positive and there shall be no tension. The factor of safety against overturning and sliding work out as 1.75 and 1.6 respectively. These factor of safeties are considered to be safe.

Stability Calculation

S.N	Item	Symbol	Data	Unit	Remark		
1	unit weight of concrete	γ_c	24	KN/m^3			
2	unit weight of soil					affluxed upstream water level	1241.27
	dry	γ_d	14.8	KN/m^3		crest level	1235.8
	wet density	γ_s	18	KN/m^3		structure bed level	1228.8
	saturated unit weight	γ_b	19.09	KN/m^3		structure bed level	1228.8
	buoyant unit weight	γ_f	9.28	KN/m^3		bottom width	12.22
3	unit weight of water	γ_w	9.81	KN/m^3		Area	9.15
4	friction coefficient	f	0.55				35.75
5	internal friction angle of soil	φ	30				12.675
6	affluxed upstream water level		1241.270				
7	crest level		1235.800				
8	structure bed level		1228.8	m			
9	height	B	7.00	m			
10	bottom length	C1	12.22	m			
11	Area 1	A1	1.30	m			
12	Area 2	A2	1.00	m			
13	Area 3	A3	0.40	m			

14	width of weir	L	30.00	m			
S.N	desc.of acting force	Acting direction	Formula	calculation data	unit	Moment arm	Moment
1	self-weight U1	↓	G=γc*B*A1	1,537.20	KN	2.29	3,516.35
	self-weight U2	↓	G=γc*B*A2	6,006.00	KN	3.58	21,471.45
	self-weight U3	↓	G=γc*B*A3	2,783.43	KN	8.84	24,605.52
2	water weight Gw	↓	Gw=γw*H*C1*B	10,464.17	KN	6.11	63,936.07
3	Upstream water pressure P1	→	$P_1=1/2\gamma_w H^2 *B$	(5,339.12)	KN	4.16	(22,192.96)
4	Uplift pressure W2	↑	$W_2=1/2\gamma_w H*B*L$	(5,232.08)	KN	8.15	(42,624.04)
	∑m						48,712.38
	∑Fv			15,558.71			
	∑Fh			(5,339.12)			
	∑Fu			(5,232.08)			
	anti-overturning moment						113,529.38
	overturning moment						(64,817.00)
	Safety factor of anti-uplifting force			2.97			
	Safety factor of anti over turning force						1.75
	Safety factor of anti-sliding force			1.60			
	eccentricity			2.98			
	Maximum stress			118.23			
	Minimum stress			29.94			

8 Remedial design of the weir

The problem related to the weir failure has been investigated on the previous chapter.in this chapter remedial design will be recommended.

As the design report indicates, for the design of Tana Beles weir a design flood of 1 in 100 year frequency (1033 m3/s) for the hydraulic design is implemented and the corresponding value estimated in this thesis is 1147.5 m3/s, which shows the value used for the design has a deficiency of 114.5 m3/s when compared with the model simulation result.

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

8.1 Design Flood Discharge

For the purposes of design of items other than free board, a design flood of 50 or 100 years frequency may be considered sufficient for hydraulic design (IS:6966) of weir and under sluices. For the design of free board a minimum of 500 year flood may be adopted. However for the remedial redesign of weir a design flood of 1 in 100 year frequency (1147.5 m^3/s) has been adopted. It is noted that by using the HEC-Hms model, a peak flood of 1147.5 m^3/s is obtained for 1 in 100 year frequency, which is significantly different from the one previously used during the design (1033 m^3/s).

8.2 Rating curve

In the absence of detailed data a preliminary rating curve has been prepared by computing the discharges at different water levels using the formula.

$$Q = 1/n\ A\ R^{2/3} S_f^{1/2}$$

Where Q = discharge in cu m/ sec

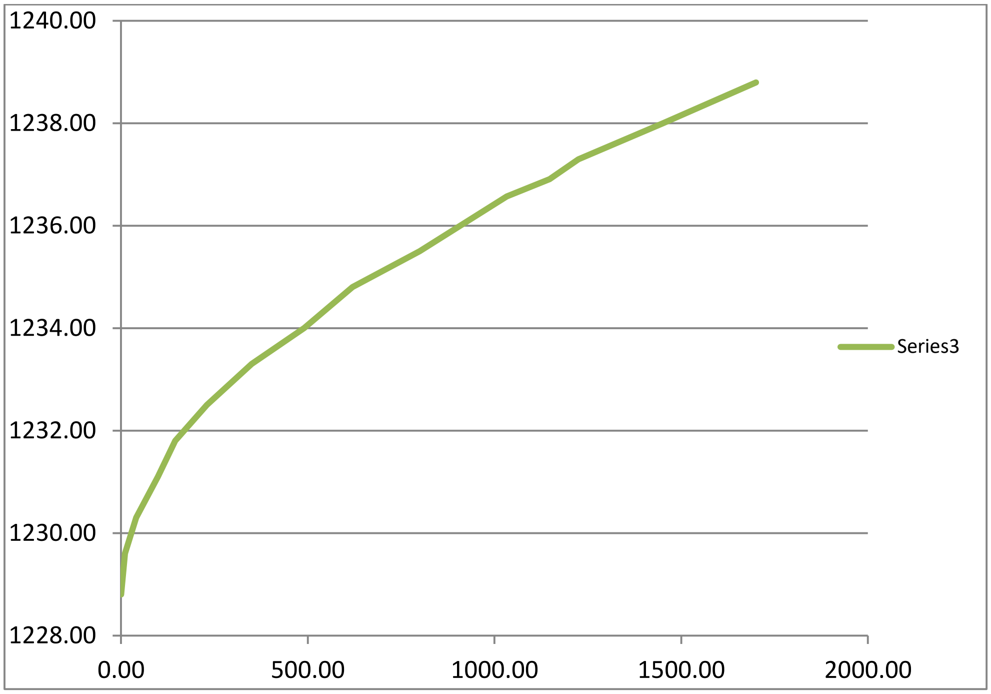
n = Rugosity coefficient

A = Area of cross section of flow in m^2

R = Hydraulic mean radius in m and

S_f= friction slope

The flood level for the design flood of 1 in 100 year frequency works out as 1236.91 m.



8.3 Pond Level

Pond level in the under slice pocket upstream of the canal head regulator and upstream of weir portion is generally obtained by adding the working head to the designed full supply level in the canal. In the present case against a FSL of 1235.3m in the canal, a Pond level of 1235.80 has been adopted.

8.4 Optimum water way and afflux

The length of water way, corresponding discharge per meter and the afflux are co - related. Afflux is generally limited to 1m but may be kept higher if permissible. In the present case, since the river training is in consideration afflux of 0.26 is provided for a design flood of 1147.5 m^3/s.

In deep and confined rivers with stable banks the overall water way of weir and under sluices (between abutments) including thickness of piers should be approximately equal to the actual width of river at the design flood.

The crest level of the under sluices is usually kept as near the bed level in the deepest channel as is practically possible. The under sluice crest is kept lower to attract a deep current in front of regulator so that dry weather current may remain near the regulator. It would be desirable to keep the crest and upstream floor level in front of under sluices at the same level. The crest level of the under sluices has been kept at 1229.3(the deepest bed level of the river at the weir site+ retrogression).

The crest level of the weir is kept at the pond level (1235.80), as no shutters or gates are proposed to be provided on the weir crest.

8.5 Adequacy of the waterway

Having tentatively decided the crest levels as well as the water way of the under sluice and the weir proper, it is necessary to check that the maximum flood discharge passes down the works without excessive afflux. The following discharge formulae have been used for this purpose. Since Ogee type of crest has been proposed for the weir, the following discharge equation has been used,

$Q=CBh^{1.5}$

Where,

Q: The discharge over the weir portion

B: The Effective water way of the weir portion

H: The Effective head (head + head due to velocity of approach)

C: The Coefficient of discharge for the ogee type crest (2.2)

Discharge Over the weir			
crest level of the weir		1235.8	m
Affluxed High Flood Level		1241.27	m
Design for flow ,Q100		1147.50	m^3/sec
Main Weir clear width, B		30.00	m
Depth of flow over main weir ,h_1		5.47	m
	take h_1	5.47	m
Discharge		767.60	m^3/sec
flow intensity ,q		25.59	m^2/sec
Discharge through scour sluice			
scour sluice width		3.30	m
scour sluice height		2.70	m
No of gates		3.00	
Scour sluice total width, Bt		9.90	m
crest level of the Under sluice		1228.80	m
Depth invert below the main weir crest		4.30	m
depth of flow over scour sluice, h1		9.77	m
Discharge		604.65	m^3/sec

flow intensity ,q												61.08	m^2/sec		
total flow over the weir												1372.25	m^3/sec		
Spare capacity												224.75	m^3/sec		

This show, the estimated flood (1147.5cum/s) can pass over the weir safely.

8.6 Design of the ogee weir profile

The profile of the ogee (weir crest) is designed for the design head. The design head is generally chosen to give the maximum practical hydraulic efficiency, in keeping with the operational requirements, stability and economy. However, crack propagation and expansion tendency along the expansion line is observed. Beside this, the plastering from the ogee weir surface is detached.

To avoid the cracks on the ogee weir surface it is required to cast a layer of C-35 reinforced concrete after chiselling the top concrete surface. To cast a constant layer of reinforced concrete on the chiselled surface, Surveying of the original elevation of the top surface of the weir is necessary. Without changing the original ogee weir profile, a constant 50cm reinforced concrete layer can be added on the original top surface of weir as shown below on the chart.

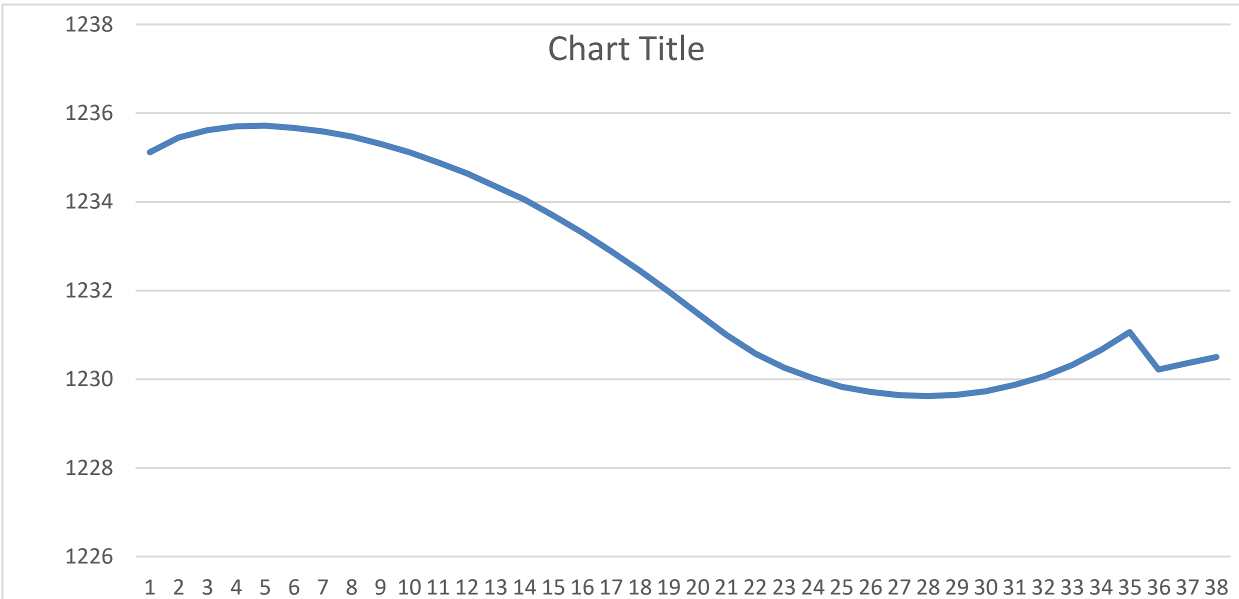
The type of ogee selected is vertical upstream face.

Table 10: ogee weir profile

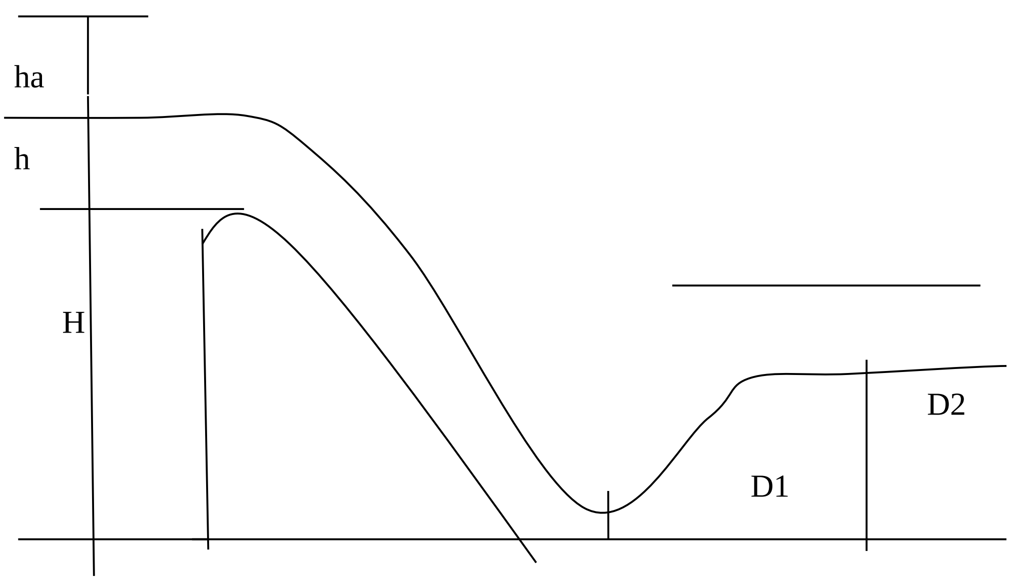
y	river bed	Top ogee level
6.75	1228.37	1235.12
7.08	1228.37	1235.45
7.25	1228.37	1235.62
7.33	1228.37	1235.7
7.35	1228.37	1235.72
7.3	1228.37	1235.67
7.22	1228.37	1235.59
7.1	1228.37	1235.47
6.94	1228.37	1235.31
6.75	1228.37	1235.12
6.52	1228.37	1234.89
6.27	1228.37	1234.64
5.98	1228.37	1234.35
5.68	1228.37	1234.05
5.32	1228.37	1233.69
4.94	1228.37	1233.31
4.52	1228.37	1232.89
4.08	1228.37	1232.45
3.61	1228.37	1231.98
3.12	1228.37	1231.49
2.63	1228.37	1231
2.21	1228.37	1230.58
1.89	1228.37	1230.26
1.65	1228.37	1230.02
1.46	1228.37	1229.83
1.34	1228.37	1229.71
1.27	1228.37	1229.64
1.25	1228.37	1229.62
1.28	1228.37	1229.65
1.36	1228.37	1229.73
1.5	1228.37	1229.87
1.69	1228.37	1230.06
1.95	1228.37	1230.32

2.29	1228.37	1230.66
2.69	1228.37	1231.06
1.85	1228.37	1230.22

1.99	1228.37	1230.36
2.13	1228.37	1230.5



8.7 Energy dissipation



Total energy line elevation above downstream river bed and upstream of weir and at hydraulic jump shall remain same. Therefore equating the total energy lines at two sections, values of v1 and d1 shall be worked out.

The required Pond Level necessary to maintain Full Supply Level in Main Canal to irrigate		
net irrigable area of 47,000 ha	1234.3	m
Peak Flood	1147.50	m^3/sec
over flow way of weir	39.90	m
distance b/n inner face of abutment	43.40	m
Discharge intensity =peak flood/ L	28.76	m^2/sec
average river bed level	1228.80	m
Downstream water level	1236.91	m
Upstream water level	1238.71	m
Downstream total energy level	1237.07	m
Upstream total energy level	1238.87	m
Head loss (H_L)	1.80	m
D/S specific energy E_{f2}(from Blench curves)	7.26	m
U/S specific energy E_{f1}	9.06	m
($E_{f1} = E_{f2} + H_L$)		
Level at which jump would form	1229.81	m
(D/S T.E.L. - E_{f2})		
Pre jump Depth D_1 corresponding to E_{f1}	3.00	m
(from energy of flow curves)		
Post jump Depth D_2 corresponding to E_{f2}	6.15	m
Length of concrete floor required beyond		
the jump = $5(D_2 - D_1)$	15.74	m
Froude No. F = q/SQRTgD_1^3	1.77	
The downstream floor may be provided at reduced level of		
	1228.30	m
with a horizontal length of	16.00	m

Design of energy dissipaters :			
though the froud number is 1.77 for better safety provide, Stilling Basin for next group of froude number			
Type of U.S.B.R Stilling Basin **(II)**	(Froude Number lies between 2.5 and 4.5)		
Chute Blocks :	Height of Chute Block	0.74	m
	Width of Chute Block	3	m
	Top length of Chute Block	1.3	m
	Spacing between Blocks	6	m

8.8 Sheet piles

Depth of sheet piles from scour considerations :			
U/S sheet pile :			
Silt Factor		2.00	
Discharge intensity 'q' at beginning of U/S floor		26.44	m^2/sec
Depth of scour 'R' = $1.36\{q^2/f\}^{1/3}$		9.58	m
Anticipated scour = 1.25 R		11.98	m
R.L. of the U/S bottom of scour hole		1226.73	m
Minimum depth of U/S cutoff below bed level			
	$y_d / 3 + 0.50$		
U/S Water Depth		9.91	m
Minimum depth of U/S cutoff below bed level		3.80	m
RL of the bottom of sheet pile		1225.00	m
The U/S sheet pile bottom El. Should be at or below the stilling basin level			
		1224.90	m
D/S sheet pile :			
Discharge intensity 'q' at end of the impervious floor		26.44	m^2/sec
Depth of scour 'R' = $1.36\{q^2/f\}^{1/3}$		9.58	m
Anticipated scour = 1.50 R		14.37	m
R.L. of the bottom of scour hole =		1222.54	m
Minimum depth of d/s cutoff below bed level			
D/S Water Depth		8.61	m
Minimum depth of d/s cutoff below bed level		4.81	m
$y_d / 2 + 0.50$(y_d is D/S water depth in m)			
RL of bottom of sheet pile		1223.50	m
The D/S sheet pile bottom at		1222.50	m

8.9 Upstream and downstream Concrete floor

Total floor length and exit gradient :					
The exit gradient should be checked for the condition when there is no water in the stilling basin					
When pond level is maintained U/S of Weir; this provides the worst static condition.					
	G_E	=	0.2		
Maximum static head =				6.00	m
Depth of D/S cut off =				5.80	m
Hence,					
	$\frac{1}{\pi\sqrt{\lambda}}$	=	$\frac{G_E * d}{H}$	0.19	
From Khosla's exit gradient curve,		α	=	4.31	
Hence requirement of total floor length b =			$\alpha * d$	24.98	m
Adopted total floor length				24.98	m
Provide downstream horizontal floor				4.00	m
The floor length shall be provided as below :					
Stilling basin length				4.00	m
ogee profile length				13.43	m
downstream curve				3.05	m
chute block				3.00	m
Upstream floor				1.50	m
Total floor length				25.00	m

8.10 Pressure calculation

Pressure calculations :							
Let us assume the floor thickness in the U/S as						1.00	m
and near the downstream cutoff as						1.25	m
Upstream sheet pile							
Let the thickness of cut off wall						0.30	m
d	=					3.90	m
b	=					25.00	m
1		d					
α	=	b				0.16	
From Khosla's Pressure curves							
λ =						3.744	
ϕ_d =						76%	
ϕ_C =						65%	
					$\phi_D - \phi_C$ =	11%	
Correction for floor thickness =						2.76%	(+ ve)
Correction for interference due to d/s sheet pile line, C = 19√D/b'.(d+D)/b							
where,							
d	=		1227.80	-	1224.9	2.90	m
D	=		1227.80	-	1222.5	5.30	m
b'	=					24.40	m
b	=					25.00	m
Then,							
C =						2.904%	(+ ve)
ϕ_C corrected =						71.09%	
Downstream sheet pile							
d						5.80	m
b						25.00	m
1		d				0.23	
α	=	b					
From Khosla's Pressure curves							
λ =						2.712	
ϕ_{E1} =						42%	
ϕ_{D1} =						28%	
					$\phi_{E1} - \phi_{D1}$ =	13%	
Correction for floor thickness =						2.86%	(- ve)
Correction for interference due to u/s sheet pile line, C = 19√D/b'.(d+D)/b							
Where, d			1227.05	-	1222.50	4.55	m
D			1227.30	-	1224.90	2.40	m
b'						24.40	m
b						25.00	m
Then,'C =						1.657%	(- ve)
ϕ_{E1} corrected =						37.02%	

8.11 Floor Thickness

Floor thickness :							
The maximum static head will occur on the floor when there is no flow over the weir							
But U/S water level is at pond level.							
Thus maximum static head =						6.00	m
The subsoil hydraulic gradient line shall be drawn for the maximum static head only							
Since the floor thicknesses are governed by this critical condition.							
Downstream floor							
(i)	Up to	1.30	m from D/S end				
	Unbalanced head =					2.305	m
	Floor thickness required =					1.92	m
	Provide floor thickness =			2.00	m in	1.30	m length D/S
(ii)	At	1.30	m from D/S end to	2.70	from D/S end (toe of D/S glacis)		
	Unbalanced head =					2.422	m
	Floor thickness required=					2.02	m
	Provide floor thickness =					2.10	m
(iii)	At	2.70	m from D/S end to	4.00m	from D/S end (toe of D/S glacis)		
	Unbalanced head =					2.531	m
	Floor thickness required					2.11	m
	Provide floor thickness					2.20	m
upstream floor							
	Provide floor thickness					1.30	m

8.12 Protection works

Protection works beyond impervious floor								
U/S Protection :								
Depth of scour 'R'	=	$1.35\{q^2/f\}^{1/3}$					9.58	m
Anticipated scour	=	1.25R					11.98	m
Upstream scour level	=						1226.73	m
Min scour depth 'D' below U/S floor								
Hence 'D' =							2.07	m
(a) Block protection								
Block protection shall be provided minimum equal to 'D' in length								
and minimum 4 rows of blocks.								
Provide	4	rows of	0.60	m X	0.60	m X	0.6	
C.C. blocks over	0.15	gravel apron in a length					of 2.40m	
(b) Launching apron								
Loose apron 1.5 D in length consisting of either boulders of not less than 40 kg or wire								
boulder 'crates should be provided so as to ensure a minimum thickness of 1 m in								
Launched position								
Thickness of launching apron=							1.00	m
(Sum of thickness of blocks & Filter)								
Quantity of launching apron required =						2.25 D		
							4.65	m
Length required =		Quantity of launching apron / Thickness of launching apron					4.65	m
Provide launching apron		1.00	m deep in a length of				4.70	m

D/S Protection :			
Depth of scour 'R'	= 1.35{q^2/f}$^{1/3}$	9.58	m^2/sec
Anticipated Scour	= 1.5R	14.37	m
Downstream scour level		1213.93	m
Scour depth 'D' below D/S floor		14.37	m
(a) Block protection			
Block protection shall be provided in a minimum length equal to '2D' which comes			
Provide 24 rows of 0.6 m X 0.6m X 0.6 m			
C.C. blocks with 5 cm gap filled with Sand over 0.6 m thick			
graded filter in a length of 15.55 m.			
(b) Inverted filter			
AS per Code of Practice, Just at the end of concrete floor on the downstream an inverted			
filter '1.5 to 2 *D* long (*D* being the depth of scour below bed), is provided below the			
Concrete block protection. The graded inverted filter of		0.6 m	thick is proposed
to be provided below the concrete block in a length of		15.55	m
The graded inverted filter should conform to the following design criteria :			
$\frac{\text{D 15 of filter}}{\text{D 15 of foundation}}$ <= 4 <= $\frac{\text{D 15 of filter}}{\text{D 85 of foundation}}$			
(c) Launching apron			
Downstream of the inverted filter, loose apron 1.5 D in length consisting of either			
less than 40 kg or wire boulder crates should be provided so as to ensure			
a minimum thickness of 1 m in launched position.			
Thickness of launching apron=		1.00	m
(Sum of thickness of blocks & Filter)			
Quantity of launching apron required =	2.25 D	32.33	
Length required =	$\frac{\text{Quantity of launching apron}}{\text{Thickness of launching apron}}$	32.33	m
Provide launching apron	1 m deep in a length of	32.40	
because the area is rocky, provide launching apron in a length of 5m			
<u>Toe wall :</u>			
Also provide a 0.9m thick and 1.00m deep masonry toe wall between			
the filter and the launching apron			

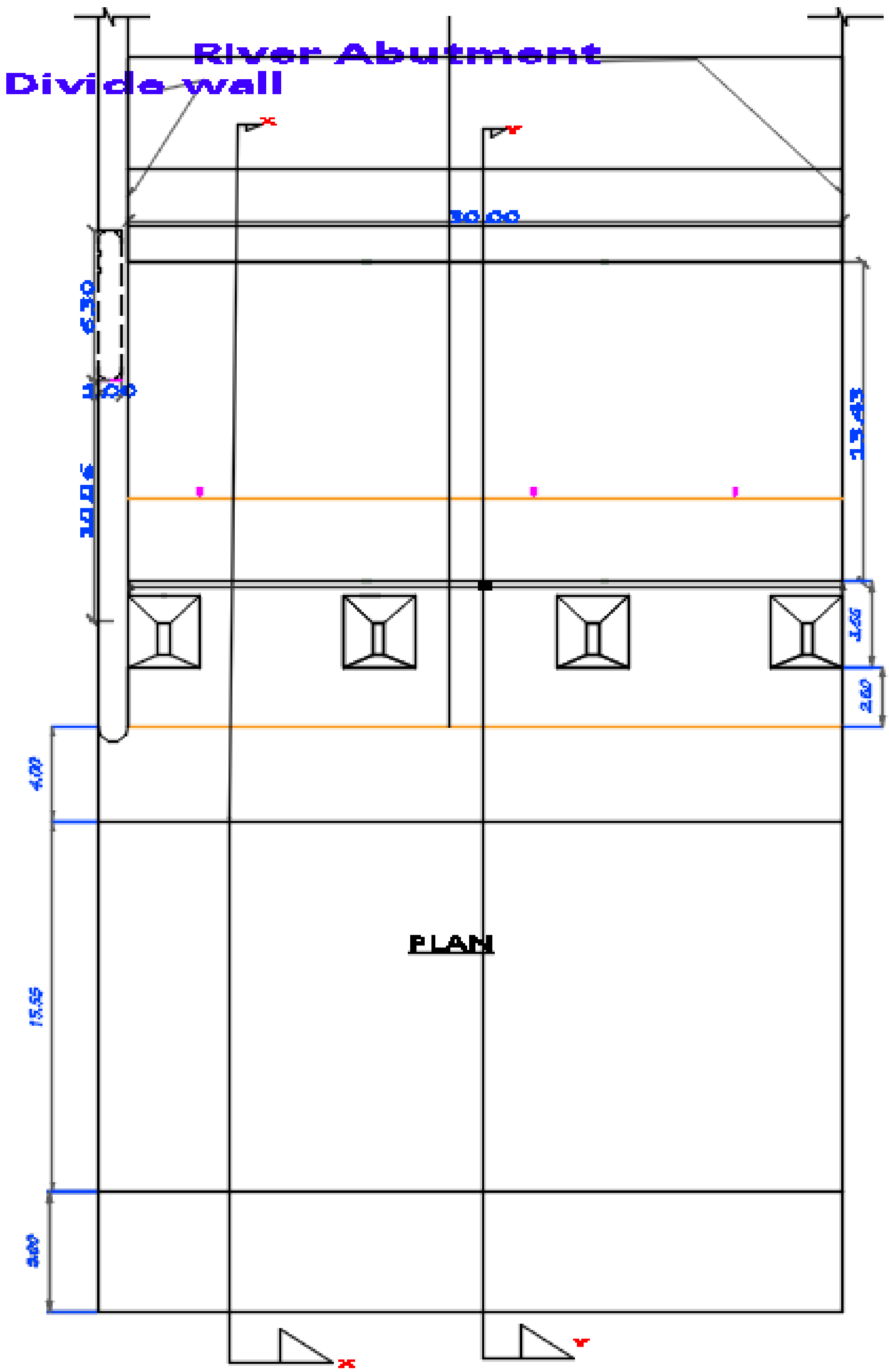


Figure 18: plan view of the weir

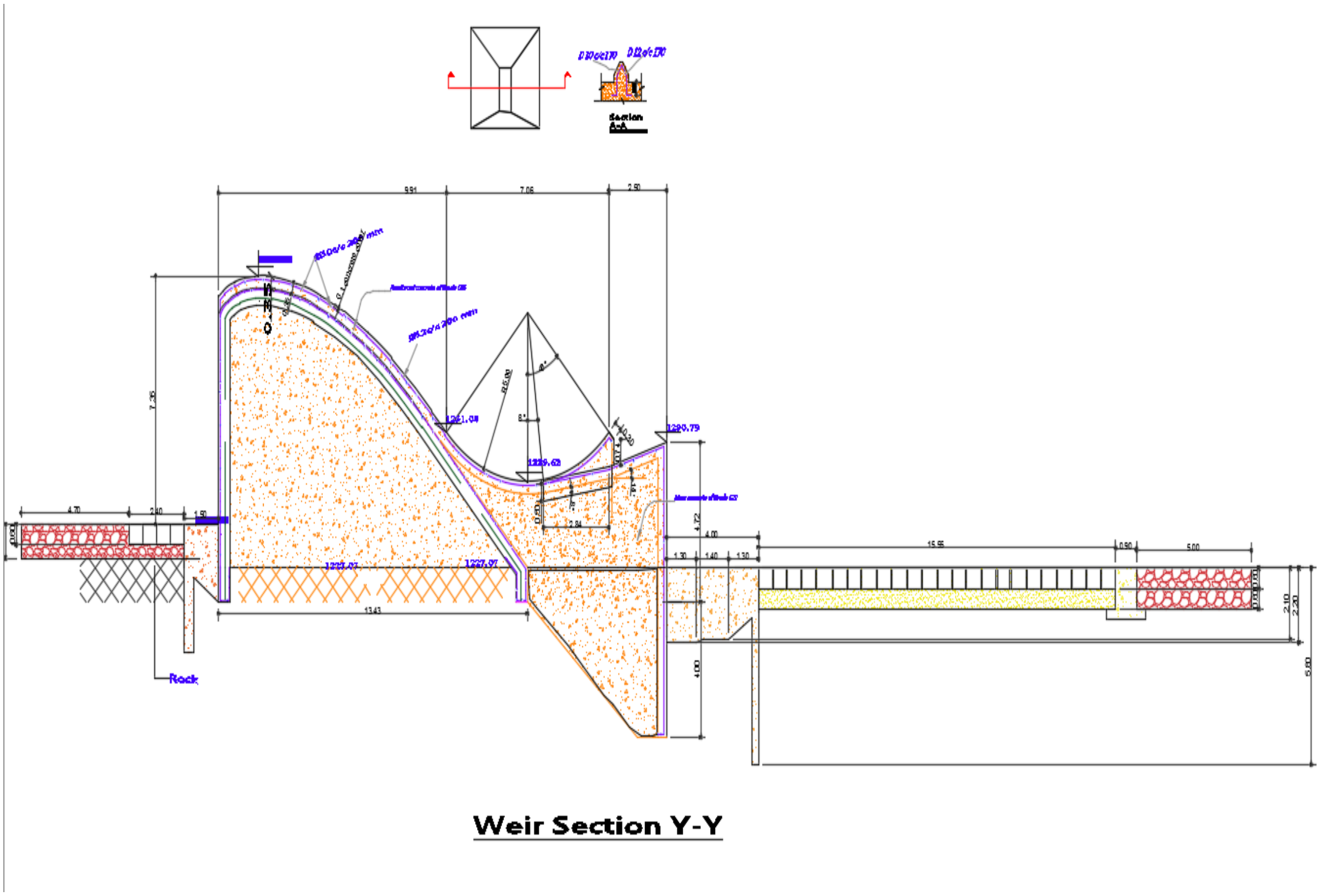


Figure 19: section view of the weir

9 Conclusion and Recommendation

9.1 Conclusion

On the hydrology analysis, It is noted that by using the HEC-Hms model a peak flood of 1147.5 m3/s is obtained for 1 in 100 year frequency, which is significantly different from the one previously used during the design (1033 m^3/s). The model result for peak flow is 989.5m^3/s and According to the study by (halcrow, 2000), the turbine release fluctuates between 64.78 and 80m^3/s. Recent information provided by the Tana Beles hydropower Station is that the maximum release is 158m^3/s, which appears to be the latest information available for the operation of the hydropower. Adding the maximum value of 158 m^3/s to the simulated result, the total peak flood will be 1147.5m^3/s.

The capacity to pass the estimated peak flood (1147.5m^3/s) safely over the weir has been checked and accordingly, the estimated peak flood can safely flow over the weir.

The hydraulic design evaluation shows the absence of the main components of hydraulic design like concrete floor with its cutoff, upstream and downstream protection works, energy dissipation and river training works plays a great role for the failure of Tana Beles weir.

The structural analysis shows that the uplift pressure was ignored in that the foundation will balance it. But absence of sufficient weight of impervious floor and piles on the design created failure due to uplift by subsurface flow.in addition to this, subsurface flow facilitate the failure by piping or undermining due to the absence of sufficient length of impervious concrete floor.

The head work site is founded on the naturally bedded basaltic rock, which extends upstream and downstream of the weir site. The abutments of the river around the weir, downstream and upstream of the weir axis are basaltic rocks, which has been considered to be quite competent to withstand the forces due to design flood flowing over the weir and it was decided to dispense with the lowering of floor up to EL. 1228 as a design standard and provision of concrete/ RCC floor on upstream and downstream. But taking into account for the foundation, concrete floor has to be provided both on the upstream and downstream of the weir. Failure by subsurface flow (piping or undermining) happen due to absence of sufficient length of impervious concrete floor and piles at upstream and downstream end of the impervious floor.

Even though the design was recommended for a foundation with basaltic rock, Physical observation of the weir site exposes part of downstream and upstream of the weir axis is not hard rocks (because it is eroded as per surveying data comparison) and design consideration (like missing of concrete floor) does not guarantee for the actual site condition. Absence of the concrete floor on the design should be taken as the potential cause for Tana Beles weir failurity.

The foundation condition of the weir site during design was taken as basaltic rock. Accordingly, No concrete floor and cutoff is provided both on the upstream and downstream of the constructed weir. But it is observed on the collected surveying data that along the flow direction, the right side of the channel bed rock foundation is eroded and replaced by alluvial deposit, which cannot balance the uplift pressure below the structure. Thus, the absence of concrete floor and cutoff can be taken as a failure cause of the weir.

As per evaluation of the foundation treatment, it is found that the foundation is not treated as per the required standard which results subsurface erosion to prevail.

Even though there is a direct maximum release of $158m^3/s$ from Tana Beles hydropower Station no scour was expected during design. Additionally, it was believed that the strata at the downstream of the weir site is rocky (vesicular basalt), hence retrogression /scour was not expected. But, it is physically observed that plastering at the surface of the weir is detached due to scour, which claims the provision of energy dissipater necessary.

In the case of Tana Beles weir design, as water flows by high energy down the sloping glacis, this maximum energy is not dissipated by flip bucket due to absence on the design and the energy will scour the downstream floor up to the main weir and this can be taken as a failure causes of Tana Beles weir.

The rock at the weir site is considered quite competent, no scour is expected and hence no additional protection works have been provided on the design of Tana Beles Weir. But, the actual observation and the collected data exposes the rock considered as competent is false and it was must to provide protection work for the actual condition. The design consideration does not guarantee for the actual site condition and Absence of the protection work on the design should be taken as the potential cause for Tana Beles weir failurity.

Malfunctioning of the Under sluice gates causes sediment deposition in front of the under sluice which apparently increase the direct flow of accumulated water into right side of the weir in a way to find low elevation and this will create pressure on the right side only. On that side of the weir, the intensity of the failure is higher when compared with the opposite side of the weir. Therefore, the Malfunctioning of the under sluice gates may be taken as a potential cause for the creation of major failure on the right side than the left side of the weir.

9.2 Recommendation

- Impervious Concrete floor with piles at the upstream and downstream end of the weir shall be included to prevent failure due to piping and uplift.
- Provision of Energy dissipater like flip bucket, protection works and river training work have to be included during maintenance of the weir.
- The under sluice gates should have to be repaired or changed to electromechanical operating system and trash racks should have to be provided for a good performance of the head work.
- Geological investigation of an area should be done with great care to get the actual output which can represent the future long year's geological condition.
- The design should represent the actual site condition of that specific structure
- To get accurate estimation of peak flood in the study area there is a need to use representative meteorological and flow data for the Project site.
- Proper hydrologic, hydraulic and structural analyses are very important which contributes to the good performance and sustainability of the Hydraulic structure.
- For future work, GIS Based Hydrologic Modeling using HEC-Geo HMS have to be done for a better accuracy.
- The peak flood estimated using any of the methods have to be considered for the design of structure in rarest condition.

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APPENDICES

APPENDIX A: Maximum Rainfall of main beles gauging station

24 Hr Duration Max. Rain fall(mm) for main beles basin			
Year	24hr duration Max. annual ppt (mm)	**Ascending**	**Y-logr**
1985	54.74	85.12	**1.93003**
1986	52.08	81.36	**1.91039**
1987	47.05	67.63	**1.83016**
1988	58.95	64.86	**1.81199**
1989	67.63	62.39	**1.79515**
1990	49.79	58.95	**1.77051**
1991	54.22	54.74	**1.73831**
1992	50.16	54.39	**1.73548**
1993	37.88	54.22	**1.73413**
1994	52.93	53.21	**1.72603**
1995	62.39	52.99	**1.72418**
1996	52.99	52.93	**1.72374**
1997	81.36	52.08	**1.71671**
1998	50.29	50.87	**1.70650**
1999	37.10	50.59	**1.70404**
2000	49.75	50.29	**1.70148**
2001	50.59	50.16	**1.70033**
2002	42.17	50.01	**1.69907**
2003	54.39	49.79	**1.69710**
2004	47.02	49.75	**1.69676**
2005	47.18	47.18	**1.67375**
2006	42.96	47.05	**1.67254**
2007	38.74	47.02	**1.67228**
2008	53.21	46.31	**1.66567**
2009	50.87	42.96	**1.63309**
2010	50.01	42.17	**1.62503**
2011	35.02	38.74	**1.58814**
2012	46.31	37.95	**1.57926**
2013	64.86	37.88	**1.57843**
2014	85.12	37.10	**1.56933**
2015	37.95	35.02	**1.54426**
ave	**51.80**	**51.80**	**1.70**
Stan deva	**11.45**	**11.45**	**0.09**
Cs			0.543
K			0.091

Summary of 24hr for main beles basin	
Return period	**Using log-Pearson Distribution**
(years)	**XT=10^YT(mm)**
2	46.11
5	58.29
10	68.09
25	82.57
50	95.00
100	108.95
500	148.65
1000	169.59

Testing for the Goodness of Fit

β	λ	ε	k
13.55	0.024	1.374	0.091

Chi square test

Interval,i	Range	n_i	$f_{s(xi)}$	y=X-ε	$F_{s(xi)}$	$F_{(xi)}$	$P_{(xi)}$	X_c^2
1	<1.6	5	0.161	0.226	0.161	0.110	0.110	0.741
2	1-6-1.7	9	0.290	0.326	0.452	0.514	0.404	0.993
3	1.7-1.8	13	0.419	0.426	0.871	0.855	0.341	0.559
4	1.8-1.9	2	0.065	0.526	0.935	0.973	0.118	0.760
5	>1.9	2	0.065	0.626	1.000	0.997	0.023	2.289
	TOTAL	31	1.000					**5.341**

$X^2_{31,0.95}$	**44.985**	**>5.341**
$X^2_{31,0.99}$	**52.191**	**>5.341**

Therefore, in both case the hypothesis is accepted

Appendix B: Maximum flow of Main Beles

Max. flow(m3/s) for main beles basin			
Year	Max. annual flow (m3/s)	**Ascending**	**Y-logr**
1985	848.32	275.80	**2.44060**
1986	849.32	355.69	**2.55107**
1987	912.36	390.68	**2.59182**
1988	795.13	416.36	**2.61947**
1989	912.36	419.35	**2.62258**
1990	926.78	428.74	**2.63219**
1991	914.66	429.58	**2.63304**
1992	745.37	438.13	**2.64161**
1993	966.60	445.65	**2.64900**
1994	666.99	537.55	**2.73042**
1995	950.05	560.36	**2.74847**
1996	2,146.50	666.99	**2.82412**
1997	873.90	669.20	**2.82556**
1998	929.90	739.84	**2.86914**
1999	739.84	745.37	**2.87237**
2000	1,192.07	795.13	**2.90044**
2001	897.26	813.99	**2.91062**
2002	537.55	848.32	**2.92856**
2003	918.38	849.32	**2.92907**
2004	669.20	873.90	**2.94146**
2005	813.99	897.26	**2.95292**
2006	429.58	912.36	**2.96017**
2007	419.35	912.36	**2.96017**
2008	438.13	914.66	**2.96126**
2009	416.36	918.38	**2.96302**
2010	428.74	926.78	**2.96698**
2011	275.80	929.90	**2.96843**
2012	445.65	950.05	**2.97775**
2013	355.69	966.60	**2.98525**
2014	390.68	1,192.07	**3.07630**
2015	560.36	2,146.50	**3.33173**
ave	**753.77**	**753.77**	**2.84**
Stan deva	**350.58**	**350.58**	**0.19**
Cs			0.013
K			0.002

Summery of 24hr	
Return period	Using log-Pearson Distribution
(years)	XT=10^YT(m3/s)
2	581.14
5	942.21
10	1242.53
25	1691.62
50	2077.33
100	2507.51

Testing for the Goodness of Fit

β	λ	ε	k
24432.15	0.001	-26.564	0.002

Chi square test

Interval,i	Range	n_i	$f_{s(xi)}$	y=X-ε	$F_{s(xi)}$	$F_{(xi)}$	$P_{(xi)}$	X_c^2
1	<2.5	1.00	0.032	29.064	0.032	0.036	0.036	0.012
2	2.5-2.6	2.00	0.065	29.164	0.097	0.103	0.067	0.003
3	2.6-2.7	6.00	0.194	29.264	0.290	0.233	0.129	0.982
4	2.7-2.8	2.00	0.065	29.364	0.355	0.422	0.189	2.543
5	2.8-2.9	4.00	0.129	29.464	0.226	0.631	0.528	9.335
6	2.9-3	14.00	0.452	29.564	0.742	0.806	0.574	0.805
7	3-3.3	1.00	0.032	29.664	0.387	0.918	0.497	13.460
10	>3.3	1.00	0.032	29.964	0.258	0.999	0.368	9.491
	TOTAL	31	1.000					**36.631**

$X^2_{31,0.95}$	**44.985**	>	36.631		
$X^2_{31,0.99}$	**52.191**	>	36.631	Difference	8.354 for log pearson for 95%
					15.560 for log pearson for 99%

Therefore, in both case the hypothesis is accepted

APPENDIX C: Maximum Rainfall of Weir site

24 Hr Duration Max. Rain fall(mm) of Weir site			
Year	24hr duration Max. annual ppt (mm)	**Ascending**	**Y-logr**
1985	89.70	119.33	**2.07677**
1986	70.20	110.84	**2.04469**
1987	63.90	107.07	**2.02965**
1988	65.16	91.02	**1.95913**
1989	79.10	90.00	**1.95426**
1990	110.84	89.70	**1.95281**
1991	90.00	88.73	**1.94805**
1992	79.26	88.36	**1.94624**
1993	78.04	88.26	**1.94575**
1994	72.14	87.89	**1.94395**
1995	82.79	84.77	**1.92826**
1996	76.81	84.66	**1.92766**
1997	88.36	84.13	**1.92497**
1998	76.87	82.79	**1.91796**
1999	91.02	80.36	**1.90502**
2000	107.07	79.55	**1.90065**
2001	119.33	79.26	**1.89906**
2002	84.66	79.10	**1.89818**
2003	79.55	78.04	**1.89229**
2004	84.13	76.87	**1.88578**
2005	87.89	76.84	**1.88558**
2006	76.84	76.81	**1.88541**
2007	84.77	75.10	**1.87567**
2008	88.26	72.14	**1.85820**
2009	66.61	70.20	**1.84631**
2010	64.49	67.37	**1.82849**
2011	75.10	66.61	**1.82356**
2012	88.73	65.16	**1.81396**
2013	80.36	64.49	**1.80950**
2014	67.37	63.90	**1.80553**
2015	59.11	59.11	**1.77165**
ave	**81.56**	**81.56**	**1.91**
Stan deva	**13.58**	**13.58**	**0.07**
Cs			0.386
K			0.064

Summary of 24hr	
Return period	Using log-Pearson Distribution
(years)	XT=10^YT(mm)
2	75.05
5	89.99
10	101.00
25	116.07
50	128.12
100	140.89
500	173.89
1000	189.79

Testing for the Goodness of Fit

β	λ	ε	k
26.78	0.013	1.545	0.064

Chi square test

Interval,i	Range	n_i	$f_{s(xi)}$	y=X-ε	$F_{s(xi)}$	$F_{(xi)}$	$P_{(xi)}$	X_c^2
1	<1.8	1	0.032	0.255	0.032	0.051	0.051	0.216
2	1.8-1.85	6	0.194	0.305	0.226	0.217	0.166	0.145
3	1.85-1.9	8	0.258	0.355	0.484	0.491	0.275	0.031
4	1.9-1.95	10	0.323	0.405	0.806	0.749	0.257	0.510
5	1.95-2	3	0.097	0.455	0.903	0.905	0.156	0.694
6	>2	3	0.097	0.505	1.000	0.971	0.067	0.420
	TOTAL	31	1.000					**2.016**

$X^2_{31,0.95}$	**44.985**	**>2.607**
$X^2_{31,0.99}$	**52.191**	**>2.607**

Therefore, in both case the hypothesis is accepted

APPENDIX D: Hydrologic Analysis

30 years of daily data for the nearby stations of the project site were collected from Ethiopian National Meteorological Agency (NMA). Rainfall records of the selected catchments were completed by correlating their long term flow rate records with other hydrological stations which have similar characteristics. A number of stations in most of the Ethiopian river basin have incomplete records. In order to make use of the partially recorded data, missing values need to be filled in sequence. To fill the missing recorded stream flow gauging data various methods are available. The missing values were filled with multiple station correlation.

Hydrologic processes such as Floods are excitedly complex natural events. They are result of a number of component parameters and are therefore very difficult to model analytically. For example the floods in a catchment depend upon the characteristics of the catchment, rainfall and antecedent conditions, each one of these factors in turn depend upon a lots of constitute parameters. This makes the estimation of flood peak a very complex problem leading to many different approaches.

In the analysis of rainfall frequency the probability of occurrence of a particular extreme rainfall (24 hr. maximum rainfall) is important and it is obtained by the frequency analysis of point rainfall depth.

The prediction of peak flows from rainfall over a catchments involves estimation of daily maximum rainfall for a given return period and conversion of the daily maximum rainfall to runoff hydrograph at the desired location.

Representative Meteorological stations are selected by thiessen polygon in which maximum rainfall data of meteorological stations located on each polygon surrounding the catchment will be weighted and used for the model simulation. Accordingly, Bahir Dar and Mandura are the only representative meteorological stations and their maximum rainfall data is converted into rainfall data of project area by aerial weighted method.

To determine the mean areal rainfall, the rainfall amount of each station is multiplied by the area of its polygon and the sum of these products is divided by the total area of the catchment.

$$P_{aver} = \frac{P1A1+P2A2+P3A3+\cdots PnAn}{A1+A2+A3+\cdots An} \quad \text{……..Eqn (1)}$$

Table: maximum daily rainfall of project area

Year of record	1980	1981	1982	1983	1984	1985	1986	1987	1988
Maximum daily rainfall						89.7	70.2	63.9	65.16
Year of record	1989	1990	1991	1992	1993	1994	1995	1996	1997
Maximum daily rainfall	79.1	110.84	90	79.26	78.04	72.14	82.79	76.81	88.36
Year of record	1998	1999	2000	2001	2002	2003	2004	2005	2006
Maximum daily rainfall	76.87	91.02	107.07	119.33	84.66	79.55	84.13	87.89	76.84
Year of record	2007	2008	2009	2010	2011	2012	2013	2014	2015
Maximum daily rainfall	84.77	88.26	66.61	64.49	75.1	88.73	80.36	67.37	59.11

Out of the commonly used frequency distribution functions for the prediction of extreme maximum values, Log Pearson is used to distribute maximum rainfall into hourly based values for each return period.

In this method the flow data is first transformed into logarithmic form (base ten) and the transformed data is then analyzed. If X is the variety of random flow series then the series of Z varieties where are obtained for this series for any recurrence interval T.

$\mathbf{Z=logXT}$ ……………………………………………………………………….Eqn (1)

$\mathbf{Z_T=\bar{Z}+K_Z\delta_Z}$……………………………………………………………………..Eqn (2)

Where, K_Z= a frequency factor which is a function of T and the coefficient of skewness, Cs

δ_Z= standard deviation of Z variety sample

$\delta_Z=\sqrt{\frac{\Sigma(Z-\bar{Z})^2}{(N-1)}}=0.07$……….……………………………………………….……Eqn (3)

$$\mathbf{Cs} = \frac{N\Sigma(Z-\bar{Z})^3}{(N-1)(N-2)(\delta_Z)^3} = 0.386 \quad \text{……Eqn (4)}$$

Where $\bar{Z}$= mean of the sample.

N= sample size = Number of year of record.

The variation of Kz =f (CS, T) is given the annex in table F

K_Z =3.482

$\bar{Z}$=1.91

$X_T = 10^{Z_T}$=140.89mm

Since the 100 year peak flood shall be computed for comparison and redesign of weir, the 100yrs, 24hr rain fall computed is 46.36mm and changed to incremental rainfall using alternate block method as shown below.

Table: maximum hourly rainfall of study area

Time	Hourly p=M*sqrt(T)	Incremental depth	Time Interval	Precipitation using Alternate Method	cumulative
hr	mm	mm	hr	mm	mm
0.5	6.69	6.69	0---0.5	1.48	0.49
1	9.46	2.77	0.5---1	1.51	0.98
1.5	11.59	2.13	1---1.5	1.54	1.49
2	13.38	1.79	1.5---2	1.58	2.01
2.5	14.96	1.58	2-2.5	1.62	2.54
3	16.39	1.43	2.5---3	1.66	3.09
3.5	17.70	1.31	3---3.5	1.71	3.65
4	18.92	1.22	3.5---4	1.76	4.23
4.5	20.07	1.15	4---4.5	1.81	4.82
5	21.16	1.09	4.5---5	1.87	5.44
5.5	22.19	1.03	5---5.5	1.94	6.08
6	23.18	0.99	5.5---6	2.01	6.74
6.5	24.12	0.95	6---6.5	2.10	7.43
7	25.04	0.91	6.5---7	2.19	8.15
7.5	25.91	0.88	7---7.5	2.30	8.91
8	26.76	0.85	7.5---8	2.43	9.71
8.5	27.59	0.82	8---8.5	2.58	10.56
9	28.39	0.80	8.5---9	2.77	11.47

9.5	29.17	0.78	9---9.5	3.00	12.46
10	29.92	0.76	9.5---10	3.30	13.54
10.5	30.66	0.74	10---10.5	3.71	14.76
11	31.38	0.72	10.5---11	4.34	16.19
11.5	32.09	0.71	11---11.5	5.45	17.98
12	32.78	0.69	11.5---12	8.42	20.76
12.5	33.45	0.68	12---12.5	20.34	27.45
13	34.12	0.66	12.5---13	6.46	29.57
13.5	34.77	0.65	13---13.5	4.80	31.15
14	35.41	0.64	13.5---14	3.99	32.47
14.5	36.03	0.63	14---14.5	3.49	33.61
15	36.65	0.62	14.5---15	3.14	34.65
15.5	37.25	0.61	15---15.5	2.88	35.59
16	37.85	0.60	15.5---16	2.67	36.47
16.5	38.44	0.59	16---16.5	2.50	37.30
17	39.01	0.58	16.5---17	2.36	38.07
17.5	39.58	0.57	17---17.5	2.25	38.81
18	40.15	0.56	17.5---18	2.14	39.52
18.5	40.70	0.55	18---18.5	2.05	40.19
19	41.25	0.55	18.5---19	1.98	40.84
19.5	41.79	0.54	19---19.5	1.90	41.47
20	42.32	0.53	19.5---20	1.84	42.08
20.5	42.84	0.53	20---20.5	1.78	42.66
21	43.36	0.52	20.5---21	1.73	43.23
21.5	43.88	0.51	21-21.5	1.68	43.79
22	44.38	0.51	21.5---22	1.64	44.33
22.5	44.88	0.50	22---22.5	1.60	44.85
23	45.38	0.50	22.5---23	1.56	45.36
23.5	45.87	0.49	23---23.5	1.52	45.87
24	46.36	0.49	23.5---24	1.49	46.36

APPENDIX E: Rating Curve