EVALUATION OF THE HYDRAULIC PERFORMANCE OF THE GEDEBA CULVERT ON THE HALABA SHASHEMENE ROAD, ETHIOPIA

M.Sc. THESIS

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HAWASSA UNIVERSITY HAWASSA, ETHIOPIA

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A THESIS SUBMITTED TO THE
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HAWASSA UNIVERSITY ADVISORS' APPROVAL SHEET

This is to certify that the thesis entitled "**EVALUATION OF THE HYDRAULIC PERFORMANCE OF THE GEDEBA CULVERT ON THE HALABA SHASHEMENE ROAD, ETHIOPIA."** submitted in partial fulfillment of the requirements for the degree of **Master's** with specialization in **Hydraulic Engineering,** the Graduate Program of the **Department of Hydraulic and Water Resource Engineering,** and has been carried out by **TEMESGEN AYANO** ID.No.: 0015/13, under our supervision. Therefore we recommend that the student has fulfilled the requirements and hence hereby can submit the thesis to the department.

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We, the undersigned, members of the Board of Examiners of the final open defense by **Temesgen Ayano** have read and evaluated his thesis entitled "**EVALUATION OF THE HYDRAULIC PERFORMANCE OF THE GEDEBA CULVERT ON THE HALABA SHASHEMENE ROAD, ETHIOPIA"**, and examined the candidate. This is, therefore, to certify that the thesis has been accepted in partial fulfillment of the requirements for the degree of Master of Science.

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DECLARATION

I hereby declare that this research work titled "EVALUATION OF THE HYDRAULIC PERFORMANCE OF THE GEDEBA CULVERT ON THE HALABA SHASHEMENE ROAD, ETHIOPIA**."** is my original work and has not been presented for a degree in any other university, and all sources of material used for this dissertation have been duly acknowledged.

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LIST OF ABBREVIATIONS

AASTHO	American Association of Highway and Transportation Officials
AHW	Allowable Headwater
CERS	Central Ethiopia Regional State
CN	Curve Number
DEM	Digital Elevation Model
DSD	DRAINAGE SERVICES DEPARTMENT
ERA	Ethiopian Road Authority
GIS	Geographic Information System
GPS	Global positioning System
HEC HMS	Hydrologic Engineering Center's Hydrologic modeling System
HEC RAS	Hydrologic Engineering Center's River Analysis System
OEH	Office of Environment and Heritage
MoWIE	Ministry of Water Irrigation and Energy
SNNP	Southern Nations Nationalities and Peoples
RVLB	Rift valley Lakes Basin
SCS	Soil Conservation Service
USACE	United State army corps of Engineers
USDT	United state Department of Transportation

ABSTRACT

*The increasing severity of climate change is causing a global rise in extreme weather events, characterized by intensified rainfall and flooding. These changes create significant challenges for drainage systems. Inadequate drainage infrastructure can lead to severe consequences, including erosion, property damage, and disruptions to transportation and essential services. The main objective of this study was to evaluate the hydraulic performance of Gedeba culvert using hydrological and hydraulic modeling analysis. To carry out the study, primary data was collected from field surveys, and secondary data was gathered from various organizations. The models and materials used in the study were HEC-HMS, HEC-RAS, GIS, and GPS. Continuous hydrologic simulation was initially done using HEC HMS model to calibrate and validate the model. The calibration and validation result indicated that there was strong relationship between simulated and observed stream flow data. Hence, based on these statistical error test criteria HEC-HMS model performance of the model is classified as very good. After model was calibrated and validated using actual observed flow data, frequency storm was generated using the annual maximum precipitation available from rainfall data and it is used as an input for HEC-HMS model to conduct event based simulation for developing flood hydrograph for different return periods. The result of event based simulation shows that, the maximum flood hydrograph for 2, 5, 10, 25, and 50 years were 87.1, 117.8, 135.4, 155, and 179.8*m^3/s *respectively. To analyze the hydraulic performance of the culvert, the HEC-RAS model was used to develop water surface and velocity profiles. The analysis indicated that, for different flood frequency, the water level exceeded the culvert crest, resulting in area, with overtopping and subsequent flooding of the main road connecting Halaba to Shashamene. This overtopping not only posed a risk to the road infrastructure but also led to erosion of the downstream area and collapse of some parts of the road. Furthermore, the analysis revealed that the velocity of the floodwaters passing over the culvert crest was significantly high, possessing erosive characteristics that worsened erosion downstream. The downstream area suffered substantial damage, with some sections of the road collapsing due to the intense flow and erosion. This underperformance of the culvert highlights critical infrastructure vulnerability, necessitating urgent attention and action. To reduce the problems of flooding, Mitigation measures, such as installing erosion control structures (e.g., riprap, check dams) and implementing vegetative buffers, can help stabilize the terrain and reduce the risk of erosion. To alleviate these problems, different type of culverts were selected for analysis to redesign the existing culvert which accommodates the floods. The analysis of various culvert types reveals significant limitations in their capacity to manage floodwaters, with existing designs inadequate for anticipated flood flows, raising risks of flooding and property damage. Modifications like widening river cross sections and using larger culverts are vital for flood management, Box culverts present an exciting option due to their moderate complexity in construction, which requires careful placement and alignment but is generally straightforward. They are designed to support heavy loads and withstand environmental stresses, making them highly durable.*

Keywords: Culvert, Hydraulic performance, Gedeba River, HEC HMS, HEC-RAS

1. INTRODUCTION

1.1. Background

Highway drainage is an important consideration in the design of many projects. The term drainage is defined in several different ways, including the process of removing surplus ground water or surface waters by artificial means, the manner in which the waters of an area are removed, and the area from which waters are drained. A project may alter the existing drainage. When this occurs, drainage features should be provided which protect the highway, adjacent land owners, and the traveling public from water, while maintaining water quality and protecting other environmental resources.(Engineering Bulletin 21-030, 2021)

Main cause of road damage, and problems with the serviceability of road networks, is excess water filling the pores of road materials in the road and in the subgrade soils. It is generally known that road structures operate well in dry conditions and because of this roads historically have been built on dry terrain. On those occasions where roads have had to be built on wet terrain, drainage structures have usually been designed to keep the road structures dry. The first roads in Europe were built about 3500 years ago. Already at that time engineers designed the road structures to take into account the importance of drainage. They paid attention to cross-fall (to help water to flow to the lateral ditches), grade line (the road surface should be above of the groundwater table and the surrounding ground) and lateral ditches (to convey water away from the road structure and prevent water table rise.(Dawson, 2008).

The highway network of any country is one of the major public investments designed to support the national economy. When well developed and maintained, the road network is expected to meet the national objectives for road transport. The road assets in Ethiopia vary extremely from a limited number of 4 lane high speed highways to low volume community roads(Engineering Bulletin 21-030, 2021). In our country, the attempt to alleviate the failures on the drainage structures is very little, even though the problem is so much large. Many times side ditches, culverts and bridges are found to be clogged, collapsed and washed away by the flood. Consequently, the quality of roads is much deteriorated and their life time is shortened. To address these problems investigations are necessary. Special attention shall be given to the failures in bridge structures since any malfunction on these structures creates a wide-ranging problem.(Mengistu A. Jemberie5 etal,2015).

The most cross drainage structures are culverts and bridges. Culverts are closed conduits hydraulic structure that allows water to flow under a road, rail road, trail, or similar obstruction from one side to the other side. Typically embedded so as to be surrounded by soil. Culverts can be constructed of a variety of materials including cast-in-place or precast concrete. Culverts are commonly used both as cross-drains for ditch relief and to pass water under a road at natural drainage and stream crossings. A culvert may be a bridge-like structure designed to allow vehicle or pedestrian traffic to cross over the waterway while allowing adequate passage for the water. Culverts come in many sizes and shapes including round, elliptical, and box-like constructions.

Bridges are vital components of the road network and they form an essential part of the infrastructure of a nation, facilitating its social and economic development by allowing the free movement of people and goods between remote locations. Bridge is normally a man built structure that shall make it possible for traffic to cross one of nature or man built obstacle.

Now a time in our country there are lots of road projects which are under construction and lot of design works are being done for the future expansion of the road network throughout the country. Bridges are one of vital component of those road networks but the country has experienced many cases of bridge failure for many years. Therefore, designing of bridge must be exercised carefully.

As the water can cause a serious impact on both the road access and its strength, an efficient drainage system is the most important part of road construction and maintenance works. Good drainage needs to be taken into consideration at the early design stages in order to secure a long life for the road. With a well-designed drainage system, future rehabilitation and maintenance works can be considerably reduced and thus limit the costs of keeping the road in a good condition. Ensuring good drainage begins when selecting the road alignment. A Centre line that avoids poorly drained areas, large runoffs and unnecessary stream crossings will greatly reduce the drainage problems. Provision of sufficient drainage is an important factor in the location and geometric design of highways. Drainage facilities on any highway or street should adequately provide for the flow of water away from the surface of the pavement to properly designed channels.

Surface drainage encompasses all means by which surface water is removed from the pavement and right of way of the highway or street. A properly designed highway surface drainage system should effectively intercept all surface and watershed runoff and direct this water into adequately designed channels and gutters for eventual discharge into the natural waterways. Water seeping through cracks in the highway riding surface and shoulder areas into underlying layers of the pavement may result in serious damage to the highway pavement. The major source of water for this type of intrusion is surface runoff. An adequately designed surface drainage system will therefore minimize this type of damage. The surface drainage system for rural highways should include sufficient transverse and longitudinal slopes on both the pavement and shoulder to ensure positive runoff and longitudinal channels (ditches), culverts to provide for the discharge of the surface water to the natural waterways. Storm drains and inlets are also provided on the median of divided highways in rural areas.

Drainage facilities are required to protect the road against damage from surface and sub-surface water. Traffic safety is also important as poor drainage can result in dangerous conditions like hydroplaning. Poor drainage can also compromise the structural integrity and life of a pavement. Drainage systems combine various natural and a man-made facility e.g. ditches, pipes, culverts, curbs to convey this water safely.

Halaba – Shashemenne road is located in the Southern Nation and Nationalities and People (SNNP) and Oromia Regional State. Halaba Shashemenne Road is part of road aimed at promoting trade and regional integration between Southern and Oromia region of Ethiopia by improving transport communications between the two regions. The development of the corridor will expand market sizes beyond national boundaries and foster a conducive and enabling environment for the private sector and for attracting foreign direct investments. In addition to enhancing trade and strengthening regional integration, the project will contribute to poverty reduction in the country by increasing access to markets and social services for the surrounding areas, and communities, and by empowering women and other disadvantaged groups through adequate roadside socio-economic infrastructure and services.

1.2.Statement of Problem

As the world continues to grapple with the impacts of climate change, many regions are experiencing increasingly severe weather patterns, including intensified rainfall and flooding. Ethiopia, particularly vulnerable to these changes, faces significant challenges in managing its road infrastructure from such climatic fluctuations. The country's road networks are crucial for economic development and connectivity, yet many are inadequately equipped to handle the demands posed by extreme weather events. In case of Halaba to Shashemenne road, these global challenges manifest in critical ways. This roadway, which serves as a vital link for local communities and economic activities, is severely hampered by the malfunctioning of drainage structures. Various locations along the road exhibit significant drainage inadequacies, primarily due to the inability of these structures to manage peak flood flows during the rainy season.

The root causes of these issues are multifaceted. Inadequate design and sizing of drainage systems fail to accommodate the volume of storm water, leading to frequent overtopping and subsequent damage to the surrounding infrastructure. Additionally, poor construction quality and inappropriate site selection have compromised the effectiveness of many drainage systems along this route. Improper alignment of drainage structures with respect to both the natural channels and the road alignment significantly intensify these problems. This misalignment increases the risk of scour, undermining the stability of the drainage systems and contributing to their failure during heavy rainfall events.

Environmental degradation further complicates the situation. Deforestation on both sides of the Halaba-Shashemenne road, driven by agricultural practices of local farmers and indigenous communities, has accelerated soil erosion. The resulting sediment accumulation in drainage systems leads to clogged culverts, which exacerbate flooding by causing storm water to overflow onto the road. In addition to sediment, debris such as logs and branches often block these drainage structures, further hindering their function and leading to flood overtopping. The consequences of these drainage failures are severe and multifaceted. Frequent flooding disrupts travel, often halting vehicles for extended periods and causing significant inconvenience. The backflow of floodwaters due to insufficient culverts poses a substantial risk to local properties, damaging homes and livelihoods. Furthermore, the heavy mud residues left on the road and in residential areas create hazardous conditions, complicating mobility for residents and emergency response efforts. Compounding these challenges is the lack of regular maintenance and cleaning of the culverts, which is often constrained by high costs, time demands, and labor requirements. This neglect extends a cycle of inefficiency, leaving the road infrastructure increasingly vulnerable to flooding and erosion.

1.3.Objective of the Study

1.3.1. General Objectives

The general objective of the study is to evaluate the performances of the existing road cross drainage structures and to propose mitigation measures the case of Gedeba cross drainage culvert Central region of Ethiopia.

1.3.2. Specific Objectives

The specific objectives of this study are:

- To estimate the design floods at road crossing site for different return period
- To evaluate hydraulic performance of the culvert for different design flood.
- To recommend mitigation option that can improve the culvert hydraulic performance

1.4.Research Questions

- What are the estimated design floods at road crossing sites for various return periods?
- How does the hydraulic performance of existing culverts vary under different design flood scenarios?
- What specific mitigation options can be proposed to enhance the hydraulic performance of culverts?

1.5.Scope of the Study

The research was concentrated on the Halaba to Shashemenne road in Ethiopia, specifically evaluating the drainage structures and culverts located at Gedeba road crossing site. The studies were involving the estimation of design floods for different return periods to understand the potential flood scenarios that the culverts must accommodate. An evaluation of the hydraulic performance of existing culverts was conducted under various design flood conditions. This include, analyzing flow capacities, potential for overtopping, and effectiveness in managing storm water runoff. The study was discovered and proposes various mitigation measures aimed at improving the hydraulic performance of the culvert. The study involved the collection of hydrological and hydraulic data, including rainfall records, flow measurements, and site surveys. The study was focused primarily on the technical and engineering aspects of drainage performance, without investigating deeply into socio-economic impacts or broader environmental factors, although these may be acknowledged in the context of the findings.

1.6.Significance of the Study

By analyzing the hydraulic performance of culverts and estimating design floods, this study will contribute to the development of more resilient road infrastructure capable of withstanding extreme weather events and reducing the risk of flooding. Addressing drainage issues will improve road safety for users by minimizing the incidence of flooding, reducing travel disruptions, and preventing accidents caused by poor road conditions during heavy rainfall. By proposing effective mitigation measures, the study aims to minimize soil erosion and sedimentation caused by inadequate drainage, thereby protecting the surrounding environment and maintaining local ecosystems. The findings will serve as a valuable reference for policymakers, engineers, and planners involved in the design and implementation of future road projects in Ethiopia, offering insights into best practices for drainage management. This research will add to the existing body of knowledge in the field of civil engineering and hydrology, providing a case study for similar regions facing drainage challenges due to climate change and infrastructural inadequacies. By addressing the flooding issues that impact local residents, the study will contribute to community resilience, allowing villagers to maintain access to essential services, protect their properties, and enhance their quality of life.

2. LITERATURE REVIEW

2.1.General

Drainage is simply defined as the natural or artificial removal of surface and subsurface water from a catchment area. The surface drainage in roads is defined as a process of removing runoff water from road surface and directing it towards a drain to disposed away from road in a water course or open area(O'Flaherty, 2002). Cross drainage involves the conveyance of surface water and stream flow across or from the highway right of way. This is accomplished by providing either a culvert or a bridge to convey the flow from one side of the roadway to the other side or past some form of flow obstruction. Road storm In highway projects, culverts are important hydraulic structures that are designed to convey water across a road corridor with minimal disturbance (Norman et al., 2001). For appropriate designing and evaluating of culverts, profound knowledge of hydrology, topography, design standards and culvert hydraulics are needed (Rowley et al., 2006). In broader scale, culvert design can be categorized into two classes: structural analysis and hydrologic-hydraulic analysis. Hydrological analysis consists of determining the referent quantities of drainage water to be used as the basis for hydraulic dimensioning (Gjesovska, 2021) . A hydraulic analysis of the flow in the culvert zone (at the inlet, in and after the culvert), as well as in the zone of the bridge structures is performed in order to evaluate the effect of the facilities upon the free water surface in conditions of stationary uneven flow, i.e., whether they have the capacity to accept these waters and safely remove them without any adverse consequences(Gjesovska, 2021).

2.2.Components of Highway Drainage System

2.2.1. Surface Drainage System

It is an essential consideration that adequate provision is made for road drainage to ensure a road pavement performs satisfactorily (O'Flaherty, 2002). (ANUNOBI, 2023) in his study emphasized that a rainwater drainage system should be designed to collect and convey runoff water generated within a catchment area during and after rainfall events, for safe discharge into a receiving watercourse. (ANUNOBI, 2023) found that the magnitude of peak flows that have to be accommodated depends primarily on the intensity of rainfall, topography, soil type, configuration and land use of the catchment area. Finn et al (Dickenson-Jones et al., 2012) stated that drains are normally located and shaped to minimize the potential traffic hazards and accommodate the anticipated surface water flows. Drainage inlets are often provided to prevent water ponding and limit the spread of water into traffic lanes. Proper design of the surface drainage system is an essential part of economic road design (O'Flaherty, 2002). The surface drainage system collects and diverts runoff water from the road surface and surrounding areas to avoid flooding. Road ditches decrease the possibility of water infiltrating into pavement layers and thus help retain the road's bearing capability. The road surface and cross-fall conduct water to surface drains, which take care of the runoff water (Faísca et al., 2009). The majority of ditches normally have a V-shaped cross section. Roadside ditches and culverts carry flow from the area around the road, especially during peak discharge events. This flow can be directed to streams by either ditches or culverts (Wemple et al., 1996). Suitable drainage dimensioning always contributes to the bearing capacity of the pavement and to road lifetime (Faísca et al., 2009).

2.2.2. Collection of Surface Water

The surface drainage may be divided in to three categories as:

a) Drainage in rural highway

There is the provision of side drains in these areas which are generally open, unlined and trapezoidal cut to suitable cross section and longitudinal slopes. Camber is applied to the pavement to drain the surface water and has to drain across the shoulders which are provided with more cross slope. Usually, drains are provided on one or both sides in embankments while drains are provided on both sides in case of roads with cutting. Open drains are dangerous in the places where space is restricted in cutting and hence covered drains are used with layers of coarse sand gravel.

b) Drains in Urban Street

Development of an urban area, involving covering the ground with artificial surfaces, has a significant effect on these processes. The artificial surfaces increase the amount of surface runoff in relation to infiltration, and therefore increase the total volume of water reaching the river during or soon after the rain.

In urban roads, underground longitudinal drains are provided due to the limitation of land width, the presence of foot path, dividing island and other road facilities. This is provided where there is lesser number of natural water courses and in the presence of impervious surfaces. Water is collected in the catch pits at suitable intervals and lead through underground drainage pipes.

c) Drainage in hill roads

In hill roads, there are complex drainage problems. Water flowing down the hill has to be efficiently intercepted and disposed of downhill side by constructing suitable cross drainage works (Adeogun et al., 2019). Catch water drains at the upper hill side, sloping drains and cross slopes are provided to drain out the water whereas side drains are provided only at the hillside. If hill roads are not properly drained, rockslides and slips may occur blocking the road during monsoon season. The shape of the side drains is made in such a way that vehicles can park at that space during emergency, crossing or parking (Njuguna, 2010).

2.2.3. Cross Drainage Structures

Subsurface drainage was developed mainly in the temperate climatic zones to control water table level, and today, it is used in semi-arid and arid zones as an integral part of the irrigated agriculture system(H. Ritzema & Schultz, 2011). The proper and lasting performance of subsurface drainage systems requires selection of appropriate materials (i.e., pipes and envelopes), adequate installation and their maintenance. Past technologies and practices can be a useful guide towards future designs (H. P. Ritzema, 2009). During the 1950s and thereafter, there were rapid developments both in installation techniques and materials.

These developments have been interdependent in that new materials prompted the development of new installation techniques, and vice versa. For centuries, engineers and inventors tried to develop rapid and low cost techniques for subsurface drainage. The selection of the appropriate drainage materials (i.e., pipes and envelopes) depends mainly on their availability, durability, and cost. (Andreas N. Angelakis et al, 2020)

Subsurface drainage systems drain water that has infiltrated through the pavement and the inner slope, but also groundwater. Subsurface drainage systems usually comprise culverts and have a direct linkage to surface drainage systems (Bruen et al., 2006). The subsurface drainage system rains subsurface water from in our roads or from the subsurface areas surrounding our roads,

.different types of structures are employed in the drainage systems are open channels whether artificial or natural convey the flows of water; surface and sub-surface drainage systems; culverts and bridges convey flows under road cross-section; energy dissipaters, used to control the velocities of flows, especially at culvert outlets (Urgessa, 2016)

Highways cross natural drainage channels or canals, and provision must be made for appropriate cross-drainage works. The alignment of a highway along ridge lines (though it may be a circuitous route with less satisfactory gradients) may eliminate the cross-drainage work, thus achieving considerable savings.

Crossing Type

Determining the type of structure for any crossing is an important consideration and there are a number of factors that need to be addressed in this process. It may be necessary to assess several options of different crossing type and size in order to appropriately meet the design requirements and objectives. There are three main types of cross drainage structures used on roads and each has particular advantages and disadvantages. The three types are bridges, culverts and fords as shown in Figure 2.1(ERA, 2013b).

Figure 2.1: Primary drainage infrastructure types

A) Culverts

Box culverts are required where precast pipes cannot be obtained in a sufficiently large size or where a box culvert configuration would better suit the available space between or adjacent to other structures or utilities. For hydraulic design of box culvert, reference can be made. The selection of the size and number of cells in a culvert depends not only on the hydraulic capacity but also on the requirement for maintenance and silting to facilitate the use of mechanical plant inside box culverts, the internal dimensions of each cell of a box culvert should not be less than 2.5 m × 2.5 m. The minimum width should be further increased if corner splays are used.(DRAINAGE SERVICES DEPARTMENT, 2018).

Pipes should be covered to a depth in accordance with the pipe manufacturer's specifications and depend on the pipe material and class of pipe. As a general rule pipes of 3.6 m diameter or less should be covered by at least 600 mm of compacted fill and pipes with a diameter greater than 3.6 m should have a depth of cover of one-sixth of the pipe diameter.

Head walls and soil stabilization measures should be used to protect the upstream and downstream fill batters surrounding the culvert pipe(s).This should be completed within five days of crossing maintenance operations. Pipe outlets should discharge onto stable surfaces. Scouring at the pipe outlet should not undermine the crossing structure or initiate gully erosion. Armouring and/or dissipaters may be required to stabilize the outlet (The State of NSW and the Office of Environment and Heritage (OEH), 2012).

Allowable Headwater (AHW) is defined as the maximum depth of water which can be tolerated at the inlet of the culvert. It is measured from the elevation at the inlet of the culvert to the water surface elevation necessary to achieve the hydraulic design requirements at the location, (Engineering Bulletin 21-030, 2021)

The minimum velocity in the culvert barrel should result in a tractive force, $\tau = \gamma dS$, greater than critical τ of the transported streambed material at a low-flow rate. The designer should use 3ft/s if the streambed material size is not known.(INDIANA DEPARTMENT OF TRANSPORTATION, 2012)

B) **Fords**

A ford, also known as a floodway, is a place in a watercourse that is shallow enough to be crossed in a wheeled vehicle. The pavement of a ford may consist of gravel, rock, bitumen or concrete, or of a stable natural surface. Fords are the preferred method for creek crossings as they are largely maintenance free once the site has been stabilized and are fish passage friendly

Operational guidance

If the maintenance of a ford results in erosion or deformation of the road surface or the bed and banks of the drainage feature, then.

• the ford crossing should be replaced with a bridge or cause way with pipe culvert(s), or

• the ford surface and approaches should be armoured with a non-erosive material. Repair or replacement of ford crossings should include all sections of the crossing and crossing approaches where erosion or deformation has occurred.(The State of NSW and the Office of Environment and Heritage (OEH), 2012)

C, Bridges

Bridges may be categorized according to their main use, such as highway bridges, railway bridges, pedestrian bridges, etc. They may also be sorted according to the material used in their construction, such as reinforced concrete bridges, steel bridges, stone bridges, timber bridges, etc. They may also be sorted in to rigid, removable and floating bridges.

The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible in elastic deformation sat the strength and extreme event limit states before failure.

The response of structural components or connections beyond the elastic limit can be characterized by either brittle or ductile behavior. Brittle behavior is undesirable because it implies the sudden loss of load-carrying capacity immediately when the elastic limit is exceeded. Ductile behavior is characterized by significant inelastic deformations before any loss of load carrying capacity occurs. Ductile behavior provides warning of structural failure by large in elastic deformations. Under repeated seismic loading, large reversed cycles of inelastic deformation dissipate energy and have a beneficial effect on structural survival. (ERA, 2013a)

Table 2.1: General Selection Factors - Structure Advantages & Disadvantages

Structure	**Advantage**	**Disadvantage**
Bridges	Waterway area generally increases with increased deck height; Provides greatest flood immunity; Large flow capacity; Fewer problems with debris; Deck widening does not affect capacity; Fewer disturbances to riparian environment about waterway.	Higher design, construction and maintenance costs; More structural maintenance required; Spill slopes can be affected by erosion(potential for costly batter protection requirements particularly for higher/ exposed approach embankments);Pier and abutment can be affected by scour; Increased buoyancy, drag and impact risks; Susceptible to stream/channel migration.
Culverts	Simplest structure to design & construct; Generally most cost effectiveoption;Canaccommodate future changes to road geometry; Less structural maintenance; Can spread flows	Generally require higher levels of general maintenance; Most susceptible to failure; Higher siltation/debris risk (blockage);Increased environmental impacts(fauna/fish passage); Potential for scour at outlet; Subject to abrasion; Future extension may reduce capacity; Potential for separation at joints; Potential for failure by piping (leading to failure of embankment).
Fords	Generally simple to design; May offer environmental advantages over culverts and bridges, since they will tend to spread flows more widely; Typically have low embankments; Risk of scour to waterway and surrounding land is reduced.	Allow water flow over road – immunity and safety issues; Increased disruption to traffic due to overtopping; Can have higher construction costs than culvert; Batter slopes can be affected by erosion / scour (particularly for higher embankments); Generally have costly batter protection requirements; Susceptible to stream/channel migration; Can have environmental impacts(fauna/fish passage);Potential for failure of embankment (depending on provided protection).

Table 2.1 (ERA, 2013b)

2.3.Factors Affecting drainage systems and Crossing Structures

2.3.1. Climate and Climate Change

The ultimate reason for having drainage is because of rain! Therefore the drainage needs and solutions will be heavily influenced by the climate where the road is built. Broadly, climate may be divided by temperature and by rain/snowfall. Typically climatologists further differentiate on temperature variation across the year and on rainfall distribution

In recent years the topic of climate change has become a consideration for almost everyone and road builders and operators are no exception. Temperature rise itself is unlikely to make a lot of difference to water in road structures in the warmer temperate areas, but in areas with seasonal freezing it could make a big difference. If the seasonal freeze period is reduced in length or lost or occurs repeatedly with intermittent thaw periods, then much longer periods of wet, non-frozen conditions (as currently experienced for shorter periods in the Autumn and Spring) can be expected, necessitating more stringent drainage requirements and causing much more frequent thaw-weakening problems. However, the shorter frozen period and the shallower penetration of the freezing front into the ground means that drainagemtrenches should more easily continue to operate year round.(Dawson, 2008)

Climate change may lead to an increase in the frequency of extreme precipitation events, floods and snowmelt periods experienced by infrastructure. The '100-year flood' will occur more frequently than before. Already today, many culverts, trenches and other drainage facilities lack the capacity to deal with the current frequency of extreme flows. An increase in the occurrence of extreme weather events will impose greater strain on the facilities for dewatering and drainage of roads. Under measured or non-functional culverts, poorly cleaned ditches and structures with limited capacity may lead to serious damage to the entire road and transport system. ,(Zahra Kalantari,2011)

2.3.2. Sediment characteristics of watershed

Hydraulic engineers and geologists have studied sediment transport in natural streams and rivers for centuries due to its importance in understanding river hydraulics. Erosion and deposition of sediment alters the hydraulic geometry of the channel and may cause an increase of flood frequency as well as navigation problems due to excessive deposition. Sediment transport in sand-bed streams and rivers is a complex process. For its quantification, numerous sediment transport functions have been introduced in the past years based on different concepts. Four basic approaches are used in the derivation of sediment transport formulae the deterministic approach, which obeys the laws of physics and usually is based on an independent variable such as slope, shear stress, stream power or unit stream power; the regression approach, which has emerged from the thought that sediment transport is such a complex phenomenon that it cannot be described by a single dominant variable; the pioneering probabilistic approach of, which highlighted the complexity and the stochastic nature of sediment transport in a rather laborious way for common usage in engineering; and the regime approach, which was developed as a result of long-term measurements in equilibrium conditions(Vasileios Kitsikoudis et al, 2015). The bed-material load should be calculated using the same sediment transport equation and same hydraulic equations that are used in the analysis of the design channel. This is automatically done in USACE SAM or HEC–RAS, if the dimensions and bed-material composition of the upstream supply reach are supplied as input data. Measured data may be used to evaluate the applicability of the Brown lie or Meyer Peter and Müller equations, but measured data should not be used as input to the analytical method.(Natural Resources Conservation Service, 2007)

2.3.3. Ground water level

The seasonal fluctuations of the water table can be a significant source of water moving into pavement sections. Although this flow varies with season, the rate of change in flow is sufficiently small so one can justifiably treat the flow as steady-state. Two possible sources of groundwater which should be considered during design of subsurface drainage systems are gravity drainage, which is water moving laterally towards the pavement section and artesian flow, which is upward flow from confined aquifers While it is feasible in some situations to intercept all of the groundwater flowing towards the pavement structure, in many instances it will not be possible, especially with regard to water originating from an artesian aquifer system. When some, but not all, of the ground water is intercepted, it is necessary to include seepage from this source while designing pavement drainage. The contribution of water flow to the pavement from these two sources of groundwater can be estimated using information about hydraulic conductivity of the underlying soil and the water pressures in the soil alongside the road and in the confined aquifer.

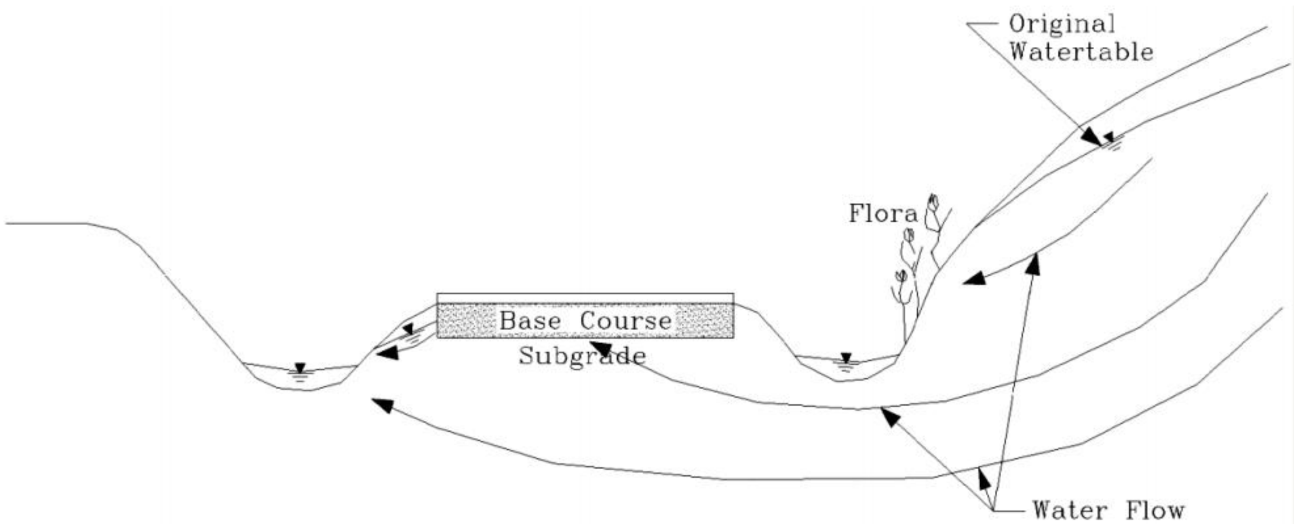

Figure 2.2: Lateral (gravity) flow of groundwater towards the roadway.

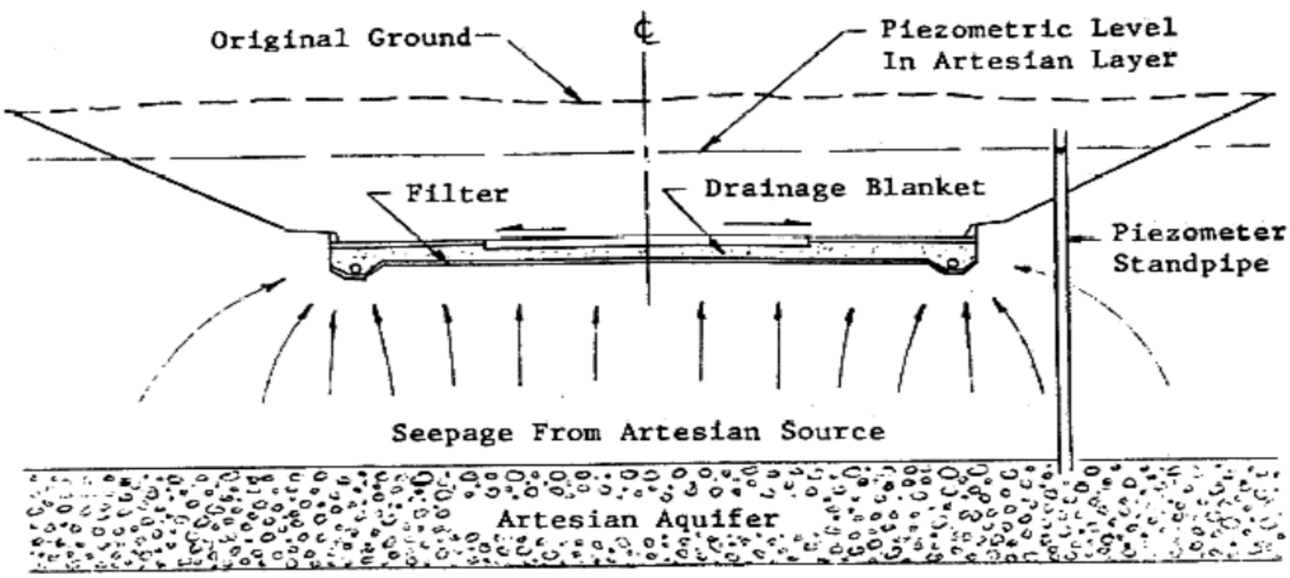

Figure 2.3: Flow of water from a confined aquifer source (Caleb N. Arika, et al, 2009)

2.3.4 Geomorphology and Stream Stability

As a rigid structure in a dynamic environment, all culverts must be designed with channel processes in mind. Effective designs consider the channel and watershed context of the crossing location. Channels are continually evolving, and an understanding of stream adjustment potential must be addressed. Without proper consideration, well-intended plans could detrimentally affect the stream system and related habitat. For example, if a head cut is progressing upstream to the culvert location and it is neither identified nor mitigated, that instability will eventually reach the site. Depending on the type of culvert installation, the head cut will likely result in a drop at the culvert outlet, destabilization of the culvert, orboth.HEC-20 provides guidelines for identifying stream instability problems at highway stream crossings. This manual covers geomorphic and hydraulic factors that affect stream stability and provides a step-by-step analysis procedure for evaluation of stream stability problems. Stream channel classification, stream reconnaissance techniques, and rapid assessment methods for channel stability are summarized. Quantitative techniques for channel stability analysis, including degradation analysis ,are provided and channel restoration concepts are introduced.(U.S. Department of Transportation, 2012).

Everyone interested in the form of stream channels is aware that some stream channels are large and others are small, and that, in general, the large stream channels carry large quantities of water and in the little ones the flows are small. It *may* be said that the stream discharge is the most obvious factor in determining stream form. A stream channel form, however, is the integrated effect of all of the factors influencing it, and since the discharge of any natural stream is constantly changing, as far as the influence of discharge is concerned, the channel form of a stream is the integrated effect of all of these discharges. It is well known that in the forms of stream investigated in this study, the change of the banks and bed goes on more rapidly with large flows than with small ones, and large flows therefore have a greater influence on the form of streams than the small ones. Recently there has been a tendency to analyze stream action on the assumption that this action is largely due to a certain discharge or small range of discharges,called the "dominant discharge" and for some purposes this is a useful concept
There is no doubt that usually a narrow range of discharge exercises the predominating influence. of discharge on stream form and this may be considered to be the dominant discharge. (U.S. Army Corps of Engineers, 1991).

2.4.Hydrologic Analysis of Road Drainage Structures

In the early days, culverts and bridges were sized by empirical methods developed from experiences with existing structures during floods. No particular recurrence intervals were associated with the resulting designs (McEnroe, 2007). But now days Modern hydrologic methods are based on statistical analyses of systematic records of stream flow or rainfall data; the desired recurrence interval is an input to the design. In the hydrologic analysis for a drainage structure, it must be recognized that there are many variable factors that affect floods. Some of the factors which need to be recognized and considered on an individual site-by-site basis are things such as: Rainfall amount and storm distribution Drainage area size, shape, and orientation, Ground cover Type of soil, Slopes of terrain and stream(s), Antecedent moisture condition, Storage potential (overbank, ponds, wetlands, reservoirs, channels, etc.), as the result. In order to optimize drainage structures design many factors are considered vital. At many sites designers have valid reason for providing safety factor in designs. These reasons include uncertainty in discharge estimate, potentially disastrous results in property damage or damage to highway from headwater elevations which exceed the allowable, potential for development upstream of structure, and the chance that the design frequency flood will be exceed during the life of the structure (Zelleke, 2007).

2.4.1. Drainage Problems

Successful drainage depends on early detection of problems before conditions require major action. Signs of drainage problems requiring attention include: puddles on the surface area, poor surface flow, slope erosion, clogged ditches, pavement edge raveling, preliminary cracking, pavement pumping, and surface settlement. These signs indicate the start of failures which occur as soil particles are gradually washed away and as excess water seeps into the roadway reducing the load carrying ability of the subgrade. Major failures caused by poor drainage conditions include washouts, slides, slip outs, road and pavement break up and flood damage. Water is the biggest enemy of roads and most experts believe that most of pavement distresses and damages are due to poor drainage. According to Ireri, eighty percent of existing road way problems can be traced to the presence of water from poor drainage either in or on the road pavement. Excessive water content in the pavement layers such as base, sub base, and subgrade soils can cause early distresses and lead to structural or functional failure of road, unless counter measures are undertaken.(Magdi M. E. Zumrawi, 2016).

2.5.Hydraulic analysis of cross drainage

The hydraulic analysis of a channel determines the depth and velocity at which a given discharge will flow in a channel or known geometry, roughness, and slope. The depth and velocity of flow are necessary for the design or analysis of channel lining and highway drainage structures. Open channel flow is defined as the flow of a free surface fluid within a defined channel. A complete theoretical analysis of culvert hydraulics based on fundamental equations can be difficult (Dickenson-Jones et al., 2012). Flow conditions vary over time for any given culvert. The barrel of the culvert may flow full or partly full depending upon upstream and downstream conditions, barrel characteristics, and inlet geometry. The basic approach presented in HEC-5 was to analyze a culvert for various types of flow control and then design for the control which produces the minimum performance. Designing for minimum performance ignores transient conditions which might result in periods of better performance. The benefits of designing for minimum performance are ease of design and assurance of adequate performance under the least favorable hydraulic conditions.(U.S. Department of Transportation, 2012)

2.6.Site assessments

Field reviews shall be made by the Hydraulics Engineer in order for him/her to become familiar with the site. The most complete survey data cannot adequately depict all site conditions or be substituted for personal inspection by someone experienced in drainage design. Factors that most often need to be confirmed by field inspection are:

- Selection of roughness coefficients;
- Evaluation of apparent flow direction and diversions;
- Flow concentration ;
- Observation of land use and related flood hazards
- Geomorphic relationships;
- High water marks or profiles and related frequencies;
- Existing structure size and type;
- Bank erosion;
- Debris problems;
- Scour and
- Existence of wetlands.

A visit to the site where the project will be constructed shall be made before any detailed hydraulic design is undertaken. This may be combined with a visit by others, such as the highway and structural designers and local road personnel. The hydraulic designer may visit the site separately, however, because of interests that are different from the others and the time required obtaining the required data.(ERA, 2013b).

An important first step in the design of a culvert is a comprehensive understanding of the site and conditions where the culvert will be located. The dynamic nature of watershed and river systems must be acknowledged and accounted for when designing a rigid, constructed structure such as a culvert. This includes watershed issues such as the amount and type of debris loading that may occur, the changes in channel conditions (size, shape and location) that will occur from the natural processes of erosion and sedimentation, and the impact of geology and soils on water quality and factors such as potential corrosion and abrasion of the selected culvert material. Other concerns include how a culvert might impact roadside safety when a vehicle leaves the roadway, or create hazardous conditions for children in urban areas (U.S. Department of Transportation, 2012).

The choice of location of bridges shall be supported by analyses of alternatives with consideration given to economic, engineering, social, and environmental concerns as well as costs of maintenance and inspection associated with the structures and with the relative importance of the above-noted concerns. Attention, commensurate with the risk involved, shall be directed toward providing for favorable bridge locations that:Fittheconditionscreatedbytheobstaclebeingcrossed;Facilitate practical cost effective design, construction, operation, inspection and maintenance; Provide for the desired level of traffic service and safety; and Minimize adverse highway impacts(AASHTO, 2012).

3. MATERIALS AND METHODS

3.1.Description of the area

3.1.1. Locations

The study was conducted on Gedeba River which is one of the sub-basins of Bilate River basin. The river crosses the road section of Halaba to Shashemenne road which is located in Central Ethiopian Regional state (CERS) and its last section is connected to the Oromia Region, Shashemenne town as shown on the figure 3.2 below. The road was constructed by Ethiopian Road Authority and it has 68 km distance between two towns. Gedeba watershed is approximately bounded between the geographic coordinate of 38°7'30"E to 38°11'30"E longitude and 7°19'0"N to 7°23'0"N latitude as shown on the figure 3.1 below and the catchment covers the total area of 35.7km^2 at Gedeba outlet and has altitude ranging between 1755 - 2158 m above sea level.

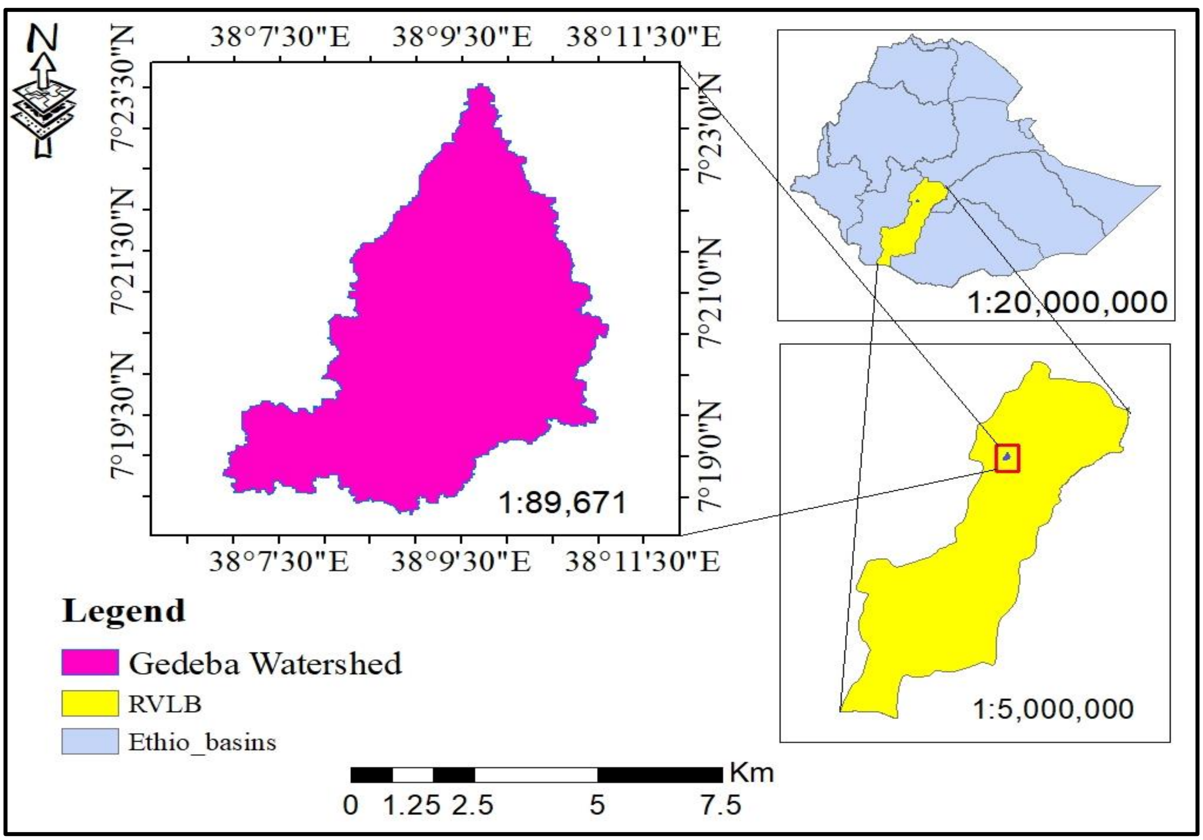

Figure 3.1: location map of Gedeba watershed

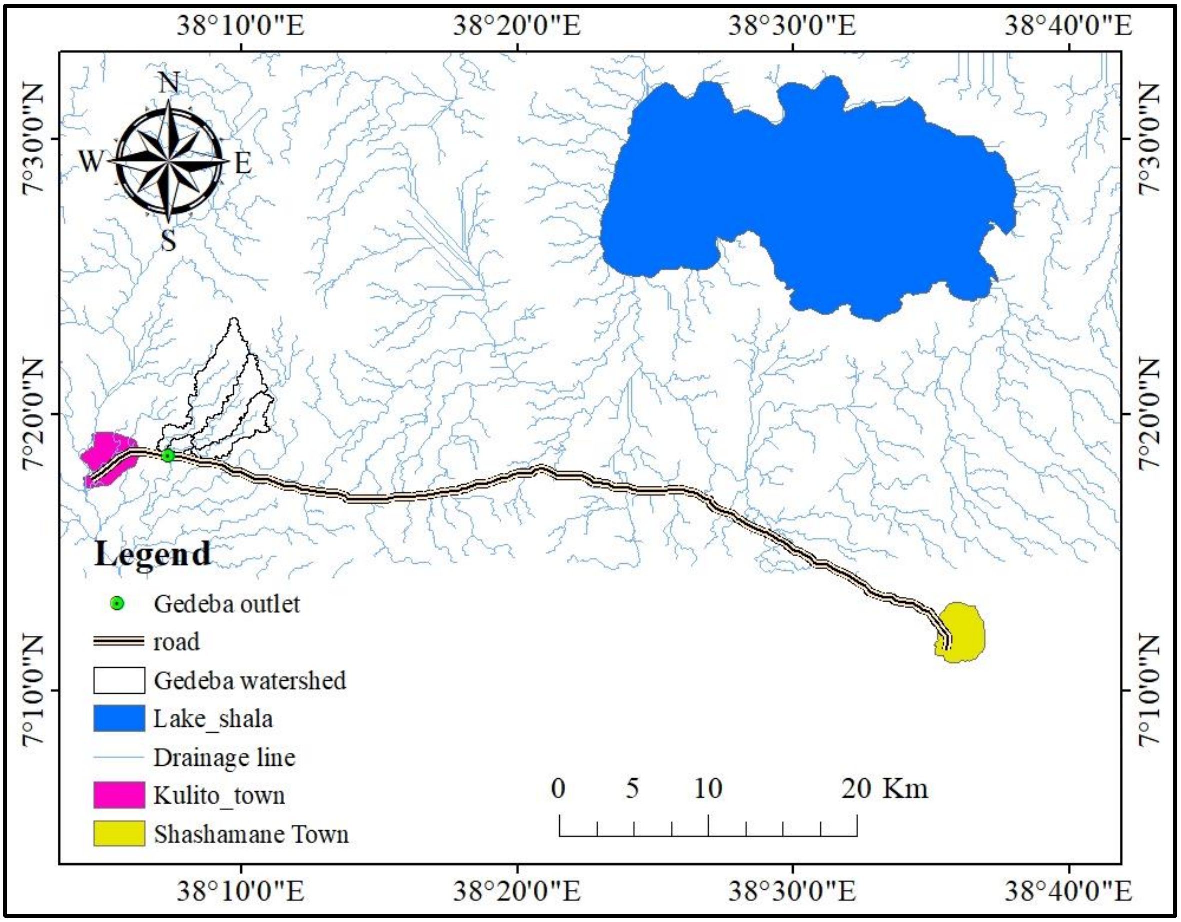

Figure 3.2:Location map of Gedeba outlet on Halaba Kulito to Shashamene road

3.1.2. Topography

Gedeba river drains from northeast part of RVLB to Bilate river. In this study, drainage area was estimated as 35.7 km^2 at Halaba to Shashamene road crossing. The elevation of Gedeba watershed ranges between 1755m a.m.s.l in the southwest and 2158m a.m.s.l in the northeast with a mean elevation of 1956.5m a.m.s.l. Figure 3.3 below shows the topography and drainage basin of the study area.

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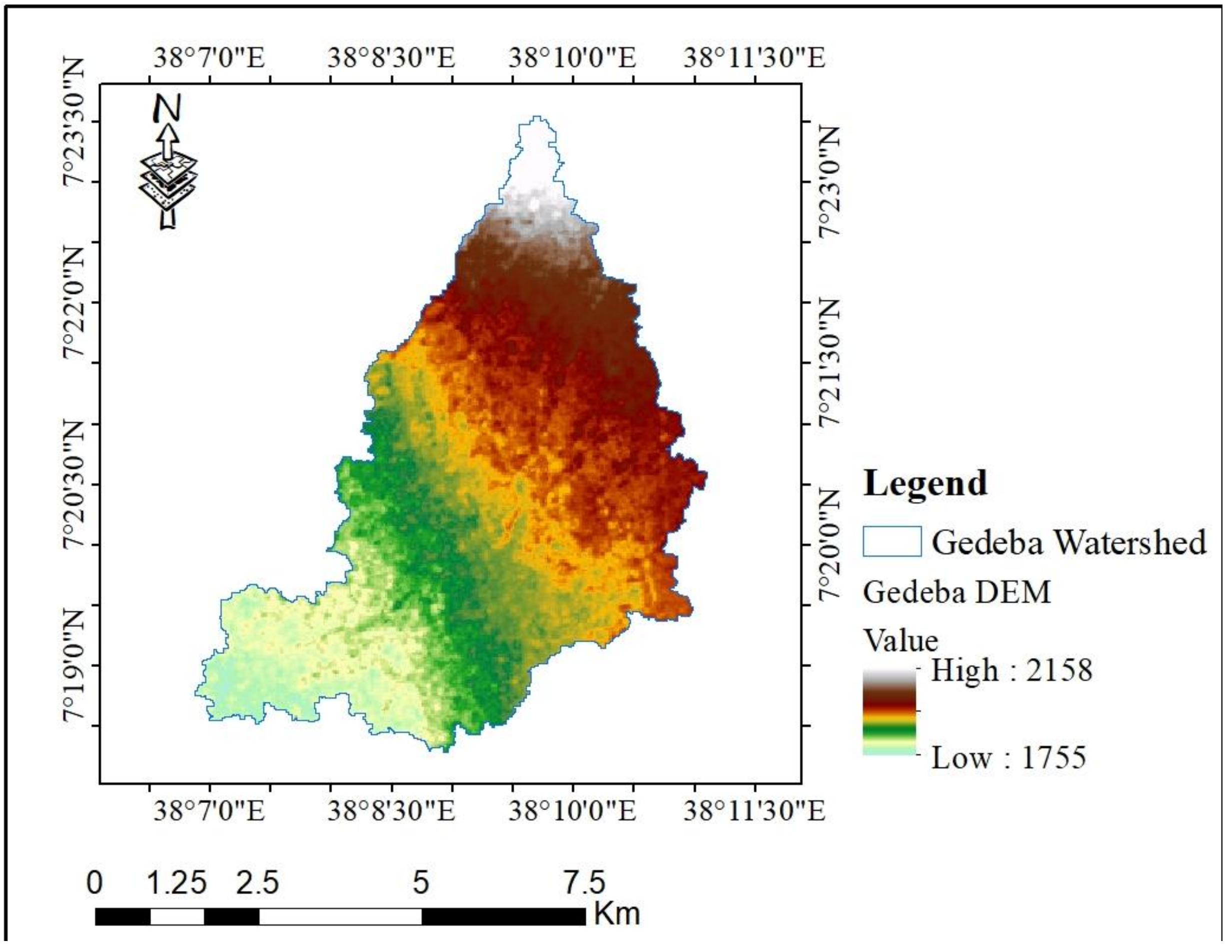


Figure 3.3: Topography of the study area

3.1.3. Soil

The soil types in the study area are classified in to two types known as Luvic Phaeozems, and Vitric Andosols, but, Vitric Andosols are generally dominating the area. Vitric Andosols are formed from volcanic ash and glassy materials. They typically develop in areas with recent volcanic activity, where the deposition of volcanic ash leads to the formation of these soils. These soils are rich in volcanic glass and minerals such as pumice, which contribute to their unique properties. Vitric Andosols are known for their excellent water retention capabilities. Their fine texture and high organic matter content contribute to moisture-holding capacity, making them beneficial for crops, especially in regions with variable rainfall. Figure 3.4 below shows the soil of the Gedeba watershed

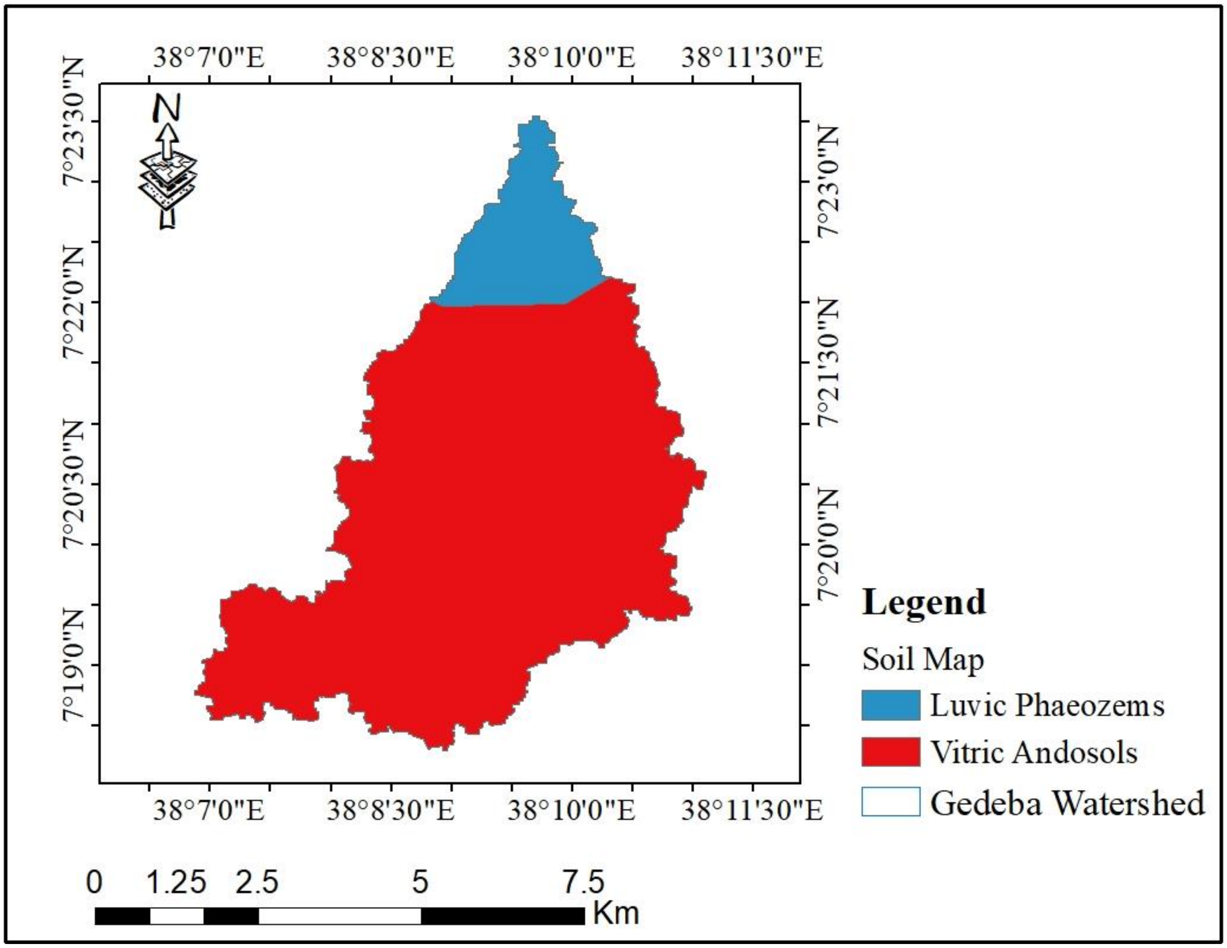


Figure 3.4: Soil of the study area

3.1.4. Land use land cover of the study area

The Gedeba watershed is characterized by a diverse array of land use and land cover types. The catchment area is predominantly utilized for intensive cultivation, featuring both perennial and annual crops. This agricultural focus is driven by the area's fertile soils, particularly the Vitric Andosols, which support high agricultural productivity. Perennial crops, such as oilseeds, spices, pulses, and various cash crops, are a significant component of the agricultural landscape. Alongside perennial crops, annual crops such as maize, teff, and legumes are widely cultivated. The cultivation of these crops is often aligned with seasonal rainfall patterns, taking advantage of the wet season for planting and harvesting. The present land use and land cover of the Gedeba watershed is shown in Figure 3.5 below.

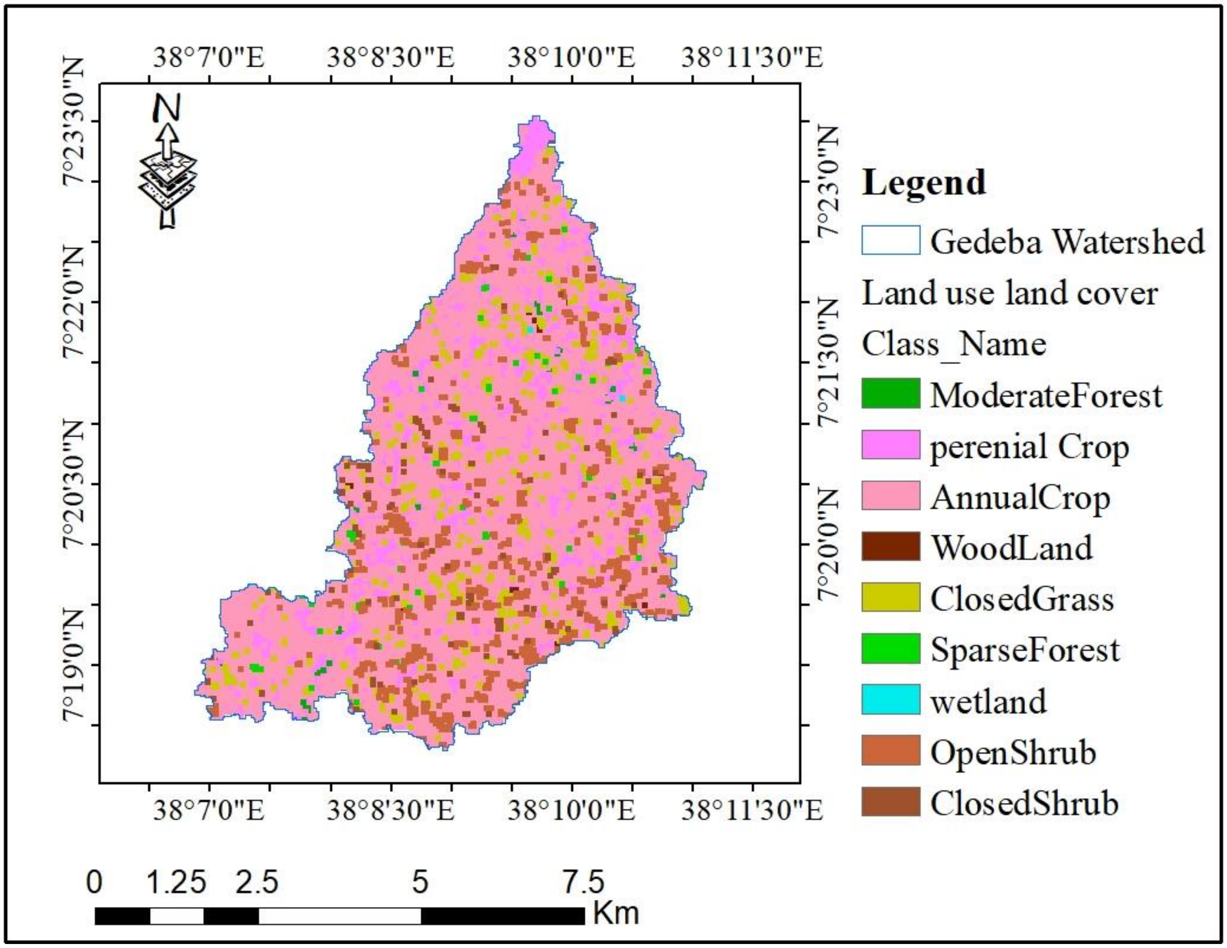


Figure 3.5: Land use of the study area

3.2.Materials

In this study, the materials and models were carefully selected to address the existing challenges and achieve the predetermined objectives effectively. Each tool plays a crucial role in the various stages of research, from data collection to analysis and visualization. Table 3.1 below shows the models and materials used in the study.

Table 3.1: Models and materials used for the study

Models	Purposes
HEC-HMS	✓ Used to simulate precipitation-runoff processes within the catchment. ✓ Enables the determination of peak flood discharges for various return periods

HECRAS	✓ Used to delineate the flood at culvert sites to see the hydraulic analysis
GIS	✓ Used for preparing raw data, mapping results, and visualizing spatial information
GPS	✓ Used to collect precise coordinate and elevation data

3.2.Data used and Sources

To achieve the designed objectives of this study, data were systematically collected and organized from both primary and secondary sources. These section summaries the general categories of data, their purposes, and the sources utilized in this research. Table 3.2 below shows data type and sources of the study area

Table 3.2: Data type and their sources

Data type	**Purposes**	**Source**
DEM (Digital Elevation Model)	To delineate the catchment drainage network	MoWE (Ministry of Water, and Energy)
Meteorological data	To describe the weather condition of the study area	National Meteorological Agency
Hydrological data	To describe hydrology of the study area	Ministry of Water Irrigation and Energy (MoWIE)
Soil Data	For Curve Number generation	Ministry of Water Irrigation and Energy (MoWIE)
Land use data	For Curve Number generation	Ministry of Water Irrigation and Energy (MoWIE)
Elevation data	For river cross sectional and longitudinal profile and other geometric data	Field survey

3.3.Data quality Analysis

Data analysis is a critical process in hydrological studies, involving the evaluation of collected data through analytical and logical reasoning. This process allows researchers to examine each component of the dataset, ensuring that the information is accurate, reliable, and relevant for subsequent modeling and decision making. One of the challenges in assessing the hydrology a given study is the quality of the database. In most cases, datasets are reported with frequent missing values. Such values need to be thoroughly evaluated before using the data in detailed analysis and designs. The quality of the data leads to the accuracy and reliability of the particular model and it's very important to run the model. Hydrological model to a large extent depends on meteorological and hydrological data. Reliability of the collected raw meteorological and hydrological data significantly affects quality of the model input data and, consequently, the model simulation.

3.3.1. Filling missing flow data

Rainfall data play a central role in developing rainfall-runoff models. Before using the recorded rainfall of a station, it is necessary to check first the data for continuity and consistency. The continuity of a record may be broken with missing data due to many reasons such as damage of instrument or fault in a rain gauge, failure of the observer to make the necessary visit to the gage during a period. The missing data can be estimated by using the data of neighboring stations (Subramanya, 2008). In this research, normal ratio method was selected to fill missing data. It is selected because of the normal annual precipitation is exceed 10% of the neighboring station for all stations in the study area. Normal ratio method is estimating daily missing data rely on the data from any adjacent stations. In this research, normal ratio method was selected to fill missing data. It is selected because of the normal annual precipitation is exceed 10% of the neighboring station for all stations in the study area. Normal ratio method is estimating daily missing data rely on the data from any adjacent stations

$$P_X = \frac{N_X}{M}\left(\frac{P_1}{N_1} + \frac{P_2}{N_2} + \cdots \frac{P_M}{N_m}\right)$$

Where P_X is the missing annual precipitation at a station X not included in the above M stations P1, P2 and Pm are the annual precipitation at neighboring M stations, N1, N2… Nm the normal annual precipitations at each of the above (M+1) stations including station X.

3.3.2. Consistency test of rainfall data

A consistent record is one where the characteristics of the record have not changed with time. Adjusting for gage consistency involves the estimation of an effect rather than a missing value. An inconsistent record may result due shifting of a rain gauge station to a new location, change in the ecosystem due to calamities, such as forest fires, landslides, and occurrence of observational error from a certain date (Subramanya, 2008). Double-mass curve was used for this research in order to check inconsistency of the record. This technique is based on the principle that when each recorded data comes from the same parent population, they are consistent.

$$P_{CX} = P_X \frac{M_c}{M_a}$$

Where Pcx is corrected precipitation at any time period t_1 at station X, Px is Original recorded precipitation at time period t_1 at station X, Mc is corrected slope of the double-mass curve, Ma is original slope of the double-mass curve

3.3.3. Statistical test of hydrological data

The quality of hydrological data was evaluated with statistical test. Independence and stationarity test, homogeneity test and outliers test were made in this research to check the quality of flow data. Independence and stationarity test is used to determine whether the mean values and variances of a series vary with time. Wold-Wolfowitz (W-W) test was selected in order to check independence and stationarity test.

For a data set x_1, x_2 … X_n the statistic R is calculated as

$$R = \sum_{i=1}^{N-1} x_i x_{i+1} + x_1 x_N$$

The normal distribution mean and variance calculate by

$\bar{R} = \frac{S^2{}_1 - S_2}{N-1}$ and

$$Var(R) = \frac{S^2{}_2 - S_4}{N-1} - \bar{R}^2 + \frac{S^2{}_1 - 4S^2{}_1 S_2 + 4S_1 S_3 + S^2{}_2 - 2S_4}{(N-1)(N-2)}$$

$$m_r = \frac{1}{N}\sum(x_i)^m$$

The statistic $|u| = \frac{R-R}{\sqrt{Var\ (R)}}$

Homogeneity test is used to check the recorded flow data comes from the same parent population or not (Rao and Hamed, 2000). Mann-Whitney (M-W) test was used for this research to check homogeneity test. The Mann-Whitney (M-W) method was tested by splitting the data set into two subsets of sizes p and q with p is less than or equal to q. The combined data series N= p+q is ranked in increasing order. Then, V and W are calculated from R as

$$V = R - \frac{p(p+1)}{2}\ ;W = pq - V$$

R is the sum of the ranks of the elements of the first sample (size p) in the combined series (sizes N). The M-W statistic U is defined by the smaller of V and W. The mean and variance Var (U)

$$Var(U) = \left[\frac{pq}{N(N-1)}\right]\left[\frac{N^3-N}{12} - \sum T\right]$$
$$\bar{U} = \frac{pq}{2} \quad and$$

Where $\bar{U}$ is mean Var (R)

3.3.4. Goodness of fit test

The D-index serves as a robust and systematic approach for selecting the most appropriate probability distribution for rainfall estimation. By focusing on extreme rainfall values and normalizing the differences the D-index is calculated as;

D-index = $\left(\frac{1}{\bar{R}}\right)\sum_{i=1}^{6}[R_i - R_i{}^*]$

where, $\bar{R}$ is the average value of the series of the recorded rainfall, R_i (for i=1 to 6) are the six highest values in the series of recorded rainfall and $R_i{}^*$ is the estimated rainfall by probability distribution. The distribution having the least D-index is considered as the better suited distribution for rainfall estimation (USWRC 1981).

3.4.Hydrological modeling of Gedeba catchment

The first step in the hydraulic design of a culvert is conducting a hydrological analysis, which involves calculating design floods and estimating discharges for historical flood events. This analysis is crucial for determining the appropriate dimensions and capacity of drainage structures, ensuring they effectively manage storm water runoff. Evaluating the peak rate of runoff is essential for understanding the maximum flow that can occur during a flood event. This information is vital for sizing the culvert to prevent overtopping and flooding. The total volume of runoff generated during a rainfall event must be calculated to ensure that the culvert can accommodate not only peak flows but also the overall water volume. This helps in designing structures that can handle prolonged rain events. Analyzing how flow rates change over time is important for understanding the duration and timing of peak flows. This information aids in determining the necessary design features of the culvert to effectively manage varying flow conditions.

3.4.1. HEC-HMS Model Development

Rainfall runoff modeling was carried out with the help of GIS in HEC-HMS that was for the calibration of hydrological model and used as input data of HEC-RAS model. Graphical view of all the Process of Rainfall Runoff Modeling in HEC-HMS is shown on figure 3.6 below with the help of schematic diagram below.

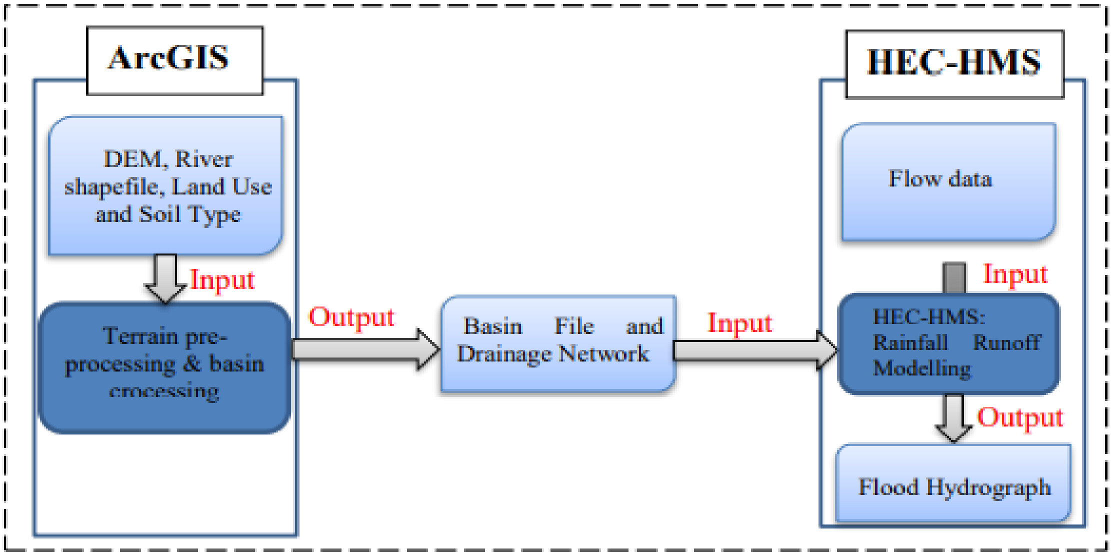


Figure 3.6: Rainfall Runoff Modeling approach

The methodology used for carrying out Rainfall Runoff Modeling can be described by categorizing them into two sections, which are as follows:

i. Creating Basin Model
ii. ii. Developing Hydrological Parameters

Creating Basin Model Basin model was created with the help of GIS in HEC HMS model. Terrain data was then loaded to HEC-HMS model to derive sub-basins and drainage network of the catchment. The steps included were fill sinks, flow direction, flow accumulation, catchment delineation. The resulting project area for Gedeba River was 35.7 Km^2. The delineated sub-basins and rivers were merged based on river junctions. For each of the sub-basins and river, physical characteristics were computed based on the refined DEM. The computed characteristics for river included river length and river slope and for basin included basin slope, longest flow path to the basin, basin centroid, centroid elevation and centroidal longest flow path. The hydrological parameters were estimated by using the land use and soil data for each sub basin. The Selected methods for HEC-HMS processes for modeling loss, transform, and routing were shown on Table 3.3 below.

Table 3.3 : Selected methods for HEC-HMS processes

HEC-HMS Processes	Selected methods

Loss	SCS Curve Number
Transform	SCS Unit Hydrograph
Routing	Muskingum

A basin model for Gedeba in HEC-HMS is given in Figure 3.7 below. The initial values of loss parameters were calculated using CN data. Likewise, all the meteorological components included for modeling of the basin were defined.

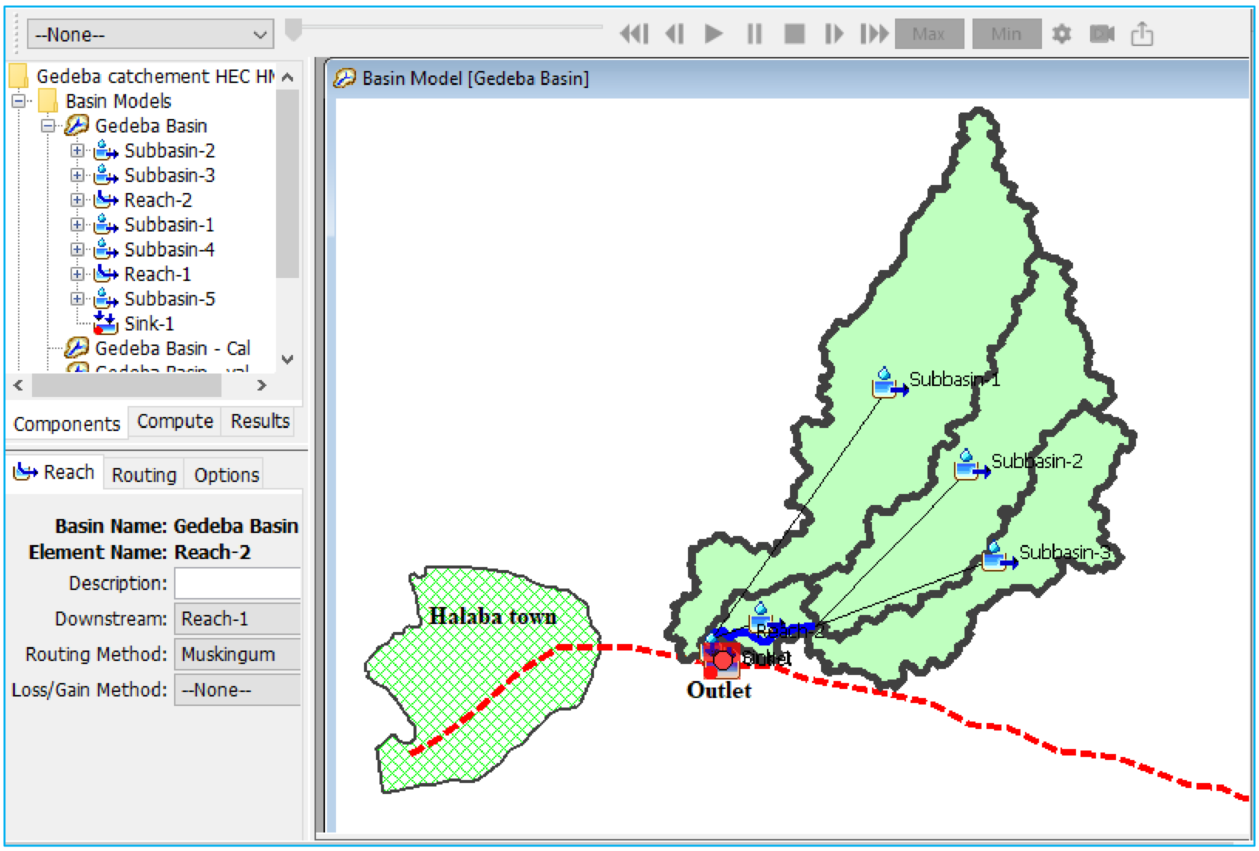


Figure 3.7: HEC-HMS Basin Model Map of Gedeba river basin

3.4.2. Soil Conservation Service (SCS) curve number (CN) method

SCS and CN method was chosen for loss estimation because it was popular, easy to understand and apply, stable and accounts for most of the runoff producing watershed characteristics.

3.4.2.1.Loss Methods

It was developed by the Soil Conservation Service (SCS), in 1972. The SCS Curve Number model is a popular method integrated into the HEC-HMS software for calculating excess precipitation or direct runoff from storm events. The computation of excess precipitation or direct runoff using the SCS-CN model is based on the following equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}$$

Where:

Q = Depth of excess precipitation or direct runoff (in inches), P = Cumulative precipitation (in inches), S = Potential maximum retention after runoff begins (in inches)

In the SCS Curve Number (SCS-CN) model, the potential maximum retention (S) is related to the Curve Number (CN) and can be calculated using the following equation:

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

Where:

S = Potential maximum retention after runoff begins (in inches), CN = Curve Number

In a watershed with different land uses and soil types, a composite Curve Number (CN) can be calculated to estimate the runoff volume. The composite CN represents an average value that considers the contributions of individual watershed subdivisions with homogeneous land use and soil type. The equation to calculate the composite CN is as follows:

$$CN_{composite} = \frac{\sum A_i\ CN_i}{\sum A_i}$$

Where:

CNcomposite = Composite Curve Number used for runoff volume estimation, i = Index of each watershed subdivision with homogeneous land use and soil type, CNi = Curve Number of the watershed subdivision I, Ai = Drainage area of the subdivision i.

3.4.2.2.Transform Method

For HEC HMS, the SCS unit hydrograph model, which is used as a transform model to convert excess precipitation into point runoff, the following relationships and equations are used:

$$U_p = C\frac{A}{T_p}$$

$$T_p = \frac{\Delta t}{2} + t_{lag}$$

$$t_{lag} = \frac{L^{0.8}\,(S+1)^{0.7}}{1900 * Y^{0.5}}$$

Where:

Up = Peak discharge of the unit hydrograph (in cubic feet per second), A = Watershed area (in acres), C = Conversion constant (2.08) - This constant is used to convert the Up to the desired units based on the unit of A (acres) and Tp (hours), Tp = Time to the UH peak (in hours), Δt = Duration of the unit of excess precipitation (in hours), t_{lag} = Basin lag (in hours), S = Maximum retention, L= hydraulic length of the watershed (longest flow path) , Y = Basin slope (%).

3.4.2.3.Route Method

Muskingum model for flood routing was selected for this study due to the availability of required data. The Muskingum method uses a simple finite difference approximation of the continuity equation:

$$\left(\frac{I_{t-1} + I_t}{2}\right) - \left(\frac{O_{t-1} + O_t}{2}\right) = \left(\frac{S_t + S_{t-1}}{\Delta t}\right)$$

Storage in the reach is modeled as the sum of prism storage and wedge storage. The storage is defined by the model as:

$$S_t = KO_t + KX(I_t - O_t) = K[XI_t + (1-X)O_t]$$

Where K = travel time of the flood wave through routing reach, and X = dimensionless weight ranging from 0 to 0.5. The quantity XIt + (1 − X) Ot is a weighted discharge. When the storage in the channel is controlled by downstream conditions, such that storage and outflow are highly correlated, X=0.0. Thus, $S = KO$ and it is the linear reservoir model. If X=0.5, the inflow, and outflow have the same weight which means that the wave does not attenuate when moving through the reach. By substituting two equations above and rearranging to isolate the unknown values at time t, we have:

$$O_t = \left(\frac{\Delta t - 2KX}{2K(1-X) + \Delta t}\right) I_t + \left(\frac{\Delta t + 2KX}{2K(1-X) + \Delta t}\right) I_{t-1} + \left(\frac{2K(1-X) - \Delta t}{2K(1-X) + \Delta t}\right)$$

3.4.3. HEC HMS Model Calibration and Validation

The calibration and validation of hydrological models are critical processes that ensure the accuracy and reliability of simulated outflow volumes, peak flows, and timing of peak flows. A calibration of model was done for total of 15 years of historical data from 1991 to 2005 were used for calibration at the Gedeba basin. Automatic calibration was used for the optimization of observed and simulated flow data using the initial parameter from watershed characteristics. After calibration, the models were validated using the same input parameters as determined by the calibration process but with different simulation time. Gedeba watershed model were validated for 8 years from 2006 to 2013.

3.4.4. Event based simulation in HEC HMS

After continuous simulation is done and the model was calibrated and validated using actual observed flow data, the event based simulation was done using the annual maximum precipitation of all years as an input. Since the available data was the 24 hrs annual maximum rainfall data, the equation Developed by ERA Drainage Manual 2013 was used to estimate the maximum annual rainfall for other selected durations.

$$R_{Rt} = \frac{t}{24}\,\frac{(b+24)^n}{(b+t)^n}$$

Where:

R_{Rt} = Rainfall depth Ratio R_t: R_{24}, R_t = Rainfall depth in a given duration’s’, R_{24}= 24 hrs rainfall depth b and n = coefficients b=0.3 and n= (0.78-1.09).

Thus, using this equation, the annual maximum rainfall depths were calculated and the data was tested for a suitable probability distribution for each duration. The log Pearson type III distribution was best fitted the data and it was used for frequency storm estimation. Log-Pearson Type III distribution is a statistical technique for fitting frequency distribution data to predict the peak flood for a river at a given site. By transforming the data into logarithmic form to the base 10, distribution can be estimated and transformed data were analyzed. This is helpful for flood modeling and designing the structures to protect against the largest expected event. For this reason, it is customary to perform the flood frequency analysis using stream flow data for 2, 5, 10, 25, and 50-year return periods. Figure 3.8 below shows the intensity-duration-frequency curves Gedeba catchment using Log-Pearson Type III distribution.

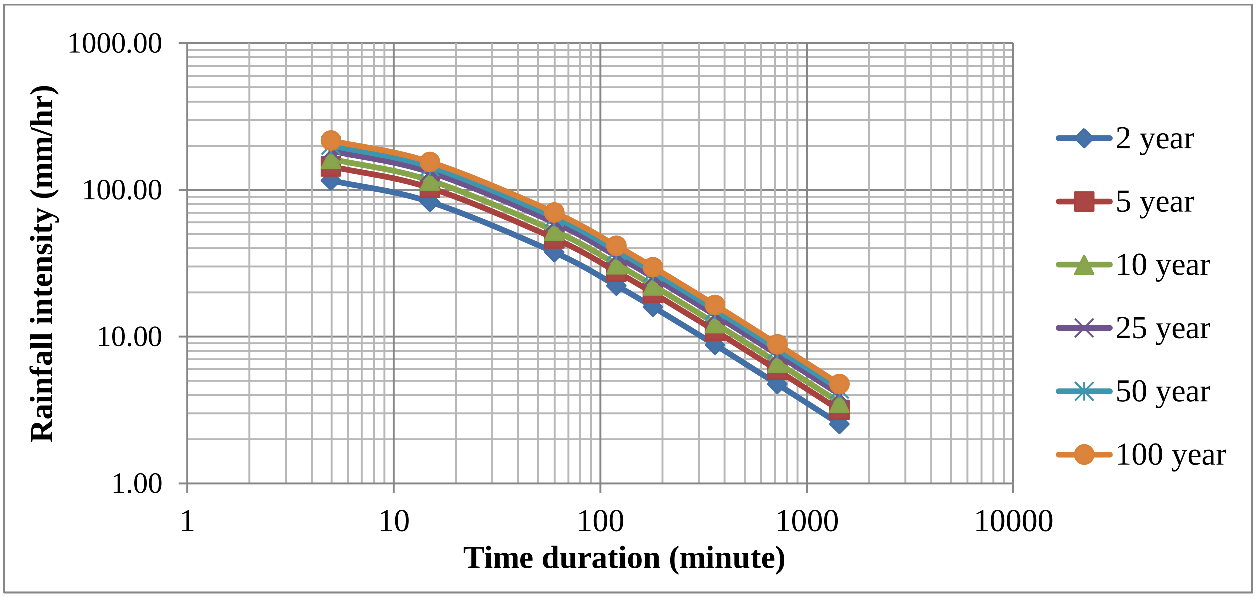


Figure 3.8: Intensity duration frequency curves for Gedeba Catchment

3.5.Hydraulic Model Development

The Hydrological Engineering Center River Analysis System (HEC-RAS) model is a software tool employed for hydraulic simulation of flow through culvert. This model conducts hydraulic calculations for unsteady flow based on the Saint Venant equations governing water flow. The flood hydrograph generated by the HEC-HMS model serves as input data for the HEC-RAS model. The figure 3.9 below shows the Gedeba River cross-sections and culvert profile in HEC-RAS Geometric Editor.

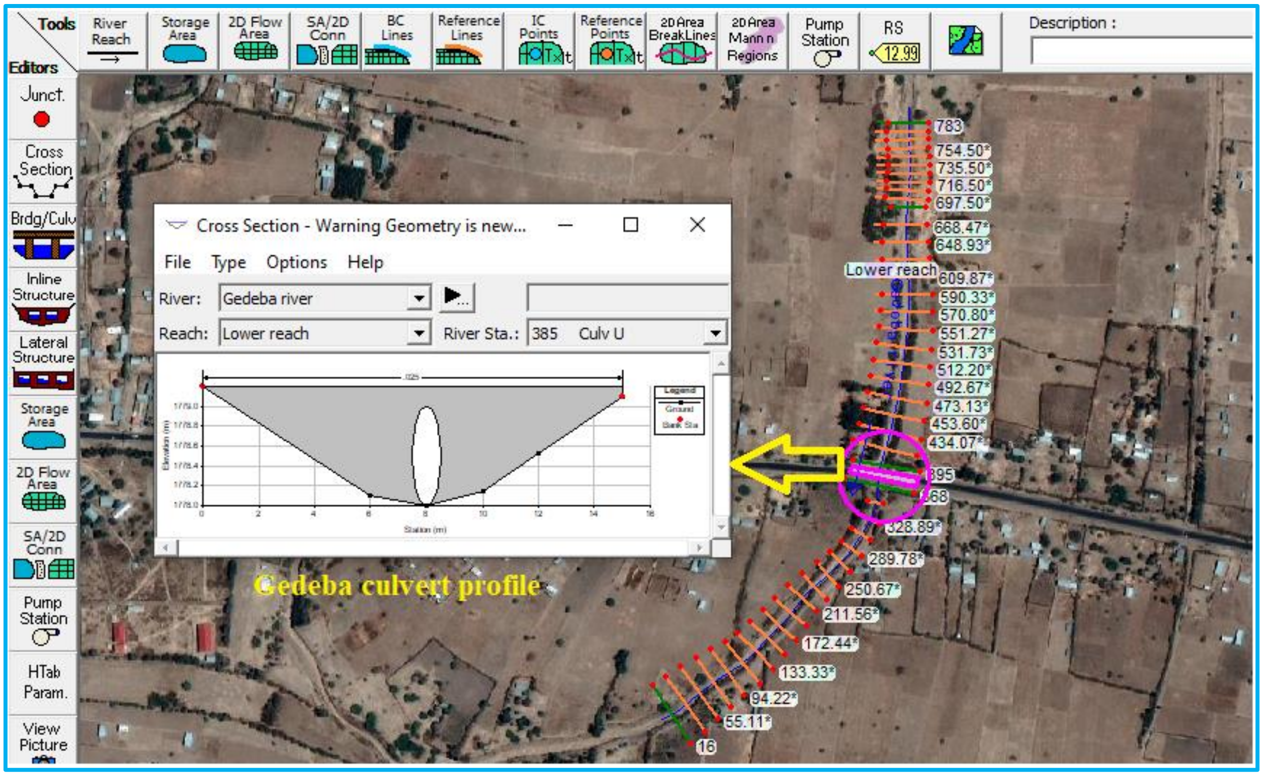


Figure 3.9: Gedeba River cross-sections and culvert profile in HEC-RAS Geometric Editor

3.5.1. Entering and editing flow data

The unsteady flow data are required in order to perform a water surface profile calculation. Each flow that needs to be simulated is called a profile in HEC-RAS. For carrying out the analysis here, the peak flood having 2, 5, 10, 25, and 50 years return period of flows were used.

3.5.2. External Boundary conditions

Boundary conditions both at the upstream and downstream ends of the cross sections are needed for the model in flood routing. There are five types of external boundary conditions that can be linked directly to river cross sections. These are Flow Hydrograph, Stage hydrograph, Normal depth, Rating curve and Precipitation boundary condition. The Normal depth and rating curve boundary condition can only be used at locations where flow will leave the cross section. The flow and stage Hydrograph boundary condition can be used for putting flow into or taking flow out of the cross section. Boundary conditions are necessary to establish the starting water surface at the ends of the river system upstream and downstream. A starting water surface is necessary in order for the program to begin the calculation. In a subcritical flow regime, Boundary conditions are only necessary at the downstream ends of the river system. If a supercritical flow regime is going to be calculated, Boundary conditions are only necessary at the upstream ends of the river system. If the mixed flow regime calculation is going to be made, then Boundary condition must be entering at all ends of the river system. HEC-RAS allows setting the water surface elevation boundary conditions by four methods. The first one is based on known water surface elevation it is based on the observed (known) water surface elevation for each of the profile to be computed. The second one using critical depth the user not required to enter any further data just the program calculate critical depth for the profile and use that as the boundary condition. The third one is using normal depth is required to enter an energy slope that will be used in calculating normal depth at that location. The energy slope can be approximated by average slope of the channel or by average slope of water surface in the vicinity of the cross-section. The last one by using rating curve it needs elevation determined from an existing stage-discharge relation curve. So that for this thesis normal depth had been selected. The average normal depth should be calculated between upstream and downstream end profile of the study area is 0.01. The upstream boundary condition can be defined by a series of cross-sections cut through or tributaries that inters into the study area. The inflow hydrographs for the upstream boundary will be considered for the flood simulations. For this study inflow hydrograph derived from HEC HMS were considered as an upstream boundary as shown on the figure 3.10 below.

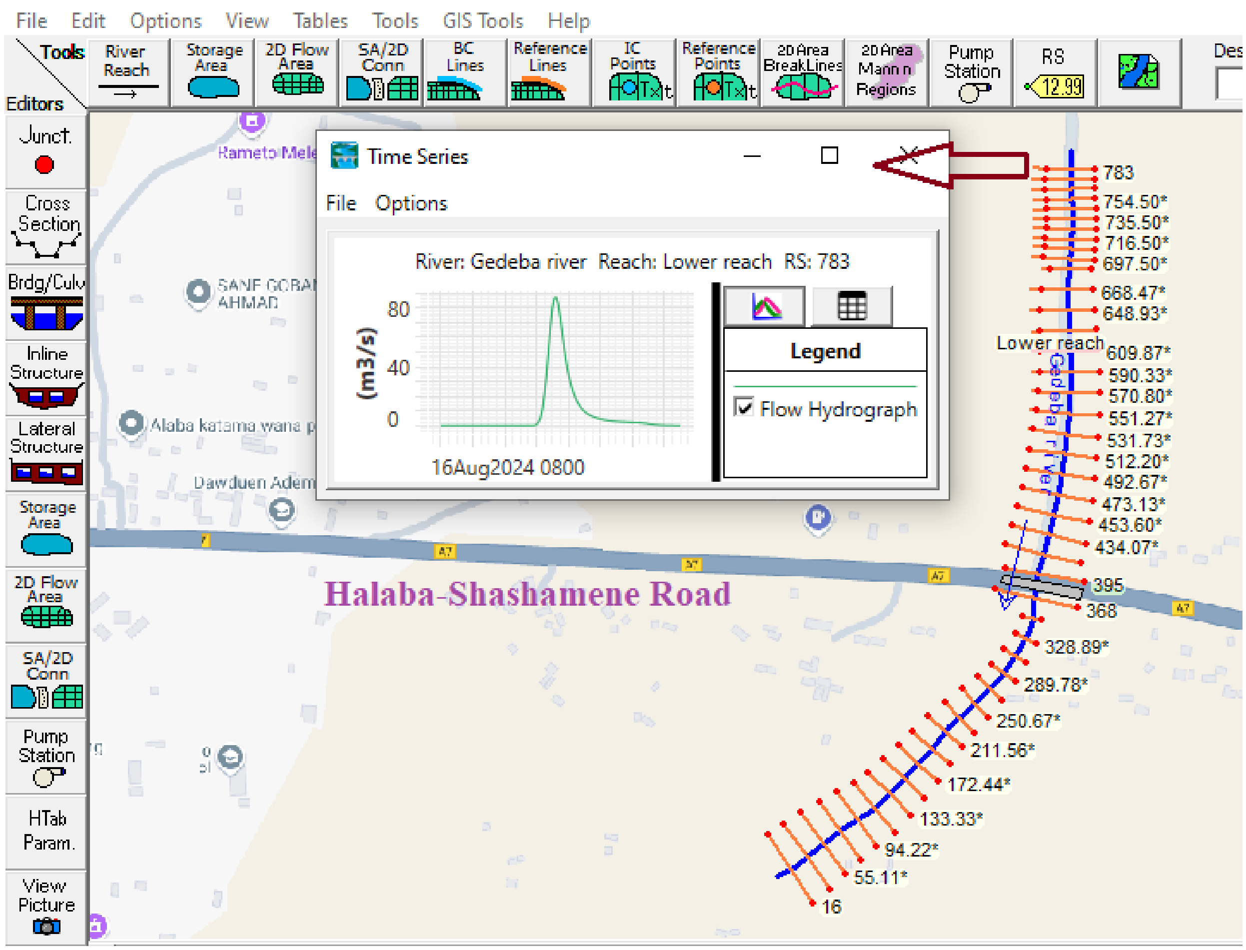


Figure 3.10: Flow hydrograph as upstream boundary condition in HEC-RAS

3.5.3. Unsteady simulation

The unsteady flow computation program in HEC RAS uses the same hydraulic calculation that HEC developed for steady flow; however, the solution of the unsteady flow equation (continuity and momentum equation) are solved.

3.5.3.1.Post Processor

The post processor is used to compute detailed hydraulic information for a set of user specified time lines during the unsteady flow simulation period. The Post processor compute detail output for a maximum stage water surface profile. The computation setting area of the unsteady flow analysis window contain the computation interval, hydrograph output interval, mapping output interval and detailed output interval. The computation interval is used in the unsteady flow calculation. This is one of the most important parameters entered into the model and selected with care due to it affects the simulation. Firstly, the interval should be small enough too accurately to describe the rise and fall of the hydrograph being routed. In this study it is taken with 20 seconds. In this study 30 minutes is used as hydrograph output interval. The detailed output interval field allows the user to write out profiles of water surface elevation and flow at a user specified interval during the simulation. One hour is used for detailed output interval. Mapping output interval this field used to enter interval at which user will be able to visualize mapping with in HEC RAS.

3.6.Culvert hydraulic performance indicators

In evaluating the hydraulic performance of the Gedeba culvert on the Halaba Shashemene Road, several key indicators were assessed. These indicators provide a comprehensive understanding of the culvert's efficiency in managing water flow and mitigating flood risks. The following sections outline each indicator in detail:

- **Flow Capacity**

The flow capacity of the culvert was determined by measuring the maximum flow rate (Q) that the culvert can handle without overtopping. This involves calculating the culvert's design capacity using hydraulic equations such as the Manning's equation for open channel flow in HECRAS.

- **Head Loss**

Head loss (hl) is the energy lost due to friction and turbulence as water flows through the culvert. This will be measured by evaluating the difference in water surface elevation between the inlet and outlet of the culvert. The Darcy-Weisbach or Hazen-Williams equations can be used to quantify head loss, taking into account factors such as the culvert length, diameter, and material roughness.

- **Velocity**

The flow velocity (V) through the culvert was taken based on the flow rate and cross-sectional area of the culvert from hydraulic analysis. Understanding the velocity is crucial as it influences sediment transport and potential erosion at the culvert's inlet and outlet.

- **Water Surface Elevation**

Water surface elevation was monitored both upstream and downstream of the culvert to evaluate how the culvert affects water levels. The hydraulic analysis of the culvert was used to see the water surface profile at culvert site, and upstream. Analyzing WSE will help to assess potential flooding risks in surrounding areas.

3.7.Mitigation option that can improve the culvert hydraulic performance

To identify and implement redesign options for the Gedeba culvert that improves hydraulic performance and ensure effective management of incoming floodwaters, Existing literature on culvert design principles, hydraulic performance indicators, and case studies of successful redesigns was reviewed. A thorough assessment of the existing Gedeba culvert was conducted, including dimensions, materials, and structural condition. Different culvert types (e.g., box, arch, semicircle, multiple boxes, and ellipse culverts) and bridges were evaluated based on site conditions and hydraulic requirements. Various sizes and shapes of culverts were assessed to determine optimal dimensions that could accommodate anticipated flood flows. Relevant parameters, including flow rates, culvert dimensions, roughness coefficients, were input into the model. Various flood scenarios, including 2-, 10-, 25-, and 50-year flood events, were modeled to evaluate the performance of both existing and redesigned culverts. The flow capacity of each redesign option was determined and compared to the expected flood flow. Hydraulic performance indicators (e.g., velocity, water surface elevation, culvert flow passing capacity) of the existing design were compared against the proposed options.

4. RESULTS AND DISCUSSIONS

4.1. HEC-HMS Model Calibration and Validation

The HEC-HMS model is calibrated and validated for the daily observed flow data of 23 years (1991 - 2013) period and the best fit parameters were obtained. Daily rainfall and flow data for the period Jan 1, 1991 to Dec 31, 2005 was used for calibration and from Jan 1, 2006 to Dec 31, 2013 was used for validation of model. The rainfall and runoff modeling for catchment was conducted using SCS curve number as loss parameter, SCS unit hydrograph as transformation parameter, and Muskingum routing as routing parameter.

4.1.1. Model Calibration

The results of the hydrological simulation demonstrate a strong agreement between the simulated and observed hydrographs, as shown in Figure 4.1. This figure illustrates the time series comparison of discharges recorded at the outlet of the entire watershed during the calibration period from 1991 to 2005. The alignment of the simulated peak flows with the observed flows suggests that the model accurately captures the dynamics and behavior of the watershed under varying hydrological conditions. To provide a more comprehensive evaluation of the model's performance, several key statistical parameters were calculated.

The value of R^2 was of 0.79 as shown on figure 4.2 which confirms that a substantial proportion of the variability in observed discharges can be explained by the model. R^2 values range from 0 to 1, with higher values indicating a better fit. An R^2 of 0.79 suggests that the model captures the relationships effectively and can be considered a reliable tool for predicting runoff in the watershed. The value of NSE was 0.78 which indicates a very good model fit. This efficiency coefficient ranges from $-\infty$ to 1, where a value of 1 represents a perfect match between observed and simulated data. An NSE of 0.78 suggests that the model explains a significant portion of the variance in the observed data, thus validating its reliability in predicting hydrological responses. The calculated PBIAS value of 0.46 reflects a slight underestimation of the observed flows. PBIAS is a measure of the average tendency of the simulated data to be larger or smaller than the observed data, with values closer to zero indicating better model performance. A value of PBIAS was 0.46 which indicates that, while there is some bias, it is relatively minor and does not significantly detract from the overall accuracy of the model. The RMSE value was 0.74 and it indicates a relatively low level of error in the simulated hydrographs compared to the observed data. RMSE is a critical measure of the differences between values predicted by a model and the actual observed values, with lower values indicating better model performance.

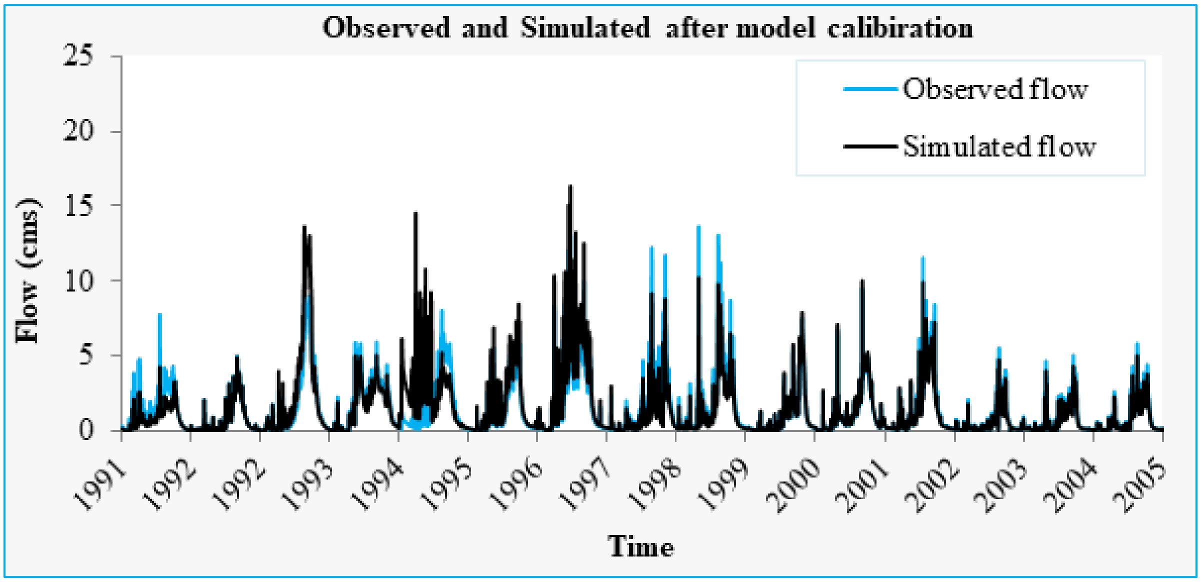


Figure 4.1: Observed and simulated flow of Gedeba River for calibration period

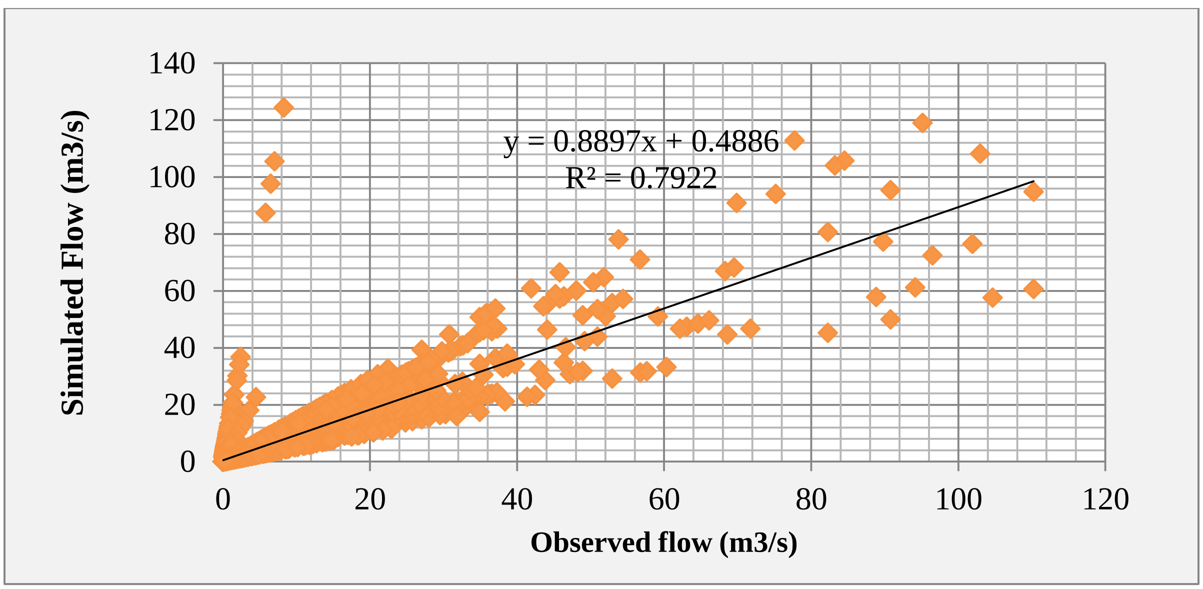


Figure 4.2: Simulated versus observed scatter diagrams during calibration

4.1.2. HEC-HMS Model Validation

Figure 4.3 presents a comparison between the simulated and observed stream flow hydrographs for the validation period. This graphical representation highlights a strong correlation between the two datasets, showcasing a close match and consistent patterns in the flow dynamics. This alignment indicates that the HEC-HMS model effectively predicts stream flow within the study area, demonstrating its reliability and robustness in simulating hydrological responses under varying conditions. The validation process used data from the years 2006 to 2013. To further evaluate the performance of the model during the validation period, several statistical parameters were computed:

The coefficient of determination (R^2) is determined to be 0.84 as shown on figure 4.4 below. This parameter indicates the proportion of variance in the observed data that is predictable from the simulated data. An R^2 value of 0.84 signifies a strong correlation, suggesting that the model effectively captures the underlying trends and variability of the stream flow data during the validation period. The NSE value for the validation period is calculated to be 0.83. This parameter is critical for assessing model performance, as it quantifies how well the predicted values match the observed values. An NSE value of 0.83 indicates a very good fit, suggesting that the model explains a significant portion of the variation in observed stream flow. Values closer to 1 signify a better model performance, and an NSE of 0.83 reflects the model's accuracy in capturing the hydrological dynamics of the watershed. The PBIAS value is calculated to be 0.69. This statistic measures the average tendency of the simulated data to differ from the observed data, providing insight into whether the model tends to overestimate or underestimate stream flow. A PBIAS value of 0.69 indicates a slight underestimation of observed flows, though the value is relatively low, suggesting that the model's predictions are generally close to the actual measured values. The RMSE is calculated as 0.65, providing a quantitative measure of the average magnitude of the prediction errors between the simulated and observed stream flows. A lower RMSE value indicates a better fit, and an RMSE of 0.65 suggests that the discrepancies between the model predictions and the observed data are minimal, reinforcing the model's credibility.

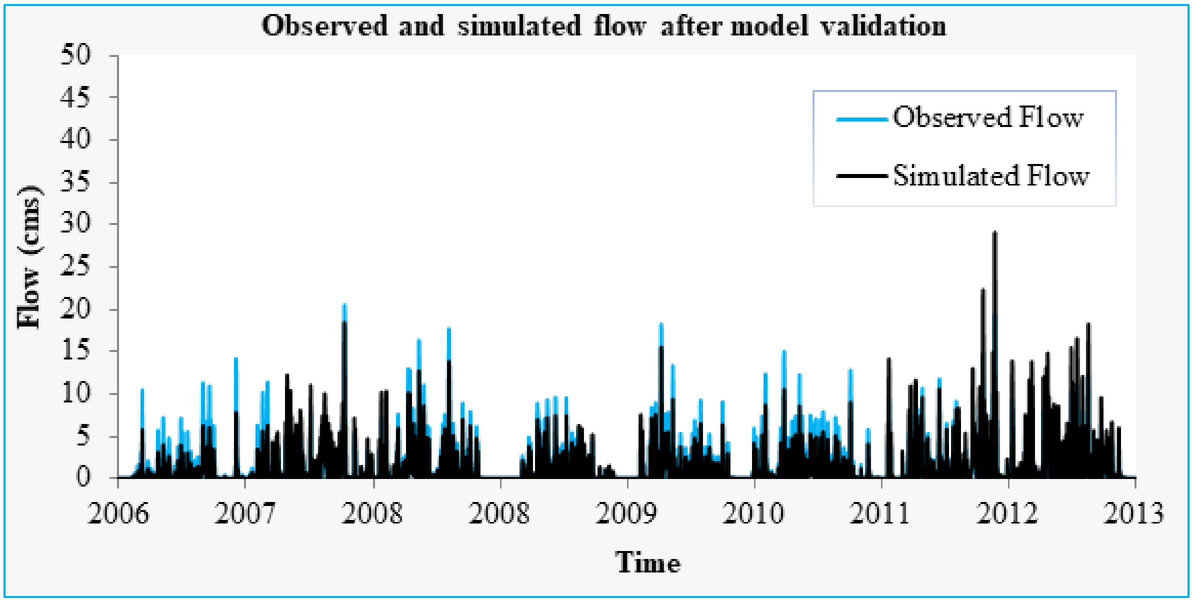


Figure 4.3: Observed and simulated flow of Gedeba River for validation period

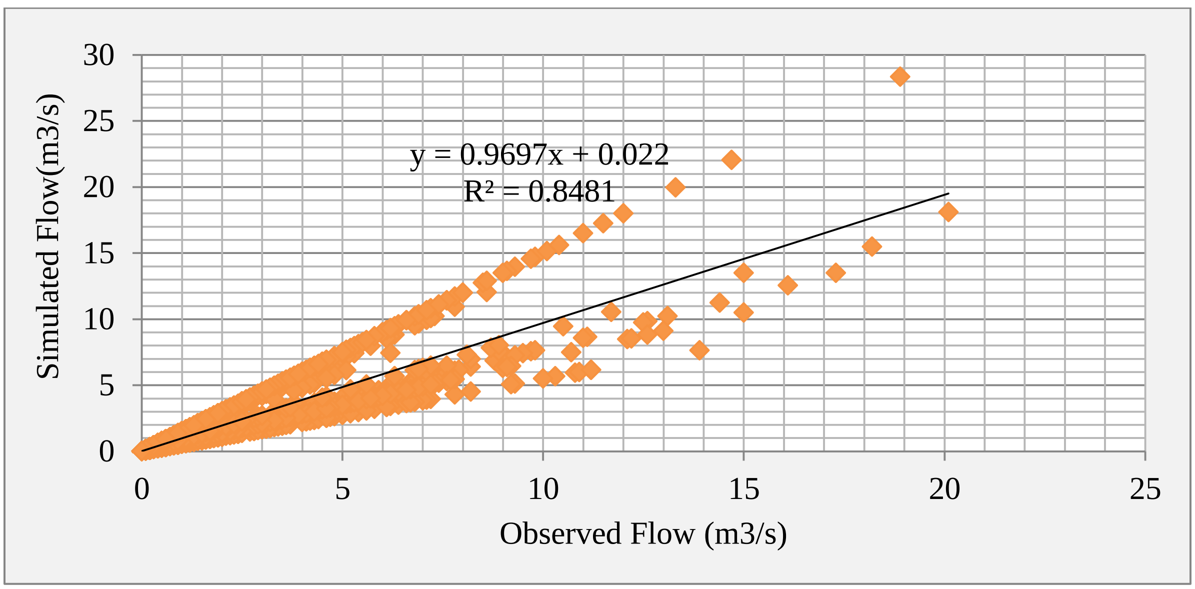


Figure 4.4: Simulated versus observed scatter diagrams during validation

4.2.HEC-HMS Event based simulation

Using the below frequency storm estimated for different return periods as an input, the maximum flood hydrograph for different selected return periods were obtained using HEC-HMS model. The result indicated that the minimum peak flow for the Gedeba River is occurred for 2 year return period for 24 hour storm duration and the maximum obtained with 50 years frequency storm for the same duration. The value being 87.1 m^3 /s and 179.8 m^3 /s for 2 year and 50 years frequency storm respectively. Figure 4.5 below shows the flood hydrographs for different return periods estimated using HEC HMS model.

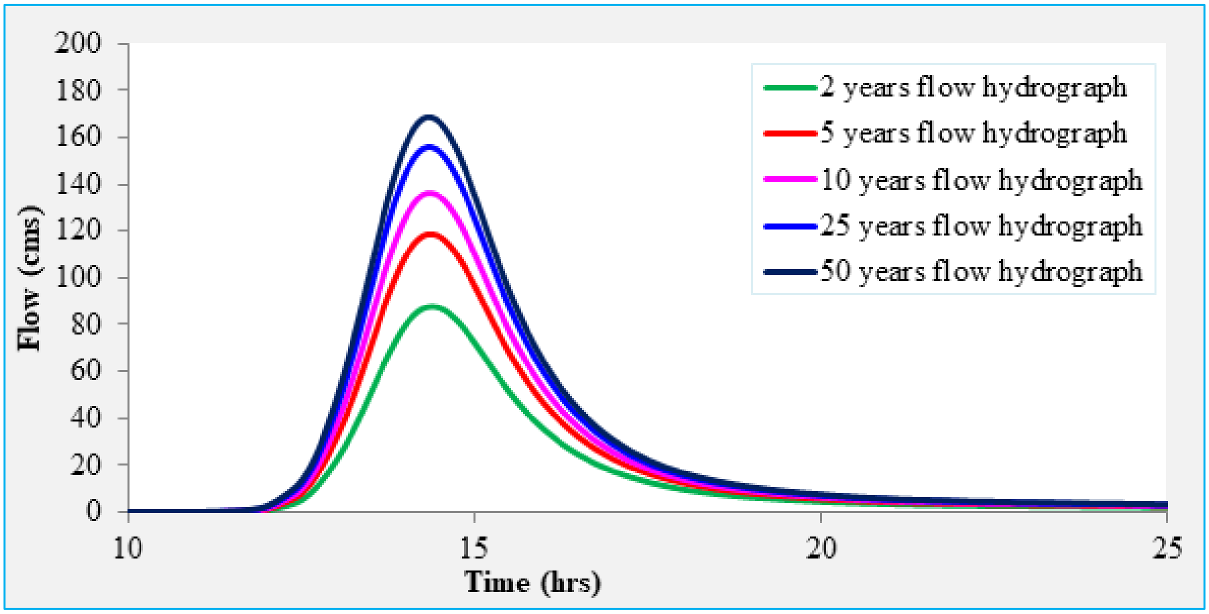


Figure 4.5: Flow hydrographs for different return periods

4.3.Unsteady Flow analysis in HEC-RAS.

The estimated design discharge from flood frequency analysis is used to determine the water surface profile. In this section we can analysis that whether the culvert opening size is adequate to pass the flood coming in any return period.

4.4.Performance analysis

4.4.1. Water surface profile

The model was run for one dimensional unsteady flow water surface profile computations for Gedeba River. The water surface profile shows how the elevation of the water surface changes along the channel. It typically includes sections with varying slopes, and it also reflects different flow conditions, including normal flow, flood events, and low-flow situations. Each condition affects the water surface elevation and flow velocity. Important features, such as culverts, bridges, and natural obstacles, are often marked on the profile. These points can significantly influence water flow and surface elevation. Figure 4.6 below shows, the water surface profile of Gedeba River at culvert cross-sections.

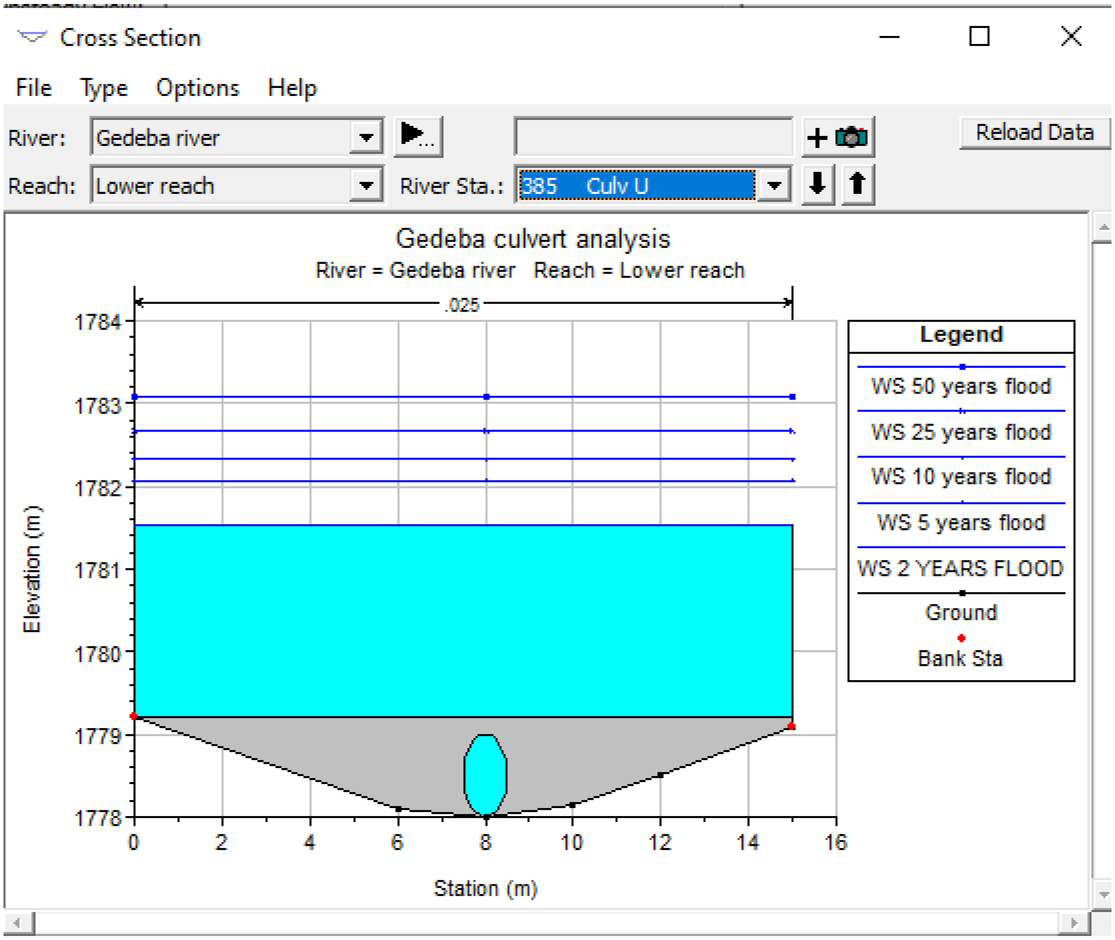


Figure 4.6: Water surface profile at culvert site

The result of cross-sectional plots generated shows that the river overtops the culvert for all flood profile which can result in the inundation of the main road and adjacent areas. The integrative water surface profile in two and three dimensional views intensifies that there is an irregularity in the flow behavior of the river as can be clearly seen in figures 4.7 and 4.8

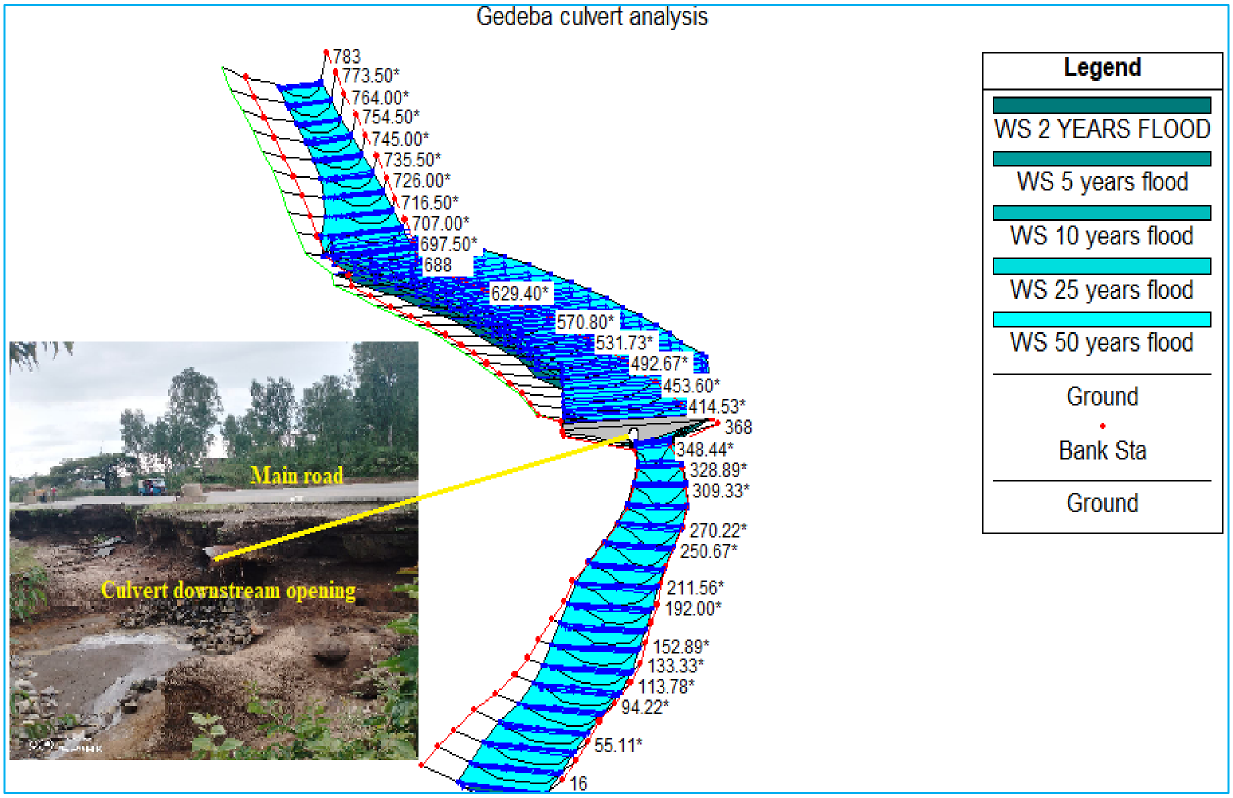


Figure 4.7: Gedeba river water surface profile in 3D view

During significant flood events, the volume of water flow can exceed the capacity of the culvert, leading to overtopping. This condition occurs when the water level rises above the culvert's inlet, spilling over into the surrounding area. Immediately downstream of the culvert, the terrain transitions from a nearly flat surface to a steep slope. This abrupt change in slope can drastically alter the flow characteristics of the overtopped water.

As the water exits the culvert and encounters the steep slope, its velocity increases. This fast flow speed can enhance the erosive power of the water, leading to greater soil displacement and degradation of the surrounding landscape. The combination of increased flow velocity and the steep gradient can result in significant erosion of the road surface and adjacent areas. The powerful flow can wash away soil and aggregate materials, undermining the stability of the roadway. Over time, continuous erosion can lead to severe undermining of the road structure. If the foundational soil is eroded sufficiently, it may cause sections of the road to collapse, posing serious risks to vehicles and pedestrians. The erosion of roads not only poses immediate safety hazards but also results in costly damage that necessitates repairs and maintenance. This can strain local resources and disrupt transportation networks.

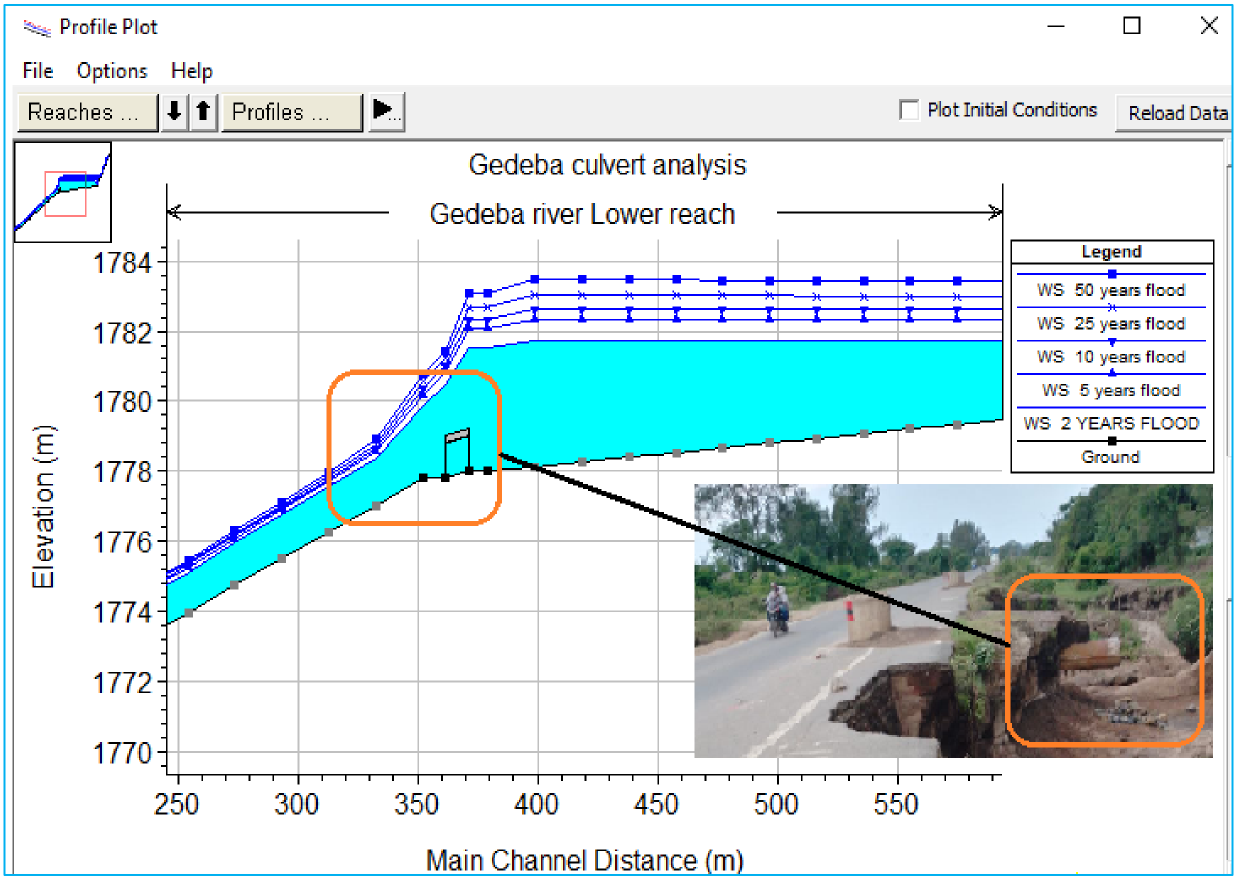


Figure 4.8: Gedeba river water surface profile in 2D view

4.4.2. Velocity Profile and Erosion Dynamics in the Main Channel

The analysis of the main channel velocity profile shows significant changes in flow characteristics that are critical for understanding flood dynamics and erosion potential. This section discusses the velocity variations along the channel, particularly in relation to the terrain slope, and the implications for erosion downstream of the culvert. In the upstream section of the main channel, the slope is notably steep. As illustrated in the figure, this gradient contributes to a higher flow velocity. The steep slope accelerates water movement, which increases the kinetic energy of the flowing water, leading to a more dynamic and turbulent flow regime. As floodwaters approach the reference station (RS) 648.93, the combination of high slope and high velocity creates conditions where the force of the water can effectively transport sediment and debris, reducing the risk of sediment deposition in this section. As the floodwaters move downstream towards RS 395, there is a significant change in terrain slope from steep to nearly gentle and eventually flat. This transition is critical as it alters the flow dynamics considerably. In this nearly flat section, the velocity of the flood wave diminishes significantly. The gentle slope reduces gravitational forces acting on the water, leading to slower flow rates. As a result, the area experiences prolonged inundation from the floodwaters, which can lead to localized flooding and water accumulation. After the floodwater passes over the culvert, there is a swift transition from a flat terrain to a steep slope. This abrupt change in gradient causes a dramatic increase in flow velocity. The water, now moving rapidly, gains significant erosive power as it cascades down the steep incline. The increased velocity downstream of the culvert elevates the erosive potential of the flow. This heightened energy can lead to severe erosion of the channel bed and banks, particularly affecting the stability of the downstream area. The erosive forces can dislodge soil and sediment, contributing to further degradation of the landscape. Figure 4.9 and 4.11 below shows the longitudinal and cross-sectional velocity profile for different years return period respectively.

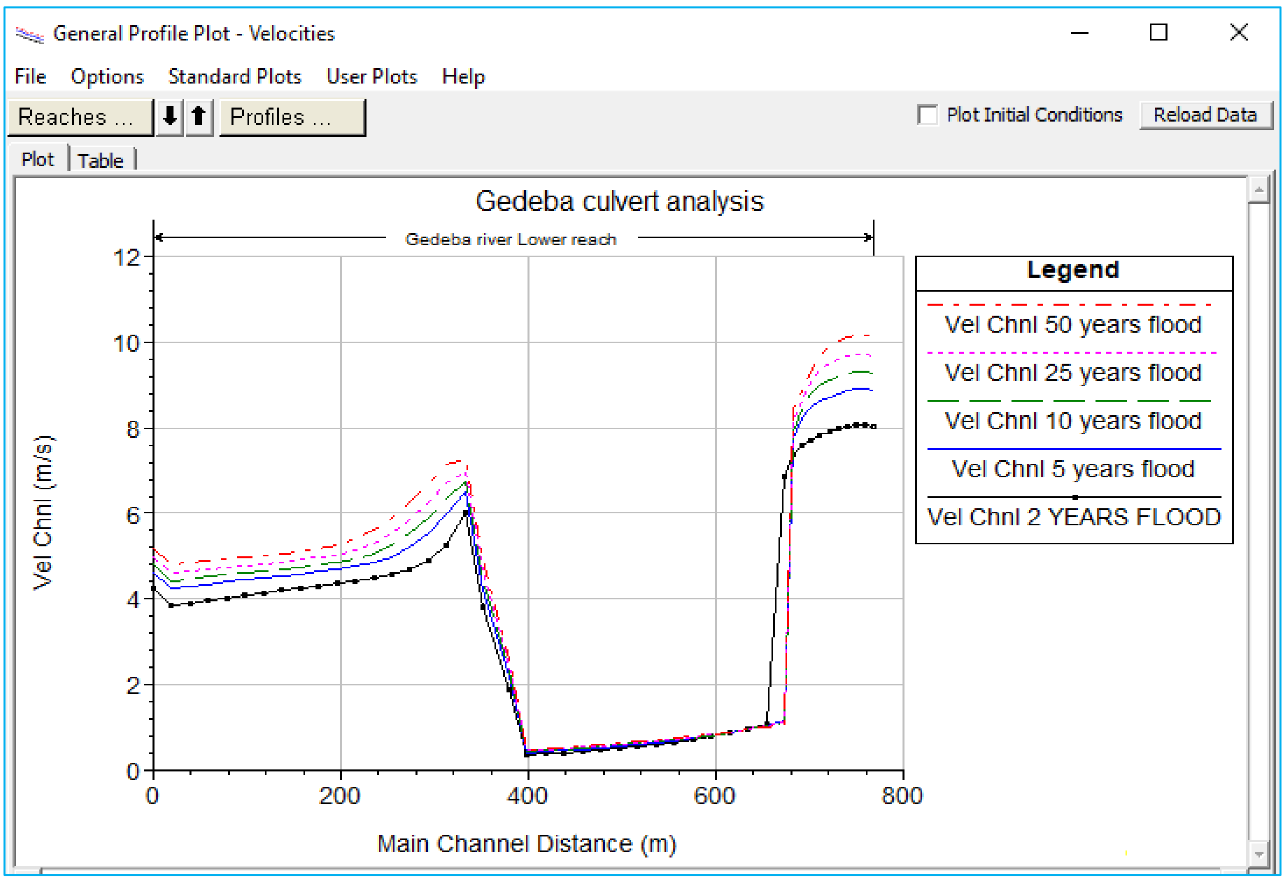


Figure 4.9: Velocity profile in main channel

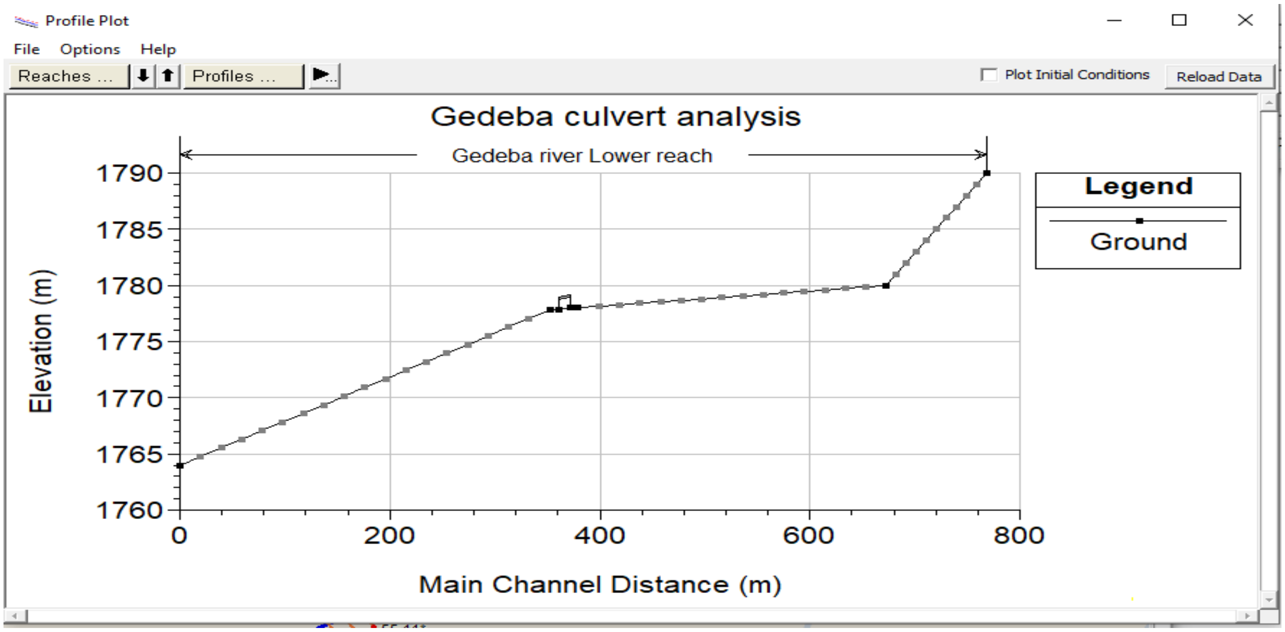


Figure 4.10: Main channel \longitudinal profile

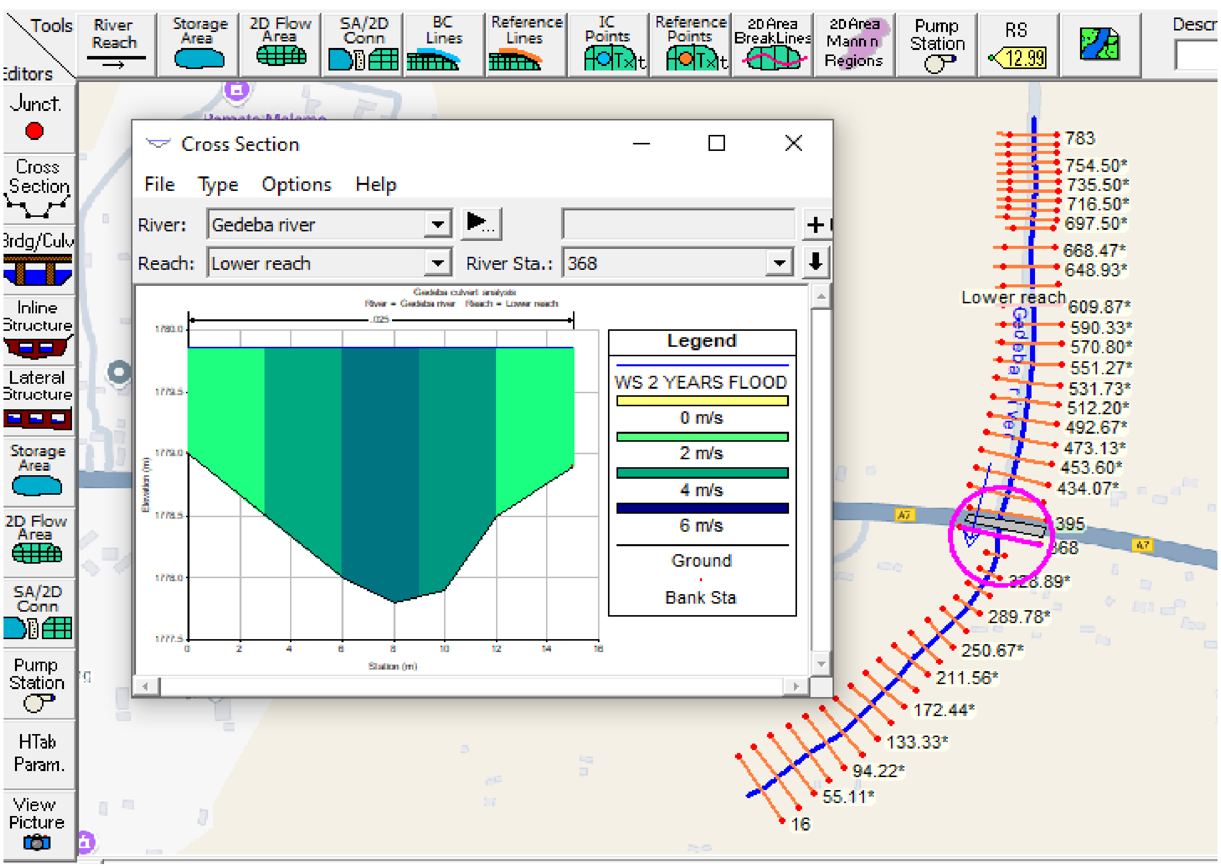
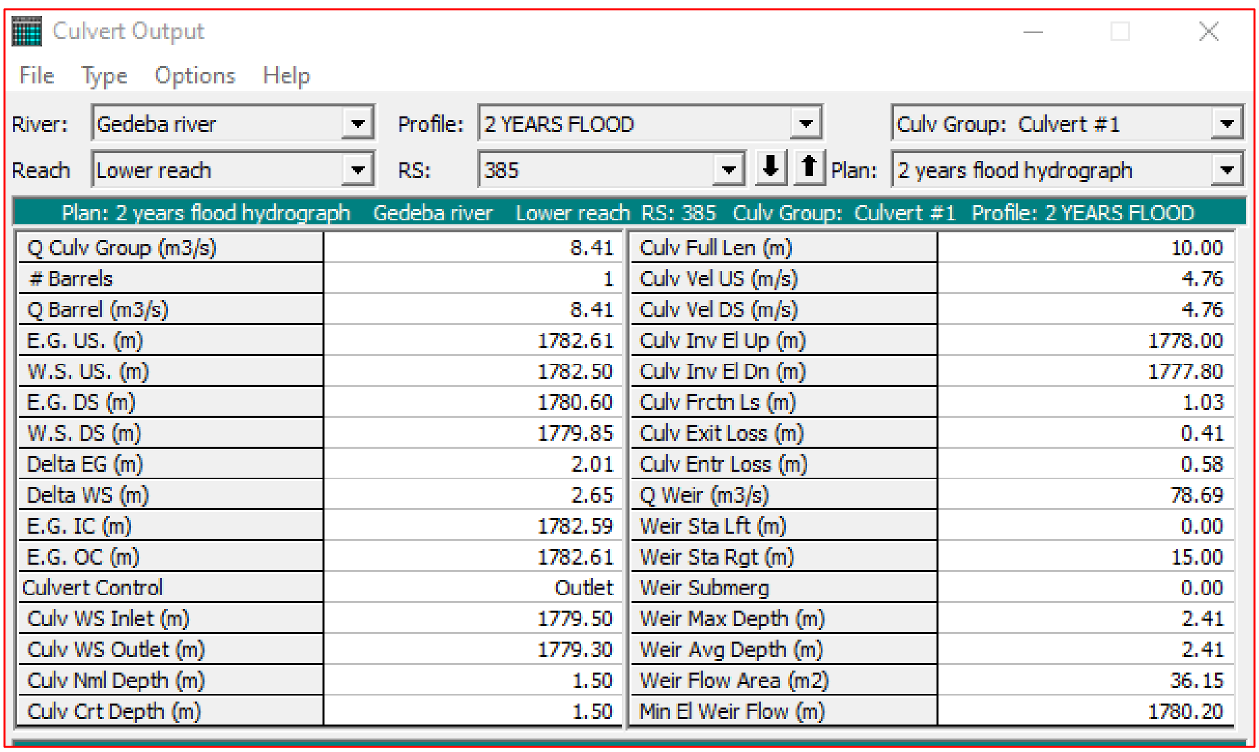


Figure 4.11: Cross-sectional velocity profile near culvert

4.4.3. Flow Capacity

The flow capacity of the culvert barrel was estimated using the hydraulic analysis of the culvert in HECRAS model. This involves calculating the culvert's design capacity using the Manning's equation for open channel flow in HECRAS. As shown on the figure below, the flow through the culvert barrel is $8.41 m^3/s$. but, the flood discharge for 2, 5, 10, 25 and 50 years were 87.1, 117.8, 135.4, 155, and $179.8 m^3/s$ respectively. According to the analysis, the culvert can pass only 9.7%, 7.1%, 6.2%, 5.4%, and 4.7%, for 2, 5, 10, 25 and 50 years respectively. These percentages indicate that the culvert's capacity is significantly lower than the required flow rates during flood events. For example, during a 2-year flood event, the culvert can only accommodate 9.7% of the incoming flow, which translates to a substantial risk of overtopping. As the return periods increase, the percentage of flow managed by the culvert continues to decline, reaching only 4.7% during a 50-year flood event. This implies that, the culvert cannot handle the incoming flow without overtopping for all flood events. Figure 4.12 shows the culvert output table in HECRAS


Culvert Output

File Type Options Help

River: Gedeba river | Profile: 2 YEARS FLOOD | Culv Group: Culvert #1

Reach Lower reach | RS: 385 | Plan: 2 years flood hydrograph

Plan: 2 years flood hydrograph Gedeba river Lower reach RS: 385 Culv Group: Culvert #1 Profile: 2 YEARS FLOOD

Q Culv Group (m3/s)	8.41	Culv Full Len (m)	10.00
# Barrels	1	Culv Vel US (m/s)	4.76
Q Barrel (m3/s)	8.41	Culv Vel DS (m/s)	4.76
E.G. US. (m)	1782.61	Culv Inv El Up (m)	1778.00
W.S. US. (m)	1782.50	Culv Inv El Dn (m)	1777.80
E.G. DS (m)	1780.60	Culv Frctn Ls (m)	1.03
W.S. DS (m)	1779.85	Culv Exit Loss (m)	0.41
Delta EG (m)	2.01	Culv Entr Loss (m)	0.58
Delta WS (m)	2.65	Q Weir (m3/s)	78.69
E.G. IC (m)	1782.59	Weir Sta Lft (m)	0.00
E.G. OC (m)	1782.61	Weir Sta Rgt (m)	15.00
Culvert Control	Outlet	Weir Submerg	0.00
Culv WS Inlet (m)	1779.50	Weir Max Depth (m)	2.41
Culv WS Outlet (m)	1779.30	Weir Avg Depth (m)	2.41
Culv Nml Depth (m)	1.50	Weir Flow Area (m2)	36.15
Culv Crt Depth (m)	1.50	Min El Weir Flow (m)	1780.20

Figure 4.12: Culvert analysis table in HECRAS

4.5. Mitigation Options for Enhancing the Hydraulic Performance of a Culvert

Culverts play a crucial role in managing water flow in drainage systems, particularly in areas prone to flooding. However, inadequate hydraulic performance can lead to significant erosion, flooding, and damage to infrastructure. This report outlines various mitigation strategies to enhance the hydraulic performance of culverts, focusing on optimizing flow dynamics, improving structural resilience, and protecting surrounding areas from erosion and flooding. Recognizing the vulnerability of areas downstream of culverts is essential for effective management. Mitigation measures, such as installing erosion control structures (e.g., riprap, check dams) and implementing vegetative buffers, can help stabilize the terrain and reduce the risk of erosion. The following mitigation measures are recommended for the bridges in areas were bridges are lying in order to mitigate the culvert performance problem

4.5.1. Different Culvert Redesign and Upgrade for existing cross section

Redesign existing culverts to increase their diameter or use larger culvert materials to accommodate higher flow rates. This helps prevent overtopping during heavy rainfall. Employ multi-barrel culvert designs, which utilize several smaller pipes instead of one large pipe. This can improve flow efficiency and reduce hydraulic head loss.

4.5.1.1. Arch culvert

The analysis of the arch culvert reveals that it can manage a flow of 18.26m^3/s as shown on Figure 4.13 and 4.14 below. While this represents an improvement over the existing culvert design, the capacity of the arch culvert still falls short when compared to the required flood discharges for various return periods. According to arch culvert analysis, the culvert can accommodate only 32%, 24%, 21%, 18%, and 16%, for 2, 5, 10, 25 and 50 years respectively. These percentages indicate that, while the arch culvert can handle a greater proportion of flood flows compared to the existing culvert, it still cannot fully manage the anticipated flood events. For instance, during a 2-year flood event, the arch culvert can accommodate 32% of the flow, which is an improvement but still leaves a significant portion unmanageable.

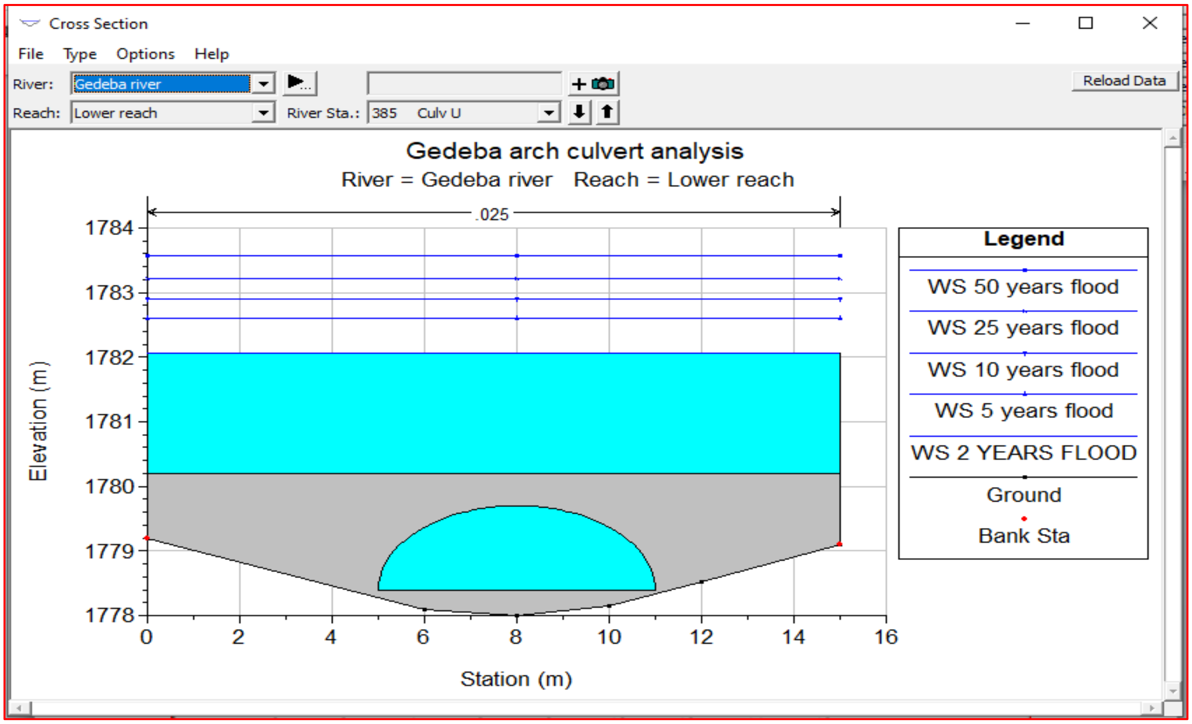
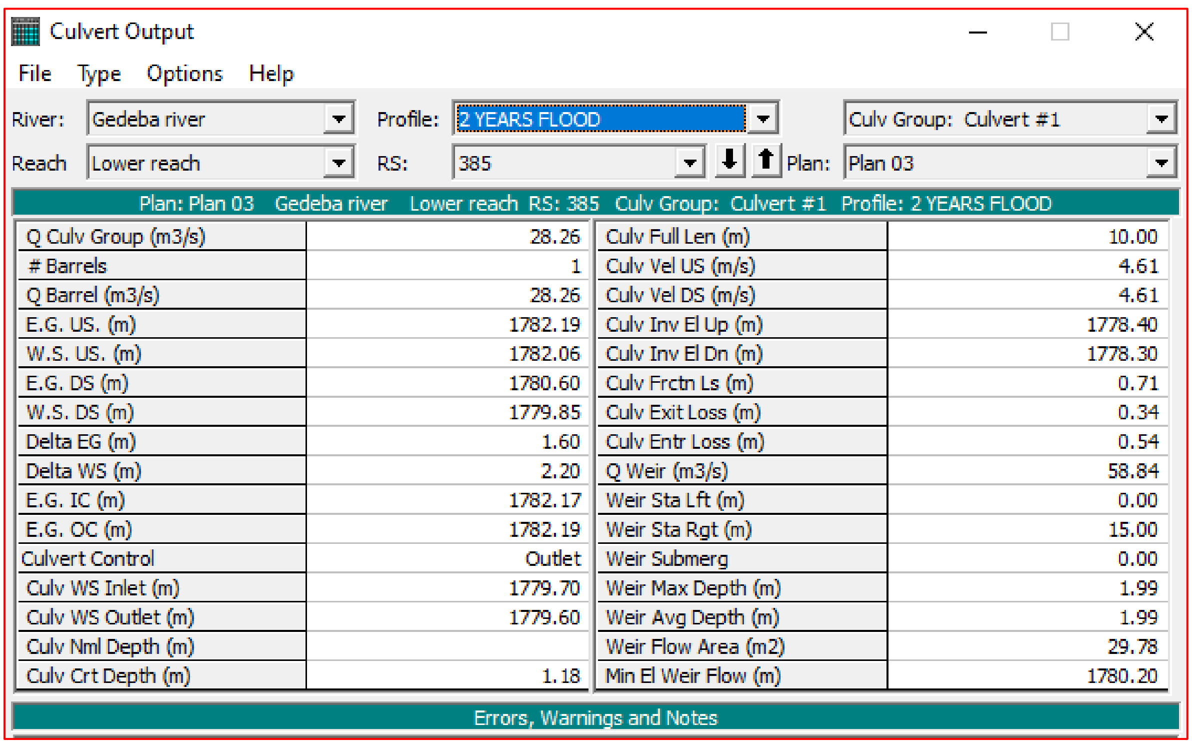


Figure 4.13: Arch culvert profile



Culvert Output

File Type Options Help

River: Gedeba river | Profile: 2 YEARS FLOOD | Culv Group: Culvert #1

Reach: Lower reach | RS: 385 | Plan: Plan 03

Plan: Plan 03 Gedeba river Lower reach RS: 385 Culv Group: Culvert #1 Profile: 2 YEARS FLOOD

Q Culv Group (m3/s)	28.26	Culv Full Len (m)	10.00
# Barrels	1	Culv Vel US (m/s)	4.61
Q Barrel (m3/s)	28.26	Culv Vel DS (m/s)	4.61
E.G. US. (m)	1782.19	Culv Inv El Up (m)	1778.40
W.S. US. (m)	1782.06	Culv Inv El Dn (m)	1778.30
E.G. DS (m)	1780.60	Culv Frctn Ls (m)	0.71
W.S. DS (m)	1779.85	Culv Exit Loss (m)	0.34
Delta EG (m)	1.60	Culv Entr Loss (m)	0.54
Delta WS (m)	2.20	Q Weir (m3/s)	58.84
E.G. IC (m)	1782.17	Weir Sta Lft (m)	0.00
E.G. OC (m)	1782.19	Weir Sta Rgt (m)	15.00
Culvert Control	Outlet	Weir Submerg	0.00
Culv WS Inlet (m)	1779.70	Weir Max Depth (m)	1.99
Culv WS Outlet (m)	1779.60	Weir Avg Depth (m)	1.99
Culv Nml Depth (m)		Weir Flow Area (m2)	29.78
Culv Crt Depth (m)	1.18	Min El Weir Flow (m)	1780.20

Errors, Warnings and Notes

Figure 4.14: Arch culvert analysis output

4.5.1.2. Box culvert

The analysis of the box culvert indicates that it has the capacity to manage a flow of $18.42 m^3/s$, as illustrated in Figure 4.15 and 4.16. This flow capacity marks an improvement over the existing culvert design, which was previously determined to be inadequate for handling anticipated flood events. However, despite this advancement, the box culvert's capacity still falls short when compared to the flood discharges required for various return periods. These percentages illustrate that, while the box culvert can manage a greater proportion of flood flows compared to the existing culvert, its overall capacity remains insufficient for larger flood events. For instance, during a 2-year flood event, the box culvert can accommodate 32.6% of the flow. Although this represents an improvement, it still means that a significant portion of the floodwaters approximately 67.4% would remain unmanageable, leading to potential overtopping and flooding in upstream areas.

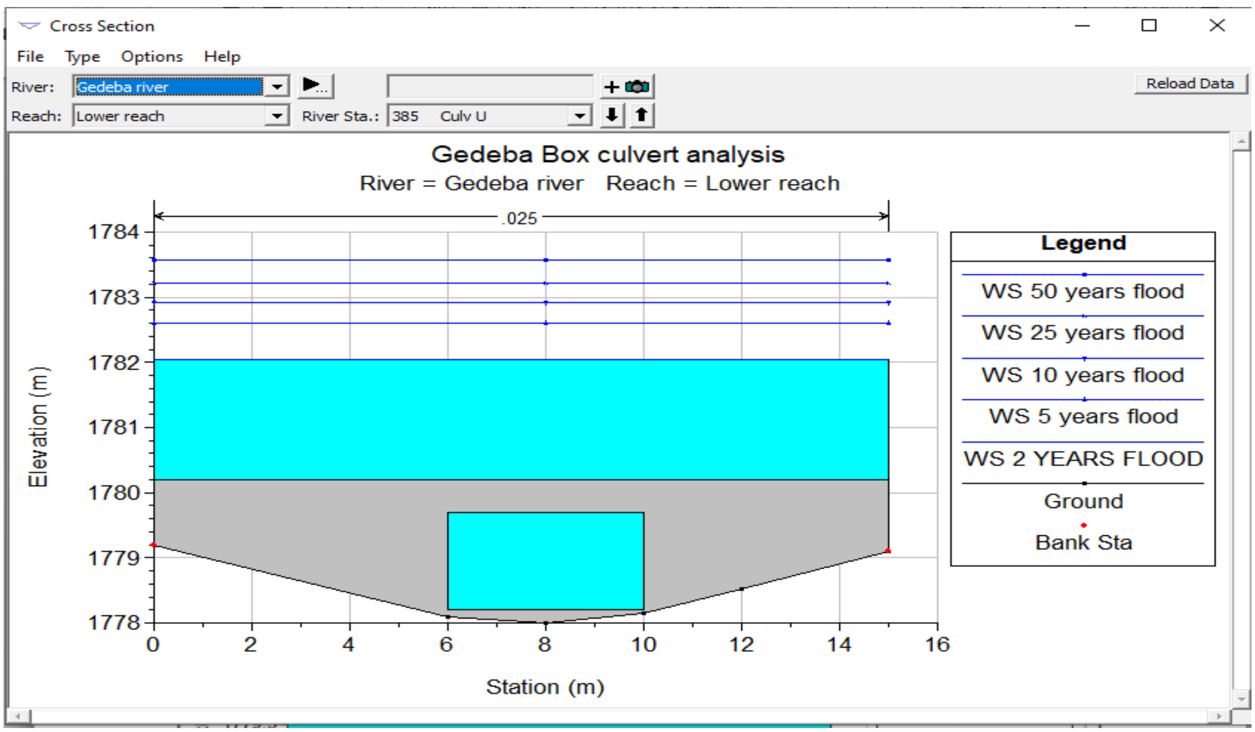
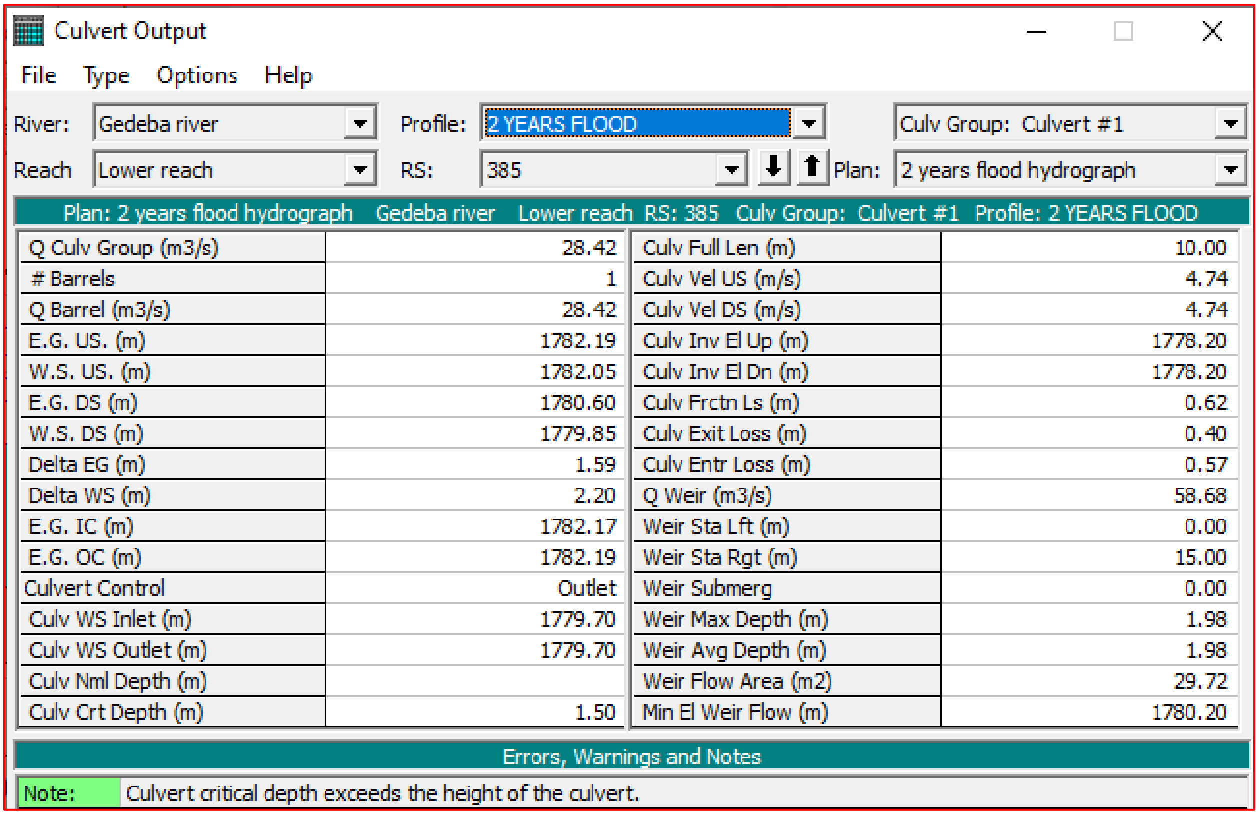


Figure 4.15: Box culvert profile



Culvert Output

File Type Options Help

River: Gedeba river | Profile: 2 YEARS FLOOD | Culv Group: Culvert #1

Reach: Lower reach | RS: 385 | Plan: 2 years flood hydrograph

Plan: 2 years flood hydrograph Gedeba river Lower reach RS: 385 Culv Group: Culvert #1 Profile: 2 YEARS FLOOD

Q Culv Group (m3/s)	28.42	Culv Full Len (m)	10.00
# Barrels	1	Culv Vel US (m/s)	4.74
Q Barrel (m3/s)	28.42	Culv Vel DS (m/s)	4.74
E.G. US. (m)	1782.19	Culv Inv El Up (m)	1778.20
W.S. US. (m)	1782.05	Culv Inv El Dn (m)	1778.20
E.G. DS (m)	1780.60	Culv Frctn Ls (m)	0.62
W.S. DS (m)	1779.85	Culv Exit Loss (m)	0.40
Delta EG (m)	1.59	Culv Entr Loss (m)	0.57
Delta WS (m)	2.20	Q Weir (m3/s)	58.68
E.G. IC (m)	1782.17	Weir Sta Lft (m)	0.00
E.G. OC (m)	1782.19	Weir Sta Rgt (m)	15.00
Culvert Control	Outlet	Weir Submerg	0.00
Culv WS Inlet (m)	1779.70	Weir Max Depth (m)	1.98
Culv WS Outlet (m)	1779.70	Weir Avg Depth (m)	1.98
Culv Nml Depth (m)		Weir Flow Area (m2)	29.72
Culv Crt Depth (m)	1.50	Min El Weir Flow (m)	1780.20

Errors, Warnings and Notes

Note: Culvert critical depth exceeds the height of the culvert.

Figure 4.16: Box culvert analysis output

4.5.1.3.Multiple Box culvert

The analysis of the multiple box culverts reveals that they can manage a flow of 24.42m³/s, as depicted in Figure 4.17 and 4.18. This capacity represents a notable improvement over the previous culvert design, which was identified as inadequate for effectively handling anticipated flood events. However, despite this advancement, the capacity of the multiple box culverts still falls.

According to multiple box culvert analysis, the culvert can accommodate only 28%, 21%, 18%, 16%, and 14%, for 2, 5, 10, 25 and 50 years respectively. This flow capacity marks an improvement over the existing culvert design, which was previously determined to be inadequate for handling anticipated flood events. However, despite this advancement, the multiple box culverts' capacity still falls short when compared to the flood discharges required for various return periods. These percentages illustrate that, while the multiple box culvert can manage a greater proportion of flood flows compared to the existing culvert, its overall capacity remains insufficient for larger flood events. These figures indicate that, while the multiple box culverts can handle a greater proportion of flood flows compared to the existing culvert design, their overall capacity remains insufficient for larger flood events. For instance, during a 2-year flood event, the multiple box culverts can accommodate only 28% of the incoming flow. This means that 72% of the floodwaters would not be managed, placing surrounding areas at risk of flooding.

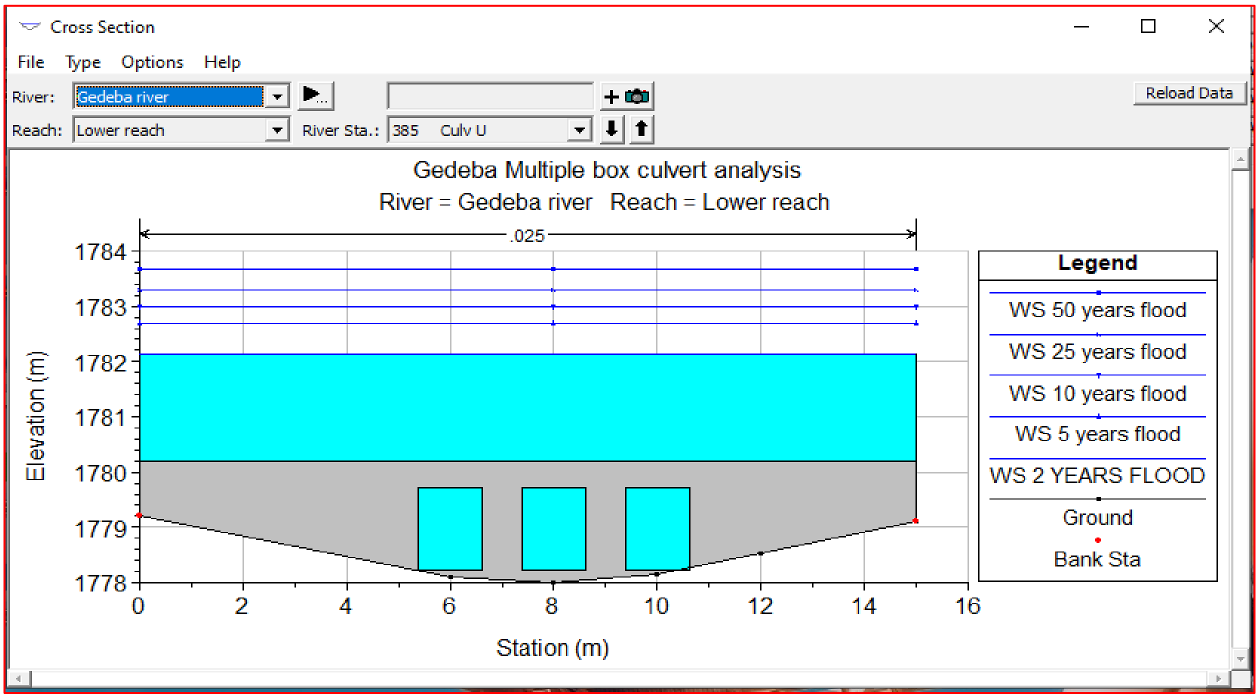


Figure 4.17: Multiple box culvert profile

Culvert Output

File Type Options Help

River: Gedeba river | Profile: 2 YEARS FLOOD | Culv Group: Culvert #1

Reach: Lower reach | RS: 385 | Plan: 2 years flood hydrograph

Plan: 2 years flood hydrograph Gedeba river Lower reach RS: 385 Culv Group: Culvert #1 Profile: 2 YEARS FLOOD

Q Culv Group (m3/s)	24.42	Culv Full Len (m)	10.00
# Barrels	3	Culv Vel US (m/s)	4.34
Q Barrel (m3/s)	8.14	Culv Vel DS (m/s)	4.34
E.G. US. (m)	1782.27	Culv Inv El Up (m)	1778.22
W.S. US. (m)	1782.13	Culv Inv El Dn (m)	1778.15
E.G. DS (m)	1780.60	Culv Frctn Ls (m)	0.97
W.S. DS (m)	1779.85	Culv Exit Loss (m)	0.22
Delta EG (m)	1.67	Culv Entr Loss (m)	0.48
Delta WS (m)	2.28	Q Weir (m3/s)	62.68
E.G. IC (m)	1782.19	Weir Sta Lft (m)	0.00
E.G. OC (m)	1782.27	Weir Sta Rgt (m)	15.00
Culvert Control	Outlet	Weir Submerg	0.00
Culv WS Inlet (m)	1779.72	Weir Max Depth (m)	2.07
Culv WS Outlet (m)	1779.65	Weir Avg Depth (m)	2.07
Culv Nml Depth (m)	1.50	Weir Flow Area (m2)	31.06
Culv Crt Depth (m)	1.50	Min El Weir Flow (m)	1780.20

Errors, Warnings and Notes

Figure 4.18: Box culvert analysis output

4.5.1.4.Semi-circle culvert

The analysis of the semi-circle culvert indicates that it has the capacity to manage a flow of 16.77m³/s, as illustrated in Figure 4.19 and 4.20. This flow capacity signifies a notable improvement over the previous culvert design, which was identified as inadequate for effectively handling anticipated flood events. However, despite this advancement, the capacity of the semi-circle culverts still falls short when assessed against the flood discharges required for various return periods.. This capacity represents a notable improvement over the previous culvert design, which was identified as inadequate for effectively handling anticipated flood events. However, despite this advancement, the capacity of the semi-circle culvert's still falls. According to analysis, the culvert can accommodate only 19%, 14%, 12%, 11%, and 9%, for 2, 5, 10, 25 and 50 years respectively. These percentages indicate that while the semi-circle culverts can manage a greater proportion of flood flows compared to the existing culvert design, their overall capacity remains inadequate for larger flood events. For example, during a 2-year flood event, the semi-circle culvert can accommodate only 19% of the incoming flow, which leaves 81% of the floodwaters unmanageable. This shortfall becomes even more pronounced for higher return periods, where the capacity to manage flow dwindles to merely 9% during a 50-year flood event.

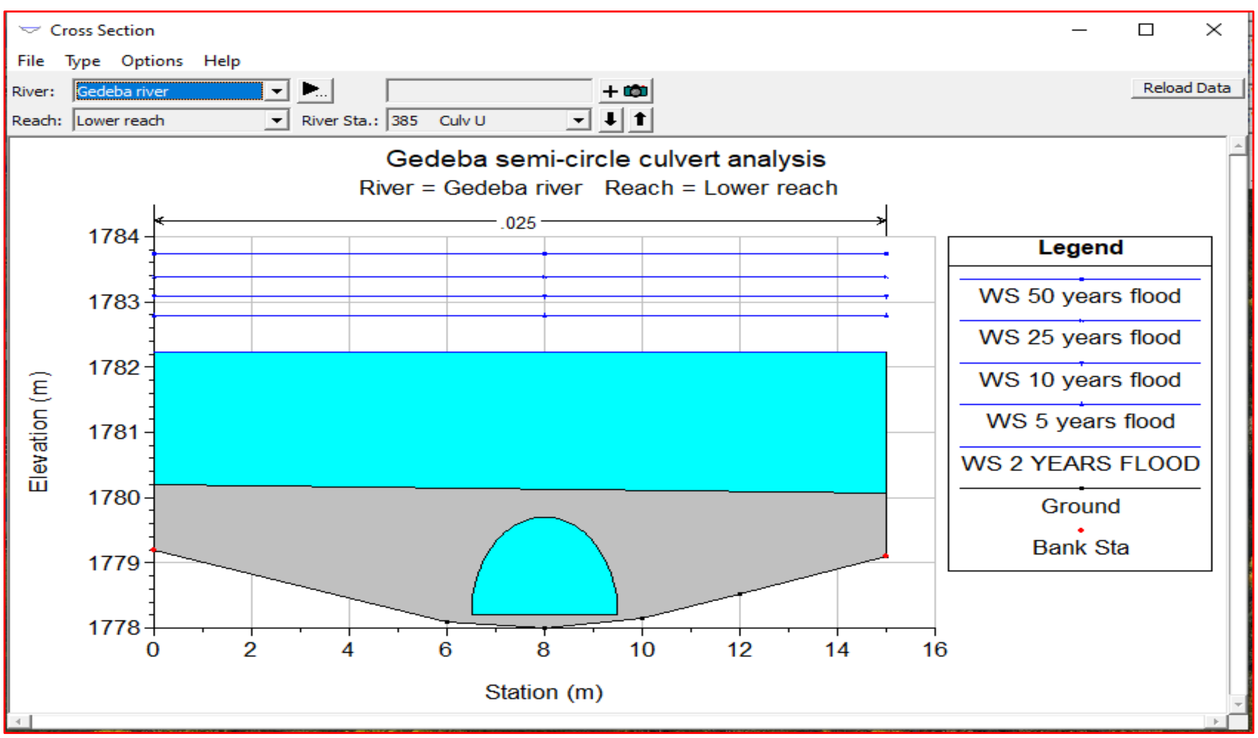
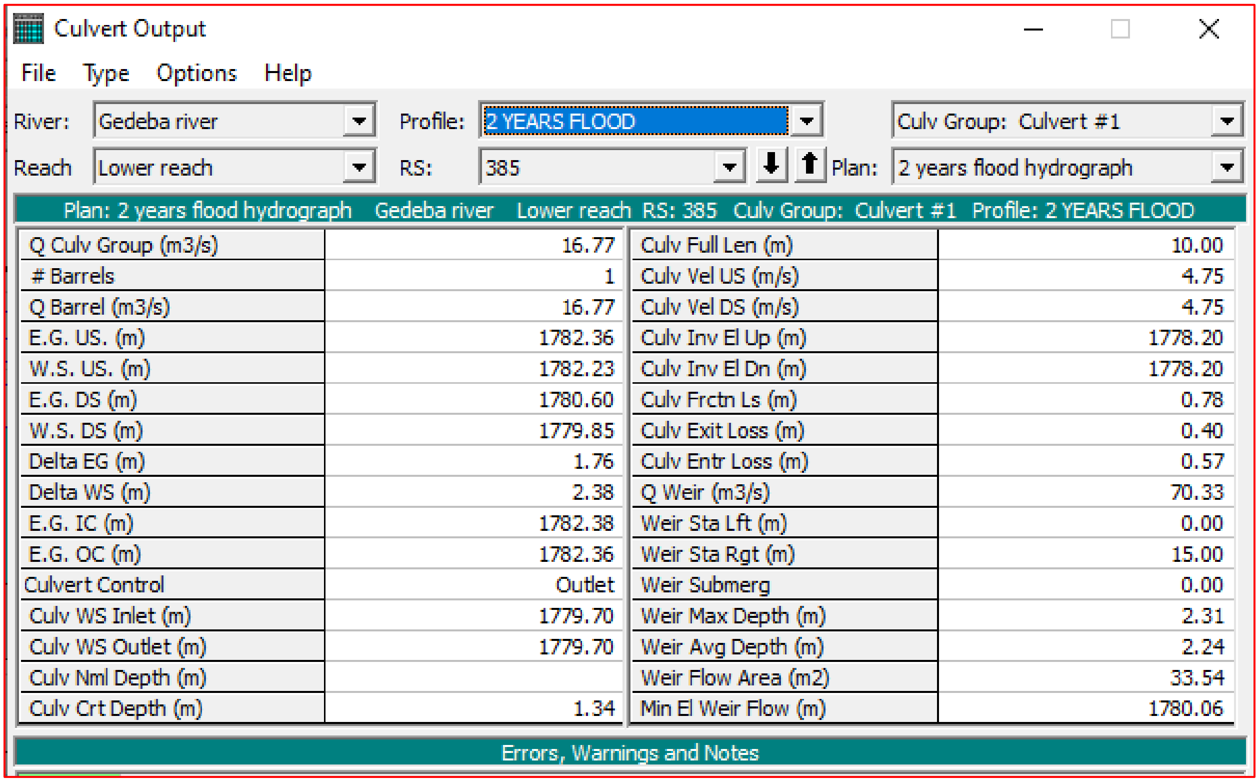


Figure 4.19:Semi-circle culvert profile



Q Culv Group (m3/s)	16.77	Culv Full Len (m)	10.00
# Barrels	1	Culv Vel US (m/s)	4.75
Q Barrel (m3/s)	16.77	Culv Vel DS (m/s)	4.75
E.G. US. (m)	1782.36	Culv Inv El Up (m)	1778.20
W.S. US. (m)	1782.23	Culv Inv El Dn (m)	1778.20
E.G. DS (m)	1780.60	Culv Frctn Ls (m)	0.78
W.S. DS (m)	1779.85	Culv Exit Loss (m)	0.40
Delta EG (m)	1.76	Culv Entr Loss (m)	0.57
Delta WS (m)	2.38	Q Weir (m3/s)	70.33
E.G. IC (m)	1782.38	Weir Sta Lft (m)	0.00
E.G. OC (m)	1782.36	Weir Sta Rgt (m)	15.00
Culvert Control	Outlet	Weir Submerg	0.00
Culv WS Inlet (m)	1779.70	Weir Max Depth (m)	2.31
Culv WS Outlet (m)	1779.70	Weir Avg Depth (m)	2.24
Culv Nml Depth (m)		Weir Flow Area (m2)	33.54
Culv Crt Depth (m)	1.34	Min El Weir Flow (m)	1780.06

Figure 4.20:Semi-circle analysis output

4.5.1.5.Ellipse culvert

The analysis of the ellipse culvert indicates that it has the capacity to manage a flow of 17.22m³/s, as illustrated in Figure 4.21 and 4.22. This flow capacity marks a notable improvement over the previous culvert design, which was identified as inadequate for effectively handling anticipated flood events. However, despite this advancement, the capacity of the ellipse culverts still falls short when evaluated against the flood discharges required for various return periods. According to analysis, the culvert can accommodate only 20%, 15%, 13%, 11%, and 10%, for 2, 5, 10, 25 and 50 years respectively. These percentages reveal that, while the ellipse culverts can manage a greater proportion of flood flows compared to the existing culvert design, their overall capacity remains inadequate for larger flood events. For example, during a 2-year flood event, the ellipse culvert can accommodate only 20% of the incoming flow, which leaves 80% of the floodwaters unmanageable. This deficiency becomes even more pronounced for higher return periods, where the capacity to manage flow diminishes to just 10% during a 50-year flood event.

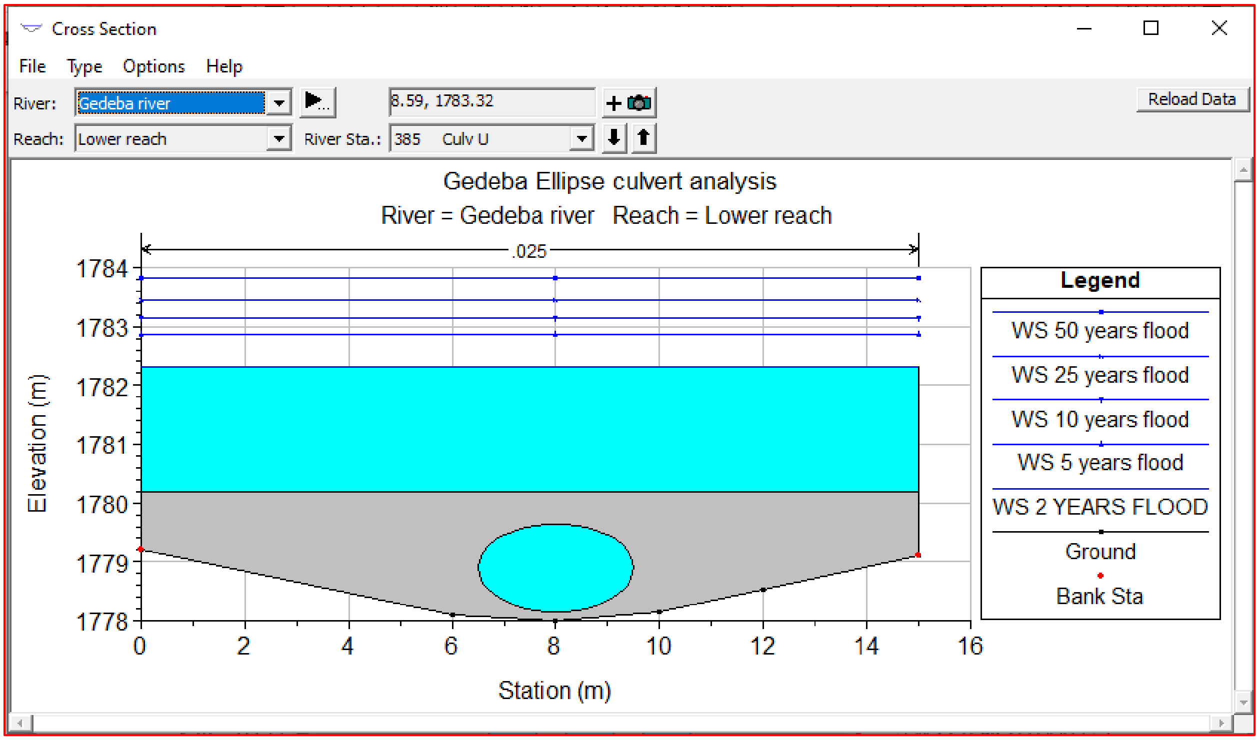
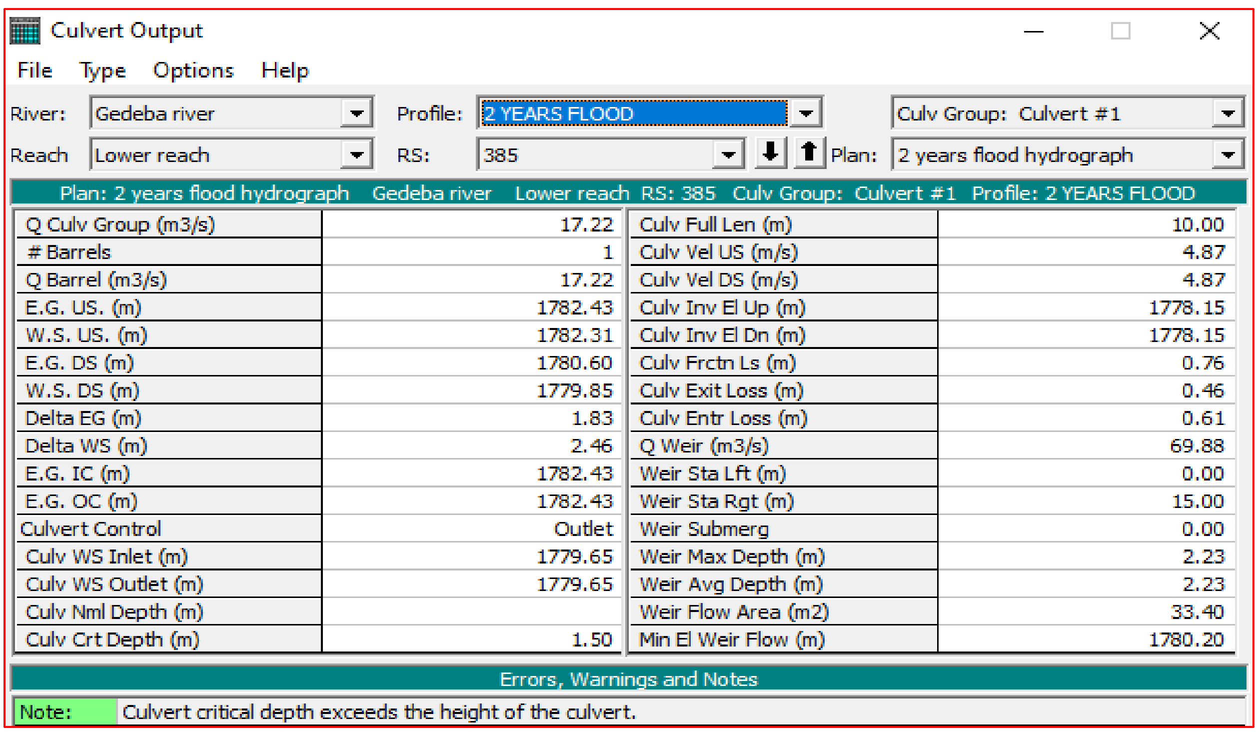


Figure 4.21: Ellipse culvert profile



Plan: 2 years flood hydrograph Gedeba river Lower reach RS: 385 Culv Group: Culvert #1 Profile: 2 YEARS FLOOD

Q Culv Group (m3/s)	17.22	Culv Full Len (m)	10.00
# Barrels	1	Culv Vel US (m/s)	4.87
Q Barrel (m3/s)	17.22	Culv Vel DS (m/s)	4.87
E.G. US. (m)	1782.43	Culv Inv El Up (m)	1778.15
W.S. US. (m)	1782.31	Culv Inv El Dn (m)	1778.15
E.G. DS (m)	1780.60	Culv Frctn Ls (m)	0.76
W.S. DS (m)	1779.85	Culv Exit Loss (m)	0.46
Delta EG (m)	1.83	Culv Entr Loss (m)	0.61
Delta WS (m)	2.46	Q Weir (m3/s)	69.88
E.G. IC (m)	1782.43	Weir Sta Lft (m)	0.00
E.G. OC (m)	1782.43	Weir Sta Rgt (m)	15.00
Culvert Control	Outlet	Weir Submerg	0.00
Culv WS Inlet (m)	1779.65	Weir Max Depth (m)	2.23
Culv WS Outlet (m)	1779.65	Weir Avg Depth (m)	2.23
Culv Nml Depth (m)		Weir Flow Area (m2)	33.40
Culv Crt Depth (m)	1.50	Min El Weir Flow (m)	1780.20

Errors, Warnings and Notes

Note: Culvert critical depth exceeds the height of the culvert.

Figure 4.22: Ellipse analysis output

4.5.2. Different Culvert Redesign and Upgrade for Modified cross section

The analysis of various culvert types has highlighted significant limitations in their capacity to manage incoming floodwaters. Hydraulic analyses consistently indicated that none of the existing culvert designs could adequately accommodate anticipated flood flows for any evaluated return periods. This deficiency raises concerns about potential flooding and associated risks in surrounding areas, including property damage, road washouts, and even loss of life. To address these inadequacies, it is crucial to explore modifications or widening of the river cross section, which can enhance floodwater management by increasing flow capacity, reducing water velocity, and minimizing erosion. Additionally, replacing existing culverts with larger, more efficient designs or implementing multi-cell culverts can significantly improve their performance. Figures 4.23 to figure 4.26 below shows the water surface profile for different flood frequencies in the modified culverts cross sections

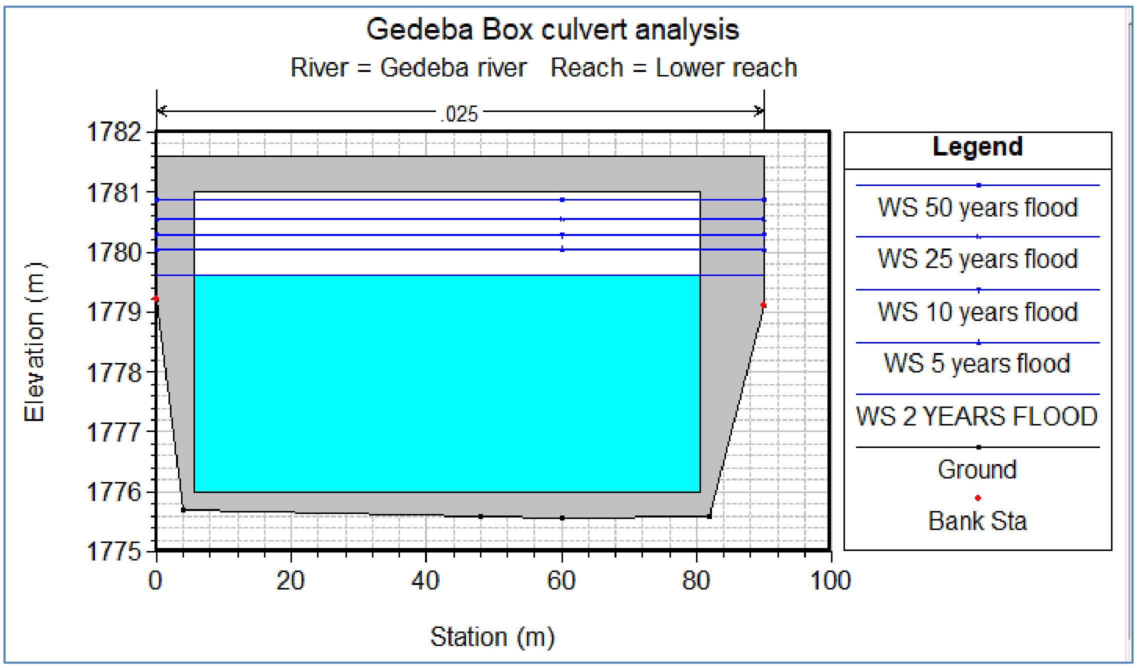


Figure 4.23 : Modified box culvert

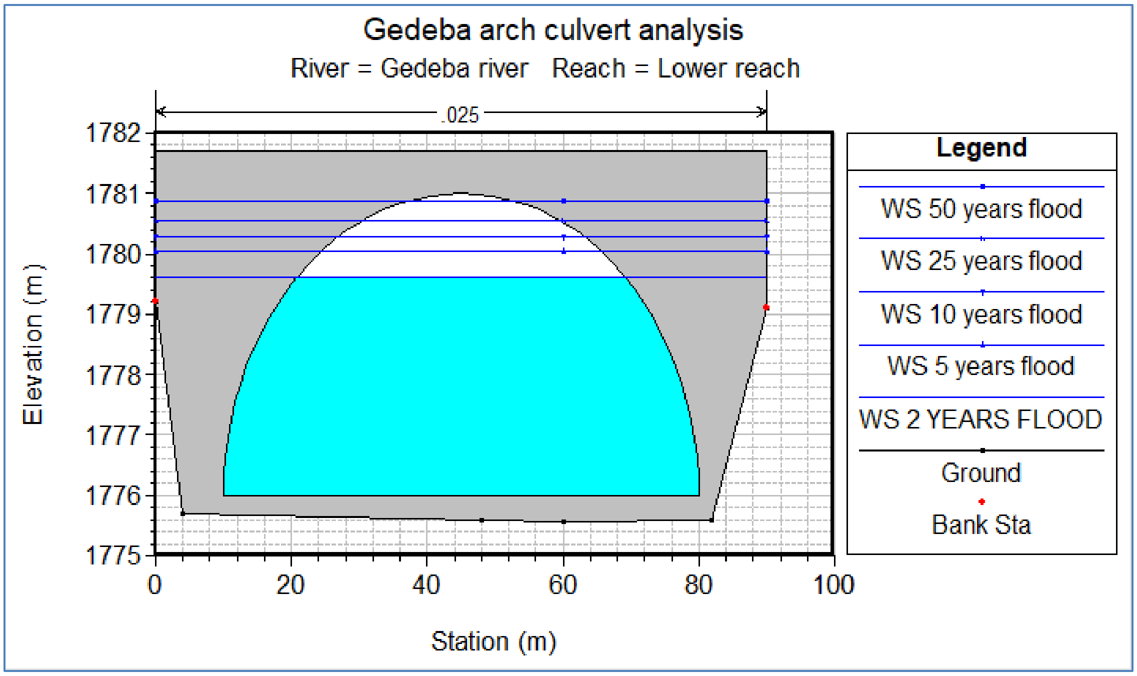


Figure 4.24: Modified Arch culvert

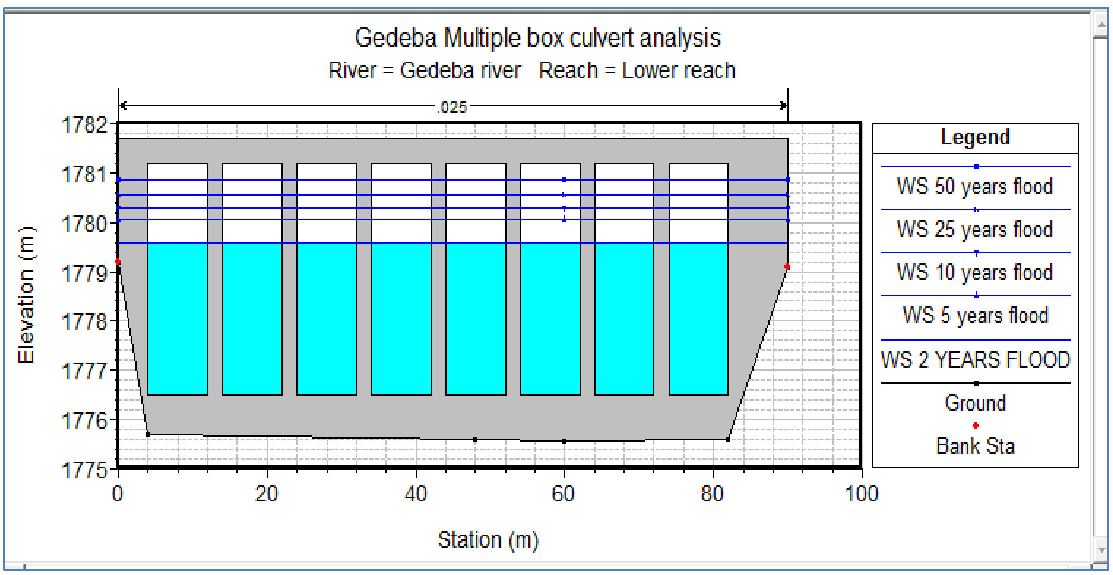


Figure 4.25: Modified multiple box culvert

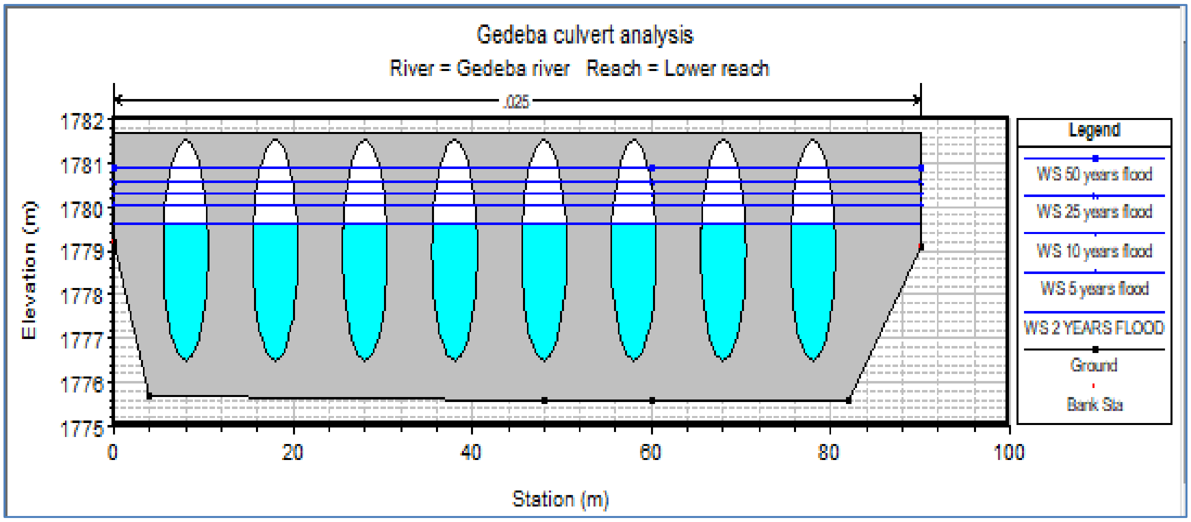


Figure 4.26: Modified pipe culvert

4.5.3. Selecting best mitigation option fit to existing condition

As the figure 4.23 to 4.26 above indicated, the modified culverts have the capacity to pass the incoming flood safely. But, when selecting the best culvert type for a site, several factors was considered, including simplicity of construction, structural durability, environmental issues, and economic viability. Based on those factors, Box culverts present an exciting option due to their moderate complexity in construction, which requires careful placement and alignment but is generally straightforward (Van Dijk & Moloisane, 2021). They are designed to support heavy loads and withstand environmental stresses, making them highly durable (Wang et al., 2022). Economically, they are typically more expensive than pipe culverts but offer long-term benefits in terms of durability and capacity (Miguez et al., 2012). In contrast, multiple box culverts can provide enhanced flow capacity and better distribution, yet they introduce increased complexity in installation due to the need for coordination among multiple units (Frankiewicz et al., 2021). While structurally durable, the higher initial costs may not always justify their use unless significantly greater flow capacity is required. Arch culverts are another strong contender, particularly when environmental sustainability is a priority. Their design allows for natural flow patterns and habitat connectivity, which can mitigate ecological impacts (Bao et al., 2023). While they may be more expensive and require precise engineering, their long-term durability and ability to handle loads effectively make them a valuable option in sensitive environments.

On the other hand, multiple pipe culverts are known for their simplicity and ease of installation, making them a quick solution for smaller drainage needs. However, their structural durability is generally moderate, and while they can be cost-effective, concerns about long-term performance under heavy loads may arise (Balkham et al., 2010). Additionally, the installation of multiple pipes can complicate flow patterns and affect habitat connectivity.

Considering all these factors, box culverts emerge as the best overall choice for most applications. They strike a balance between structural durability, hydraulic capacity, and relative ease of construction, while also being adaptable to environmental considerations. If the project site prioritizes ecological sustainability, arch culverts may be the preferred option, despite their higher costs. Ultimately, the specific site conditions, including anticipated flow rates, environmental sensitivity, and budget constraints, will guide the final decision, but box culverts generally provide the most versatile and reliable solution for effective flood management.

4.5.4. Non-Structural Measures

Community Engagement and Education: Conduct educational initiatives to inform communities about the importance of maintaining culvert functionality and recognizing signs of blockage or damage. Create channels for residents to report issues with culverts, such as blockages or erosion, ensuring timely responses from authorities.

Land Use Planning and Zoning: Implement zoning regulations that restrict development in high-risk floodplain areas, minimizing exposure to flood hazards and protecting infrastructure. Establish vegetative buffer zones around culverts to filter runoff, improve water quality, and reduce sedimentation within drainage systems.

Monitoring and Data Collection: Install sensors and monitoring systems to track water levels and flow rates in and around culverts. This data can provide insights for timely interventions during flood events. Conduct thorough assessments after significant rainfall or flooding to evaluate culvert performance and identify areas for improvement.

Natural Solutions: Implement vegetative buffers along waterways to stabilize soil, reduce runoff, and enhance water quality. Native plants can help absorb excess water and filter pollutants. Restore nearby wetlands to increase water retention and reduce peak flow rates, thereby alleviating pressure on culvert systems.

Emergency Preparedness Planning: Develop and regularly update emergency response plans that include protocols for culvert management during flood events. This ensures communities are prepared for quick action.

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5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

As climate change continues to worsen, many regions are witnessing increasingly severe weather patterns, including intensified rainfall and flooding. Ethiopia, being particularly susceptible to these changes, faces significant challenges in managing its road infrastructure amid such climatic fluctuations. This study focuses on the Gedeba River, a tributary of the Bilate River, to evaluate the hydraulic performance of an existing cross drainage culvert. The main objective of this study was to evaluate the hydraulic performance of the existing cross drainage culvert whether it pass the flood water safely without over topping and areal inundation, eroding the road embankments. In order to achieve the objectives of this study, existing Meteorological, hydrologic and topographic data collected from different organizations were used. These data are, meteorological data, stream flow data, manning's roughness coefficient of the site, topographic and land use/land cover data. The meteorological data has been collected from NMA, The flow data was collected from Hydrology Department of Ministry of Water and energy (MoWE), whereas the Geometric data such as cross section data, culvert data was collected from the field**.** The materials and tools used for study are: HEC HMS, HEC-RAS model, GIS software, GPS, Metering tape. In order to evaluate the hydraulic performance of the culvert, the inflow hydrograph for different return period was developed using HEC-HMS. Daily rainfall data were collected over a period of 1991- 2021 and Observed stream flow data were collected over a period of 1991-2013 available from MoWE was used. The model was calibrated and validated using actual stream flow data. The calibration and validation result indicated that there was strong relationship between simulated and observed stream flow data. Hence, based on these statistical error test criteria HEC-HMS model performance of the model is classified as very good. After model was calibrated and validated actual observed flow data, frequency storm was generated using the annual maximum precipitation available from rainfall data and it is used as an input for HEC-HMS model to develop flood hydrograph for different return periods. The result of event based simulation for developing flood hydrograph for different period shows that, the maximum flood hydrograph for 2, 5, 10, 25, and 50 years were 87.1, 117.8, 135.4, 155, and 179.8m3/s respectively. To analyze the hydraulic performance of the culvert, the HEC-RAS model was employed to develop water surface, culvert capacity and velocity profiles. The analysis indicated that, during significant flood events, the water level exceeded the culvert crest, resulting in overtopping and subsequent flooding of the main road connecting Halaba to Shashamene. This overtopping not only posed a risk to the road infrastructure but also led to erosion of the downstream area and collapse of some parts of the road. Furthermore, the analysis revealed that the velocity of the floodwaters passing over the culvert crest was significantly high, possessing erosive characteristics that exacerbated erosion downstream. The downstream area suffered substantial damage, with some sections of the road collapsing due to the intense flow and erosion. This underperformance of the culvert highlights critical infrastructure vulnerability, necessitating urgent attention and action. To alleviate these problems, different type of culverts were selected for analysis to redesign the existing culvert which accommodates the floods.

The analysis of various culvert types reveals significant limitations in their capacity to manage floodwaters, with existing designs inadequate for anticipated flood flows, raising risks of flooding and property damage. Modifications such as widening river cross sections or replacing culverts with larger designs are essential for improving floodwater management. Among the options, box culverts are preferred due to their balance of structural durability, hydraulic capacity, and ease of construction, while arch culverts are suitable for environmentally sensitive areas despite higher costs. Multiple pipe culverts offer simplicity but may compromise long-term performance and habitat connectivity. Ultimately, site-specific conditions will dictate the best choice, but box culverts generally present the most versatile solution for effective flood management.

5.2.Recommendation

Based on the findings of the study, several key recommendations can be made to enhance the resilience and functionality of the road infrastructure in the face of climate change:

Culvert Capacity Assessment and Redesign: Conduct a comprehensive assessment of the existing culvert's design capacity to determine its adequacy in handling projected flood flows. Based on the study’s findings, it is evident that the culvert cannot safely manage floodwaters for significant return periods. A redesign or reconstruction of the culvert to increase its capacity is essential.

Implementation of Multi-Stage Drainage Solutions: Explore the installation of multi-stage drainage systems that can effectively manage varying levels of flood intensity. This may include additional culverts or spillways that can divert excess water away from critical infrastructure, reducing the risk of overtopping.

Regular Maintenance and Monitoring: Establish a routine maintenance program for existing drainage structures to ensure they remain clear of debris and sediment, which can impede flow. Regular monitoring of water levels and flow rates during rainfall events will help in making informed decisions regarding immediate interventions.

Enhancement of Erosion Control Measures: Implement erosion control measures downstream of the culvert, such as vegetation planting, riprap, or geotextiles, to stabilize the soil and mitigate the impact of high-velocity floodwaters. This will help protect the road infrastructure from further erosion and damage.

Community Awareness and Emergency Preparedness: Engage local communities in awareness programs regarding flood risks and emergency preparedness measures. Providing information on safe evacuation routes and flood response strategies can help minimize risks during extreme weather events.

Investment in Sustainable Infrastructure: Advocate for investment in sustainable and resilient infrastructure that incorporates climate adaptation strategies. This may include green infrastructure solutions, such as permeable pavements and rain gardens, that can help manage storm water effectively.

Collaboration with Relevant Authorities: Foster collaboration among government agencies, local authorities, and community stakeholders to ensure a unified approach to flood management and infrastructure resilience. Joint planning efforts can enhance resource allocation and implementation of best practices.

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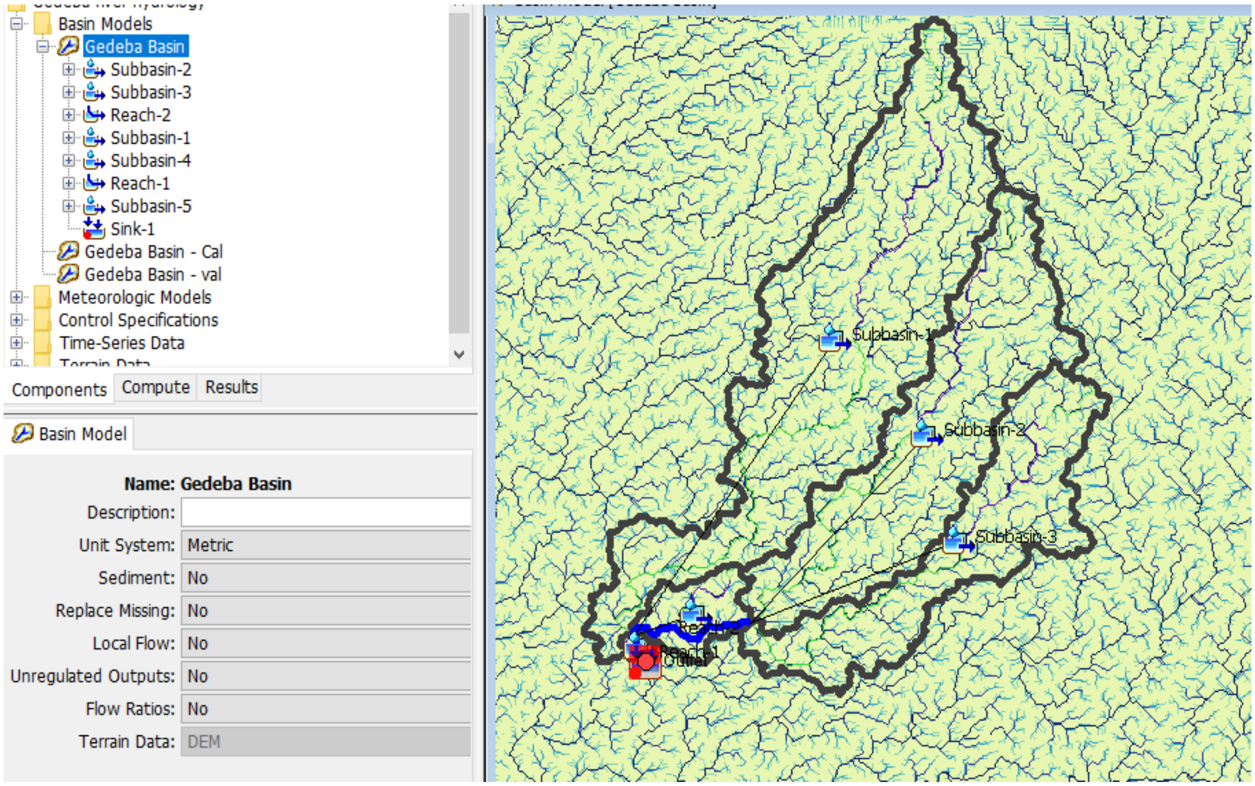
7. APPENDIX

Appendix A: Annual maximum 24hr rainfall (mm) Halaba meteorological center

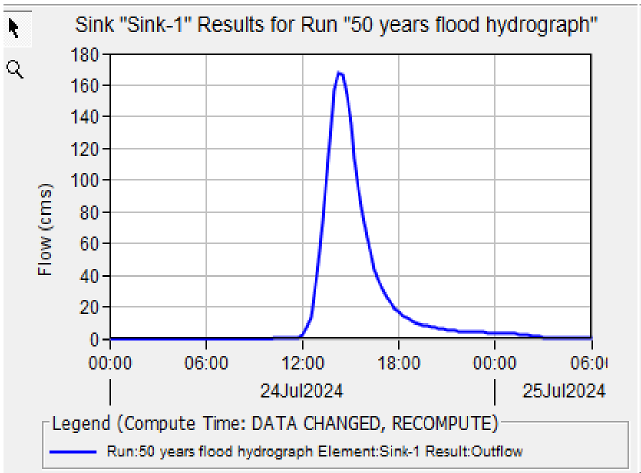
No.	***Year***	***Maximum rainfall***	***Descending Order***	***Rank***	***Logarithmic Value(Yo)***	**(Yo-Yo)^2**	**(Yo-Yo)^3**
1	1991	56.40	75.40	1	1.8774	0.0204044	0.0029146
2	1992	74.70	75.00	2	1.8751	0.0197498	0.0027755
3	1993	71.80	74.70	3	1.8733	0.0192636	0.0026737
4	1994	37.90	73.70	4	1.8675	0.0176731	0.0023495
5	1995	52.40	71.80	5	1.8561	0.0147859	0.0017979
6	1996	51.30	68.00	6	1.8325	0.0096004	0.0009407
7	1997	75.40	59.00	7	1.7709	0.0013195	0.0000479
8	1998	48.20	59.00	7	1.7709	0.0013195	0.0000479
9	1999	59.00	57.00	9	1.7559	0.0004557	0.0000097
10	2000	45.40	56.60	10	1.7528	0.0003345	0.0000061
11	2001	57.00	56.40	11	1.7513	0.0002806	0.0000047
12	2002	48.40	55.70	12	1.7459	0.0001283	0.0000015
13	2003	55.40	55.70	12	1.7459	0.0001283	0.0000015
14	2004	55.00	55.40	14	1.7435	0.0000807	0.0000007
15	2005	55.70	55.30	15	1.7427	0.0000672	0.0000006
16	2006	37.80	55.00	16	1.7404	0.0000341	0.0000002
17	2007	54.00	54.00	18	1.7324	0.0000046	0.0000000
18	2008	73.70	53.80	19	1.7308	0.0000140	-0.0000001
19	2009	41.40	52.40	20	1.7193	0.0002309	-0.0000035
20	2010	55.70	51.40	21	1.7110	0.0005553	-0.0000131

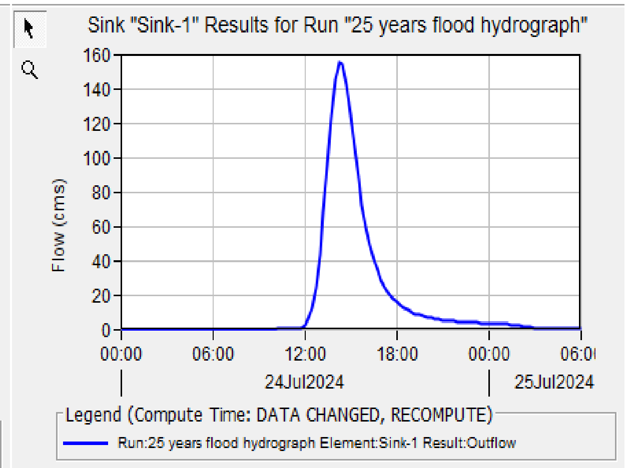
21	2011	53.80	51.30	22	1.7101	0.0005958	-0.0000145
22	2012	34.80	48.40	23	1.6848	0.0024683	-0.0001226
23	2013	51.40	48.20	24	1.6830	0.0026502	-0.0001364
24	2014	86.00	45.40	25	1.6571	0.0060018	-0.0004650
25	2015	40.30	41.40	26	1.6170	0.0138126	-0.0016234
26	2016	55.30	40.30	27	1.6053	0.0166984	-0.0021578
27	2017	38.50	38.50	28	1.5855	0.0222209	-0.0033124
28	2018	59.00	37.90	29	1.5786	0.0243011	-0.0037883
29	2019	56.60	37.80	30	1.5775	0.0246602	-0.0038725
30	2020	75.00	34.80	31	1.5416	0.0372290	-0.0071833
Mean			55.65		1.7345	0.0096	0.0000
Standard deviation			12.76		0.0995		
Skewness coefficient			0.478		-0.0406		

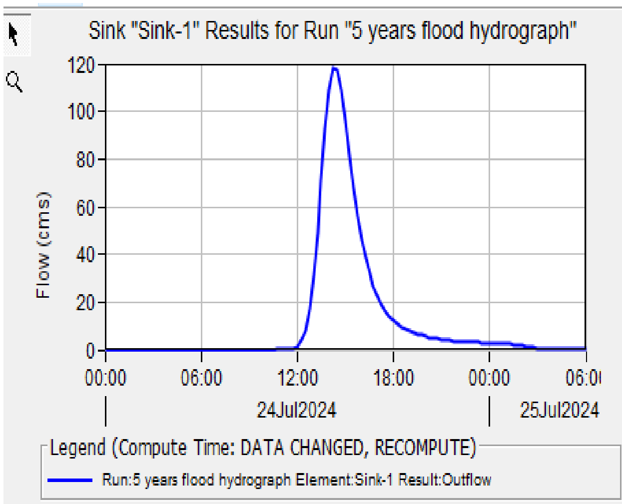
Appendix B: Gedeba basin map in HEC-HMS model

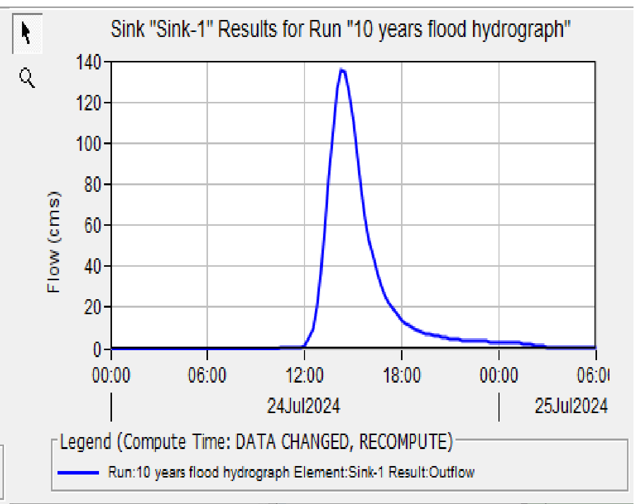


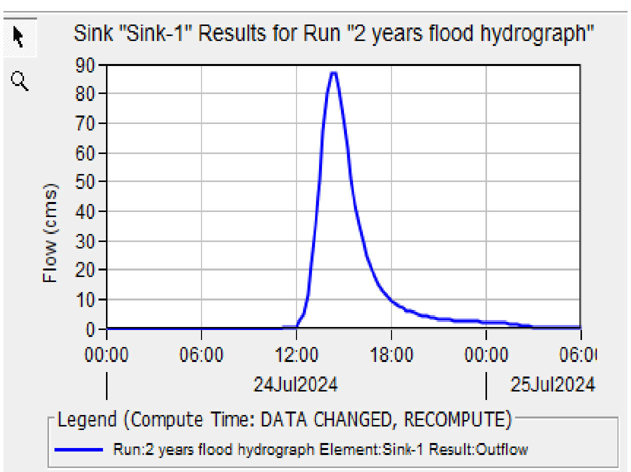
Appendix C: Flow hydrographs for different return periods in HEC-HMS



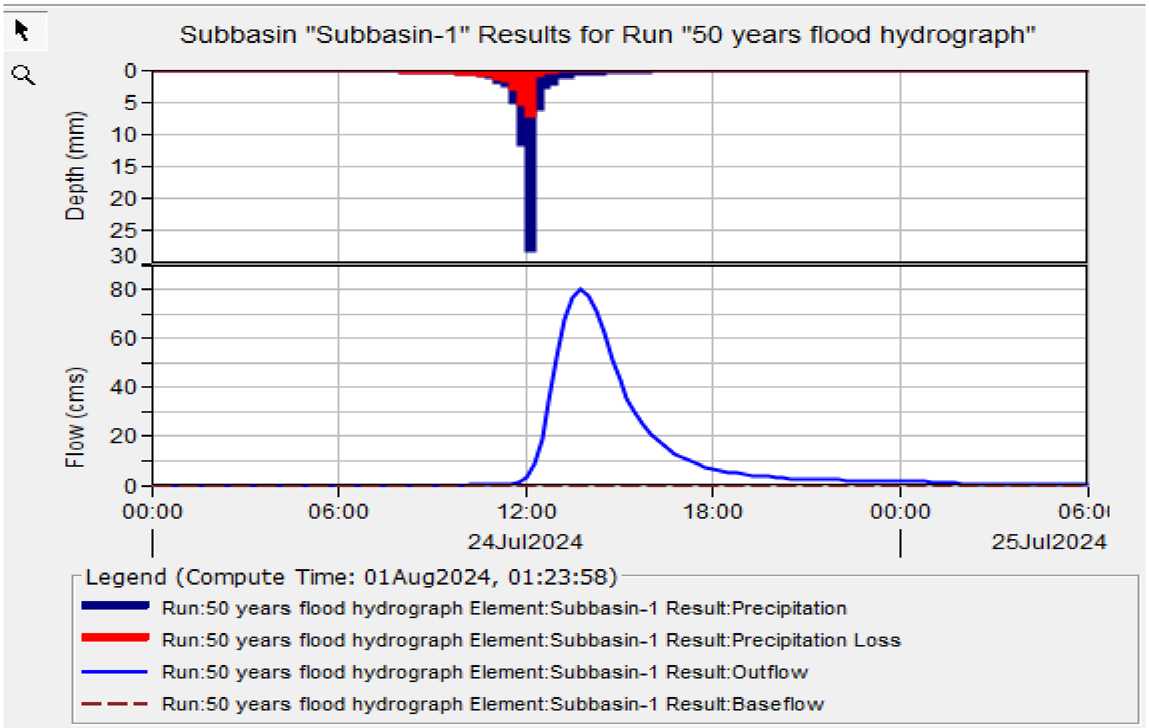


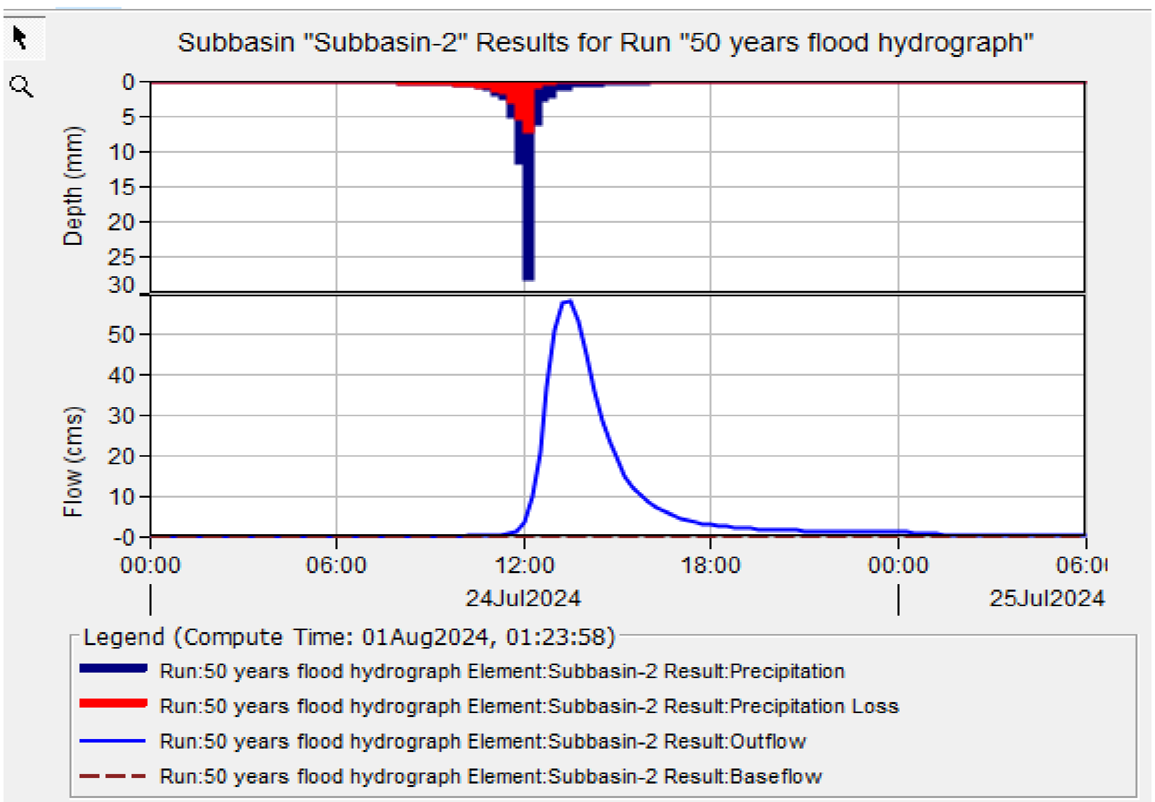




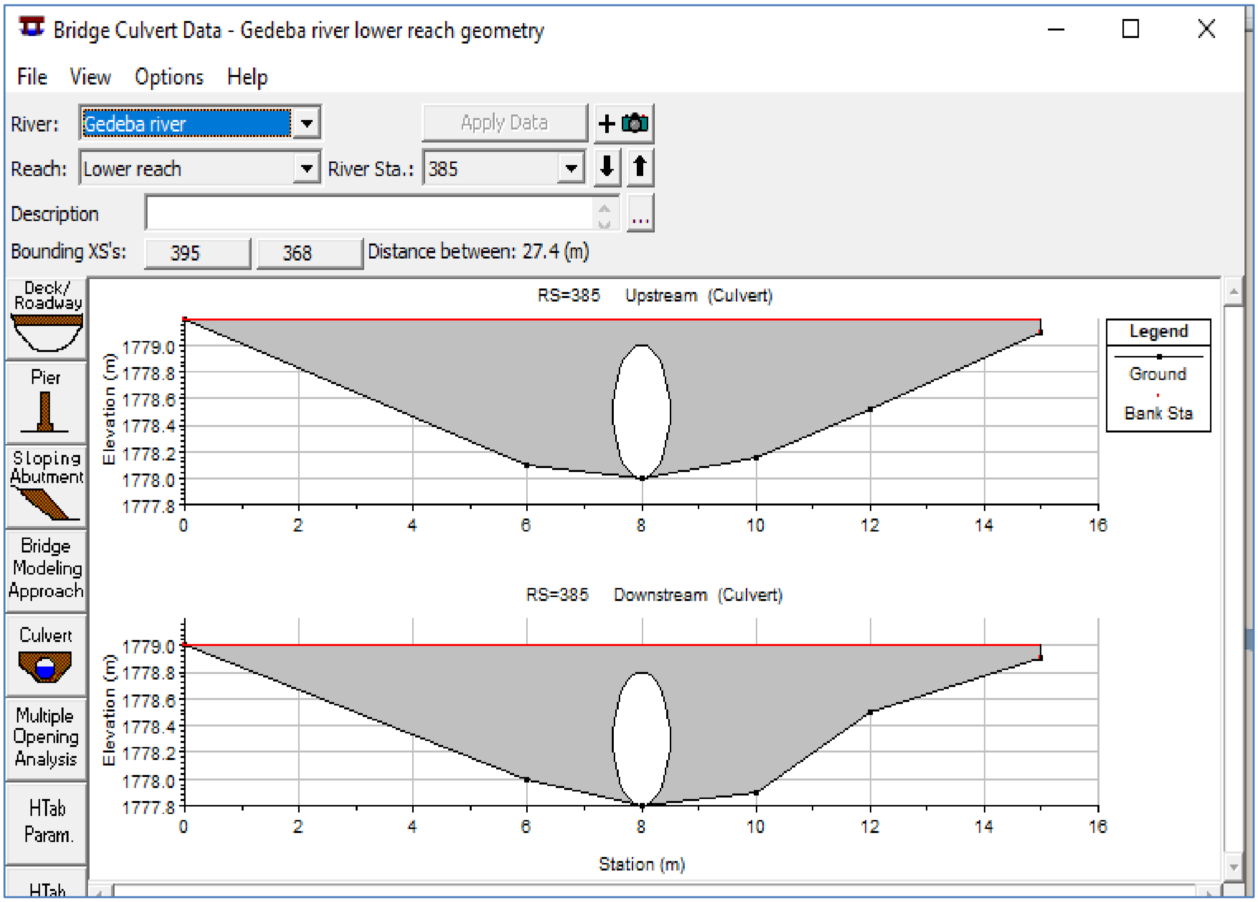


Appendix D: Sub basins contributions for flood hydrographs for 50 years flood

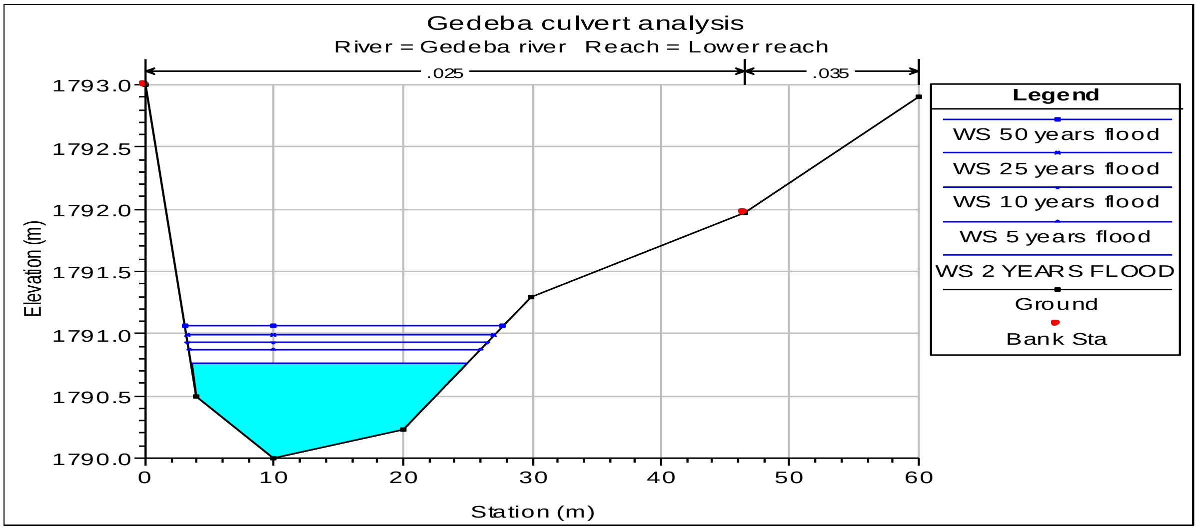


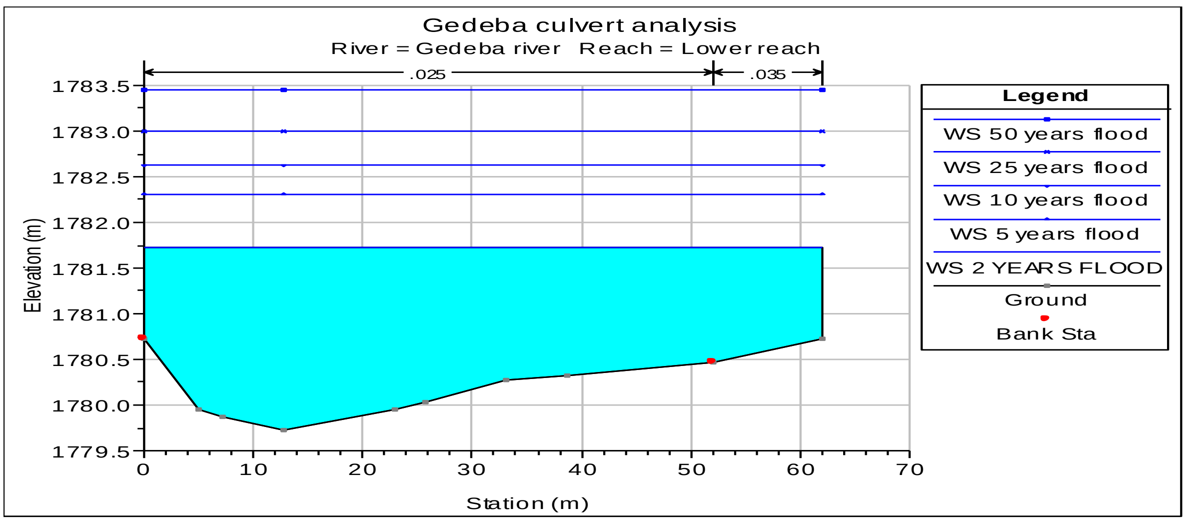


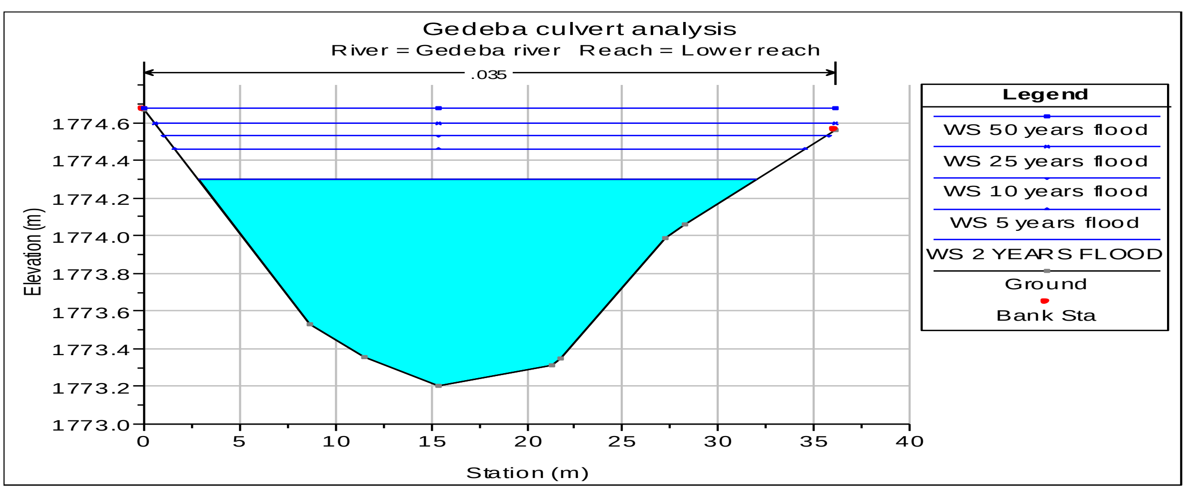
Appendix E: Culvert profile in Geometric data Editor



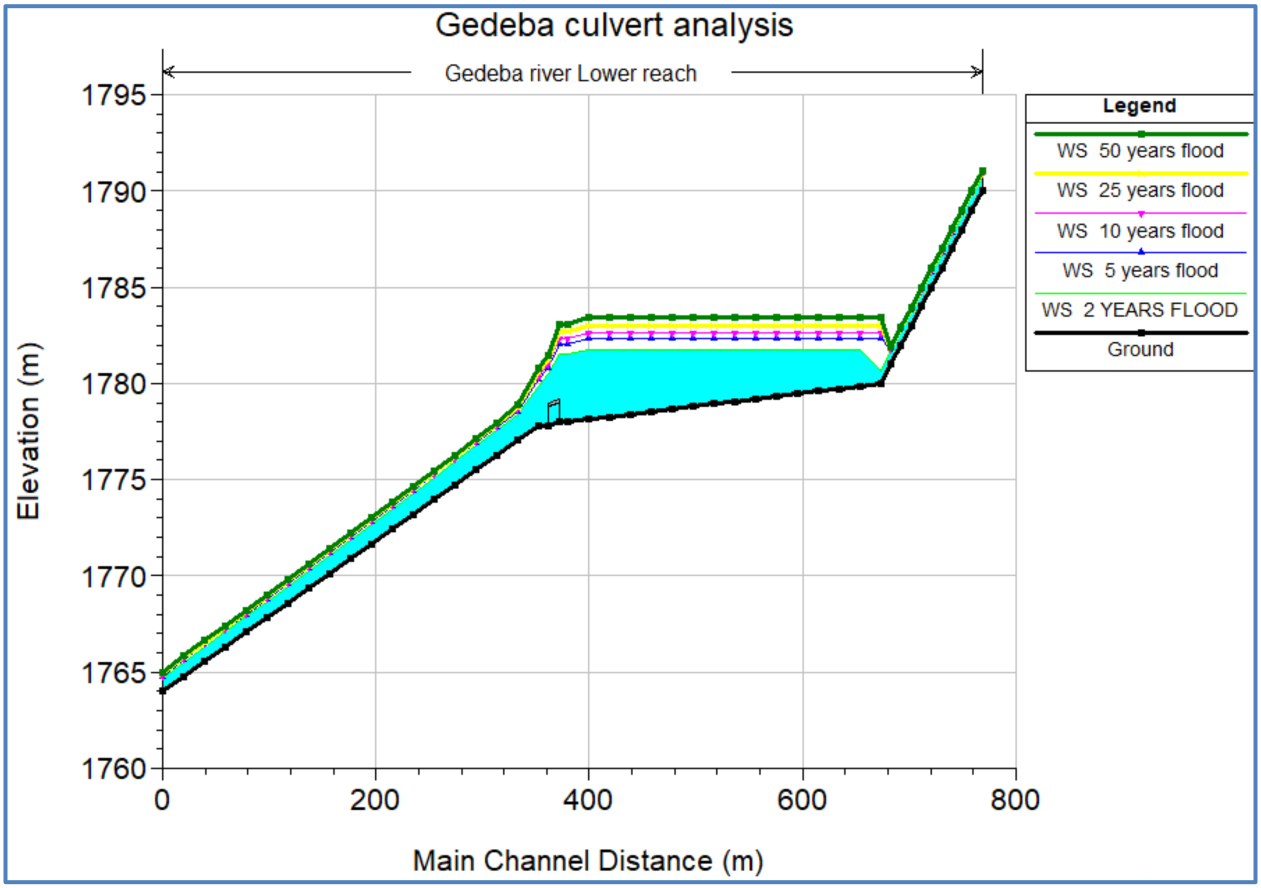
Appendix F: Water surface profiles at upstream, middle and downstream cross-sections



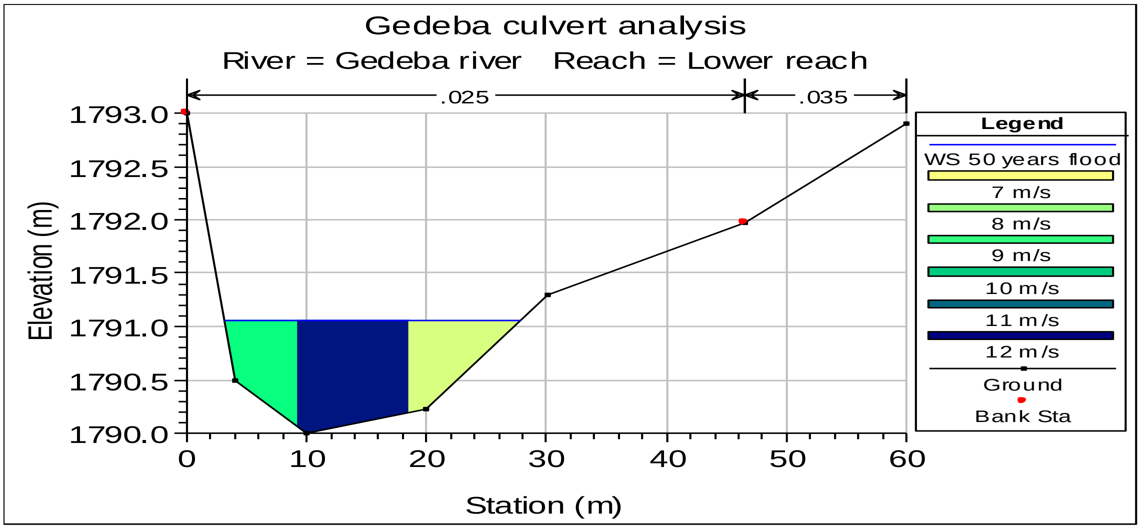


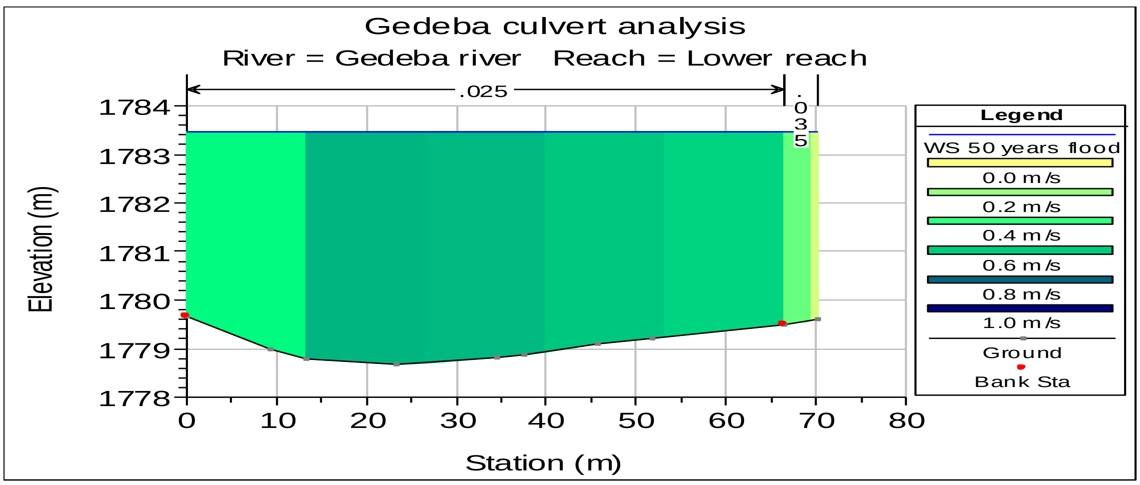


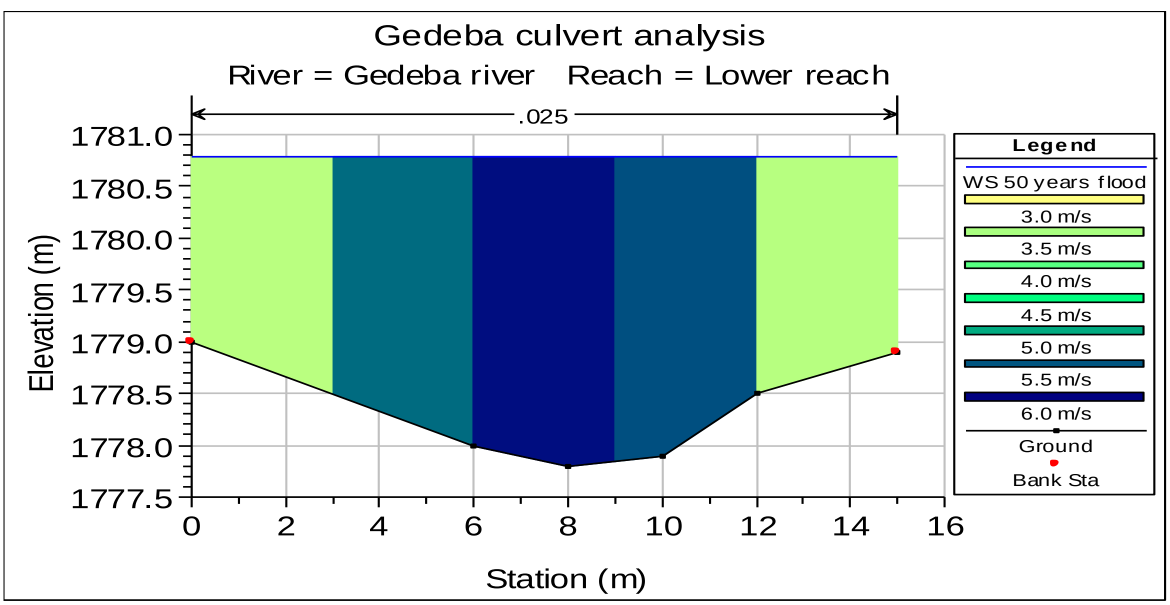
Appendix G: Water surface profiles along main channel in 2D view



Appendix H: Velocity profiles at upstream, middle and downstream cross-sections







Appendix I: Detailed output at cross sections

Cross Section Output

File Type Options Help

River: Gedeba river Profile: 50 years flood

Reach Lower reach RS: 783 Plan: 2 years flood hydrograph

Plan: 2 years flood hydrograph Gedeba river Lower reach RS: 783 Profile: 50 years flood

E.G. Elev (m)	1796.28
Vel Head (m)	5.22
W.S. Elev (m)	1791.06
Crit W.S. (m)	1792.07
E.G. Slope (m/m)	0.100137
Q Total (m3/s)	179.80
Top Width (m)	24.65
Vel Total (m/s)	10.12
Max Chl Dpth (m)	1.06
Conv. Total (m3/s)	568.2
Length Wtd. (m)	9.55
Min Ch El (m)	1790.00
Alpha	1.00
Frctn Loss (m)	0.98
C & E Loss (m)	0.00

Element	Left OB	Channel	Right OB
Wt. n-Val.		0.025	
Reach Len. (m)	9.54	9.55	9.54
Flow Area (m2)		17.77	
Area (m2)		17.77	
Flow (m3/s)		179.80	
Top Width (m)		24.65	
Avg. Vel. (m/s)		10.12	
Hydr. Depth (m)		0.72	
Conv. (m3/s)		568.2	
Wetted Per. (m)		24.88	
Shear (N/m2)		701.58	
Stream Power (N/m s)		7097.09	
Cum Volume (1000 m3)		88.07	5.60
Cum SA (1000 m2)		34.69	1.76

Appendix J: Summary output tables by profile

Profile Output Table - Standard Table 1

File Options Std. Tables Locations Help

HEC-RAS Plan: 2 years flood hydrograph River: Gedeba river Reach: Lower reach Profile: 50 years flood

Reload Dat

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(m3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m2)	(m)	
Lower reach	783	50 years flood	179.80	1790.00	1791.06	1792.07	1796.28	0.100137	10.12	17.77	24.65	3.80
Lower reach	773.50*	50 years flood	179.80	1789.00	1790.04	1791.03	1795.30	0.104149	10.15	17.71	25.17	3.87
Lower reach	764.00*	50 years flood	179.80	1788.00	1789.02	1789.98	1794.28	0.107598	10.16	17.70	25.79	3.91
Lower reach	754.50*	50 years flood	179.80	1787.00	1788.01	1788.92	1793.23	0.110301	10.12	17.76	26.52	3.95
Lower reach	745.00*	50 years flood	179.80	1786.00	1786.99	1787.87	1792.14	0.112816	10.05	17.89	27.49	3.98
Lower reach	735.50*	50 years flood	179.80	1785.00	1785.98	1786.83	1790.99	0.119814	9.91	18.14	29.78	4.06
Lower reach	726.00*	50 years flood	179.80	1784.00	1784.97	1785.78	1789.73	0.128190	9.67	18.60	33.38	4.14
Lower reach	716.50*	50 years flood	179.80	1783.00	1783.95	1784.73	1788.37	0.134961	9.31	19.31	38.15	4.18
Lower reach	707.00*	50 years flood	179.80	1782.00	1782.93	1783.65	1786.93	0.141069	8.85	20.31	44.79	4.20
Lower reach	697.50*	50 years flood	179.80	1781.00	1781.90	1782.57	1785.54	0.128481	8.44	21.37	49.39	4.01
Lower reach	688	50 years flood	179.80	1780.00	1783.45	1781.50	1783.50	0.000181	1.09	177.10	60.00	0.20
Lower reach	668.47*	50 years flood	179.80	1779.87	1783.45		1783.50	0.000149	1.02	189.03	61.03	0.18
Lower reach	648.93*	50 years flood	179.80	1779.73	1783.45		1783.49	0.000123	0.95	201.25	62.06	0.17
Lower reach	629.40*	50 years flood	179.80	1779.60	1783.45		1783.49	0.000102	0.89	213.72	63.08	0.15
Lower reach	609.87*	50 years flood	179.80	1779.47	1783.45		1783.49	0.000086	0.84	226.46	64.11	0.14
Lower reach	590.33*	50 years flood	179.80	1779.33	1783.45		1783.48	0.000072	0.79	239.47	65.14	0.13
Lower reach	570.80*	50 years flood	179.80	1779.20	1783.45		1783.48	0.000061	0.74	252.75	66.17	0.12
Lower reach	551.27*	50 years flood	179.80	1779.07	1783.46		1783.48	0.000052	0.70	266.30	67.19	0.11
Lower reach	531.73*	50 years flood	179.80	1778.93	1783.46		1783.48	0.000045	0.67	280.15	68.22	0.10
Lower reach	512.20*	50 years flood	179.80	1778.80	1783.46		1783.48	0.000039	0.63	294.24	69.25	0.10
Lower reach	492.67*	50 years flood	179.80	1778.67	1783.46		1783.48	0.000033	0.60	308.62	70.28	0.09
Lower reach	473.13*	50 years flood	179.80	1778.53	1783.46		1783.47	0.000029	0.57	323.28	71.31	0.09
Lower reach	453.60*	50 years flood	179.80	1778.40	1783.46		1783.47	0.000025	0.54	338.19	72.34	0.08
Lower reach	434.07*	50 years flood	179.80	1778.27	1783.46		1783.47	0.000022	0.52	353.40	73.36	0.07
Lower reach	414.53*	50 years flood	179.80	1778.13	1783.46		1783.47	0.000019	0.49	368.90	74.39	0.07
Lower reach	395	50 years flood	179.80	1778.00	1783.09	1780.94	1783.44	0.000979	2.60	69.13	15.00	0.39
Lower reach	385		Culvert									
Lower reach	368	50 years flood	179.80	1777.80	1780.79	1780.79	1782.00	0.006112	4.88	36.84	15.00	0.99
Lower reach	348.44*	50 years flood	179.80	1777.03	1778.91	1779.71	1781.60	0.048697	7.25	24.78	18.53	2.00
Lower reach	328.89*	50 years flood	179.80	1776.27	1777.96	1778.71	1780.55	0.055524	7.13	25.22	22.05	2.13
Lower reach	309.33*	50 years flood	179.80	1775.50	1777.11	1777.78	1779.39	0.053598	6.69	26.86	25.58	2.09
Lower reach	289.78*	50 years flood	179.80	1774.73	1776.29	1776.88	1778.29	0.050470	6.27	28.68	29.11	2.02
Lower reach	270.22*	50 years flood	179.80	1773.97	1775.48	1776.01	1777.26	0.048064	5.92	30.37	32.64	1.96
Lower reach	250.67*	50 years flood	179.80	1773.20	1774.67	1775.16	1776.29	0.046328	5.63	31.92	36.16	1.91
Lower reach	231.11*	50 years flood	179.80	1772.43	1773.87	1774.32	1775.36	0.044776	5.41	33.25	39.12	1.87
Lower reach	211.56*	50 years flood	179.80	1771.67	1773.07	1773.50	1774.47	0.043407	5.25	34.26	41.26	1.84
Lower reach	192.00*	50 years flood	179.80	1770.90	1772.26	1772.68	1773.62	0.042627	5.16	34.83	42.42	1.82
Lower reach	172.44*	50 years flood	179.80	1770.13	1771.45	1771.87	1772.78	0.042351	5.11	35.22	43.41	1.81
Lower reach	152.89*	50 years flood	179.80	1769.37	1770.64	1771.05	1771.94	0.042108	5.05	35.58	44.36	1.80
Lower reach	133.33*	50 years flood	179.80	1768.60	1769.84	1770.24	1771.12	0.041925	5.01	35.91	45.25	1.79
Lower reach	113.78*	50 years flood	179.80	1767.83	1769.03	1769.44	1770.29	0.041759	4.96	36.22	46.09	1.79
Lower reach	94.22*	50 years flood	179.80	1767.07	1768.23	1768.63	1769.47	0.041614	4.93	36.51	46.91	1.78
Lower reach	74.67*	50 years flood	179.80	1766.30	1767.43	1767.83	1768.65	0.041523	4.89	36.77	47.67	1.78
Lower reach	55.11*	50 years flood	179.80	1765.53	1766.63	1767.02	1767.83	0.041280	4.85	37.08	48.48	1.77
Lower reach	35.56*	50 years flood	179.80	1764.77	1765.84	1766.22	1767.02	0.041348	4.82	37.31	49.31	1.77
Lower reach	16	50 years flood	179.80	1764.00	1764.98	1765.42	1766.36	0.026811	5.19	34.64	49.02	1.97

Appendix K: Field photo taken during the study