Improving the design of road hydraulic structures for water harvesting:

The case of Freweyni – Hawzien – Abraha – We – Atsbeha road, Tigray, Ethiopia

A thesis submitted in partial fulfillment of the requirements for the degree of Masters of Science in Civil Engineering

(Hydraulic Engineering stream)

By

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DECLARATION

I certify that this thesis submitted for the degree of Master of Science in Civil Engineering (Hydraulic Engineering stream) is the result of my own research, except where otherwise acknowledged, and that this thesis (or any part of the same) has not been submitted for a higher degree to any other university or institute.

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Abstract

For safe disposal of water, roads are provided with hydraulic structures to safely convey the water from catchments. Road hydraulic structures are, however, designed without considering the effect of concentrated water downstream of the cross-drainage structures. This study explores effect of design improvements of road hydraulic structures for water harvesting. The study was conducted with a case study in the Freweyni-Hawzien-Abraha-We-Atsibha road network. Road design improvement scenarios were also developed for different channel characteristics depending on the location of the road drainage structures. In this study, HEC-HMS was used to simulate rainfall-runoff processes for the sub-catchments of road drainage structures. Floodplain analysis was done using ArcGIS, HEC-GeoRAS and HEC-RAS. HEC-HMS and HEC-RAS were used also to assess the effect of the design scenarios at selected sites in the case study areas. Runoff was estimated either using the Rational or the SCS curve number method depending on the size of catchment areas. Runoff for the side channels was, moreover, routed using the Muskingum method. Rainfall-runoff simulation was conducted using 24 hour rainstorm and stream flow covering two years (2014 and 2015) of data. Out of these, 10 events are selected for model calibration and the remaining 5 for model validation. Nash Sutcliffe (NS) and Relative volume error (RVE) are used for performance evaluation of the model calibration results; in which the values were between 0.48 and 0.70 for NS and 0.52 and 0.79 for RVE. The modeling results showed an increase in peak runoff but decrease in flood risk and flood inundation extents for the various scenarios which results an average 10% increase in runoff volume which results a safe flood convey and an average of 25% increase in water harvesting. Culvert site with the highest water harvesting potential was culvert site 8 with 263,650.6 m^3 and culvert site 4 has the smallest water harvesting potential with 13,630.46 m^3annually, showing applying new ideology like roads for water and artificial ground water recharges are nowadays food warranty to enhance sustainable development.

Key words: Runoff, Water harvesting, sediment, HEC-HMS, HEC-RAS, Road drainage

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Abbreviations

CN	Curve Number
DEM	Digital Elevation Model
EA	Actual evapotranspiration in the HBV model
EMA	Ethiopia Meteorological Agency
EMWR	Ethiopian Ministry of Water Resource
EP	Potential evapotranspiration
ERA	Ethiopian Road Authority
FAO	Food and Agriculture Organization
FC	Field Capacity
GIS	Geographic Information System
GPS	Global Positioning System
Ha	Hectare
HSG	Hydrologic Soil Groups
IDF	Intensity-Duration-Frequency
M.A.S.L	Meters above sea level
MCM	Million cubic meters
MFL	Maximum flow length
MUSLE	Modified Universal Soil Loss Equation
NS	Nash Sutcliffe coefficient
RVE	Relative Volume error
RWH	Rain Water Harvesting
SCS	Soil Conservation Service
SPSS	Statistical Package for the Social Sciences
USLE	Universal Soil Loss Equation
UTM Universal	Transverse Mercator Coordinate System

List of Symbols

e_a	Actual vapour pressure
e_s	Saturation vapour pressure
A	Catchment area
C	Runoff Coefficient
Cf	Frequency Factor
g	Acceleration due to gravity
I	Runoff intensity
Pcorr	General Precipitation correction factor.
Q	Rate of Runoff, Discharge
R	Roughness coefficient
R2	Coefficient of determination
rfcf	Rainfall correction factor,
Rn	Net radiation at the crop surface
sfcf	Snow fall correction factor
Tc	Time of concentration

CHAPTER ONE

1.0 INTRODUCTION

1.1. Background of the study

Water is the most essential compound and source of all life on earth, the essential part of all living things in the world, shapes and keeps the hydrologic cycle for assuring human existence and continuation of generation to generation (Chow et al., 1988).

In Tigray, the northernmost state of Ethiopia, the rainy season lasts little more than two months. The drought-prone area specifically in Senkata, Hawzen and Wukro was affected by shortage of food for many years and there was less developed water harvesting system, so the local farmers have faced failure of crop production due to shortage of rainfall and occurrence of erratic rainfall. But today, methods of water harvesting technologies and sustainable water management systems are changing the region into safe and guaranteed food availability.

In the past, the highlands of northern Ethiopia have always belonged to the most drought-prone areas of the country. The irrigation system employed in this region is with poor water harvesting technique because they don’t have capacity to build large and permanent diversion structures to harvest the flowing water generated from the watershed during dry season for irrigation purpose. The vulnerability towards drought and famine caused in these regions is because of the rain lasts only two to three months, strong population growth, massive deforestation and overgrazing leading to land degradation. Like many other villages of Tigray, Abraha - We - Atsbeha, Hawzien and Freweyni was continually dependent on food aid.

The increasing demand for water has increased awareness towards the use of artificial recharge to augment groundwater supplies. It is essential to understand the hydrological response of the catchment in order to know water resource potential and suggest better land and water management practices. Therefore, understanding the hydrological processes of different parts of a watershed is crucial to make decisions on water and land resources management through artificial recharges (Sivapalan, 2008). It refers to the movement of water through man-made systems from the surface of the earth to underground water bearing strata where it may be stored for future use. Artificial recharge in this thesis is a way to store water underground from road hydraulic structures in times of water surplus to meet demand in times of shortage.

It is known that road hydraulic structures like bridges, road side channels and culverts are constructed primarily for passage of water from upstream to downstream. Road drainage structures can also be used for harvesting concentrated for different agricultural uses. It is, however, necessary to adapt the design of the drainage structures in such a way that the water is safely disposed and effectively harvested downstream. To effectively use road drainage structures for water harvesting, the U/S of the drainage structures can be adjusted in order to minimize erosion in the drainage structures, conveyance systems and sedimentation in the water harvesting structures. Therefore, hydraulic and hydrologic models can be used to simulate the effect of different drainage design options on drainage structures as well as water harvesting structures. There is still a knowledge gap on the design of road drainage structures for efficient harvest of water from road catchments. Optimizing the design of road drainage structures can enhance efficiency of water harvesting and decrease erosion and sedimentation in road drainage structures, water channels and water harvesting structure

1.2. Statement of the problem

Road construction has become a common practice worldwide. This is primarily because of the increase of the transportation systems and traffic loads following the urbanization and expansion of networking among cities (ERA, 2002). In order to minimize failure and damage to the road structure, drainage structures are provided in road networks and help to collect and dispose concentrated water safely. The general objective of a drainage system is to protect the road system from being damaged by runoff. Despite road drainage structures help to reduce the effect of water on road sections, little attention is given to damages caused by concentrated water downstream of the drainage structure. These damages could be erosion and gully formation, flooding of land, sediment deposition on farmlands. It is, however, possible that the concentrated water from road drainage structures can be harvested near or downstream of the drainage structures. Water could be harvested in different water harvesting structures such as deep trenches, check dams, percolation ponds, etc. The damage from concentrated water downstream of the drainage structures such as culverts could be decreased by reducing the energy of water. This could be done by either reducing the conveyed volume through for instance spreading mechanisms or reduce the flow velocity (Temmink, 2016). As the water cause a serious impact on both the road access and its strength, an efficient drainage system is the most important part of rural road construction and maintenance works (Rono, 2014).

Despite several studies on water harvesting from road catchments (e.g., Grum et al, 2014,), optimizing the location of culverts for water harvesting (Temmink, 2016) and optimizing intensified runoff from roads for supplemental (Teweldebirhan, 2014), there is still a practical knowledge gap how water from road catchments can be harvested without causing damages. Water could be effectively harvested from road catchments without causing downstream damage if proper design of road drainage structure and water channels is adopted. The objective of this study is to explore ways of improving design of road design structures and water channels so that water could be effectively harvested downstream for different uses. This study will be implemented in the Freweyni-Hawzien-Abreha-we-Atsibha road network as case study to develop and test different drainage design options and evaluate their benefits. The design options will be tuned in a way that the effect of peak discharges is attenuated to reduce erosion as well as reducing sedimentation in water channels and water harvesting structures. Moreover, possibilities of sediment reduction mechanisms will be explored.

1.3. Research questions

- ✓ How much concentrated flood can be generated at the outlets of bridges, culverts from road catchments?
- ✓ How should drainage structures be designed in such a way that water can be safely harvested downstream of drainage structures?

1.4. Research objectives

1.4.1. Overall (General) Objective of the study

- ✓ Improving the design of road hydraulic structures for effectively harvesting water from road catchments.

1.4.2. Specific objectives of the study

- ✓ To evaluate the performance of HEC-HMS for event-based hydrological modelling in road drainage structures in northern Ethiopia.
- ✓ To develop different design scenarios of road drainage structures for harvesting water, reducing erosion and sedimentation.
- ✓ To improve design of road hydraulic structures for effective water harvesting.

1.5. Scope of the study

This research was geographically limited to the Freweyni – Hawzien - Abrha -we - Atsbeha road network. The study generally addresses issues related to rural road surface drainage and the integration between drainage and road infrastructures with hydraulic structures in the road zone. The specific focus is exploring existing condition of road and drainage structures, their network condition, safety of road and drainage infrastructures, impacts of road and drainage infrastructures integration on road performance and associated flood prone areas in the study area. At last, the major focus is harvesting the concentrated water safely which is coming from the road hydraulic structures like bridges, culverts and ditches for different uses.

1.6. Thesis structure

The main issues addressed in this paper are: (a) hydrological modeling, (b) hydraulic modeling, and (c) assessment of road catchments for water harvesting. The thesis has been divided into five chapters. The first chapter begins by giving a brief overview of the general background. It will then go on to research objectives and problem statement. Chapter two presents a literature review on the history, art, and approaches of water harvesting technology from roads. It gives the descriptions of water harvesting from road hydraulic structures. The review of the water harvesting structures and impact of road runoff and its method of estimations are also presented at the end of this chapter. Chapter three describes the study area and data used for the hydrological modeling and the assessment of hydraulic modeling for road water harvesting. Chapter four discusses the methodology for building up of models, model parameters, calibration, validation, and HEC-RAS model construction. Chapter five presents the discussions followed by results of this study. The last chapter summarizes the research findings and recommendations for further study.

CHAPTER TWO

2.0 LITERTURE REVIEW

2.1 Introduction

If a roadway is constructed along an alignment that meets a river or stream, then a suitable crossing is required to provide continuity of the roadway across the stream. For ecological continuity, if a bridge or culvert crossing is provided, then the opening that accommodates the stream must have a width, slope, and surface treatment that provides for the free flow of the "ecological traffic" through the structure. The flow through the structure includes the passage of water, sediment, and natural debris, and in addition both the upstream and downstream movement of aquatic organisms. Road drainage structures that cross the rivers and valleys are vital components of the road network that contributes greatly to the national development and public daily life. Any damage or collapse of these structures can cause the risk of the lives of road users as well as create serious influence to the entire country economic development (Kassa, 2013). Many people, indeed many engineers, who are not familiar with the subject, imagine that constructing a bridge/culvert across a river is entirely a problem in structural engineering. They assume that the bridge/ culvert opening can be made so large that it will completely span the river at such a height that floodwater will never rise anywhere near the deck. But in reality, Economics often dictate the length of span and therefore how many piers have to be located in the river, the geography of the site or the nature of the crossing may impose some restriction on the maximum permissible elevation of the deck. Studying the bridge& culvert hydraulics is important because of; nobody can be allowed to build a new bridge/culvert in a river without first being able to prove by calculation or modeling that the resulting backwater will not cause flooding of land and property upstream. At locations where there is an existing bridge/culvert and significant flooding, an analysis may be required to determine how much of the flooding is caused by the bridge/culvert and how much by others.

The two main types of water flows that can be considered are the flows that usually crossing the area that could be diverted by the presence of the road, and the flows generated by the runoff of the rainwater falling on the carriageway and its surroundings. The basic design techniques in roadway drainage system should be developed for economic design of surface drainage structures including ditches, culverts and bridges (ERA, 2002).

A hydraulic investigation and analysis of both the upstream and downstream reaches of the watercourse is necessary to determine the best location, size, and elevation of the proposed crossroad structure, whether a culvert or a bridge.

2.2 Types of road drainage structures

2.2.1 Culverts

Culverts are shallow passages that are fitted under roads that allow water to pass beneath them. They can be made of either steel, plastic or concrete. A culvert helps move water under a road or driveway to a stream, lake or detention basin. The purpose of culverts is to safely convey water from one side of the road way to the other. The water may be from natural streams or run off surface water from the road structure or areas close to the road. A culvert must be durable and have sufficient hydraulic capacity to carry a predetermined quantity of water for a given time.

- **General considerations in design of culverts**

Culverts conveying cross drainage flow from outside should be located on the natural drainage path of the flow. When the natural drainage path of the flow is a wide overland flow area, the designer should evaluate the need for multiple culverts in order to prevent concentrated flow at a single location. The proposed cross culvert must be aligned with upstream and downstream channels. The designer must analyze the existing flow conditions of the areas located upstream and immediately downstream of proposed cross culverts. Land use conditions in upstream and downstream areas should be clearly document ted in the Drainage Report, including photo documentation of the areas, if possible. This documentation of the existing conditions on the adjacent drainage areas, prior to construction, could provide useful information for subsequent adjacent property owner inquiries.

2.2.2 Roadside and median channels

Roadside channels and median channels are part of the storm drain system and are commonly used with uncurbed roadway sections to convey runoff from the road pavement and from areas which drain toward the road. Due to right of way limitations, roadside channels cannot be used in most urban areas. These channels also provide temporary storage of storm water to prevent serious inundation problems during major storms.

2.2.3 Bridge drainages

A bridge is a structure built to span physical obstacles without closing the way underneath such as a body of water, valley, or road, for the purpose of providing passage over the obstacle. There are many different designs that each serve a particular purpose and apply to different situations. Designs of bridges vary depending on the function of the bridge, the nature of the terrain where the bridge is constructed and anchored, the material used to make it, and the funds available to build it. It is noted that the procedure for arriving at final design for a bridge crossing over a river is a complex one in which structural, geotechnical and hydraulic factors are adjusted iteratively to achieve a bridge configuration which is satisfactory functionally, economically and aesthetically. It is also noted that the extent of the investigation to determine the required structural, geotechnical and hydraulic factors depends mainly on the size of the proposed bridge works and the nature of the bridge site. In acknowledgment of these aspects, AACRA requires a bridge site investigation to be undertaken for each bridge which involves stages like topographic survey, waterway investigation and geotechnical investigation.

2.3 Impact of concentrated flood on road drainage structures

When a road hydraulic structure is placed in a river it forms a narrowing of the natural channel and obstacles to the flow. This results in a loss of energy as the flow contracts, passes through the bridges, culverts and re-expands back to the full channel width. To provide the additional head necessary to overcome the energy loss, the upstream water level increases above that which would be usually experienced without the bridge or culvert. This additional head is called the afflux, and its variation with distance upstream is called the backwater profile. If the constriction is very severe the flow is usually subcritical, with gradually varied flow upstream and downstream of the structure and rapidly varying flow at the bridge or culvert. The figure below shows how road hydraulic structures affect the river flow. Scour can be defined simply as the excavation and removal of material from the bed and banks of streams as a result of the erosive action of flowing water. The most common cause of bridge and culvert failures is from floods eroding bed material from around their foundations by expansion and constriction of the flow through the structure. The effect of scouring on road hydraulic structures cross-sectionally and longitudinally by expansion and constriction of the flow has significant variation in the foundation and abutments of the structures.

Scour in its widest sense may also include lateral erosion of the riverbanks in the vicinity of a bridge or culvert. This may result in the flow approaching the bridge at a skewed angle instead of perpendicularly, greatly increasing the potential for failure of the piers, abutments and highway embankments. Scour is the engineering term for the erosion of soil, alluvium or other materials surrounding bridge and culverts generally for road hydraulic structure foundations (piers and abutments) by flowing water.

2.3 Damage of concentrated runoff from road drainage structures

2.3.1 Precipitation

The transfer of water from the atmosphere to the land is called precipitation and is the most important part of hydrological cycle. Precipitation can be in form of rain, snow, hail and sleet. Precipitation in the form of rain is the driving force of the land phase of hydrologic cycle. It is characterized by both high spatial and temporal variability. Rainfall is random or probabilistic in nature. Part of the precipitation is intercepted by natural vegetation cover. The intercepted precipitation is either redistributed through runoff or evaporates back to the atmosphere. Precipitation also moves into the soil through the process of infiltration. Some of this infiltrated water percolates deep down into the ground to recharge the ground water reservoir. Several methods have been developed to estimate precipitation. Some of these methods include recording and non-recording gauges.

The recording gauges produce a continuous plot of rainfall against time and provide valuable data of intensity and duration of rainfall for hydrological analysis. These gauges automatically record the depth of rainfall in intervals ranging from as little as one minute in duration while non-recording gauges are read manually at longer time interval at 12:00 am and 12:00 pm. There are three types of recording gauges in general use. These are the weighing-bucket type, float type and the tipping bucket type. These gauges have been described in several hydrologic books (Subramanya, 1984; Maidment, 1993). The two types of non-recording gauges are the standard and the storage type. They are the most widely used rainfall data measuring devices in hydrology. Rainfall data need to be checked for continuity and consistency before being used. The continuity of the records may be broken with missing data due to reasons such as damage or fault in a rain gauge during a period.

The missing data is usually estimated using the data of the neighboring stations. In this calculation, the normal rainfall is used as a standard of comparison. The normal rainfall is the average value of rainfall at a particular date, month or year over a specified 30 year period. The methods applied are the simple arithmetic method and normal ratio method. The latter is used if the normal precipitation varies considerably. The spatial rainfall is usually used in hydrology for various applications. Several procedures have been developed to convert point rainfall data into spatial format. The methods are: Simple arithmetic method, Isoyetal and Thiessen polygon. The arithmetic method is usually applied when the rainfall measured for various stations show little variation. The average precipitation over the catchment is taken as the arithmetic mean. In Isohyetal method the catchment area is drawn to scale and the rain gauge stations are marked. The isohyets are drawn; these are lines joining points of equal rainfall magnitude. The area between two adjacent isohyets is determined and the average rainfall indicated by the two isohyets is assumed to be acting over the inter-isohyetal area. Isohyetal method is superior to the other two methods, when stations are large in number. For Thiessen Polygon method, the rainfall recorded at each station is given weightage on the basis of an area closest to the station. The catchment area is drawn to scale and the stations marked on it. Stations are joined to form a network of triangles and perpendicular bisectors for each of the sides of the triangles are drawn. These bisectors form a polygon around each station.

The area of each polygon is determined and used to calculate the average weighted rainfall. The formula is given as:

$$P = \frac{\sum_{i=1}^{n} PiAi}{Ai} \qquad (2.1)$$

Where P is the average rain fall over the catchment, P1… Pn are the rainfall magnitude recorded by each station, A1… An, are the polygon areas.

Thiessen polygon method is more superior to the arithmetic method because it applies some weightage. This weightage is given to the rainfall station on a rational basis. Due to this reason and the use of fixed area polygons it is preferred average rainfall and especially because it lends itself to computer analysis.

2.3.2 Infiltration

Infiltration is the passage of water through macro pores from the surface to the subsurface and determines the amount of runoff that causes erosion (Mao et al., 2008). It is the downward movement of water from the land surface into soil or porous rock (Maidment, 1993). Initial rate of infiltration depends on the moisture content of the soil prior to the introduction of water on the soil surface. The final rate of infiltration is equivalent to the saturated hydraulic conductivity of the soil. In order to calculate or determine the infiltration rate, a number of methods have been proposed. These include Green and Ampt infiltration method. It was developed to predict infiltration assuming excess water at the surface at all times (Chow et al., 1988). The equation assumes that the soil profile is homogenous and antecedent moisture is uniformly distributed in the profile. As water infiltrates into the soil, the model assumes the soil above the wetting front is completely saturated and there is a sharp break in moisture content at the wetting front. Green and Ampt infiltration method is given as:

$$f = K\left[1 + \frac{(\phi - \theta i)Sf}{F}\right] \tag{2.2}$$

Where f is the infiltration rate, K is the effective hydraulic conductivity, Sf is the effective suction in the wetting front, Φ is the soil porosity, θi is the water content and F is the accumulated infiltration. Amount of water entering the soil profile is calculated as the difference between the amount of rainfall and the amount of surface runoff.

2.3.3 Design of road drainage structures

Runoff is that portion of precipitation that does not evaporate or infiltrate. It makes its way towards stream channels, lakes and oceans as surface or subsurface flow. It is the essential factor in determining the hydrologic response change in a catchment that is affected by land use changes. Land use change is an important factor in the runoff process that affects infiltration, erosion and evapotranspiration. Due to rapid land development, land cover is subjected to changes causing soils to become impervious surfaces. This leads to decrease in the soil permeability, and consequently increase the amount and rate of runoff. It is possible to describe the catchment characteristics when determining runoff response to rainfall input. Several methods have been developed for estimating runoff from a given catchment.

One of these methods is the Soil Conservation Service (SCS) curve number method. The curve number model is stated as:

$$Q = \frac{(P - Ia)^2}{(P - Ia + S)} \tag{2.3}$$

Where Q is the runoff in mm, P is the rainfall in mm, Ia is the initial abstraction in mm and S is the potential maximum retention after the runoff begins in mm.

The retention parameter varies spatially due to changes in soils, land use, management and slope, and temporarily due to changes in soil water content. The retention parameter is defined as:

$$S = \frac{25400}{CN} - 254 \tag{2.4}$$

Where: CN is the curve number.

The initial abstraction Ia is all losses before runoff begins. It includes water retained on the surface depression, water intercepted by vegetation, evaporation and infiltration. Ia is highly variable; however it is commonly approximated as 0.2S. By substituting this approximate variable into equation 2.5, the equation reduces to:

$$\mathrm{Q} = \frac{(P - 0.2S)^2}{(P + 0.8S)}, \text{for P} > 0.2S \tag{2.5}$$

Runoff will only occur when the P > Ia.

Major factors that determine CN are hydrologic soil group, cover type, treatment, and antecedent soil condition. The hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. Runoff becomes stream flow when it is concentrated in a channel. It is possible to measure the amount of water in this phase of the cycle as it leaves the catchment (Linsley and Franzini, 1989). The stream flow data is an important indicator of biophysical changes in the catchment. For instance, the stream flow rate at a particular point in time and location on a drainage system, integrates all the hydrologic processes and storages upstream of that location. The rate of stream flow depends on several factors such as: rainfall events, the seasonal distribution, type and transpiration of the vegetation.

2.3.4 Water yield

Water yield is the total water outflow from a catchment during a given time. One way of determining effects of land cover change on water yield from the catchment is by use of paired catchment. The paired catchment studies have been widely used as a means of determining the magnitude of water yield change resulting from changes in vegetation cover (Stednick, 1995). These paired catchments studies involve the use of two catchments. The catchments must be similar hydrologically and adjacent to each other.

This characteristic might not be achieved as there are few catchments which are totally the same. In Malaysia a paired catchment study which involved three catchments was carried out by the forest conversion normally leads to increase in water yield from the catchment. Li et al. (2007) concluded that there is no significant impact on the water yield and river discharge when the deforestation percentage is below 50% or grazing percentage below 70% for savanna and 80% for grassland areas. However, it was observed that the water yield increases drastically when land cover change exceeds these thresholds.

2.3.5 Sediment yield

Sediment yield is the total sediment outflow from a catchment during a given time. Sources of sediment include soil erosion usually carried as suspended loads and material eroded from the stream channel. Many factors influence the sediment production in natural catchments. The major controlling factors for sediment yield are: climate, vegetation, catchment size, elevation and relief, rock and soil type, and human activities, all which in turn determine soil erosion rate and stream capacity. There have been several methods that were developed to estimate sediment produced from catchments. One among such methods which is inbuilt in Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) is the Modified Soil Loss Equation (MUSLE). The MUSLE equation was modified from Universal soil loss Equation (USLE) developed by Wischmeier and Smith (1978). USLE predicts average annual gross erosion as a function of rainfall energy. In MUSLE, the rainfall energy factor is replaced with a runoff factor. This improves the sediment yield prediction and eliminates the need for delivery ratios. The Modified Universal Soil Loss Equation is shown in equation 2.6.

✓ Modified Universal Soil Loss Equation (MUSLE)

The MUSLE was developed by applying statistical multivariate regression techniques to the large data bases collected by the USDA Agricultural Research Service. The erosion risk (S=annual sediment yield) is calculated from a number of factors that have been measured for all climates, soil types, topography and kinds of land. This technique helps to predict erosion. It also identifies erosion-sensitive areas. The factors are combined in a number of formulae of the Modified Universal Soil Loss Equation, which returns a single number, the tolerance factor, equivalent to predicted erosion in ton/ha: The equation below shows that sediment yield varies directly with variation in discharge implying that discharge estimates could give an indication of sediment yield.

$$S = 11.8 * (Q * q * A)^{0.56} * K * C * P * LS \qquad (2.6)$$

Where S is the sediment yield, Q is the surface runoff volume, q is the runoff rate, A is the area of the hydrological response, K is the USLE soil erodability factor, C is the USLE cover and management factor, P is the USLE support practice factor and LS is the USLE topographic factor. The above equation shows that sediment yield varies directly with variation in discharge implying that discharge estimates could give an indication of sediment yield. Erosion, sediment transport and deposition are major environmental issues that affect the environment through reduction of reservoir, siltation of rivers and streams, intensification of both water pollution and flood. Water resource management requires sediment yield information in order to make and implement sustainable catchment management policies. The increase in sediment yield from many catchments has resulted from changes in land use. The related increase in urban areas and road construction has increased the impervious surfaces hence reducing the infiltration capacity. This has resulted in high runoff which transports sediment from the catchment to the receiving water bodies.

2.4 Hydrologic and hydraulic models

Hydrologic modeling has proved to be a powerful tool that can be applied to understand and explain the effects of land use and land cover change on hydrologic response of a catchment. It allows generation of runoff data in order to make forecasts and calculate the probable maximum flood (PMF) (Fleischbein et al., 2006).

Hydrologic models are relatively complex mathematical description of the hydrologic cycle (Singh and Woolhiser, 2002). They describe the actual physical processes of the hydrologic cycle and represent the behavior of the catchment in transforming a hydrologic input (rainfall) into output (stream flow or runoff). Stream flow models are therefore mathematical expressions that simulate stream flow or runoff in a manner similar to the way a catchment would operate on the same rainfall event. However, in developing a hydrological model, assumptions are made in applying the physical laws and equations that govern the processes to simplify the larger and more complex hydrologic systems. Hydrologic models are broadly categorized into stochastic and deterministic models. The stochastic models are mathematical models of sequence of hydrologic variables governed by probability laws. They are generally used for time rainfall-runoff analysis and have outputs that are at least random (Chow et al., 1988). On the other hand, the deterministic models seek to simulate part of the hydrologic cycle at a point (Freeze, 1978). Furthermore, deterministic models have physical and conceptual parameters and can be classified as lumped, semi distributed and distributed. Lumped models treat the whole catchment or a portion of it as if it was homogeneous in character and that it is subject to uniform. These models do not consider the spatial variation of parameters and other hydrologic processes. However, lumped models are relatively simple and less complex in application (Fleischbein et al., 2006).

2.4.1 Hydrologic system modeling using HEC-HMS

The Hydrologic Modeling System is designed to simulate the precipitation-runoff processes of dendritic watershed systems. It is designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems. This includes watershed runoff. Hydrographs produced by the program are used directly or in conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation. The program is a generalized modeling system capable of representing many different watersheds. A model of the watershed is constructed by separating the hydrologic cycle into manageable pieces and constructing boundaries around the watershed of interest. In most cases, several model choices are available for representing each flux. Each mathematical model included in the program is suitable in different environments and under different conditions.

2.4.2 Hydraulic river system analysis using HEC-RAS

HEC-RAS is hydraulic modeling software developed by the U.S. Army Corps of Engineer´s Hydrologic Engineering Center. In this study, version 4.1.0 of HEC-RAS was used. The software is capable of performing one-dimensional (1-D) steady and unsteady-flow simulations and comprises a graphical user interface, separate hydraulic analysis components, data storage and management capabilities as well as graphics and reporting facilities. In inundation analysis, flow modeling is used to simulate the flow of a flood wave through a river reach and its floodplains. In one-dimensional flow routing, flow through the river channel and the floodplains is treated only in the longitudinal direction parallel to the conduit. Even though in reality, the flow in a natural channel is never truly 1-D, these flow models were found to deliver acceptable results for predicted hydraulic parameters in many applications. In the 1-D HEC-RAS flow model, the geometry of the channel and the floodplains is represented by a series of cross sections along the reach.

2.5 Event-based hydrological modeling

An event model simulates a single storm. The duration of the storm may range from a few hours to a few days. This distinction applies primarily to models of watershed-runoff processes. Event hydrological modeling reveals the how a basin responds to an individual rainfall event (e. g. quantity of surface runoff, peak, timing of peak, detention etc.). Fine-scale event hydrological modeling is particularly useful for understanding detailed hydrologic processes and identifying the relevant parameter that can be further used for coarse-scale continuous modeling, especially when long-term intensive monitoring data are not available or the data are incomplete.

Selection of rainfall-runoff events is a critical step for event hydrologic modeling and model calibration/validation. Selection depends on many factors, such as rainfall characteristics (magnitude, duration, intensity, temporal and spatial variability etc.), watershed properties (size, land use/covers, soil types etc.) and availability and completeness of rainfall and stream monitoring data. The following criteria were applied for selecting individual rainfall-runoff events suitable for the calibration and verification of the HEC-HMS model according to the recommendations given in USACE-HEC manual:

- Maximum spatio-temporal data density of the observed daily stream flow and rainfall records.
- Uniform rainfall distribution throughout the period of effective precipitation over the entire watershed.
- Rainfall-runoff events generated by the same rainfall event.
- Stream flow peaks representing all runoff due to the selected rainfall event.
- The duration of rainfall events exceeding the time of concentration of the basin.
- The magnitude of rainfall events selected for calibration approximately equal the magnitude of rainfall events the model is intended to analyze.

2.6 Hydrology of un-gauged catchments

Due to different reasons that can be appropriate to practical or cost-effective aspects, there are some catchments that are un-gauged. There should be a means so that the discharge of these catchments can be produced. Engineers and scientists have studied such cases and find the adequate means to the difficulty. Deriving a unit hydrograph for an un-gauged catchment requires a relation between the physical geometry of the area and the resulting hydrographs. Three approaches have been used formulas relating hydrograph features to obtain characteristics, transportation of unit hydrographs, and storage routing. Basin characteristics formulas usually pertain to time of peak, peak flow, and time base of the unit hydrograph. When these features are established, the hydrograph can be sketched to provide the necessary unit volume. There are different ways of estimating runoff from ungauged catchments: The first is area proportion method: if you have a gauged catchment with similar watershed characteristic then you can simply predict the flow from the ungauged catchment by area proportion. This is much uncomplicated but strength fully helpful. The second is using hydrological modeling: If there are few gauged river basins and if rainfall data is available, it is good calibrate those gauged catchments. After calibration, you can develop a regional model which relates model parameters with watershed characteristics. Using watershed characteristics and model parameters simulations of the gauged catchments, it is possible to determine the design discharge of the ungauged catchments according to the meteorological, hydrological, morphologic and geomorphologic aspects among the combination (Montanari et al., 2013).

Even if estimating the ungauged catchments is the most difficult task but, there is area transformation for designing essential projects while there is no time (Hrachowitz et al., 2013). If precipitation and soil types are comparable in the ungauged catchments as the larger catchment having the ungauged catchments, a simple method which is a ratio of the areas and uses that to figure out what portion of huge catchments discharge is the lesser ungauged catchment discharge. But this needs a similar meteorological data, similar soil characteristics, land use and no huge differences river surface formations between the catchments (Bárdossy, 2007). If the hydrological data's and methods between the ungauged and gauged catchments differ the area proximity is not valid for all hydrological phenomena's. However, for catchments with few or no discharge measurements of the ungauged catchments, the area transformation assumes that catchments with comparable individuality show a related hydrological characteristics and thus a transposition of model parameters from similar characteristics of basins should be done carefully In substitute, identifying a relative between the river discharge and the physical and hydrological characteristics of the corresponding drainage area of the ungauged basin is good to operate a proper translation. However, this is quite difficult because it needs to analyze deeply climate, land use, slope, and soil type in order to identify valid relationships and the results may be affected by a very large uncertainty. Therefore most scientists agreed that, similarity in between the basins must be greater than 75 % for good and certain result of area proximity.

CHAPTER THREE

3.0 DESCRIPTION OF THE STUDY AREA

3.1 Geographic location and boundary of the study areas

The project areas are located in northern part of Ethiopia, east Tigray regional state. Geographically the study areas are bounded administratively by N13038' and N13058' latitude and E38058' and E39025' longitudes. Hawzien E39025' 21" N13058'21" and Fireweyni E390 34'33" N140 03'11".

Geographical location of the study areas

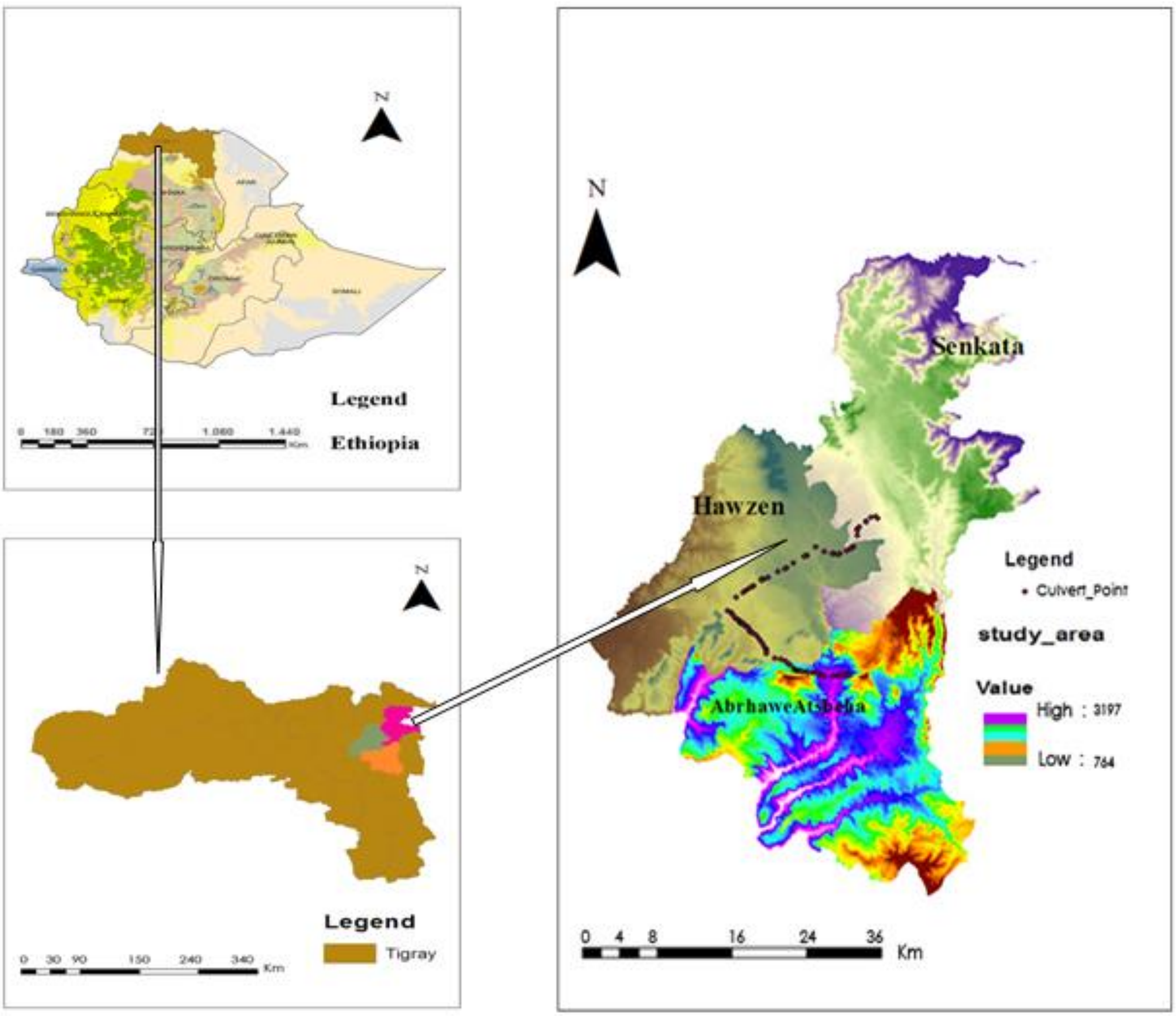

Figure 3.1: Location map of the study areas

3.2 Road span of the study areas

The road network which spans from Freweyni to Abreha-we-Atsbeha has a total length of 52 km. The road has different topographic characteristics and land forms

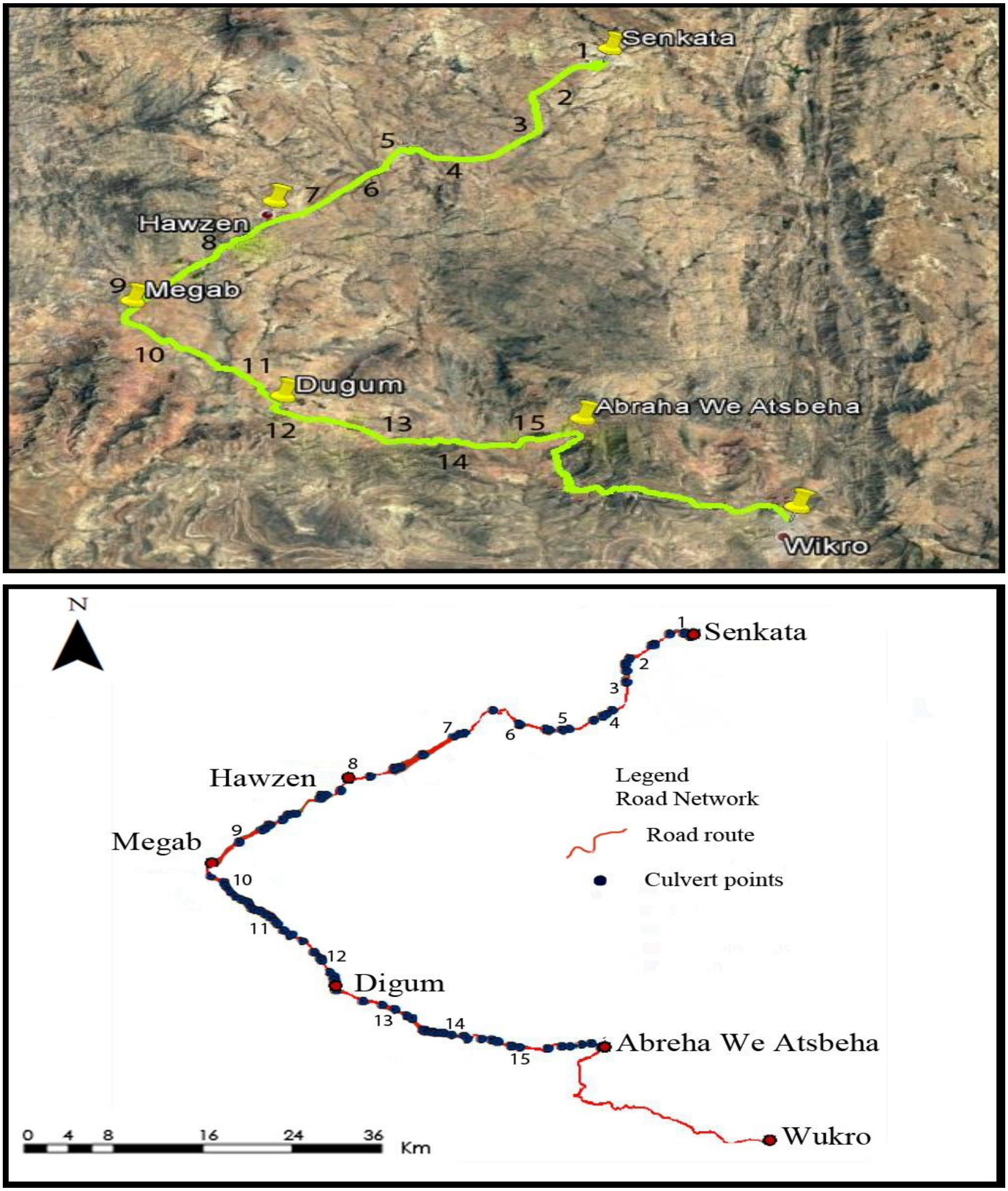

Figure 3.2: Road section of Frewyni-Hawzien-Abraha-we-Atsbeha

3.3 Study areas and land use land cover

As the watershed reports of the study areas shows, the catchment areas are covered by different land uses. The land use classification shows that the major land use types in these watershed areas are cultivated land, shrub land, wood land, and bare land areas occupied with low volume of natural forests and water bodies. The dominant land use type is cultivated land that covers 37.8% of the total areas. It has poor vegetation cover. The second largest land use type is shrub land, which covers 15.6%. The wood land has coverage of 14.6% of the total catchment area. The bare land has an area of 12.5%. The study areas and land uses representing 15 culvert sites are shown in the figure below:

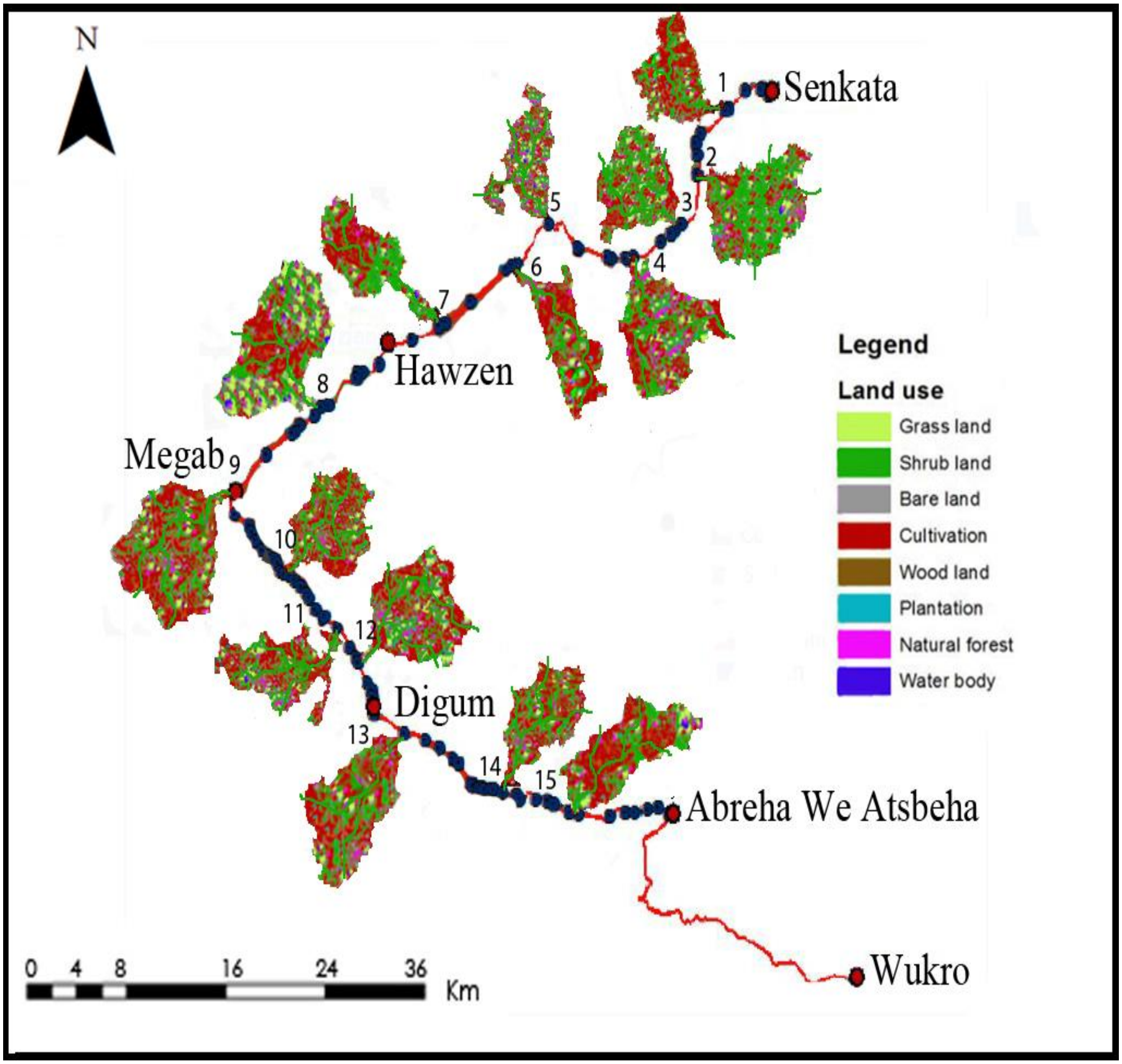

Figure 3.3: Study areas with Land uses of all culvert sites

3.4 Soil characteristics

The major soil textures found in this watershed areas from FAO soil map are silt clay, silt loam and sandy loam. From such type of soils, the dominant one is silt loam, which covers 46.5% of the total areas. Such soil type is commonly found in moderately steep and sloping cultivated land, shrub and wood lands. It has soil depth of ranging 0.25 m up to 0.5 m. In such soil type, where there is moderate vegetation cover, gully erosion is commonly occurred affecting the existing land resource. The second soil texture, which covers about 38.2%, is silt clay. This soil type is situated in cultivated land with soil depth range of 0.50 m up to 0.75 m. There is some gully and rill erosion in such area. The sandy loam soil texture that covers 15.3% is commonly found in steep and very steep terrain. It is found in grazing lands and area closures.

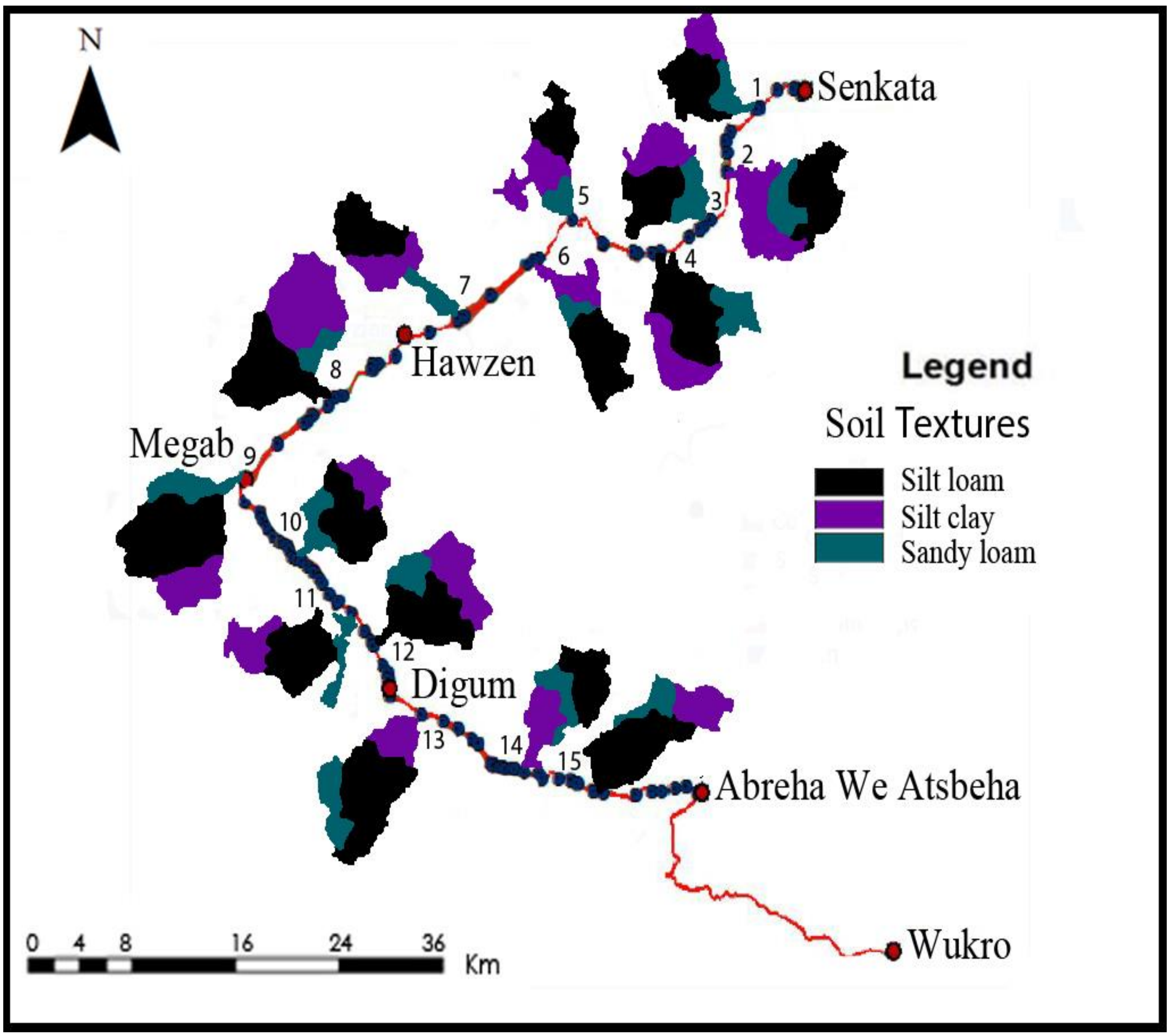

Figure 3.4: Soil map of the study areas

CHAPTER FOUR

4.0 MATERIALS AND METHODS

4.1 Data acquisition and processing

The data acquisition and processing required for the study included: Geophysical, hydrologic and hydraulic data. Geophysical data were: topographic and soil survey maps, and Google earth imagery. They were used to derive raster layers. The hydrologic data were precipitation and stream flow. Stream flow was acquired for calibration and validation of the model. Rainfall data was used as an input to simulate catchment hydrologic response from the meteorological data’s. The hydraulic data were length, width and sediment yield of the river bank.

4.2 Meteorological data

4.2.1 Recorded data

Recorded daily rainfall and temperature data’s from weather stations within the catchments was obtained from Ethiopian Meteorological department, Tigray - Mekelle branch for Wukro, Freweyni and Hawzien stations. The data used was from 1971 to 2016 G.C. Table 4.1 shows a summary of the data’s that was available for each station within the study area.

Table 4.1: Rainfall gauging stations

Station Name	Station ID	Data Duration (Years)
Wukro Station	NMA 126	1992 - 2016
Senkata Station	NMA 74	1973 - 2016
Hawzien Station	NMA 62	1971 – 2016

4.2.2 Estimation of missing data and Areal rainfall

Location of each station was georeferenced using the Global Positioning System (GPS) and a point map of the rainfall stations was prepared. The missing data was filled arithmetically based on the above gauges that have data for the day of interest and rainfall data was prepared in the format accepted by the model. Unweighted rainfall data was used with the point map to obtain a spatial distributed rainfall data using Thiessen polygon method (Figure 4.1).

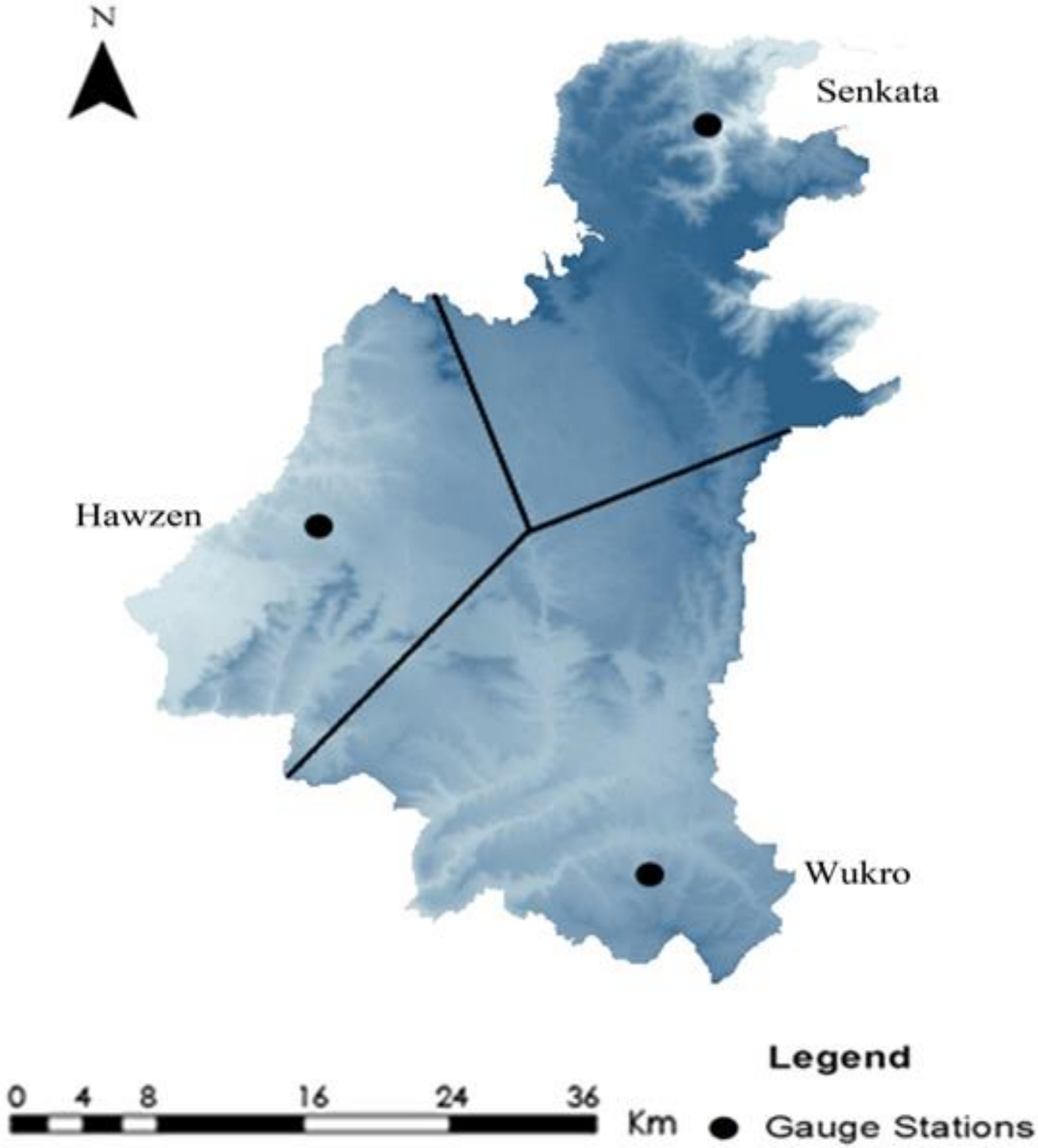

Figure 4.1: Rainfall gauging stations and Thiessen polygon map

Figure 4.1 show the locations of the rainfall stations in the culvert site catchments. They are mainly concentrated at the lower part of the catchments which is Freweyni. In the upper part there is small concentration with rain gauge station at Wukro therefore in developing the spatial rainfall distribution, it was assumed that the rainfall data from these gauges was representative.

4.2.3 Data quality analysis

Quality data analysis was carried out to ensure that the data used in the study were of good quality. Measured data is not error free, as it was noted in the literature. During collection of data errors may be introduced in several ways such as: erroneous reading, recording, copying and by instrument defects (Shaw, 1996). Also errors may be introduced if the gauging station is moved to another location. Therefore any collected data need to be analyzed and the necessary corrections done. The Excel software was used in the study to carry out homogeneity and consistency tests. Hawzien, Wukro and Freweyni stations had complete rainfall data and therefore it was used as the base station in homogeneity and consistency tests.

4.2.4 Rainfall

For the selected catchment case studies the rainfall data's for stations Freweyni, Hawzien and wukro are taken from Tigray national metrology agency, Mekelle branch. The daily maximum and mean monthly values of rainfall for the study areas are in the table below.

Table 4.2: Daily maximum rainfall, average of three stations

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Maximum daily rainfall (mm)	17.4	38.5	42.6	75.5	61.9	62.4	94.4	114.2	31.7	52.8	43.4	10.4

Table 4.3: Mean monthly rainfall, average of three stations

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Monthly mean rainfall (mm)	2.8	3.7	11.6	16.2	15.8	12.9	34.75	38.4	14.3	5.8	4.7	0.82

4.2.5 Temperature

The temperature data of the sites are taken from the national meteorology agency of Mekelle branch for stations Freweyni, Hawzien and wukro, which are collected for the past 40 years. A maximum and minimum monthly temperature (average of three stations) for the study areas are tabulated in the below.

Table 4.4: Maximum monthly temperature, average of three stations

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Max daily Tem(^{0}c)	24.4	24.6	26.2	26.8	26.4	27.5	22.9	23	25.2	23.7	22.8	23.2

Table 4.5: Minimum monthly temperature, average of three stations

Month	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Mini daily Tem(^{0}c)	9.4	10.6	12.1	13.5	13.6	13.5	12.9	12.7	12.1	11.1	10.5	9.6

4.2.6 Areal rainfall

The design point rainfall can be selected based on the suitable return period. The return period of any scheme depends mainly on type of the structure, degree of risk to be accepted, and the importance of the structure. Based on ERA manual for small road drainages and culverts, a return period of 20 years is proposed. For this scheme, a return period of 20 years is selected. The one-day maximum point rainfall has been used to estimate the design flood. In most design of small and large watersheds, if there is no any recorded data of flood in the area, precipitation are made in use. Thus, for this particular scheme, frequency analysis of rainfall records is performed in estimating the design areal rainfall, which is then converted to the design flood. Hence the design areal rainfall has been computed 94.4 mm, 73.6 mm and 88.2 mm for Senkata, Hawzen and Wukro area respectively.

4.3 Hydrological data

4.3.1 Hydrology and drainage

The rivers in which the Culvert sites are to be located are not gauged. In order to know the potential of the streams of the culvert sites, it is a procedure to collect the flow in the dry and summer months of the surrounding gauged river catchments in particular. The maximum observed flow of Genfel, Sulluh and Agula catchments in mm are 900 mm/day, 905 mm/day and 935 mm/day respectively. The measurement conducted in the above catchments is using gauged stage method on the month of summer flow, but the above catchments have huge areas which is not comparable with the culvert sites so Gule gauged catchment which is 12 km^2 and located in between Freweyni and Abreha We Atsbeha is used for ungauged catchments of the Culvert sites. Besides rainfall and other meteorological data, the daily runoff data for the two years 2014, and 2015 recorded at Gule gauged watershed outlet using water level stage recorded was collected. The flood hydrographs recorded were analyzed for water stages at one day interval which was required for event based rainfall-runoff simulation.

4.3.2 Stream flow data of Gule station

The Gule gauged sub-watershed is located in the upper Geba watershed, part of the Tekeze river basin in northern Ethiopia (13°52'49''N, 39°28'59''E). The sub-watershed has a catchment area of around 12 km^2. The observed runoff data of Gule gauged station are presented in Appendix A.

4.3.3 Areal transformation from gauged catchment

There is no any recorded flow data at the culvert sites without Gule gauged station which has an area of 12 km^2 nearby the catchments. If rainfall and land cover are similar in the ungauged catchment as the larger catchment containing the ungauged catchment, a simple approach would be to take a ratio of the areas and use that to figure out what fraction of large catchment discharge is the smaller ungauged catchment discharge. However, this requires similarity almost greater than 75% which have a similar spatiotemporal rainfall, similar soil properties and land cover (similar vegetation types, farms, urban), and no major differences in water abstractions between the two catchments. Therefore, design flood estimation for this project is carried out by using the areal transformation method from Gule catchment using two years of 2014 and 2015 flow data having 12 km^2 area because of the similar characteristics between gauged and the ungauged catchments is greater than 75% of the qualification criteria.

4.4 Geophysical data

4.4.1 Soil survey

Soil samples from different locations of the catchment areas are taken at a depth of 20 cm, 30 cm and 50 cm for good average results for all culvert sites (three samples for each catchment without bias using triangulation method).

Figure 4.2: Soil sampling @ different depths

4.4.2 Field work on water harvesting techniques

The farmers have been using different water harvesting techniques from hand dug wells and artificial recharges, so water harvesting from roads will give them an additional purpose for their good productivity.

Figure 4.3: Varieties of water harvesting uses

4.4.3 Geometric data

The geometric data of the existing drainage structures are collected from all culvert sites as shown in the figure below:

Table 4.6: Geometric data of all culvert sites

Culvert Item's	Diameter of Concrete Pipe (m)	Area of the Opening (m2)	Length of Pipe Culvert (m)	Minimum Embankment Cover (m)	Head Water Depth (m)	Slope of Pipe Culvert (%)
Culvert site 1	1.20	1.13	7.0	0.25	1.48	2.2
Culvert site 2	1.00	0.85	7.0	0.25	1.30	2.2
Culvert site 3	1.00	0.85	7.0	0.25	1.35	2.2
Culvert site 4	1.20	1.13	7.0	0.25	1.75	2.2
Culvert site 5	1.00	0.85	7.0	0.25	1.84	2.2
Culvert site 6	1.20	1.13	7.0	0.25	1.32	2.2
Culvert site 7	1.20	1.13	7.0	0.25	1.40	2.2
Culvert site 8	1.00	0.85	7.0	0.25	1.71	2.2
Culvert site 9	1.20	1.13	7.0	0.25	1.65	2.2
Culvert site 10	1.00	0.85	7.0	0.25	1.15	2.2
Culvert site 11	1.00	0.85	7.0	0.25	1.50	2.2
Culvert site 12	1.20	1.13	7.0	0.25	1.44	2.2
Culvert site 13	1.00	0.85	7.0	0.25	1.36	2.2
Culvert site 14	1.20	1.13	7.0	0.25	1.28	2.2
Culvert site 15	1.20	1.13	7.0	0.25	1.18	2.2

4.5. Data analysis and office works

The different activities like post field works and office works carried out during this study are soils lab investigation taken from study areas, data analysis of all culvert sites using Excel and applying GIS (Arc Map) in Combination with Hydrologic (HEC-HMS) and Hydraulic (HEC-RAS) Models are made.

4.5.1 Sampling

The research area was focused in 15 culvert sites (6 culvert sites having steep slope catchments, 4 having gentle slopes and 5 having medium slopes) and having good downstream areas for water harvesting and cultivation areas from the total 92 culvert sites.

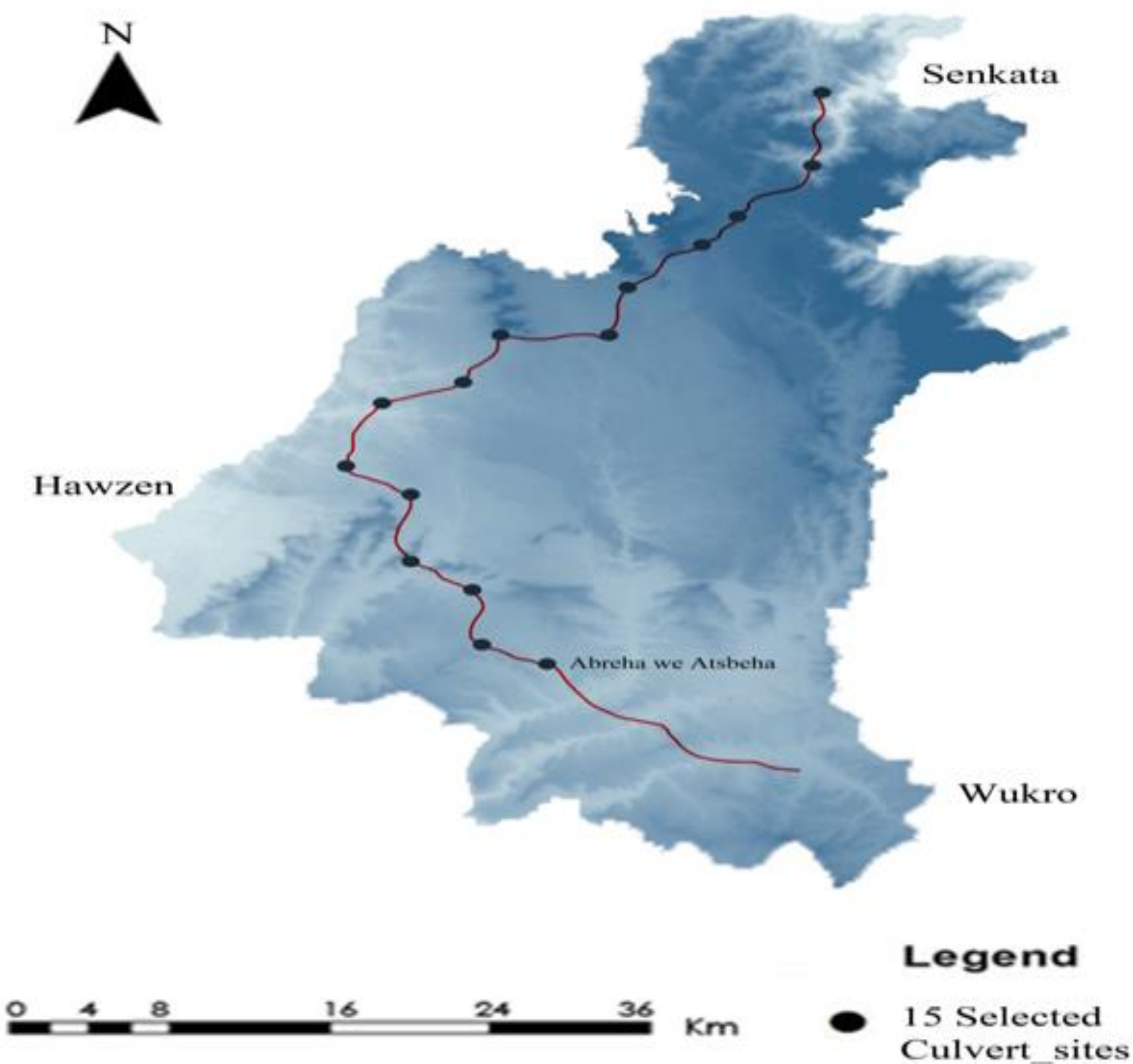

Figure 4.2: Selected research Culvert case studies

The table below shows the coordinates, types, symbols, catchment areas and run off simulation method which are representative for the respective areas of selected Culvert site case studies; Therefore Culvert site 1, Culvert site 2, Culvert site 3, Irish Bridge 2, Culvert site 7, Culvert site 9 and Culvert site 10 are simulated using SCS method under HEC-HMS software and the eight Culvert sites are simulated using rational method according to their area limitation for this paper as shown in the table below:

Table 4.7: Table 4.6: Description of culvert case study sites and method of runoff computations

No	X	Y	Type	Symbol	Catchment area (ha)	Run off (Q) method
1	543601	1543197	Culvert	C 1	57.3	SCS method
2	543066	1542845	Culvert	C 2	115.6	SCS method
3	540453	1540067	Culvert	C 3	74.1	SCS method
4	558678	1551701	Culvert	C 4	17.4	Rational method
5	555202	1548116	Irish Bridge	IBR 1	46.8	Rational method
6	543875	1543430	Culvert	C 5	37.4	Rational method
7	541007	1539718	Culvert	C 6	39.5	Rational method
8	541299	1539270	Irish Bridge	IBR 2	347.0	SCS method
9	542292	1538275	Culvert	C 7	85.6	SCS method
10	542623	1538193	Culvert	C 8	28.7	Rational method
11	544502	1536532	Culvert	C 9	50.3	SCS method
12	547995	1533025	Culvert	C 10	76.0	SCS method
13	549059	1532432	Culvert	C 11	49.9	Rational method
14	549847	1531656	Culvert	C 12	37.5	Rational method
15	550710	1531451	Irish Bridge	IBR 3	19.5	Rational method

4.5.2 Research design

Field work data and GIS map analysis has designed to be tools for describing of the research area by conducting field work survey of culvert sites (soil, river morphology, topography, extracting coordinate systems and site visit), office (software input/outputs, literature review area and final paper analysis), are basic parameters from to the thesis paper. The diagrammatic methodology employed includes modeling and field observation as well as consulting advisory literature formal and informal communication with respective organizations, stakeholders' interview, and focal group discussions. The figure below shows research design diagram.

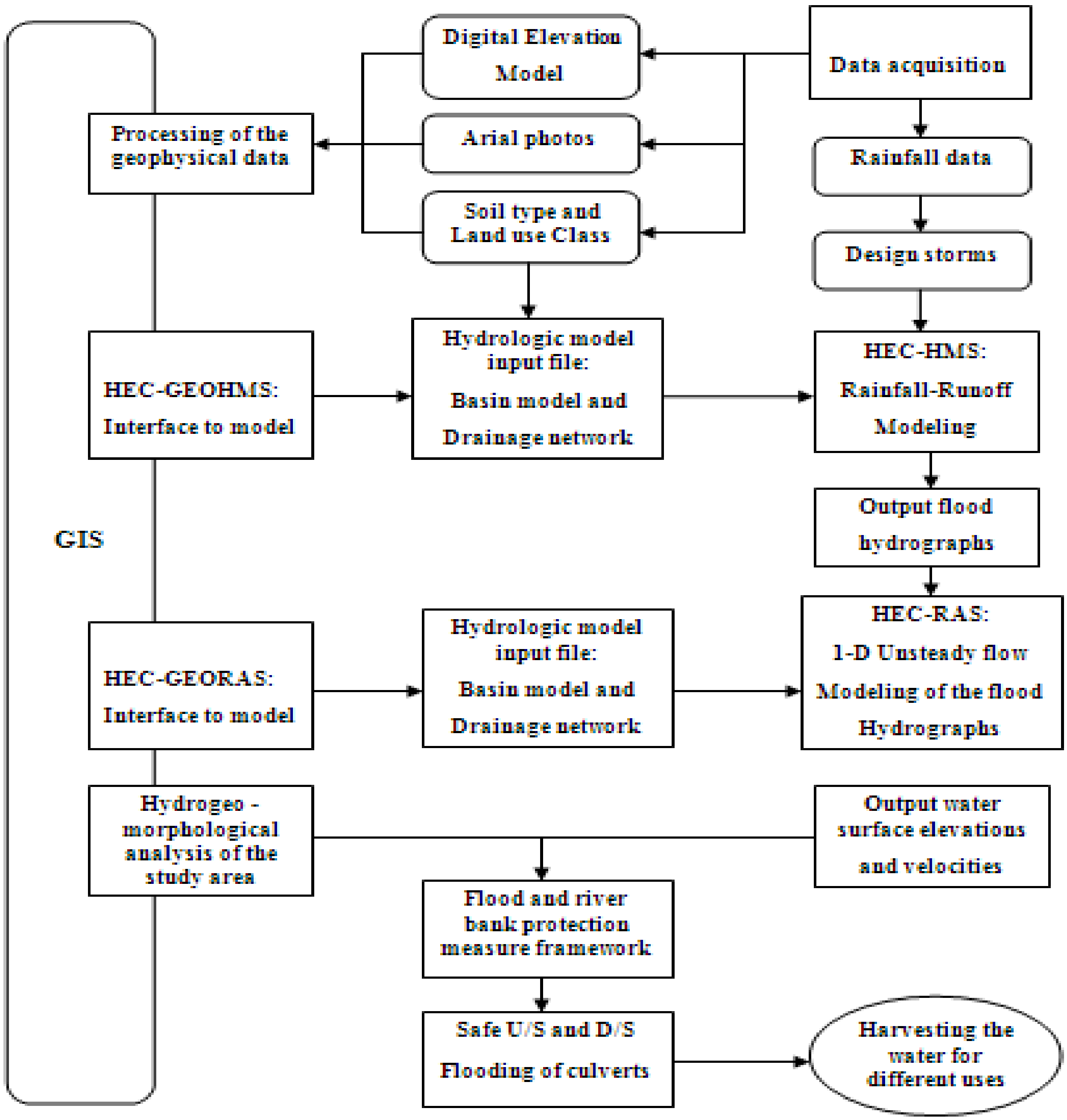

Figure 4.3: Research Design Diagram

Figure 4.3 above shows a conceptual flow chart of the approach that is applied for the accomplishment of the study goals. The Hydrologic Engineering Center´s (HEC) Hydrologic Modeling System (HEC-HMS) is used to model the rainfall-runoff process. A physical model of all watersheds that are relevant for area is created with the Arc Map extension HEC-GeoHMS using a 30 by 30 meter resolution digital elevation model (DEM).

Design storms with statistical record periods of 24, 43, and 45 years for Wukro, Freweyni and Hawzien stations are respectively generated from IDF curves (ERA manual) for the rational formula. These storms are then transformed into flood hydrographs at the outlet of each watershed by the rainfall-runoff model in HEC-HMS which comprises the NRCS Curve Number loss method, the NRCS Unit Hydrograph transform method and the Muskingum-Cunge routing method (for the road side channels only). The HECs´ River Analysis System (HEC-RAS) is used for one-dimensional (1-D) hydraulic modeling of the flood signal in the watershed areas (15 culvert sites), which is the main watercourse in the culvert structure. Together with the analysis of watershed's hydro geomorphology through on-site observations and remote sensing techniques, the results of the H&H modeling are incorporated into the flood correlation between discharge, flow depth and velocity. Based on the estimated flood situation, the flood protection measure framework work of the culvert site will be developed.

4.5.3 Data analysis

The collected data from primary and secondary has been analyzed in qualitative and quantitative data forms. Generally, the data was analyzed using different software in maps, graphs and tables. Table 4.4 shows the analysis of data by different software.

Table 4.8: Software's used for Analysis

No.	Software type	Input	Output
1.	GIS	Maps of Google, Areal CAD files and coordinates	Maps, Mutual integration to HEC-RAS and HEC-HMS
2.	Excel Words	Raw Data	Graphs and Tables
3.	HEC-HMS	Metrological data, Spatial data and area delineated from HEC-GeoHMS	Rainfall – Runoff simulation
4.	HEC-RAS	Width, elevation, shape, location and length of river profile.	River geometrical analysis
5.	HEC-GeoHMS	Out let coordinate points	Input to HEC - HMS
6.	HEC-GeoRAS	Hec-Ras model out puts	Input to HEC - RAS

4.6 GIS, hydrologic and hydraulic models

This sub - section provides the theoretical background for the understanding of the data processing and modeling procedures used in this study. The three software solutions and the mathematical models used in this study are presented in detail. Arc Map is used for all GIS related tasks, HEC-HMS for hydrologic- and HEC-RAS for hydraulic modeling. HEC-GeoHMS and HEC-GeoRAS serve as the interface between GIS and the H&H modeling. At the end of this section a brief result and discussion with final conclusion, output review gives an overview about the applicability and limitations of the applied models.

4.6.1 Geographical Information system (GIS)

For all GIS related tasks, the Environmental Systems Research Institute´s (ESRI) Arc Map software, version 10.0 was used in this study. Arc Map is the main component of ESRI´s ArcGIS suite of geospatial processing software. Most of the GIS tasks were performed based on the functionality of the Arc Map extensions HEC-GeoHMS and HEC-GeoRAS. These GIS extensions are used to prepare a consistent model input file for both HEC-HMS and HEC-RAS within the Arc Map software environment. In addition to that, HEC-GeoRAS can be used for the visualization of the hydraulic modeling results in the form of inundation depth maps.

4.6.2 Rainfall-runoff model: HEC-HMS

4.6.2.1 Fundamentals

HEC-HMS is open source software for the modeling of the rainfall-runoff process developed by the U.S. Army Corps of Engineering´s Hydrologic Engineering Center. The software includes a graphical user interface for the management and analysis of the model data. It is important to mention that HEC-HMS itself is not an actual hydrological model rather than software that enables the user to perform hydrological modeling based on a wide selection of common mathematical models used in hydrology. This simplified representation of the runoff process does not account for the storage and movement of water vertically within the soil layer. It is however sufficient to model a flood hydrograph as the result of a storm (Heimhuber, 2013). For modeling purposes, this simplified hydrologic cycle is further divided into four components, which are modeled separately. The models included in the software can thus be categorized as follows:

Loss method: A model to compute the runoff volume is often referred to as the loss method since it accounts for the losses that occur during a rainfall event as a result of infiltration and evapotranspiration. For each time interval in the modeling process, the loss method calculates the amount of water that contributes to the runoff in the river (effective rainfall).

Transform method: Models of direct runoff are also called transform method, since they convert the effective rainfall over a watershed into a hydrograph at the outlet of the watershed. These models account for the surface roughness and geometry of the watershed.

Base flow method: Base flow models are used to simulate the fraction of the runoff contributed by groundwater.

Routing method: If the analyzed watershed is divided into sub-watersheds, the flow at the outlet of a certain upstream watershed has to be routed through the river channel in the downstream watershed. The models used to simulate this routing process are therefore called routing methods. They account for the geometry and roughness of the relevant river channel.

4.6.2.2. Data requirements and inputs

The main inputs to the model include:

- Watershed stream network and size,
- Infiltration loss method i.e. Initial and Constant, Deficit and Constant, Exponential, Green-Ampt, Smith Parlange, Soil Moisture Accounting, SCS curve Number,
- Transform method for transforming excess precipitation into runoff i.e. SCS, Clark or Snyder unit hydrographs, Kinematic wave, ModClark, User specified unit hydrograph,
- Routing methods i.e. Muskingum, Kinematic Wave, Lag, Modified Puls, Muskingum Cunge, and Straddle Stagger,
- Meteorological data i.e. precipitation, and
- The time span of the simulation.

The outputs from the model include:

- Hydrographs
- Flow volume

4.6.2.3 Model components

In the control specifications, the computational time step and the date of the simulation run are defined. The meteorological model is the representation of the rainfall event that is intended to be modeled. The physical basin model is essentially a simplified physical representation of the watershed which is prepared with HEC-GeoHMS in this study. The main features of the basin model are sub-basins, reaches and junctions. The modeling results comprise runoff hydrographs for each sub-basin as well as graphical and numerical representations of rainfall, losses and direct runoff for each sub basin.

4.6.2.4 Hydrologic model selection and description

Depending on the situation that is being modeled and the available data, an adequate mathematical model for each of the previously defined four components of the rainfall-runoff process needs to be chosen. In this study, the hydrologic modeling is performed primarily to generate flood hydrographs with certain statistical return periods resulting from single design storm events with the same statistical return periods. Since base flow does not occur in the analyzed watercourses for event based small watersheds, it can be neglected in the modeling process. Furthermore, the rivers and the watersheds are ungauged and due to their location in a remote and impoverished area, complex field surveys are not possible. Based on this background, the models shown in Table 4.5 were chosen for each of the four components of the runoff process.

Table 4.9: Hydrologic model selection and categorization

Component	Chosen Model	Categorization
Loss Method	NRCS Curve Number	Event, lumped, empirical, fitted parameter
Transform Method	NRCS	Event, lumped, empirical, fitted parameter
Routing Method	Muskingum-Cunge (for side channels)	Event, lumped, quasi conceptual, measured parameter
Base flow Method	None	None

All the chosen models are designed to model single storm events rather than continuous precipitation data. Furthermore, they are lumped models, meaning that spatial variations of processes and characteristics are not considered explicitly rather than averaged for each sub-watershed. The NRCS Curve Number (CN) and SCS models are both of empirical nature meaning that they are based on observations of the in- and output of a certain system without trying to represent the actual conversion processes as done in conceptual models.

- **Loss method: NRCS curve number method**

The U.S. Natural Resource Conservation Service (NRCS) (formerly the Soil Conservation Service (SCS)) Curve Number method used in this study estimates the effective rainfall as a function of the cumulative rainfall, the land use, the soil type and the antecedent moisture condition of the soil. Apart from the input precipitation, the method uses a single parameter, the CN to characterize the watershed. The CN quantifies the infiltration capacity and theoretically ranges between 0 (100% of the total rainfall infiltrate) to 100 (0% of the total rainfall infiltrate). The basic runoff equation of the CN method is shown in Eq. 4.1.

$$Q = \frac{(P - Ia)^2}{(P - Ia + S)} \tag{4.1}$$

Where: Q = runoff (mm)

P = rainfall (mm)

S = potential maximum retention after runoff begins (mm)

Ia= initial abstraction

The initial abstraction comprises all the losses that occur before surface runoff begins. According to the, it includes water retained in surface depressions as well as water intercepted by vegetation, evaporation and infiltration. In the CN model, Ia is assumed to be correlated to S through Eq. 4.2.

$$Ia = 0.2\,S \tag{4.2}$$

The maximum retention S is further related to the soil and cover conditions of the analyzed watershed through the CN by Eq. 4.3:

$$S = \frac{25400}{CN} - 254 \tag{4.3}$$

In the HEC-HMS modeling process, the incremental excess rainfall for each computation time interval is computed as the difference between the accumulated excess at the end of and the beginning of the period. The cumulative excess Pe is computed with Eq. 4.4.

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}, \text{for P} > 0.2S \tag{4.4}$$

- **Transform method: SCS method**

The SCS runoff equation was developed to estimate total storm runoff from total storm rainfall that is, the relationship excludes time as a variable. Rainfall intensity is ignored. The SCS method for calculating rates of runoff requires much of the same basic data as the rational method namely catchment area, a runoff factor (curve number), time of concentration, and rainfall. However, the SCS method also considers the time distribution of the rainfall, the initial rainfall losses to interception and storage, and an infiltration rate that decreases during the course of a storm. It is therefore, potentially more accurate than the rational method and is applicable when the catchment area is larger than 50 hectares.

The time to peak TP is related to the duration of the unit of excess precipitation D through the following equation:

$$Tp = \frac{\Delta D}{2} + L \tag{4.5}$$

ΔD is the excess precipitation duration which is also the computation interval of the hydrologic modeling process in HEC-HMS and L is the lag time. The lag time is defined as the time difference between the center of mass of rainfall excess and the time to peak of the UH. In the case of ungauged watersheds such as the one examined in this study, it suggests that the lag time is related to the time of concentration as:

$$L = 0.6 * Tc \tag{4.6}$$

The time of concentration is defined as the time for runoff to travel the distance from the hydraulically most distant point in the watershed to the outlet, also referred to as the longest flow path (LFP).

Besides the length of the longest flow path, Tc depends on the surface roughness, the channel shape and the slope in the watershed. According to, the time of concentration is a quasi-physically based parameter that can be estimated as:

$$Tc = Tsheet + Tshallow + Tchannel \tag{4.7}$$

Where: T sheet = sum of travel time in sheet flow segments over the watershed land surface

T shallow = sum of travel time in shallow flow segments, rills and rivulets

T channel = sum of travel time in channel segments

Sheet, shallow and channel flow can be calculated based on the watershed characteristics as follows:

A. Sheet flow

Sheet flow, is flow over the land surface, before the water reaches a channel. According to sheet flow occurs in the headwater of streams and turns into shallow concentrated flow after a maximum of 30.5 m. The sheet flow travel time can be estimated through Eq. 4.8. The NRCS provides a table for the estimation of the roughness coefficients for sheet flow which is presented in Chapter 5.

$$Tsheet = \frac{0.007\ (NL)^{0.8}}{(P2)^{0.5} S^{0.4}} \tag{4.8}$$

Where: N = overland-flow roughness coefficient

L = flow length

P_2 = 2-year, 24 hour rainfall depth

S = Slope of hydraulic grade line (approximately the land slope)

B. Shallow flow

After a maximum of around 100 m, sheet flow turns into shallow concentrated flow. The average velocity of shallow flow is a function of the watercourse slope and type of channel. In HEC-HMS, the shallow flow velocity V is computed as:

$$V = \begin{cases} 16.1345\sqrt{S}\ for\ unpaved\ surfaces \\ 20.3282\ \sqrt{S}\ \ for\ paved\ surfaces \end{cases} \tag{4.9}$$

Based on the average flow velocity V, the travel time T_{travel} is computed as:

$$Ttravel = \frac{L}{V} \tag{4.10}$$

C. Channel flow:

Open channels are assumed to begin, where channels are visible on aerial photographs. For the computation of the travel time in channels, HEC-HMS uses Manning´s equation for uniform flow in open channels. The travel time can be estimated with Eq. 4.11.

$$V = \frac{R^{\frac{2}{3}}S^{\frac{1}{2}}}{n} \tag{4.11}$$

Where: V = average velocity

R = hydraulic radius

S = slope of the energy grade line (approximated as channel bed slope)

n = Manning´s roughness coefficient

4.6.2.5 Catchment areas

Catchment areas can be determined from topographic maps and field surveys. For this thesis, the catchment areas are determined from topographic map of the study area. For large catchment areas, it is necessary to divide the areas into sub-catchment areas to account for major land use changes, obtain analysis results at different points within the catchment area, or locate drainage structures and assess their effects on the flood flows. For this thesis, a field inspection of existing or proposed drainage systems has been made to determine if the natural drainage divides have been altered. These alterations could make significant changes of the size and slope of the sub-catchment areas. However, it is obtained that the alterations do not occur. In general, the catchment areas can be determined from topographic maps and field surveys. However, for large catchment areas, it is necessary to divide the area into sub catchment areas to account for major land use changes. After unit peak discharge is obtained, design peak discharge is determined using the formula:

$$\text{Design Peak Discharge,} \quad Qp = Qu * Q * A \tag{4.12}$$

Where, Qp= Design Peak Discharge, m3/sec

Qu=Unit Peak Discharge, m3/sec/100ha/mm

Q= Direct Runoff, mm

A= Area of the catchment, ha

❖ **Routing method: Muskingum-Cunge (channel routing)**

The movement of a flood wave through a river reach or reservoir is simulated by flow routing. Chow et al. (1988) define flow routing as a mathematical procedure for the prediction of the changes in magnitude, speed and shape of a flood wave during its flow along a watercourse. Thus, any routing model computes a downstream hydrograph based on a given upstream hydrograph as a boundary condition. The Muskingum-Cunge method achieves that by solving simplified versions of the basic equations of open-channel flow. The continuity equation is derived from the basic law of mass conservation and accounts for the volume of water in a reach of an open channel. It sums up the water that flows into the reach, the water flowing out of the reach and the water stored in the reach. In the Muskingum-Cunge model, the finite difference approximation of the continuity equation shown in Eq. 4.13 is used (Chow et al. 1988).

$$\left(\frac{I_{t-1} + I_t}{2}\right) - \left(\frac{Q_{t-1} + Q_t}{2}\right) = \left(\frac{S_t - S_{t-1}}{\Delta t}\right) \tag{4.13}$$

Where: I = Inflow

Q = Outflow

S = Storage

t = incremental time step

The volume of prism storage can be expressed as KQ, were K is the travel time through the reach and Q is the outflow rate. The volume of wedge storage is approximated as KX (I - Q) (The weighted difference between inflow and outflow, multiplied with the travel time). The weighting factor X ranges from 0 for reservoir-type storage to 0.5 for a full wedge. The total storage between the up and downstream sections of the modeled reach is thus given as:

$$S = KQ + KX\,(I - Q) \tag{4.14}$$

or at a given time t:

$$S_t = K[XI_t + (I - X)Q_t] \tag{4.15}$$

If this equation is substituted into the continuity equation (Eq. 4.13) and the unknown values at time t are isolated, the result is the routing expression of the model (Eq. 4.16).

$$Q_t = \left(\frac{\Delta t - 2KX}{2K(I - X) + \Delta t}\right) I_t + \left(\frac{\Delta t + 2KX}{2K(I - X) + \Delta t}\right) I_{t-1} + \left(\frac{2K(I - X) - \Delta t}{2K(I - X) + \Delta t}\right) Q_{t-1} \qquad (4.16)$$

In HEC-HMS, this routing equation is solved for each time step t to compute the outflow hydrograph for a given inflow hydrograph. Besides the inflow hydrograph, the initial outflow Qt=0, the travel time K and the weighting factor X are necessary input parameters for the model. According to, X will approach 0 for channels with mild slopes and overbank flow while for steeper streams with well-defined channels X approach 0.5. For situations without flow measurements like in this study, Cunge suggested a method for the estimation of X and K with Eq. 4.17 and 4.18 (Chow et al. 1988).

$$K = \frac{L}{V_W} \qquad (4.17)$$

where L is the distance between the inflow and outflow section and Vw is the speed of the flood wave, which depends on the channel geometry. The weighting factor X is estimated as:

$$X = \left(1 - \frac{Q_o}{BS_o c\Delta X}\right) \qquad (4.18)$$

Where: Qo = Reference flow from the inflow hydrograph

B = top width of flow area

So = friction slope or bed slope

∆x = length of the reach

c = flood wave speed

The reference flow Qo is the flow between the base flow and the peak flow of the input hydrograph. In HEC-HMS, most of the steps for the calculation of the input parameters for Muskingum-Cunge flow routing are automated. The only input parameters to be defined are the average channel geometry (length, cross section geometry, and slope) and the roughness of the reach that is aimed to be modeled. Apart from the channel´s roughness coefficient, these input features can be derived from a digital elevation model and aerial photographs using GIS.

- ❖ **Base Flow Method: None**

In HEC-HMS, the base flow model is applied both at the start of simulation of a storm event, and later in the event as the delayed subsurface flow reaches the watershed channels. A user-specified threshold flow defines the time at which the recession model defines the total flow. For this thesis report the routing method is not applicable because the model used is event based hydrological simulation.

4.6.3 Hydraulic Model: HEC-RAS

4.6.3.1 Data Requirements and inputs

- ✓ The main inputs to the model are:
    - River geometric data: width, elevation, shape, location, length,
    - River floodplain data: length, elevation,
    - The distance between successive river cross-sections,
    - Manning ‘n’ value for the land use type covering the river and the floodplain area,
    - Boundary conditions e.g. slope, critical depth, • Stream discharge values

Model input data for flow simulations in HEC-RAS comprises a geometric representation and the relative surface roughness of the analyzed flow channel and its overbanks, flow data and data about hydraulic structures such as bridges, culverts, levees or weirs. The base of any geometric model in HEC-RAS is the River System Schematic which is a combination of the river network that defines the connectivity and orientation of all sub-reaches and a series of cross sections that span across the reach and its overbanks and define the boundary geometry. Each cross section can further contain a variety of geometric attributes.

4.6.3.2 Results and outputs from the model

- ✓ The outputs from the model include
    - Water surface elevations
    - Rating curves
    - Hydraulic properties i.e. energy grade line slope and elevation, flow area, velocity

The cross sectional elevations represent the ground surface at various points along each cross section. The bank stations mark the border between the main channel and the floodplain areas. Reach lengths are defined for the left overbank, the main channel and the right overbank. They define the distance between cross sections which is necessary for the energy loss calculations in the modeling process. Furthermore, cross sections can contain the location of the above stated features such as levees or ineffective flow areas. When using HEC-GeoRAS, these cross section features are created automatically based on the location of the digitized channel features. The roughness of the channel and the overbanks is usually defined through appropriate Manning coefficients (n-values).

4.6.4 Model Performance evaluation methods

4.6.4.1 HEC-HMS Model performance

The evaluation of the HEC-HMS model performance is usually done using the traditional R^2 values, the relative volume error (RVE) and the R^2 computed for logarithmic discharge values (IHMS, 2006). But for this paper the model performance is evaluated using the relative volume error (RVE) and Nash-Sutcliffe coefficient (NS) for their simplification and their accuracy.

- ✓ **Relative volume error (RVE)**

There are different functions believed as a measure for the performance of the model. Relative volume error is one among the functions and can vary between ∞ and -∞.The relative volume error performs well when the value of 0 is generated;-it shows there is no difference between simulated and observed discharge. As such, this objective function should always be used in combination with another objective function that considers the overall shape agreement. The formula used to calculate the relative volume error is shown below in equation [4.19].

$$RV_E = \left(\frac{\sum_{i=1}^{n} Q_{sim(i)} - \sum_{i=1}^{n} Q_{obs(i)}}{\sum_{i=1}^{n} Q_{obs(i)}} \right) 100\% \qquad (4.19)$$

Where: RV_E: Relative volume error, $Q_{sim(i)}$: Simulated flow and $Q_{obs(i)}$: Observed flow.

- ✓ **Nash-Sutcliffe coefficient (NS)**

The NS coefficient (with values ranging from -∞ to 1) measures the efficiency of the model by finding the relationship between the goodness-of-fit of the model and the variance of the measured data. A NS efficiency of 1 implies that the modeled discharge is perfectly similar to the observed data. Owing to the frequent use of this coefficient, it is generally accepted that when values between 0.6 and 0.8 are generated, the model performance is reasonable. Values between 0.8 and 0.9 mean that the model performs well and values between 0.9 and 1 imply that the performance of the model is extremely good (Deckers, 2006).

$$NS = 1 - \frac{\sum_{i=1}^{n}\left(Q_{sim(i)} - Q_{obs(i)}\right)^2}{\sum_{i=1}^{n}\left(Q_{obs(i)} - \overline{Q_{obs}}\right)^2} \tag{4.20}$$

Where: NS: Nash-Sutcliffe coefficient, $Q_{sim(i)}$: Simulated flow, $Q_{obs(i)}$: Observed flow and $Q_{\overline{obs}}$: Average of observed flow.

Model validation was carried out for the period 2014 to 2015 of five selected events used for catchment road drainage parameterization. Validation results indicate that the model is capable of fairly predicting the catchment response. Figure 5.2 indicates that the model under predicted the flows except for 2014 and 2015.The NS coefficient shows that the model can predict the catchment response with acceptable accuracy. However, the performance is slightly lower than that for calibration. In their work the calibration results showed a better match than validation. Regardless of the low performance during validation, the results indicate that the model could with fair accuracy simulate the catchment hydrologic response. The graphs for validation results for the study area are shown in Figure 5.2.

4.6.4.2 HEC-RAS model performance

- ✓ **HEC-RAS Calibration/Validation**

The five selected USGS gages are calibrated individually by adjusting their Manning's n values until the model water surface elevations match that of USGS gages. Then the calibrated parameters i.e. Manning's roughness coefficient, are validated for four different peak events over the last 20 years to determine if the water surface elevations are comparable.

CHAPTER FIVE

5.0 RESULTS AND DISCUSSIONS

5.1. Introduction

In this chapter, the hydrologic and hydraulic modeling results as well as the water harvesting scenarios and the problem preventive measure framework are presented. The validity of the hydrologic modeling results is evaluated by analyzing the model loss and transform calculations as well as by discussing the appropriateness of the applied runoff. The accuracy of the hydraulic modeling results is validated in the frame of a sensitivity analysis. The development of the flood hazard zoning is based on the results of the hydraulic modeling and a detailed analysis of the hydro-geomorphology of the study area. Based on the downstream harvest area, a framework of integrated flood protection measures was developed in which a variety of practical solutions to the most crucial flood related issues are presented along with basic constructional guidelines.

5.2. Hydrologic modeling results

5.2.1 Analysis of physical parameters of watershed

The geographic parameters of the watershed have significant effects on runoff, erosion and sediment yield in a watershed. The morphologic parameters of this watershed under study are reported in Table 5.1. The morphologic parameters of watershed such as area of watershed area, perimeter of watershed, total stream length, maximum basin length, maximum basin width etc., are presented. Areas and perimeters of the sub-watershed in culvert site 2 are 115.6 ha and 4.67 km, respectively. The total stream length of the sub-watershed is the sum of lengths of all streams of all orders in arable and non-arable areas. The longest flow path and stream length ratio of the sub-watershed are 1.12 km and 0.69 respectively. The maximum basin length and basin width of the sub-watershed are 1200 m and 625 m, respectively.

The shape of a watershed is generally expressed by three factors, i.e., form factor, circulatory ratio and elongation ratio and these values for the sub-watershed are 0.28, 0.47, and 0.59, respectively. These factors are dimensionless and refer to the shape of outline of the watershed. The table below shows the morphological parameter of the sub-watershed at culvert site 2 and the parameters of other culvert sites are presented in Appendix A.

Table 5.1: Morphologic parameters of the sub-watershed of culvert site 2.

Sr. No.	Parameter	Values
1	Drainage site	Culvert site 2
2	Longest Flow Path	1.12 km
3	Stream length ratio	0.69
4	Perimeter of watershed	4.67 km
5	Area of watershed	115.6 Ha
6	Elongation ratio	0.62
7	Length of overland flow	0.072 km
8	Stream channel slope	4.3 %
9	Maximum length of watershed	1.12 km

5.2.2 Meteorological data event

All the rainfall events were methodically scrutinized and the events were selected reasonably from the collected data. Flood events of various durations and different peak flows were selected to cover a wide spectrum of duration and peaks. For this study, 15 events were selected. Among these 15 events, 10 events were used for calibration and remaining 5 events were used for validation. For the selected 15 events ten for calibration and five for the validation the starting time and end time are recorded by the gap of the daily rainfall event and the peak discharges in m^3/s and the out flow volume in m^3 are analyzed according to the daily rainfall recorded events up to 24:00 hrs.

The selected events from the collected data were used to calibrate the loss rate parameters by calibration. These calibrated parameters were further used for validation. The details of the selected flood events are given in the table below.

Table 5.2: Period of selected storm events for all Culvert sites

Sr. No.	Events	Start date	Start time	End date	End time	Calibration/ Validation
1	Event 1	05 July 2014	12:34:26	05 July 2014	24:00	Calibration
2	Event 2	09 July 2014	14:15:00	09 July 2014	24:00	Calibration
3	Event 3	30 July 2014	13:50:00	30 July 2014	24:00	Calibration
4	Event 4	31 July 2014	08:44:46	31 July 2014	24:00	Calibration
5	Event 5	01 August 2014	12:27:51	01 August 2014	24:00	Calibration
6	Event 6	03 August 2014	14:41:53	03 August 2014	24:00	Calibration
7	Event 7	05 August 2014	16:23:35	05 August 2014	24:00	Calibration
8	Event 8	09 August 2014	13:10:07	09 August 2014	24:00	Calibration
9	Event 9	16 August 2014	14:50:40	16 August 2014	24:00	Calibration
10	Event 10	03 Sep 2014	14:20:48	03 Sep 2014	24:00	Calibration
11	Event 11	18 August 2014	14:30:00	18 August 2014	24:00	Validation
12	Event 12	29 August 2014	14:47:23	29 August 2014	24:00	Validation
13	Event 13	31 August 2014	12:11:41	31 August 2014	24:00	Validation
14	Event 14	06 Sept. 2014	14:16:57	06 Sept. 2014	24:00	Validation
15	Event 15	10 Sept. 2014	14:02:41	10 Sept. 2014	24:00	Validation

5.2.3 Model application and HEC-HMS model performance

The model performance results for calibration of discharge at the 15 culvert sites are presented in the table below ranging 0.45 – 0.72 for NS and 0.50 – 0.79 for RVE.

Table 5.3: Model calibration performance results for discharge at 15 culvert sites.

Culvert Sites	NS (-)	RVE (%)
Culvert Site 1	0.64	0.71
Culvert Site 2	0.72	0.79
Culvert Site 3	0.65	0.70
Culvert Site 4	0.68	0.77
Culvert Site 5	0.71	0.74
Culvert Site 6	0.54	0.58
Culvert Site 7	0.50	0.50
Culvert Site 8	0.69	0.72
Culvert Site 9	0.66	0.68
Culvert Site 10	0.48	0.52
Culvert Site 11	0.70	0.75
Culvert Site 12	0.68	0.71
Culvert Site 13	0.55	0.60
Culvert Site 14	0.61	0.65
Culvert Site 15	0.59	0.67

The optimized model parameter values will be used in validation process for sub-watershed in the selected sites. All model parameters were estimated for each calibrated event. The parameters for all the individual events, along with mean values are presented in Table 5.4. It is observed from Table 5.4 that loss rate for initial abstraction varies from 12.15 mm to 18.45 mm and curve number from 61.26 to 77.47 for different storm events. The mean value of Ia is 14.89 mm and that of CN is 67.40.

Table 5.4: Optimized model parameter values for culvert site two

Parameters / Events	Ia (mm)	CN	Tc (Hr)	R	Qo (m^3/s)	Rc	Qt (m^3/s)	K (hr)	X
Event 1	14.56	69.45	0.220	0.015	0.510	0.000	0.568	0.154	0.025
Event 2	15.21	63.45	0.220	0.015	2.254	0.000	2.315	0.220	0.076
Event 3	13.56	70.50	0.220	0.015	1.614	0.000	1.686	0.089	0.257
Event 4	16.48	60.78	0.220	0.015	2.475	0.000	2.522	0.155	0.091
Event 5	12.15	67.25	0.220	0.015	1.658	0.000	1.694	0.099	0.105
Event 6	16.80	61.26	0.220	0.015	1.542	0.000	1.588	0.085	0.024
Event 7	13.45	77.47	0.220	0.015	0.548	0.000	0.583	0.078	0.086
Event 8	13.74	71.24	0.220	0.015	1.186	0.000	1.341	0.088	0.148
Event 9	18.45	64.28	0.220	0.015	3.358	0.000	3.405	0.112	0.066
Event 10	14.50	68.11	0.220	0.015	0.850	0.000	0.915	0.054	0.031
Mean Value	**14.89**	**67.40**	**0.220**	**0.015**	**1.600**	**0.000**	**1.662**	**0.113**	**0.091**

The transform parameters, that is, time of concentration (Tc) and storage coefficient (R) are constant having values 0.222 hours and 0.015, respectively. The initial surface discharge (Qo), total flow (Qt), and recession constant (Rc) are the total flow parameters which varied from 0.017 to 0.019 m^3/s, 0.113 to 0.125 m3/s and 0.470 to 0.759, respectively. Mean value computed from ten events for initial runoff (Qo), total flow (Qt) parameters are 1.600 m3/s, 1.662 m3/s respectively and recession constant (Rc) is zero for base flow . Muskingum method parameter K varies from 0.028 to 0.456 hours with mean value of 0.131hours. Parameter X varies from 0.039 to 0.500 and with mean value of 0.205. The value of X is dimension less and generally varies from 0 to 0.5.

5.2.4 Model calibration

Ten events (Event-1, Event-2, Event-3, Event-4, Event-5, Event-6, Event-7, Event-8, Event-9, and Event-10) were used for calibration of the model parameters whereas the remaining five events (Event-11, Event-12, Event-13, Event- 14 and Event-15) were used for validation.

The observed direct surface runoff hydrographs and the values of peak total outflow volume of the observed direct surface runoff hydrographs were compared with the simulated values were compared with the hydrographs computed by the model (both before calibration and after calibration) for the selected events used for calibration. The data presented below in table 5.5 reveals model simulations for peak discharge and total outflow volume after calibration are similar to observations. The observed peak discharge was minimum for the Event- 1 with a value 0.510 m^3/sec and for the same event the peak discharge simulated before calibration was 0.415 m^3/sec and after calibration it is 0.420 m^3/sec. Observed peak discharge was maximum for Event-4 which is 2.475 m^3/sec. For the same event, the simulated peak discharge before calibration was 1.945 m3/sec and that of after calibration was 1.986 m3/sec.

Table 5.5: Comparison of simulated and observed peak discharge and total outflow volume for the calibration of culvert site two

Event	Peak discharge (m3/s)			Total outflow volume (1000 m3)		
	Simulated		Observed	Simulated		Observed
	Before calibration	After calibration		Before calibration	After calibration	
Event 1	0.415	0.420	0.510	17.10	17.28	20.98
Event 2	1.322	1.354	2.254	46.40	47.53	79.12
Event 3	0.952	0.988	1.614	34.84	36.16	59.07
Event 4	1.945	1.986	2.475	106.8	109.1	135.9
Event 5	1.558	1.615	1.658	64.70	67.07	68.86
Event 6	1.456	1.508	1.542	48.76	50.50	51.64
Event 7	0.660	0.652	0.548	18.07	17.86	15.01
Event 8	1.254	1.248	1.186	48.90	48.66	46.25
Event 9	3.324	3.218	3.358	109.6	106.1	110.7
Event 10	0.480	0.475	0.850	16.68	16.51	29.54

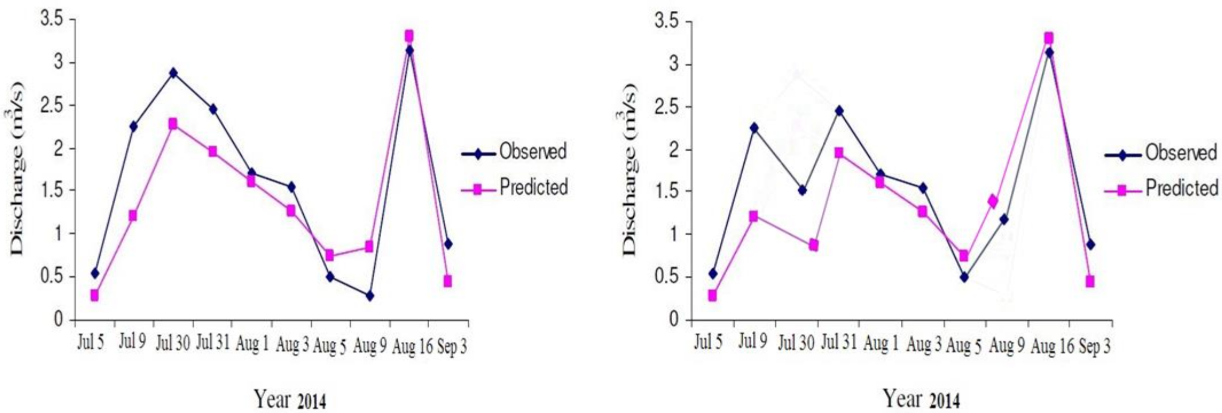

Figure 5.1: Simulated and observed peak discharges for culvert sites 1 and 2

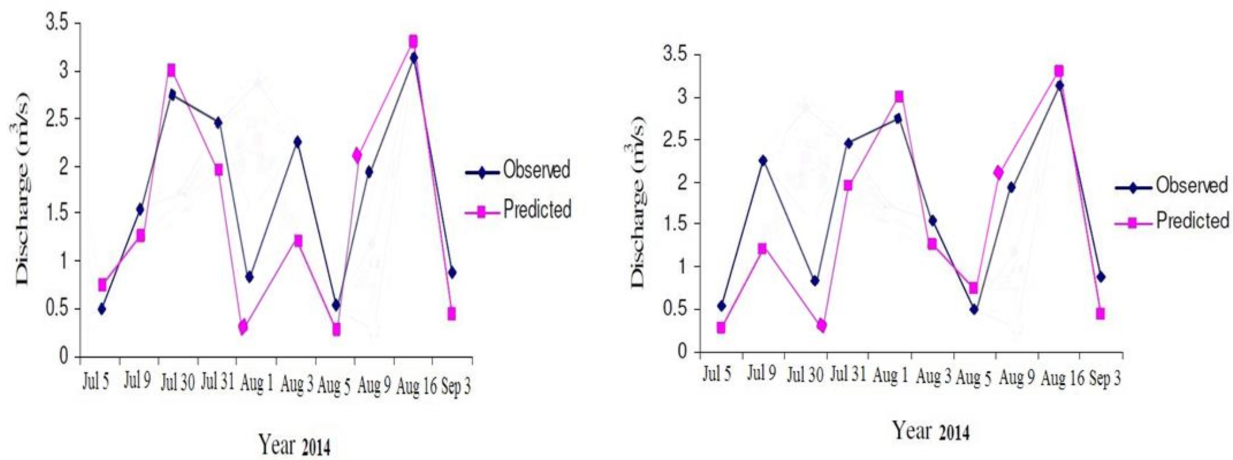

Figure 5.2: Simulated and observed peak discharges for culvert sites 3 and 4

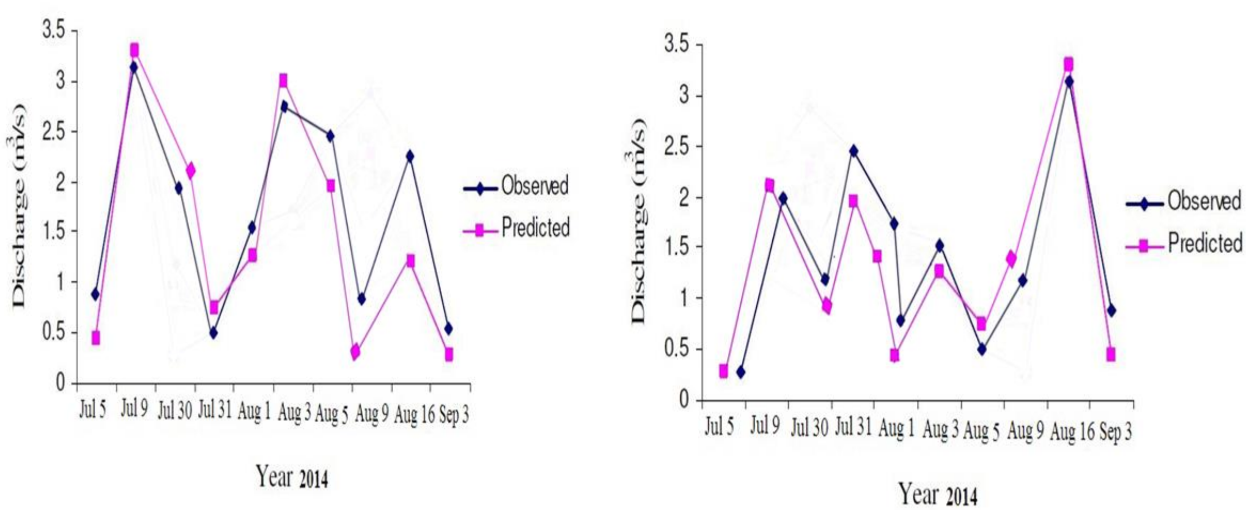

Figure 5.3: Simulated and observed peak discharges for culvert sites 5 and 6

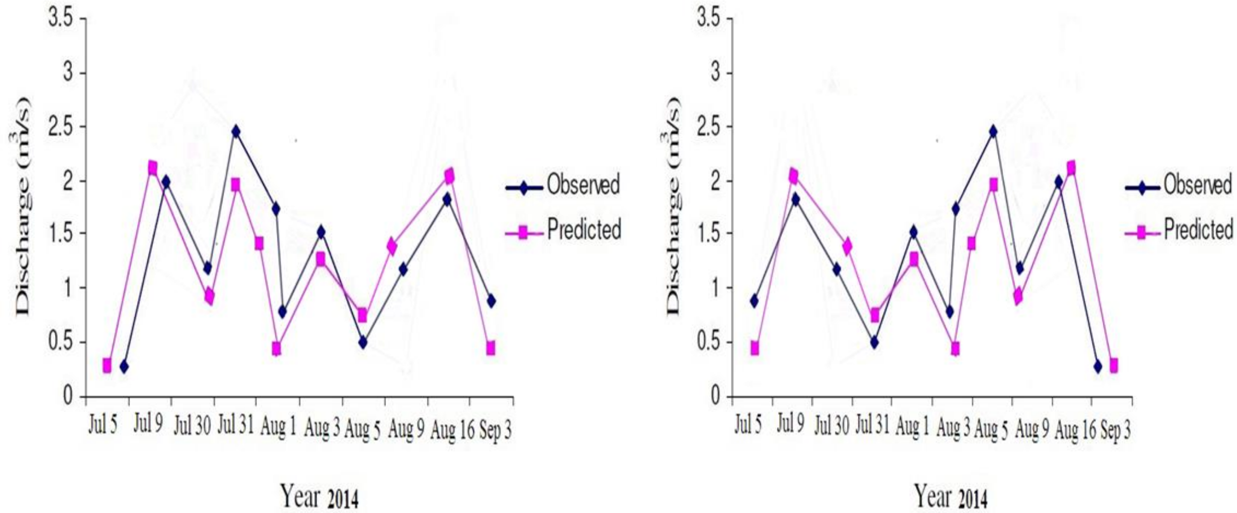

Figure 5.4: Simulated and observed peak discharges for culvert sites 7 and 8

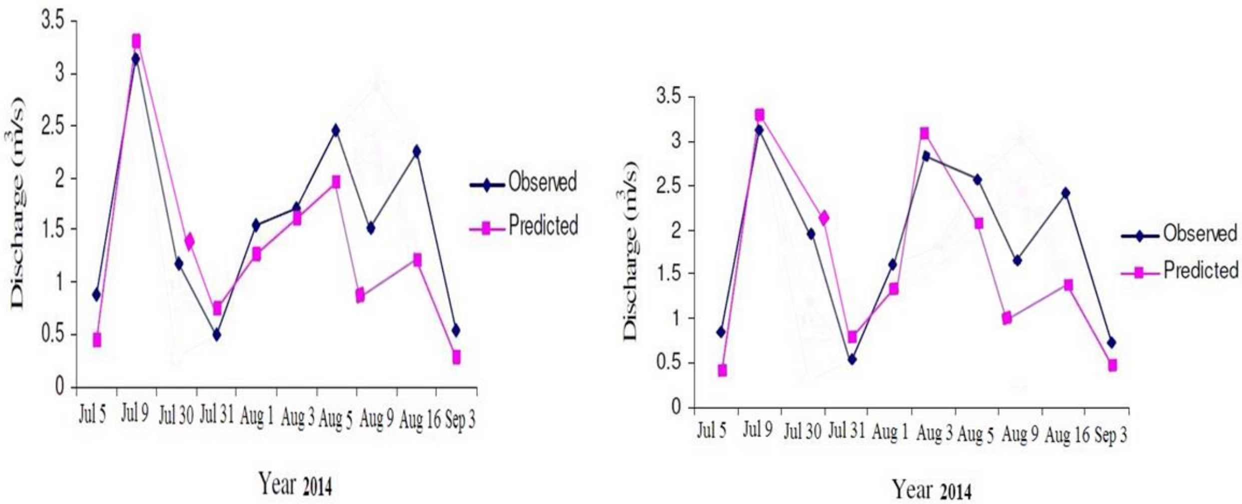

Figure 5.5: Simulated and observed peak discharges for culvert sites 9 and 10

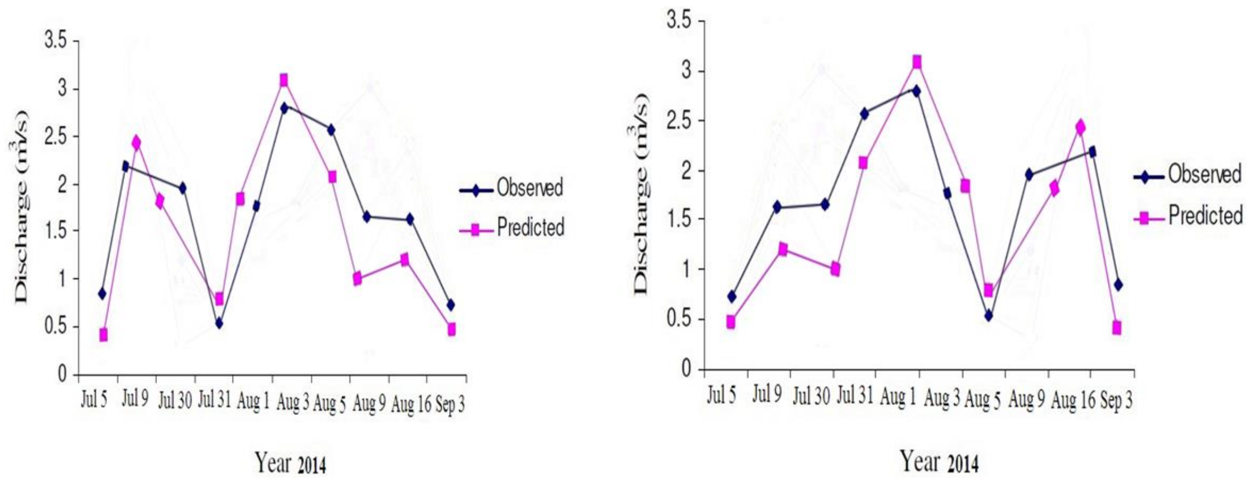

Figure 5.6: Simulated and observed peak discharges for culvert sites 11 and 12

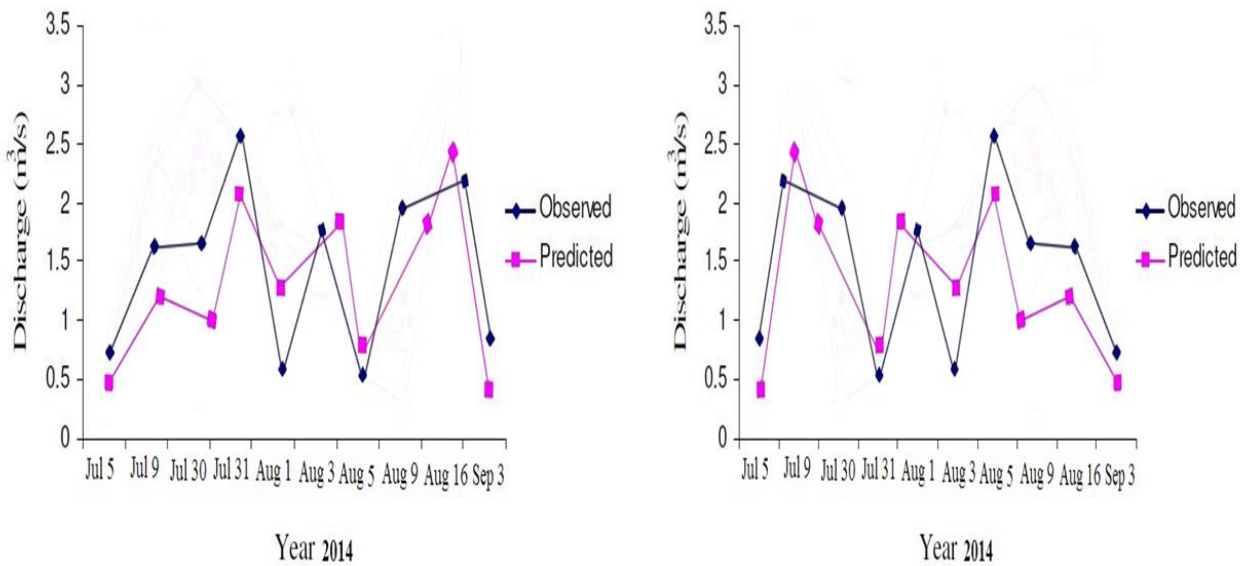

Figure 5.7: Simulated and observed peak discharges for culvert sites 13 and 14

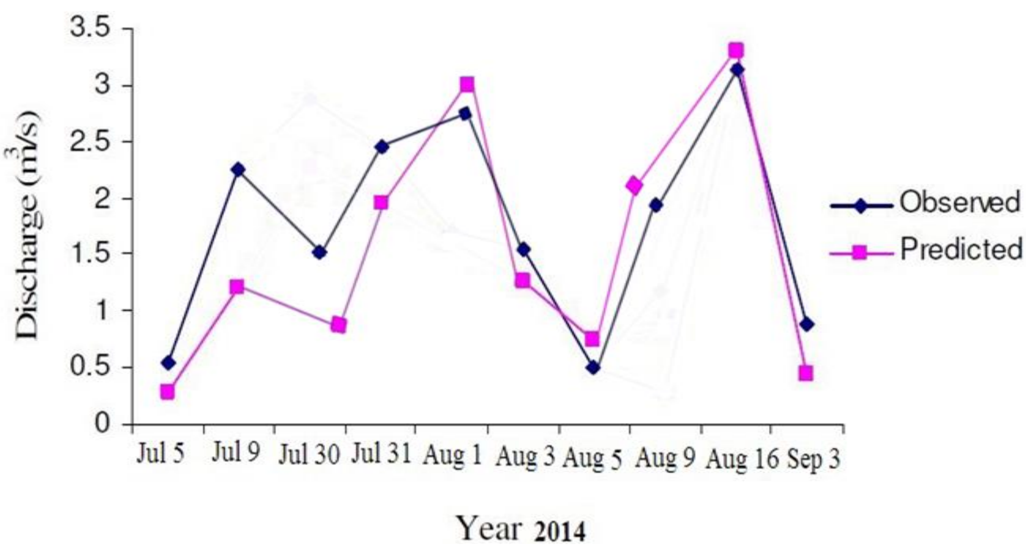

Figure 5.8: Simulated and observed peak discharges for culvert site 15

5.2.5 Model validation

The calculated NS efficiency and RVE were 0.59 and 0.67 respectively for all culvert sites. The NS coefficient shows that the model can predict the catchment response with acceptable accuracy. However, the validation model validation performance was slightly lower than the calibration. Regardless of the low performance in validation, the results indicated that the model could with fair accuracy simulate the catchment hydrologic responses. The hydrographs for validation results for the study areas are shown in Figure below.

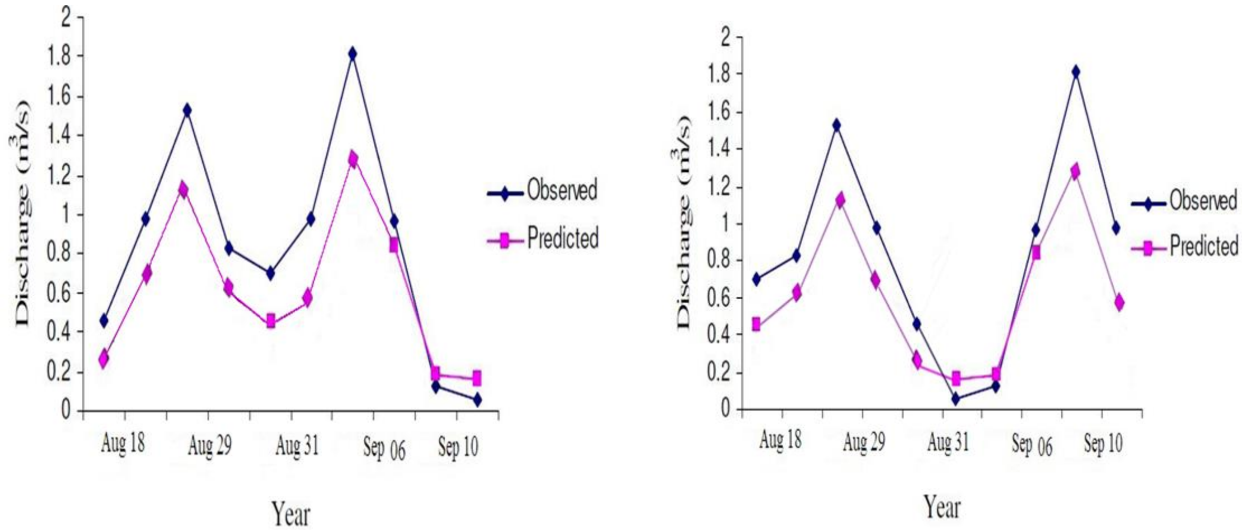

Figure 5.9: Simulated and observed peak discharges for culvert sites 1 and 2

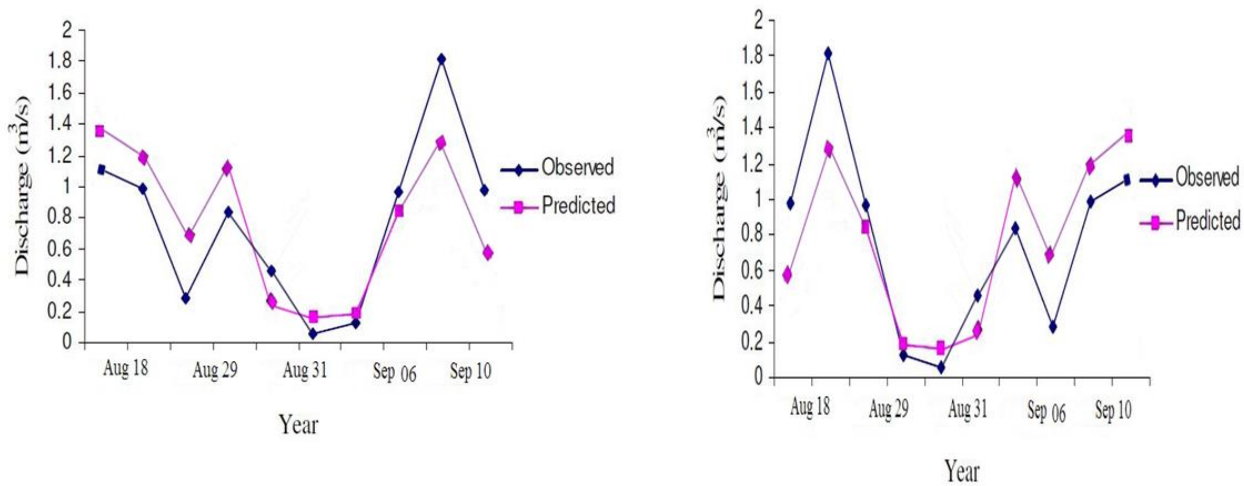

Figure 5.10: Simulated and observed peak discharges for culvert sites 3 and 4

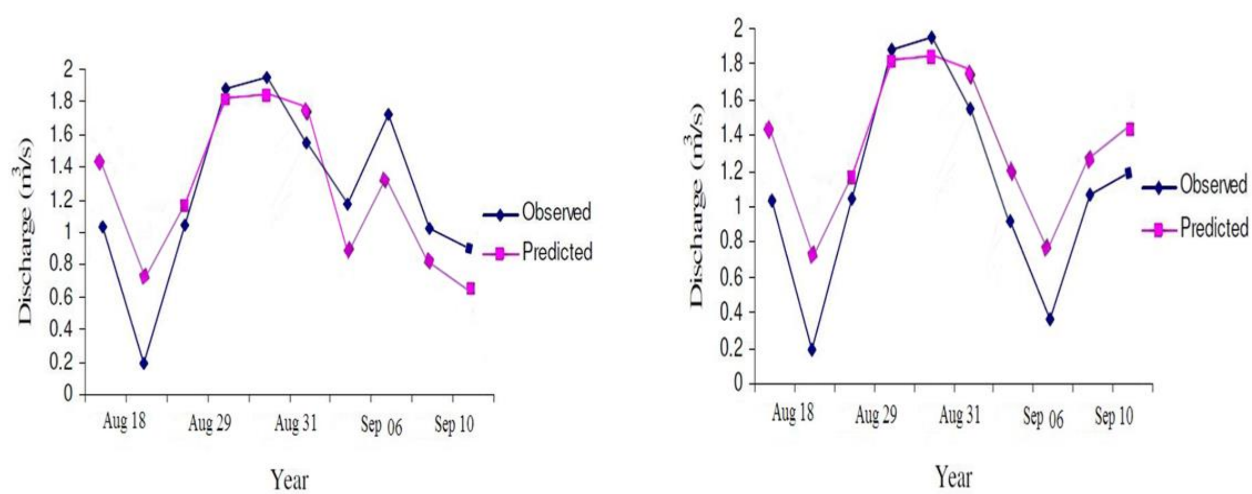

Figure 5.11: Simulated and observed peak discharges for culvert sites 5 and 6

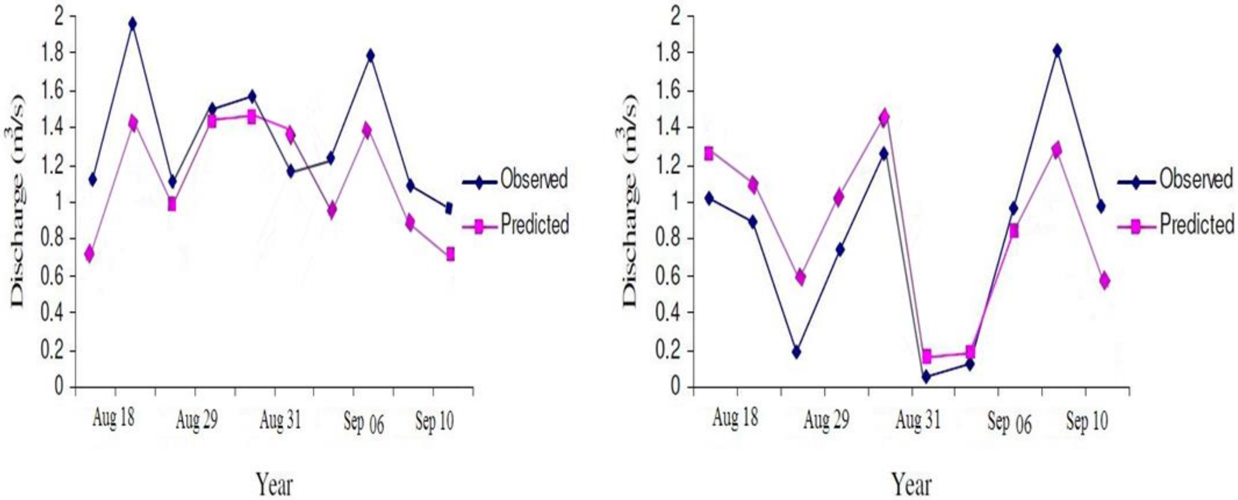

Figure 5.12: Simulated and observed peak discharges for culvert sites 7 and 8

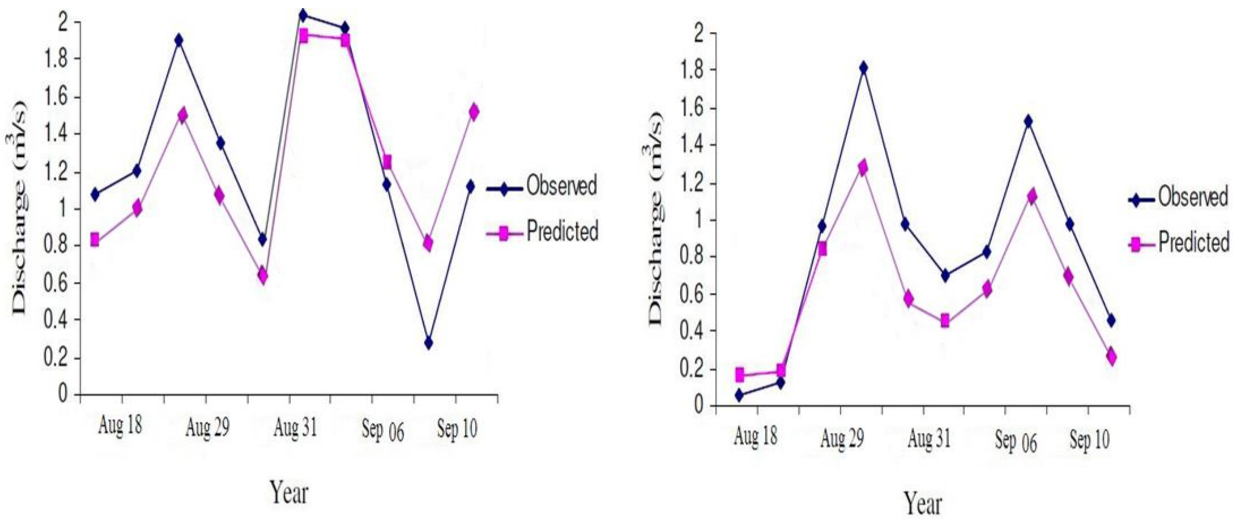

Figure 5.13: Simulated and observed peak discharges for culvert sites 9 and 10

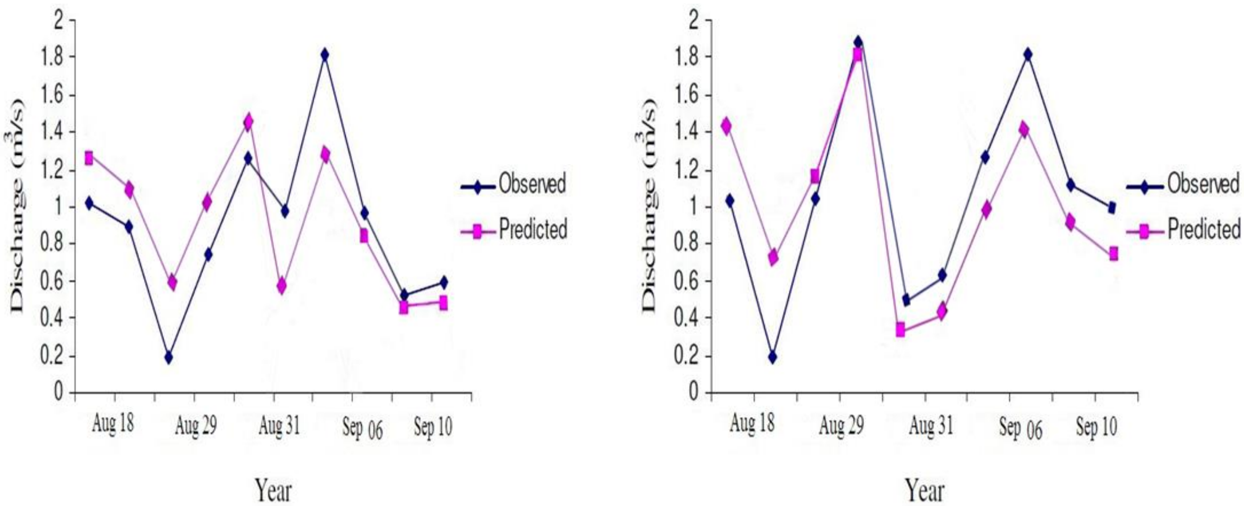

Figure 5.14: Simulated and observed peak discharges for culvert sites 11 and 12

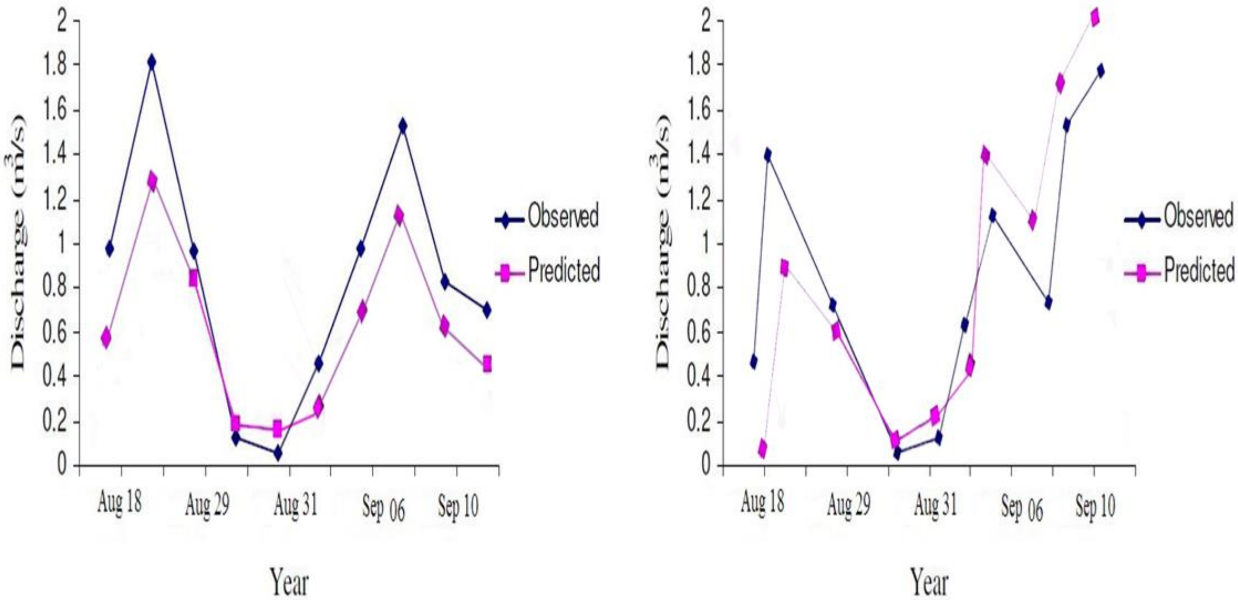

Figure 5.15: Simulated and observed peak discharges for culvert sites 13 and 14

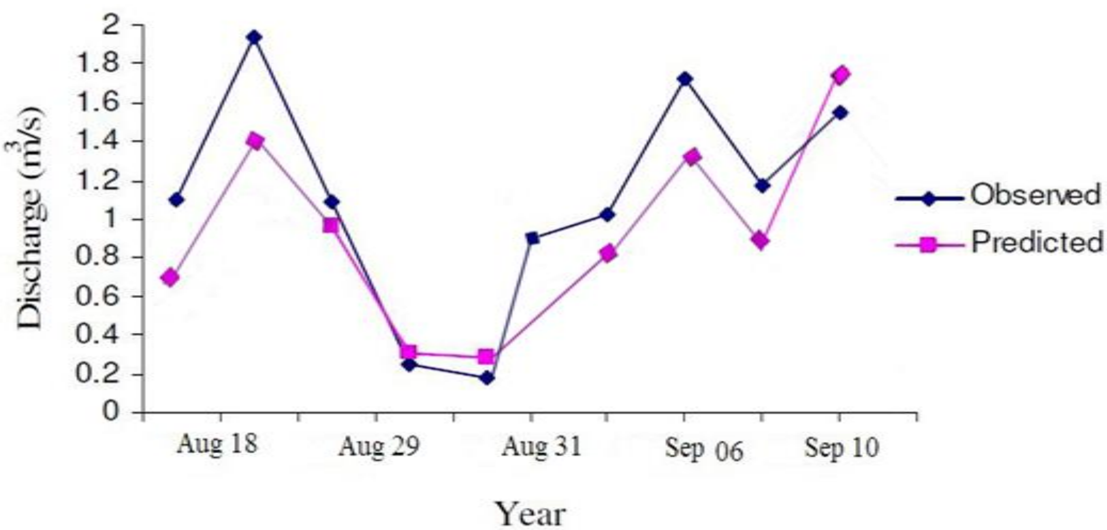

Figure 5.16: Simulated and observed peak discharges for culvert site 15

The observed and simulated runoff hydrographs for the selected 10 events are also shown in Appendix B. A perusal of the Appendix B shows that the peaks both in terms of magnitude and time to peak are best simulated by HEC-HMS model after parameter calibration for Event-1, Event-3, Event-7, and Event-9. For the Event-2, Event-4, and Event-10 although there is a lag in time to peak, yet the discharge is simulated well and the shape of the hydrograph is symmetric with the observed hydrograph.

5.3. Sediment yield

Culvert site 8 has the huge annual soil loss with 1075.7 ton and culvert site 4 with 22.45 ton has the smallest annual soil loss. As per the MUSLE, the annual soil loss (S) is in table below:

Table 5.6: Total sediment loss for 15 culvert sites.

Culvert Item's	**Catchment area (ha)**	**Average annual soil loss (ton/ha)**	**Total annual soil loss per watershed area (ton)**
Culvert site 1	57.3	1.54	88.11
Culvert site 2	115.6	1.93	223.11
Culvert site 3	74.1	1.61	119.31
Culvert site 4	17.4	1.29	22.45
Culvert site 5	46.8	1.45	67.86
Culvert site 6	37.4	1.38	51.61
Culvert site 7	39.5	1.40	55.3
Culvert site 8	347.0	3.10	1075.7
Culvert site 9	85.6	1.71	146.38
Culvert site 10	28.7	1.33	38.17
Culvert site 11	50.3	1.51	75.95
Culvert site 12	76.0	1.64	124.64
Culvert site 13	49.9	1.49	74.35
Culvert site 14	37.5	1.38	51.75
Culvert site 15	19.5	1.32	25.74

5.4 Water harvesting potential

Culvert site 8 has the huge water harvesting potential with 263,650.6 m^3 and culvert site 4 with 13,630.46 m^3 has the smallest harvesting potential using 80% annual dependable rainfall.

Table 5.7: Total water potential for all culvert sites

No	Type	Symbol	Catchment area (ha)	Weighted runoff coefficient , C	Dependable rainfall (mm)	Runoff volume (m^3)
1	Culvert	C 1	57.3	0.16	315.0	28,879.2
2	Culvert	C 2	115.6	0.19	342.6	75,248.66
3	Culvert	C 3	74.1	0.19	342.6	48,234.65
4	Culvert	C 4	17.4	0.24	326.4	13,630.46
5	Irish Bridge	IBR 1	46.8	0.22	326.4	33,606.14
6	Culvert	C 5	37.4	0.19	326.4	23,193.98
7	Culvert	C 6	39.5	0.16	379.9	23,952.8
8	Irish Bridge	IBR 2	347.0	0.20	379.9	263,650.6
9	Culvert	C 7	85.6	0.15	379.9	48,836.2
10	Culvert	C 8	28.7	0.19	377.5	20,585.08
11	Culvert	C 9	50.3	0.20	377.5	37,976.5
12	Culvert	C 10	76.0	0.24	360.1	65,682.24
13	Culvert	C 11	49.9	0.22	360.1	39,531.78
14	Culvert	C 12	37.5	0.24	360.1	32,409.00
15	Culvert	C 13	53	0.26	374.8	51,647.44

5.5 Hydraulic modeling results

The results of the calibration and validation process are close in values with percent difference ranging from -0.39 % to 0.58 % at the culvert inlets when the flow is constricted and at the selected river elevation profiles respectively as shown in Table 5.10 and Table 5.11.

Table 5.8: HEC-RAS model calibration for all culvert sites at culvert inlets

		Water Elevation			**Manning n**	
Culvert site items	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**	**Main Channel**	**Overbanks**
Culvert site 1	2.88	1.55	1.53	0.34	0.056	
Culvert site 2	3.55	1.37	1.37	0.06	0.035	0.14
Culvert site 3	3.07	1.38	1.34	0.58	0.087	0.08
Culvert site 4	1.84	1.88	1.82	-0.35	0.083	
Culvert site 5	2.49	1.99	2.02	-0.41	0.02	
		Water Elevation			**Manning n**	
Culvert site items	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**	**Main Channel**	**Overbanks**
Culvert site 6	2.14	1.29	1.34	-0.37	0.047	0.05
Culvert site 7	2.19	1.37	1.37	0.00	0.01	
Culvert site 8	4.65	1.82	1.75	0.43	0.02	
Culvert site 9	3.15	1.64	1.60	0.39	0.03	
Culvert site 10	2.05	1.14	1.19	-0.38	0.05	0.05
		Water Elevation			**Manning n**	
Culvert site items	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**	**Main Channel**	**Overbanks**
Culvert site 11	2.63	1.48	1.48	0.00	0.02	
Culvert site 12	3.11	1.56	1.53	0.30	0.01	
Culvert site 13	2.58	1.40	1.46	-0.39	0.02	
Culvert site 14	2.17	1.32	1.30	-0.26	0.03	0.02
Culvert site 15	2.07	1.23	1.25	0.26	0.15	0.02

The calibration and validation results have good performance of the observed and simulated values having the model performance results R^2 and NS were 0.68 and 0.63 respectively.

Table 5.9: HEC-RAS model validation for selected culvert sites at river cross-sections.

Validation				
Culvert site 1	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**
Elevation at 2606	2.88	0.99	1.09	-2.44
Elevation at 2526	2.88	0.63	0.61	0.42
Elevation at 2466	2.88	0.16	0.23	-1.54
Elevation at 2346	2.88	0.39	0.69	-6.73
Culvert site 5	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**
Elevation at 2547	2.49	1.45	1.46	0.01
Elevation at 2487	2.49	1.41	1.29	2.70
Elevation at 2367	2.49	1.63	1.49	2.90
Elevation at 2367	2.49	1.47	1.57	-1.78
Culvert site 8	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**
Elevation at 2274	4.65	1.02	1.35	-5.52
Elevation at 2234	4.65	1.00	1.01	-0.13
Elevation at 2174	4.65	0.51	0.85	-6.08
Elevation at 2134	4.65	0.75	0.21	6.43
Culvert site 10	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**
Elevation at 2184	2.05	1.46	1.60	-3.01
Elevation at 2144	2.05	1.36	1.33	0.51
Elevation at 2084	2.05	0.93	1.11	-4.66
Elevation at 2064	2.05	1.67	1.24	6.44
Culvert site 12	**Flow (m3/s)**	**Observed (m)**	**Simulated (m)**	**%Difference**
Elevation at 2066	3.11	1.40	1.33	1.05
Elevation at 1986	3.11	1.10	1.19	-1.05
Elevation at 1980	3.11	0.58	0.84	-5.73
Elevation at 1970	3.11	0.92	0.40	6.62

The figures shown below are Stage/area curve and stage/velocity curve for Gule gauged catchment. All the rating curves and the out puts obtained from hydraulic models are presented in Appendix B.

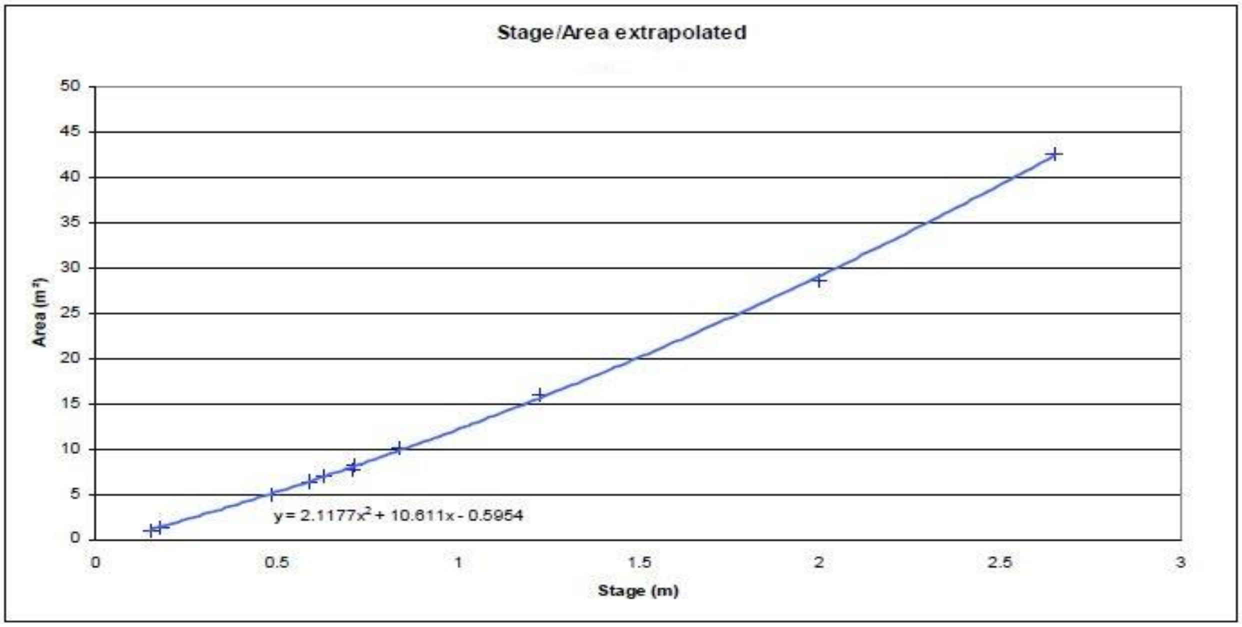

Figure 5.17: Stage/area curve for Gule gauged catchment extended by leveled data

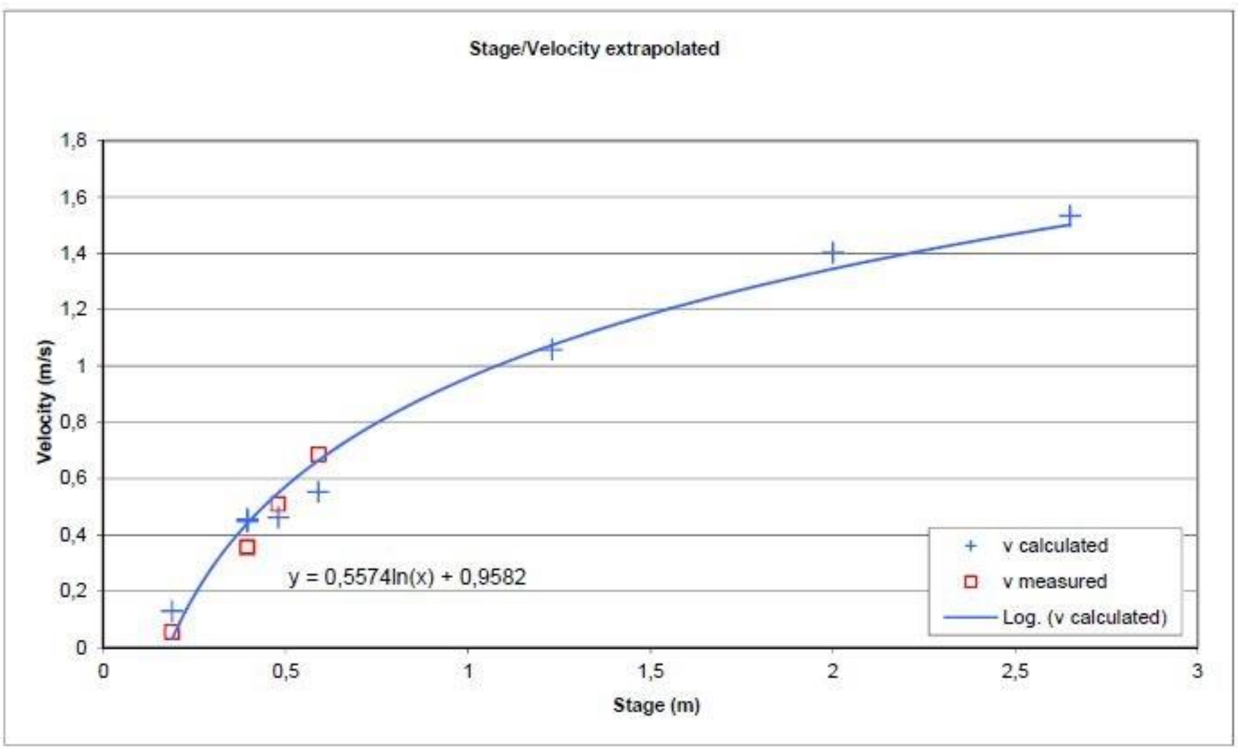

Figure 5.18: Extrapolated stage/velocity curve for Gule gauged catchment.

5.6 Design modification scenarios and options for water harvesting techniques

5.6.1 Road drainage design improvement

In order to provide a practical overview of the outcome of the hydrologic and hydraulic modeling, the results were incorporated into a water harvesting scenarios from roads. It is important to note that water harvesting in comparison to a flood passage does not account for the potential economic and social damages resulting from flooding. Due to the high concentration of flood, the general approach is to convey the flood runoff within uniform impact channels safely through the culverts. The key element of this strategy is an overall improvement of the artificial fraction of all culvert sites so that it can handle a concentrated flood runoff safely and with a possibility of harvesting the water downstream. Another key element of the framework is the guidance of the flood runoffs and scouring velocity from existing drainage Culvert sites 4, 7, 10 and 11 (see Appendix D) along the gentle various potential flow paths into the most steep watersheds (Culvert sites 1, 5, 8, 9, 12 and 14) and moderately sloping culvert sites 2, 3, 6, 13 and 15.

These simple measures could reduce the risk related to debris flows in the holes of the culvert sites to a minimum. In addition to that, a simple bed load scour in the area of the outlet of sub-watersheds of all culvert sites could reduce the velocity and solid load of the dangerous and highly concentrated flood runoffs produced by these watersheds. Since such a varieties of watersheds are not expected to be zero scour in the inlets and outlets of the culvert sites due to its size, slope and flood concentration from the catchments, the bed load has to be managed downstream of the culverts. The areas of all culvert sites that were found to be potentially available for this purpose are outlined in (Appendix D1 – D3). However, in the long run, the abundance of erosion by high scour velocity and concentrated flood in the sub-watersheds should be addressed by adequate preventive measures. In the following sections, scenarios and measure frameworks of all culvert sites are described along with basic guidelines of each watershed.

5.6.2 Design scenarios for the design improvement

A. Scenarios for Culvert sites 4, 7, 10 and 11 (having Gentle slope catchments)

Appendix D shows a specific view of the scenarios and measures for all culvert sites

As mentioned above, the culvert sites frequently suffer from torrential debris and soil deposition caused by stagnant flows generated from flat lands and watersheds having gentle slopes.

After the watersheds of all culvert sites, these watersheds have the smallest catchment areas with a total area of 17.4 ha up to 50.3 ha. In additions to that, the area in the outlet of these watersheds are shown in Appendix D3 are particularly gentle with a gradient in the range of 3%. In combination with the abundance of erosion and active landslides in the watershed, these features favor the formation of torrential debris flows. The flood hydrographs for the outlet of the above culvert sites are presented in Appendix B. They show that a peak flow in the range of 1.35 up to 1.82 m^3/s is expected here in the case of a 20 year flood. It is important to mention that the breach of a blockage and deposition in the culvert inlets resulting from active erosion in these watersheds might lead to greater peak discharges and over flow on the road surface than the ones predicted by the hydrological model. Due to these characteristics, a bed load artificial settling basin in the natural river having 6.4 m length of settling, 2.8 m width of settling, 1.6 m depth of settling and 0.5 m flushing flume with a combination longitudinal guide structure at the upstream of the culvert inlets are suggested for controlling the flood over flow and partial deposition caused by stagnant flow situation in this culvert sites.

The bed load settling basin reduces the solid load along with the flow speed of the flood runoff before it enters the settlement area in the culvert inlets. The longitudinal guide structure serves the purpose of keeping the runoff in the original channel bed so that the settlement in this area is no longer affected by flood events. The technical feasibility of the bed load cross-sectional settling basin was not examined in detail but the potential location of the settling basin was defined based on the relative narrowness of the channel located at 10 m to 20 m upstream of the culvert inlets. The soil eroded from the catchment reaching into the channel in this area of watersheds however imposes some risk in relation to the suggested settling basin construction. Due to the relatively low costs and the ease of construction, a simple gabion barrage with a fixated longitudinal guide bank that can convey the design runoff safely is suggested. Since such types of barrages are not self-flushing, the deposited bed load would have to be removed manually from time to time. Alternatively to the suggested bed load settling basin, the natural channel area between left bank and right bank could serve as a bed load deposition area. A reduced channel slope of 6% in combination with the channel width of up to 10m provides suitable conditions.

The lateral guide structure serves as an indication of where lateral constructions might be necessary. Before the construction, flood runoffs from sub-watersheds of all culvert sites were entirely conveyed within the flow channel along the defined guide structure. The definition of the exact locations of where the natural channel banks need to support structures would require a detailed on site examination. Similar to the bed load settling basin, the guide structure could be constructed using gabions. Alternatively, large lime stone boulders from nearby quarries could be used to construct or reinforce the channel banks wherever necessary. For the fixation of the channel banks with boulders, the following guidelines should be followed. The lowest boulders (1) should be fixed under the channel bottom to avoid scouring whereas the highest boulders (2) have to exceed the elevation of the maximum water level corresponding to the design runoff. Additionally, the usage of concrete for the fixation of the boulders is not recommended (3) in order to ensure a certain level of flexibility of the structure. Based on these considerations, the suggested guide structures are expected to require a minimum of constructional effort, cost and expertise while providing a high level of security to the relevant areas and increases the downstream water harvesting by 25%.

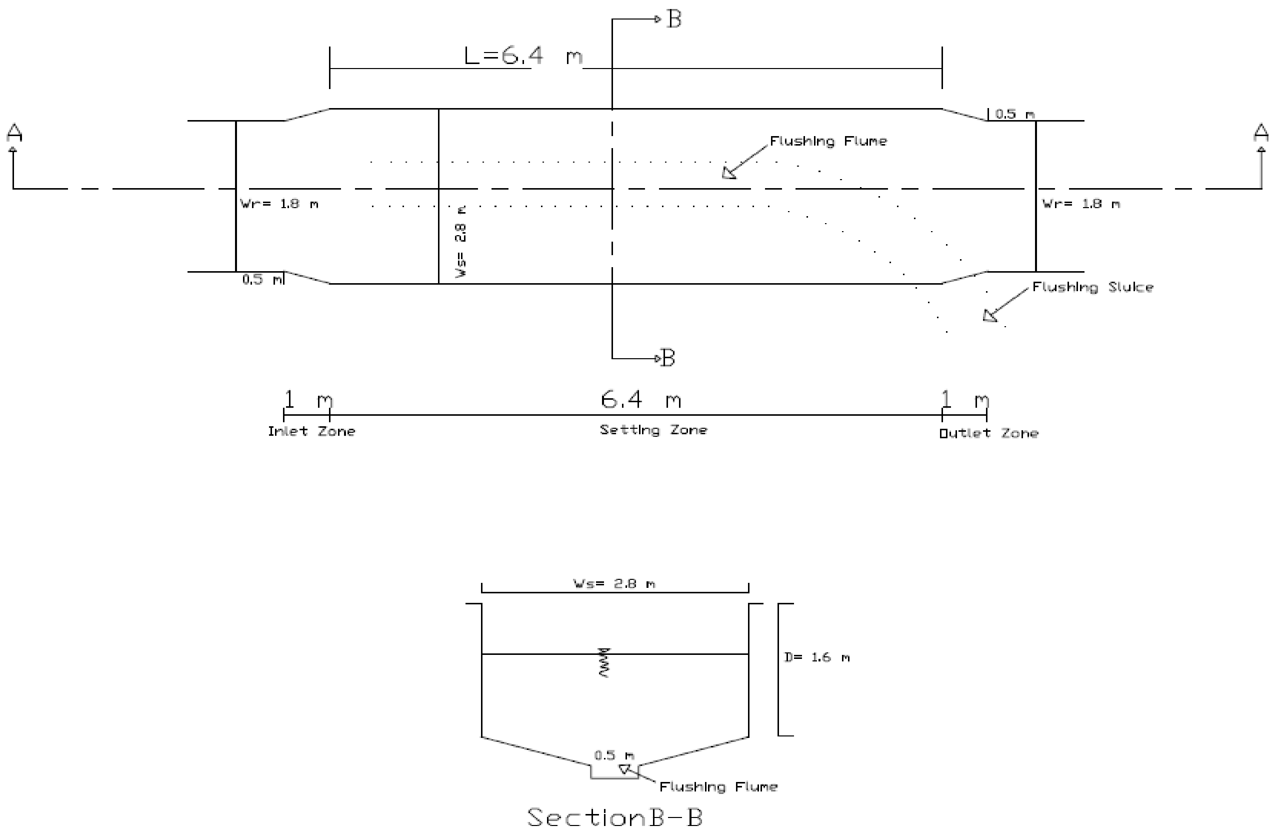

Figure 5.19: Design of settling basin for scenario A

B. Scenarios for Culvert sites 2, 3, 6, 13 and 15. (having Moderate slope catchments)

As seen in Appendix C, the runoffs downstream of protection for the above culvert sites still have the potential to affect large areas of the settlement. A simple solution to this situation is a relatively short redirection of these runoffs in the form of an excavated channel with 0.3 m by 0.3 m rectangular channel along with fixated banks. Similar to the previously introduced protection measures and scenarios, the structures like mentioned in the first scenario (scenario A) applied for gentle catchments are not only representing a schematic indication of the potential constructions. Due to difference in elevation of 6 m up to 30 m from the upper (catchment inlet) to the lower end (catchment outlet) of the indicated channel, the redirection is considered to require little effort at this location.

This redirection could be achieved by an excavation of the new channel in combination with a fixated channel bank or guide structure based on the constructional principles presented in the previous scenario along with the abutment of the culverts. Even though, this measure leads to additional cost to separate the runoff flow from deposited sediments in the culvert inlets, a large increase in the volume of water is expected for pure downstream water harvesting, since the soil loss of culvert sites 6 and 13 are smaller than the other culvert sites 2 and 3. The additional runoff and volume of water will however be considered as positive goal in the design improvement of the downstream water harvesting for the culvert sites 6 and 13. Another problem related to the suggested redirection is that the new rectangular channel would cross the roads under the culverts near the abutments of the culverts that lead to narrowness of the culvert hole acting as cross drainage work. Since runoff in the channel only occurs in response to severe rainfall events, a fixated road passage with ramps on either side of the channel or selecting Irish bridge instead of box culverts could solve this problem with minimal constructional effort. As an alternative to the redirection, the original flow path could be used instead. However, due to the constant and uncontrolled densification of the settlement, this would require the construction of many fixated channel sections and improved road passages along the flow path through the settlement**.** The another alternative measure and scenario that would help for this culvert sites is a uniform and fixated flow channel is used to safely convey the flood runoff into the downstream channel. Similar to the above scenario, the sediments retained by the suggested barrage would have to be removed manually in order to restore the sediment retaining capacity.

C. Scenarios for Culvert sites 1, 5, 8, 9, 12 and 14 (having steep slope catchments)

As shown in Appendix D2 and D3, also the flood runoffs generated from sub-watersheds 1, 5, 8, 9, 12 and 14 require the development of drainage infrastructure, downstream drop structures and energy dissipaters. Appendix D2 and D3 briefs a close-up recommended solution for sediment reduction and water harvesting of the suggested drainage concept for this area using the results from the model. The key element of this concept are fixated impact channels like drop structures and energy dissipaters that are designed to safely convey the peak discharges of the 20 year flood generated in Culvert site 1 (2.88 m3/s), Culvert site 5 (2.49 m3/s), Culvert site 8 (4.65 m3/s), Culvert site 9 (3.15 m3/s), Culvert site 12 (3.11 m3/s) and Culvert site 14 (2.17 m3/s) at rainy season. The hydrographs for return period of 20 years are presented in Appendix B.

In the case of this culvert sites it can be seen that the culvert bed will either have to be completely fixated, or stabilized by concrete in frequent intervals in order to control the culvert bed erosion. Due to the high solids load and energy of the resulting flow regime, a reduction of the bed load and flow speed is necessary before the watershed flow crosses the road and enters into the culverts. Therefore, an increase of the upstream energy dissipaters across with the channel width and downstream drop structures with end of bed load settling basin as described in Appendix D 3 are suggested for this areas. Alternatively to the upper measures, an artificial enlargement of the channel width (10 – 20 m upstream of the culvert inlets) will lead to a reduction of the flow velocity so that sedimentation can occur at the end of the wider channel section, a bed load barrage or settling basin similar to the one presented for protection measure and scenario A is used to further reduce the flow speed and to retain the majority of the bed load. The crest of this settling basin has to be designed to resist the 20 years flood peak discharge.

The required settling basin drainage channel is an elongation of the already existing drainage channel that was constructed along with the culvert abutments. For the discharges of the above culvert sites, two alternatives or scenarios are considered feasible. The current flow path of the runoffs is preserved by scenario 1, which would require a longitudinal guide structure and a fixation and enhancement of the already existing flow channel. Alternative 2 represents a shortcut connection of the culvert sites channel along a currently inactive channel. The shortcut has a slope of 6% up to10% in the suggested drainage channel. This scenario would thus require a support structure at the conjunction. In order to assess which alternative is more appropriate, a detailed on-site analysis is necessary. The constructional guidelines for the suggested guide structures are the same as for protection measure and scenario B. The construction of the suggested impact channels comprises excavations along with a fixation of the channel bed and banks with large boulders.

D. General scenarios and measure frameworks for future works

As mentioned above in the three scenarios, the improvement of the road hydraulic structures by adding and constructing artificial part of the upstream and downstream channel structures is the key element of the flood protection framework of the culverts for acceptable downstream water harvesting.

Depending on the above scenarios and protection measures the additional constructed channels with all culvert sites has to be designed to handle the peak discharge of the 20 years flood safely. For large channels from steep catchments, the depth of surface water in is commonly designed to not exceed the elevation of the culvert hole resulting from the design flood by a certain constant value. This so called freeboard should be in the range of 0.2 to 0.5 m. The extended and location of the suggested improved channel is described in Appendix D 1. Therefore the most suitable construction method for all catchments with steep channels would be excavation along with a complete fixation of the channel with large boulders and upstream energy dissipaters across with the channel width and downstream drop structures with end of bed load settling basin. For the catchments having moderate channel slops a short redirection of these runoffs in the form of an excavated channel with closed conduits or pipes along with fixated banks is appropriate to reduce bed load deposition. The culvert sites having gentle slope catchment channels are frequently suffer from torrential debris and soil deposition caused by stagnant flows generated from flat lands. Bed load settling basin in combination with a longitudinal guide structure at the upstream of the culvert inlets are suggested for controlling the flood over flow and partial deposition caused by stagnant flow situation in this culvert sites and the bed load settling basin is located at 10 m to 20 m upstream of the culvert inlets. Generally based on the hydrologic and hydraulic modeling results, the required size of the additional impact channels has to be designed to fulfill the capacity requirements of all the culvert sites to maximize the water harvesting potential.

5.7 Discussions

A number of studies have demonstrated the importance of low velocity areas in culverts for acceptable scour and safe water passage (Maidment, 1993). It is important that these low-velocity areas are available at relatively large flows since the flow passes through culverts during the rising and falling limbs of flood hydrographs. Maintaining low-velocity areas at these higher flows is difficult because of the increased constriction and expansion of flow at higher discharges. Modeled velocities in culverts for the study sites in this study (15 culvert sites) exceeded 4.5 meter per second for some of them but the other sites at less than 25% the higher.

Although velocities at the study sites exceed this threshold, lower velocity areas near culvert/channel boundaries or in shallow flow areas may be within the range of safe passage abilities. It was beyond the scope of this study to evaluate the spatial distribution of velocities in channels and culverts. It is important to acknowledge that water harvesting does not account for any type of runoff produced in the area of all culvert sites. However, due to the very small size of the culvert inlets in the areas, significant problem is expected around the culvert structures. As mentioned in the general scenarios, the high solid loads coming from the flood runoffs in the study area are expected to pose an increased level of deposition to the affected areas. In order to account for this, areas where bed load deposition is expected during flood events are outlined in the downstream water harvesting techniques. The additional artificial part of the channels was found to provide protection only to flood events in the frequent flood range. Flood events of the rare and exceptional range exceed the channels capacity and will thus cause severe flooding in the outlined areas. The watersheds of the culvert sites were observed to be frequently affected by smaller scale highly concentrated flows. Due to a reduction of the channel slopes along the various flow channels, these concentrated flows eventually spread out on the inlets so that their impact is expected to decrease drastically from this point on. Therefore, areas that are expected to experience spreading flows are outlined separately.

Based on the flood water harvesting benefits as the outcome of the hydrologic and hydraulic modeling, a scenario and framework of flood protection measures was developed. These scenarios are intended to provide practical and feasible solutions to most of the flood related problems that the culvert sites are exposed to. Each measure that is outlined in the scenario is further presented in detail in the above sections including general construction related guidelines. Due to the ongoing densification of the settlement, any type of flood protection infrastructure should be designed to handle a 20 years flood events which are the standard for rural areas in many developing countries. The modeling results showed an increase in peak runoff but decrease in flood risk and flood inundation extents for the various scenarios which results an average 10% increase in runoff volume for all the land use and design storm events. The water elevations between the culverts decrease by 0.54 m comparing each culvert and maximum increase of 15 % in the water elevations for all culverts. Handling the above constraints approves a safe flood convey and an average of 25% increase in water harvesting.

CHAPTER SIX

6.0 CONCLUSION AND RECOMMENDATION

6.1 Conclusions

Hydrological modeling was successfully performed using HEC- HMS model. After developing the basin model component using HEC GeoHMS in ArcGIS environment, populating the meteorological model and defining the control specifications, The HMS was run while calibrating the parameters. The model output results were the quantified runoff floods that resulted from input rainfall data. The hydraulic model was calibrated and then used to simulate the 20 year flood to determine maximum channel flood depths for all river cross sections of all culvert sites. Based on the specific objectives the conclusions below are presented.

The HEC-HMS model performance is estimated 0.48-0.70 for NS and 0.50-0.79 for RVE which shows good performance to the event based hydrological model. The sediment loss estimated for all culvert sites is 22.45-1075.7 tons per year which shows there is high erosion but decreasing the yield by 15% will increase the water potential by 25%.

The peak flood and river analysis were generated as shown in above concluding remark. From the flood model developed, the most flood risk culvert is culvert site 1 with steep catchment and difficult topography and the culvert site with the highest water harvesting potential was culvert site 8 with 263,650.6 m^3 and culvert site 4 has the smallest water harvesting potential with 13,630.46 m^3. Although some culvert sites are affected by high erosion and deposition the design scenarios will be good for reduction of erosion and sedimentation in the land use areas.

6.2 Recommendation

Based on the results of the study and the conclusions thereof, the following recommendations are made.

- ✓ The use of models in simulating catchment response should encompass extensive application of GIS and remote sensing. These tools will ensure that geophysical parameters of the catchment are effectively incorporated in the simulation. Consequently the simulation results and the parameters both physical and conceptual will not be unique for the catchment under study.
- ✓ If the low flow passage is a major concern, additional structures like U/S settling basin and energy dissipaters with stone ripraps may need to be placed accordingly in low, moderate and steep gradient culverts to maintain a low flow channel. The bed had little cross-sectional variation in this analysis and may not provide enough depth for catchments at low flows.
- ✓ This assessment was focused primarily on conditions at the 15 culvert sites. A further investigation of trends among different road sites would help to further understanding of the response of culvert sites to improve the design. Predictor variables such as culvert size, culvert shape, culvert slope, catchment slope and the size of placed bed material can be related to observed scour, predicted scour, and flood conditions in order to evaluate how components of culvert design affect scour and deposition among all sites.
- ✓ The government of Ethiopia and concerned professionals should work with the Ministry of Water, Irrigation and Electricity to avoid water scarcity of the country, by applying new ideology like roads for water, artificial ground water recharges and ground water wells to enhance sustainable development.

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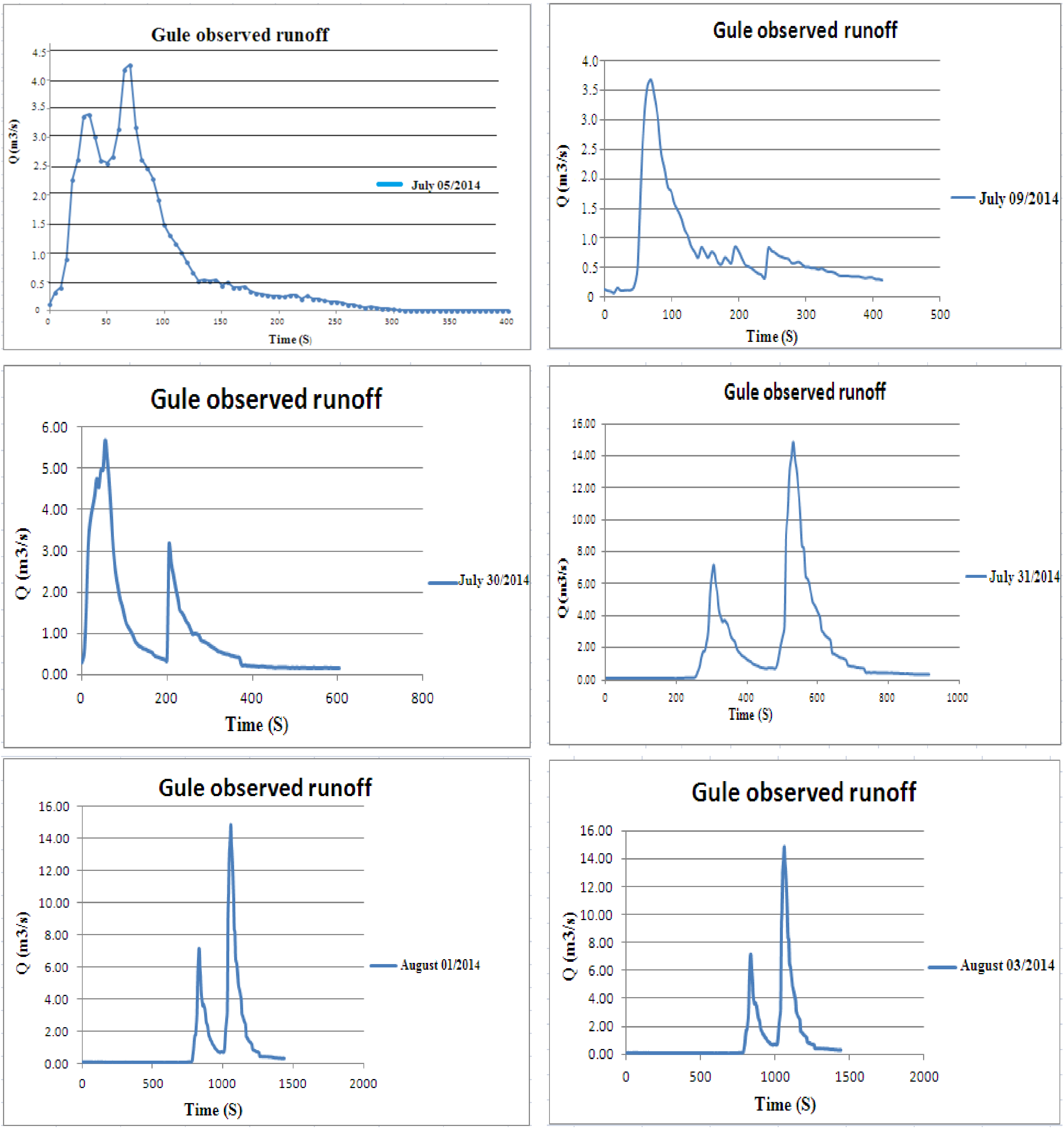
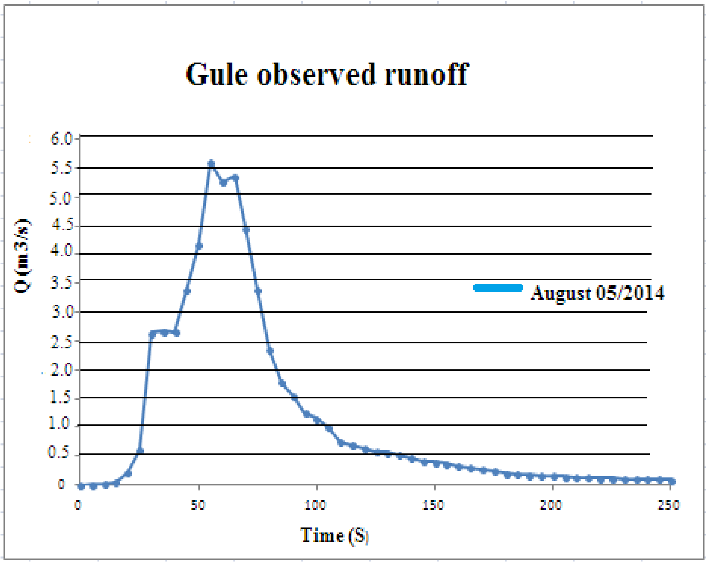
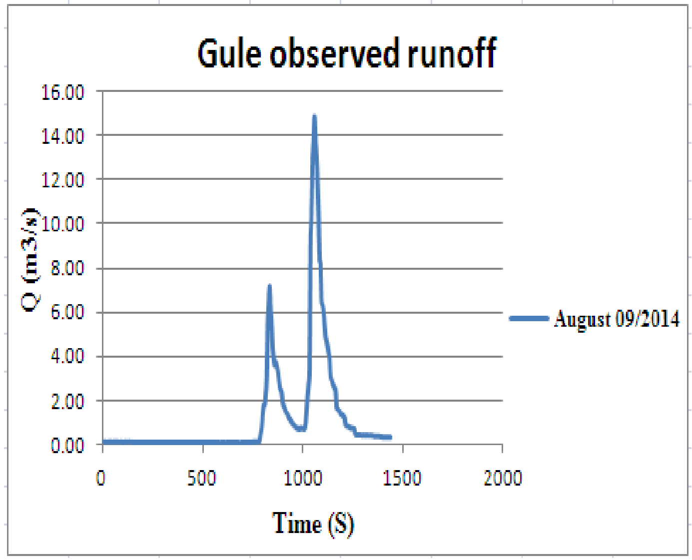
APPENDICE

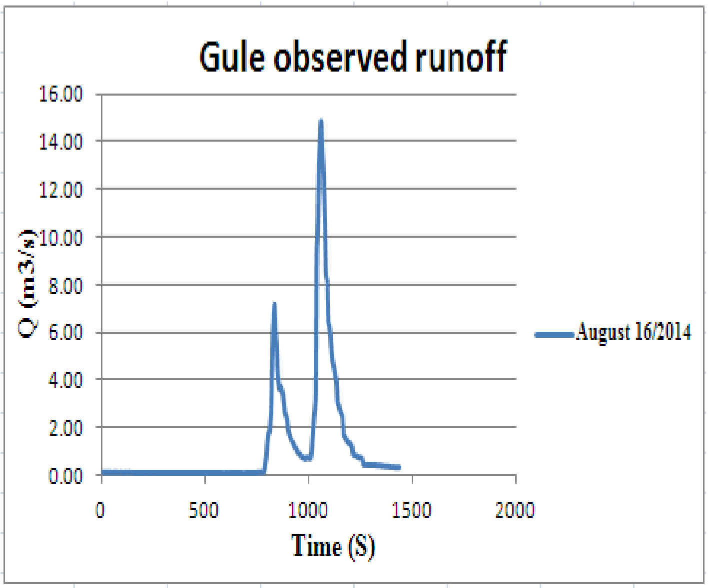
Appendix A: Runoff data of Gule gauged watershed

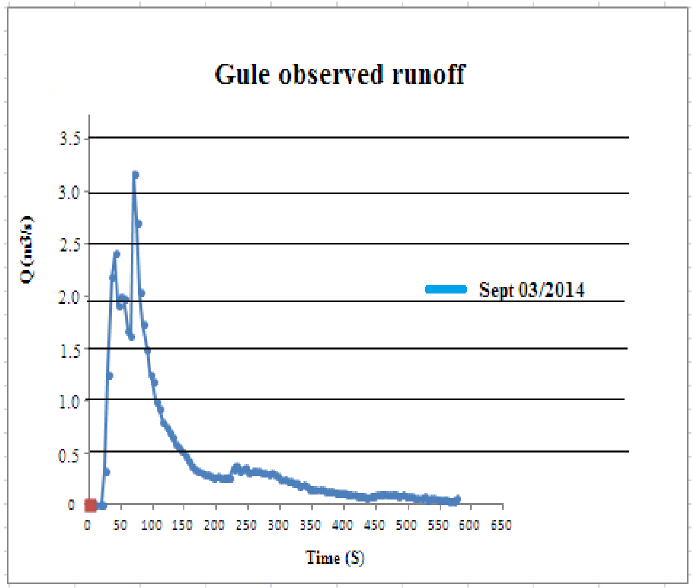
Figure A-1: Observed 15 selected runoff events of Gule gauged station

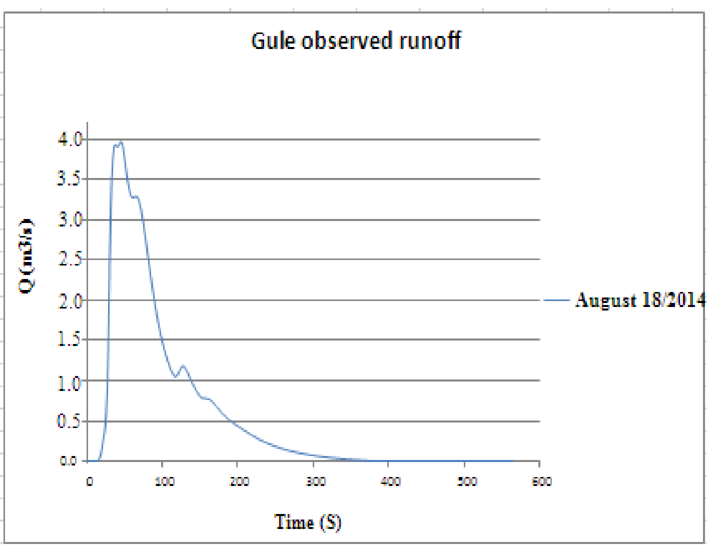


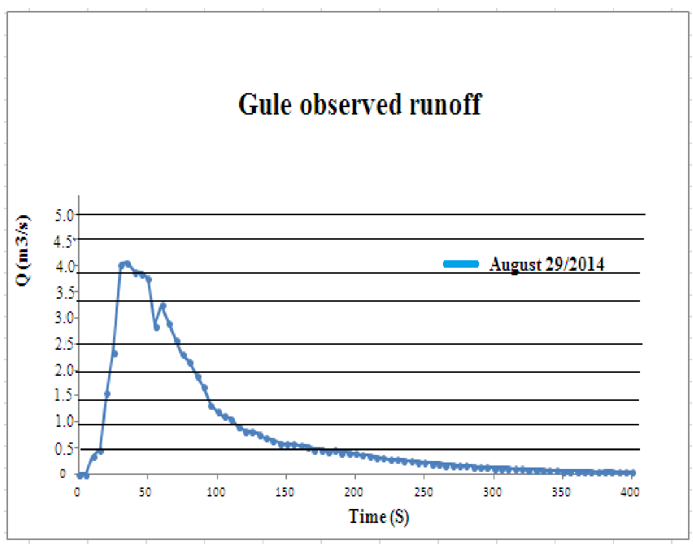

Gule observed runoff
Q (m3/s)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0
50
100
150
200
250
Time (S)
August 05/2014

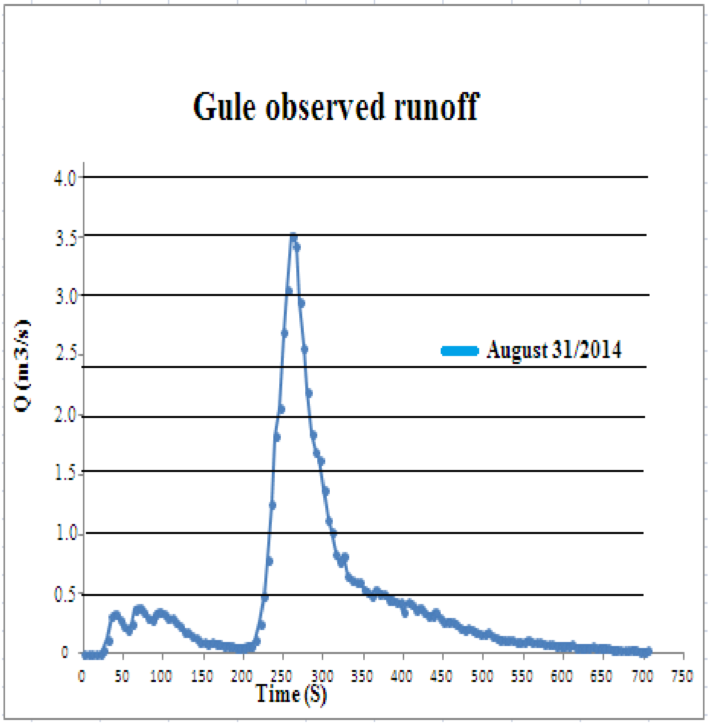

Gule observed runoff
Q (m3/s)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
500
1000
1500
2000
Time (S)
August 09/2014

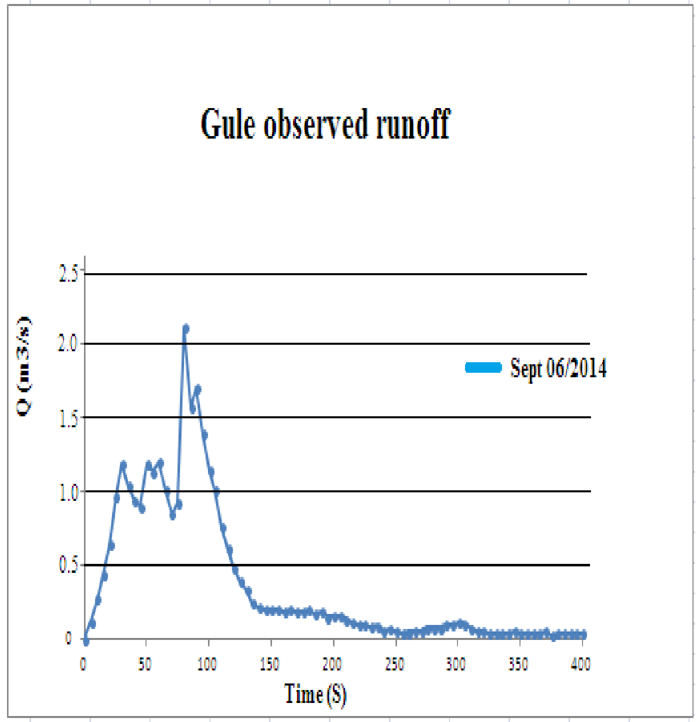

Gule observed runoff
Q (m3/s)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
500
1000
1500
2000
Time (S)
August 16/2014

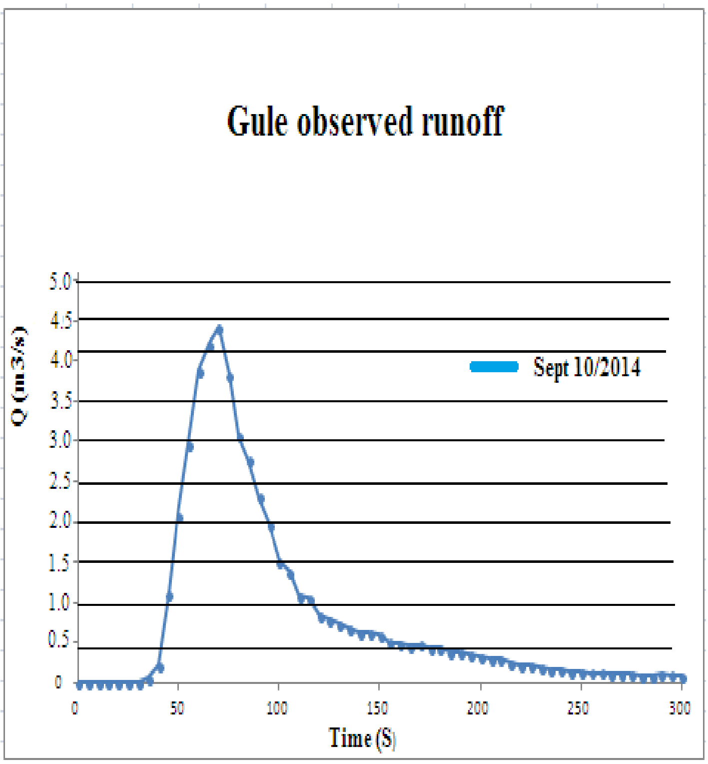

Gule observed runoff
Q (m3/s)
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0
50
100
150
200
250
300
350
400
450
500
550
600
650
Time (S)
Sept 03/2014


Gule observed runoff
Q (m3/s)
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
100
200
300
400
500
600
Time (S)
August 18/2014


Gule observed runoff
Q (m3/s)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0
0
50
100
150
200
250
300
350
400
Time (S)
August 29/2014


Gule observed runoff
Q (m3/s)
August 31/2014
Time (S)


Gule observed runoff
Q (m3/s)
Sept 06/2014
Time (S)


Gule observed runoff
Q (m3/s)
Sept 10/2014
Time (S)

Appendix B: Hydrologic outputs and data used

Table B-1: Morphologic parameters and soil types of all Culvert sites

<table>
<tr><th>Sr. No.</th><th colspan="2">Parameters</th><th>Values</th></tr>
<tr><td>1</td><td colspan="2">Stream Number (Nu)</td><td>Culvert Site 1</td></tr>
<tr><td>2</td><td colspan="2">Longest Flow Path (L)</td><td>0.64 km</td></tr>
<tr><td>3</td><td colspan="2">Stream Length Ratio (Rl)</td><td>0.64</td></tr>
<tr><td>4</td><td colspan="2">Perimeter of Watershed (P)</td><td>3.15 km</td></tr>
<tr><td>5</td><td colspan="2">Area of Watershed (A)</td><td>57.3 Ha (0.573 km2)</td></tr>
<tr><td>6</td><td colspan="2">Elongation Ratio (Re)</td><td>0.52</td></tr>
<tr><td>7</td><td colspan="2">Length of overland flow (Lo)</td><td>0.064km/km2</td></tr>
<tr><td>8</td><td colspan="2">Stream Channel Slope (S)</td><td>4.1 %</td></tr>
<tr><td>9</td><td colspan="2">Max. Length of Watershed (Lb)</td><td>0.84 km</td></tr>
<tr><td rowspan="3">10</td><td rowspan="3">Percentage of soils texture in the culvert site</td><td>Silt loam</td><td>56.4 %</td></tr>
<tr><td>Silt clay</td><td>32.8 %</td></tr>
<tr><td>Sandy loam</td><td>10.8 %</td></tr>
<tr><th>Sr. No.</th><th colspan="2">Parameters</th><th>Values</th></tr>
<tr><td>1</td><td colspan="2">Stream Number (Nu)</td><td>Culvert Site 2</td></tr>
<tr><td>2</td><td colspan="2">Longest Flow Path (L)</td><td>1.12 km</td></tr>
<tr><td>3</td><td colspan="2">Stream Length Ratio (Rl)</td><td>0.69</td></tr>
<tr><td>4</td><td colspan="2">Perimeter of Watershed (P)</td><td>4.67 km</td></tr>
<tr><td>5</td><td colspan="2">Area of Watershed (A)</td><td>115.6 Ha (1.156 km2)</td></tr>
<tr><td>6</td><td colspan="2">Elongation Ratio (Re)</td><td>0.62</td></tr>
<tr><td>7</td><td colspan="2">Length of overland flow (Lo)</td><td>0.072km/km2</td></tr>
<tr><td>8</td><td colspan="2">Stream Channel Slope (S)</td><td>4.3 %</td></tr>
<tr><td>9</td><td colspan="2">Max. Length of Watershed (Lb)</td><td>1.12 km</td></tr>
</table>

10	Percentage of soils texture in the culvert site	Silt loam	60.8 %
		Silt clay	20 .1 %
		Sandy loam	19.1 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 3**
2	Longest Flow Path (L)		0.655 km
3	Stream Length Ratio (Rl)		0.67
4	Perimeter of Watershed (P)		4.1 km
5	Area of Watershed (A)		74.1Ha (0.741 km2)
6	Elongation Ratio (Re)		0.59
7	Length of overland flow (Lo)		0.066km/km2
8	Stream Channel Slope (S)		3.9 %
9	Max. Length of Watershed (Lb)		0.76 km
10	Percentage of soils texture in the culvert site	Silt loam	58.5 %
		Silt clay	32.7 %
		Sandy loam	8.80 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 4**
2	Longest Flow Path (L)		0.41 km
3	Stream Length Ratio (Rl)		0.49
4	Perimeter of Watershed (P)		2.1 km
5	Area of Watershed (A)		17.4Ha (0.174 km2)
6	Elongation Ratio (Re)		0.47
7	Length of overland flow (Lo)		0.054km/km2
8	Stream Channel Slope (S)		4.0 %

9	Max. Length of Watershed (Lb)		0.29 km
10	Percentage of soils texture in the culvert site	Silt loam	44.9 %
		Silt clay	37.8 %
		Sandy loam	17.3 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 5**
2	Longest Flow Path (L)		0.71 km
3	Stream Length Ratio (Rl)		0.48
4	Perimeter of Watershed (P)		0.29 km
5	Area of Watershed (A)		46.8Ha (0.468 km2)
6	Elongation Ratio (Re)		0.58
7	Length of overland flow (Lo)		0.061km/km2
8	Stream Channel Slope (S)		3.95 %
9	Max. Length of Watershed (Lb)		0.66 km
10	Percentage of soils texture in the culvert site	Silt loam	34.8 %
		Silt clay	26.3 %
		Sandy loam	38.9 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 6**
2	Longest Flow Path (L)		0.57 km
3	Stream Length Ratio (Rl)		0.51
4	Perimeter of Watershed (P)		0.315 km
5	Area of Watershed (A)		37.4Ha (0.374 km2)
6	Elongation Ratio (Re)		0.38
7	Length of overland flow (Lo)		0.056km/km2

8	Stream Channel Slope (S)		2.6 %
9	Max. Length of Watershed (Lb)		0.74 km
10	Percentage of soils texture in the culvert site	Silt loam	40.4 %
		Silt clay	39.2 %
		Sandy loam	20.4 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 7**
2	Longest Flow Path (L)		0.49 km
3	Stream Length Ratio (Rl)		0.54
4	Perimeter of Watershed (P)		0.28km
5	Area of Watershed (A)		39.5Ha (0.395 km2)
6	Elongation Ratio (Re)		0.36
7	Length of overland flow (Lo)		0.045km/km2
8	Stream Channel Slope (S)		3.0 %
9	Max. Length of Watershed (Lb)		0.71 km
10	Percentage of soils texture in the culvert site	Silt loam	56.8 %
		Silt clay	17.6 %
		Sandy loam	25.6 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 8**
2	Longest Flow Path (L)		1.7 km
3	Stream Length Ratio (Rl)		0.81
4	Perimeter of Watershed (P)		5.28 km
5	Area of Watershed (A)		347.0 Ha (3.470 km2)
6	Elongation Ratio (Re)		0.74

7	Length of overland flow (Lo)		0.075km/km2
8	Stream Channel Slope (S)		0.48 %
9	Max. Length of Watershed (Lb)		2.15 km
10	Percentage of soils texture in the culvert site	Silt loam	50.0 %
		Silt clay	28.4 %
		Sandy loam	21.6 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 9**
2	Longest Flow Path (L)		0.71km
3	Stream Length Ratio (Rl)		0.7
4	Perimeter of Watershed (P)		4.0 km
5	Area of Watershed (A)		85.6 Ha (0.856 km2)
6	Elongation Ratio (Re)		0.54
7	Length of overland flow (Lo)		0.06km/km2
8	Stream Channel Slope (S)		3.45 %
9	Max. Length of Watershed (Lb)		0.78 km
10	Percentage of soils texture in the culvert site	Silt loam	37.7 %
		Silt clay	32.9 %
		Sandy loam	29.4 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 10**
2	Longest Flow Path (L)		0.50 km
3	Stream Length Ratio (Rl)		0.47
4	Perimeter of Watershed (P)		0.29 km
5	Area of Watershed (A)		28.7 Ha (0.287 km2)

6	Elongation Ratio (Re)		0.40
7	Length of overland flow (Lo)		0.057km/km2
8	Stream Channel Slope (S)		2.9 %
9	Max. Length of Watershed (Lb)		0.59 km
10	Percentage of soils texture in the culvert site	Silt loam	41.4 %
		Silt clay	28.2 %
		Sandy loam	30.4 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 11**
2	Longest Flow Path (L)		0.61 km
3	Stream Length Ratio (Rl)		0.59
4	Perimeter of Watershed (P)		3.05 km
5	Area of Watershed (A)		50.3 Ha (0.503 km2)
6	Elongation Ratio (Re)		0.52
7	Length of overland flow (Lo)		0.062km/km2
8	Stream Channel Slope (S)		4.15 %
9	Max. Length of Watershed (Lb)		0.76 km
10	Percentage of soils texture in the culvert site	Silt loam	56.3 %
		Silt clay	39.4 %
		Sandy loam	4.30 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 12**
2	Longest Flow Path (L)		0.7 km
3	Stream Length Ratio (Rl)		0.68
4	Perimeter of Watershed (P)		4.0 km

5	Area of Watershed (A)		76.0 Ha (0.76 km2)
6	Elongation Ratio (Re)		0.62
7	Length of overland flow (Lo)		0.071km/km2
8	Stream Channel Slope (S)		3.65 %
9	Max. Length of Watershed (Lb)		0.81 km
10	Percentage of soils texture in the culvert site	Silt loam	44.6 %
		Silt clay	35.5 %
		Sandy loam	19.9 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 13**
2	Longest Flow Path (L)		0.64 km
3	Stream Length Ratio (Rl)		0.58
4	Perimeter of Watershed (P)		2.9 km
5	Area of Watershed (A)		49.9 Ha (0.499 km2)
6	Elongation Ratio (Re)		0.50
7	Length of overland flow (Lo)		0.059km/km2
8	Stream Channel Slope (S)		4.4 %
9	Max. Length of Watershed (Lb)		0.68 km
10	Percentage of soils texture in the culvert site	Silt loam	24.8 %
		Silt clay	30.7 %
		Sandy loam	44.5 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 14**
2	Longest Flow Path (L)		0.47 km
3	Stream Length Ratio (Rl)		0.57

4	Perimeter of Watershed (P)		0.26km
5	Area of Watershed (A)		37.5 Ha (0.375 km2)
6	Elongation Ratio (Re)		0.41
7	Length of overland flow (Lo)		0.049km/km2
8	Stream Channel Slope (S)		3.5 %
9	Max. Length of Watershed (Lb)		0.59 km
10	Percentage of soils texture in the culvert site	Silt loam	61.4 %
		Silt clay	24.6 %
		Sandy loam	14.0 %
Sr. No.	**Parameters**		**Values**
1	Stream Number (Nu)		**Culvert Site 15**
2	Longest Flow Path (L)		0.44 km
3	Stream Length Ratio (Rl)		0.52
4	Perimeter of Watershed (P)		2.45 km
5	Area of Watershed (A)		19.5 Ha (0.195 km2)
6	Elongation Ratio (Re)		0.42
7	Length of overland flow (Lo)		0.051km/km2
8	Stream Channel Slope (S)		3.3 %
9	Max. Length of Watershed (Lb)		0.33 km
10	Percentage of soils texture in the culvert site	Silt loam	41.2 %
		Silt clay	38.4 %
		Sandy loam	20.4 %

Appendix C: Graphs and Outputs obtained from Hydrologic Events

Figure C-1: Comparison of runoff hydrograph for 1st event (5th July, 2014)

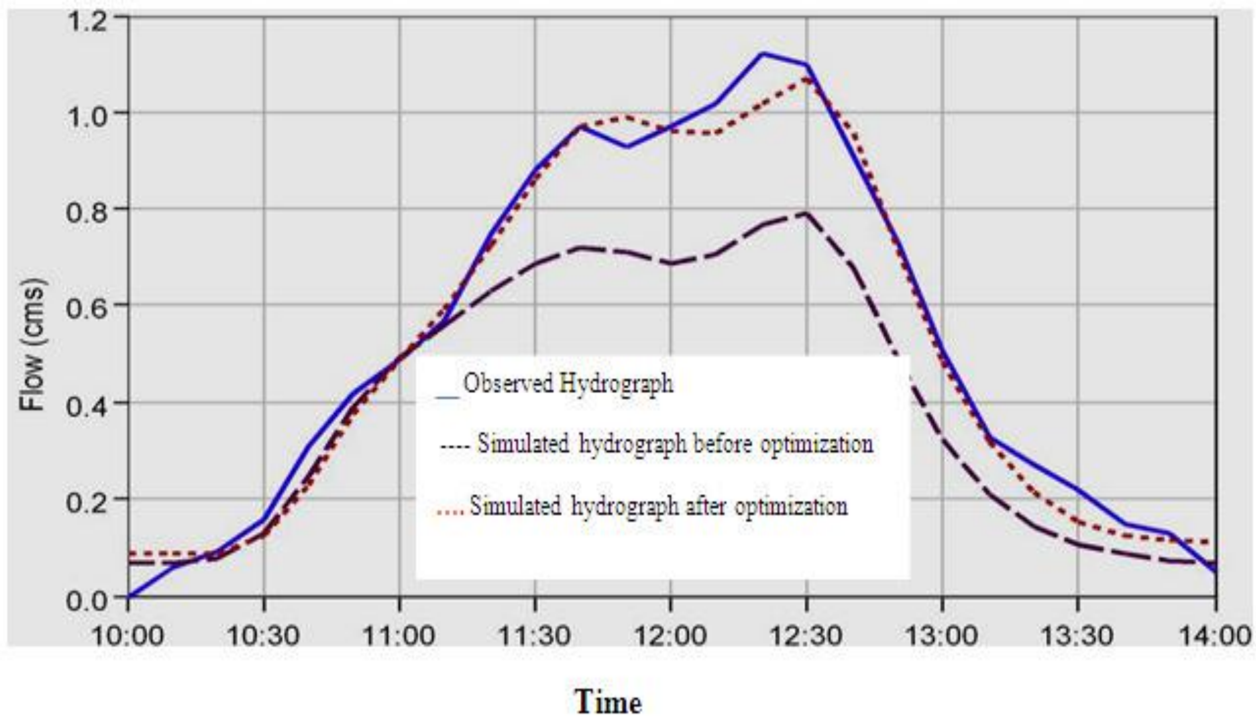


Figure C-2: Comparison of runoff hydrograph for 3rd event (30th July, 2014)

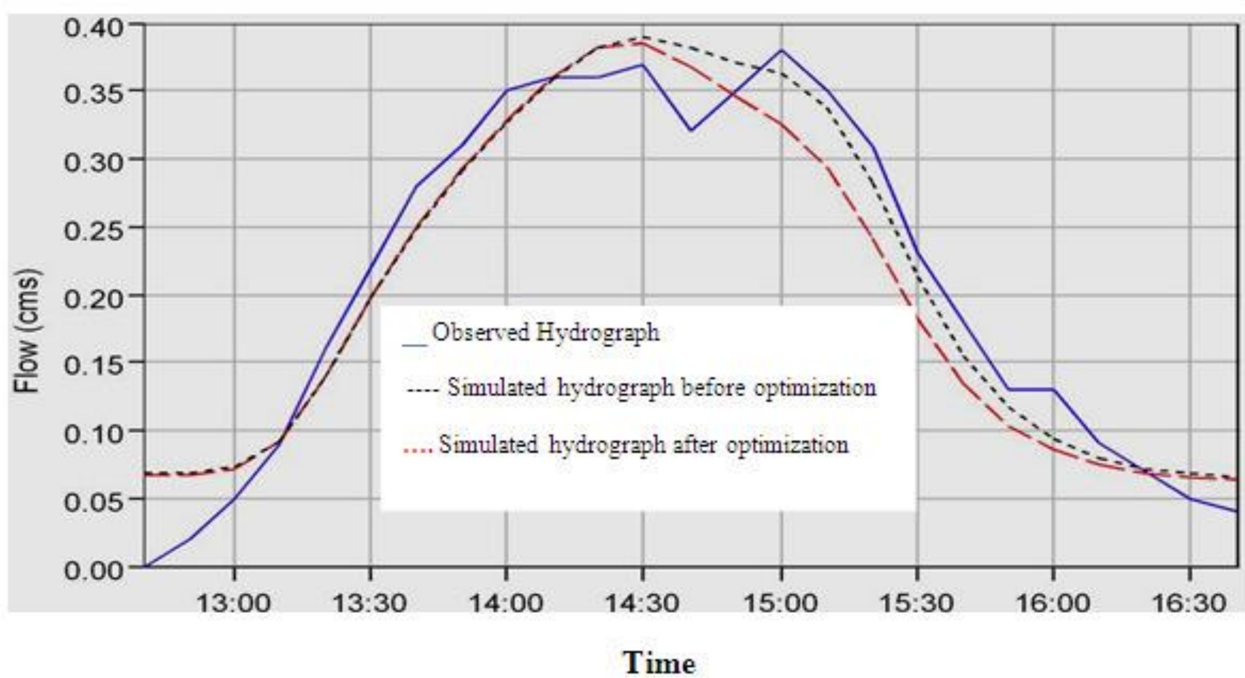


Figure C-3: Comparison of runoff hydrograph for 7th event (5th August, 2014)

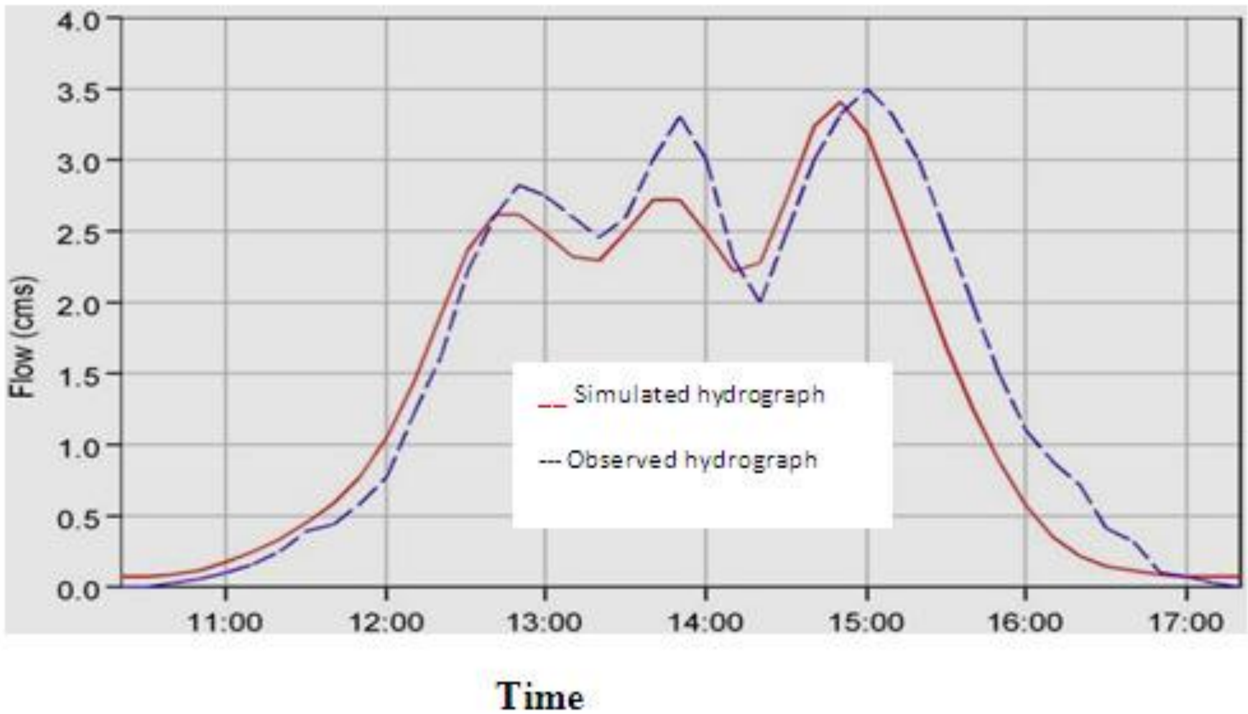
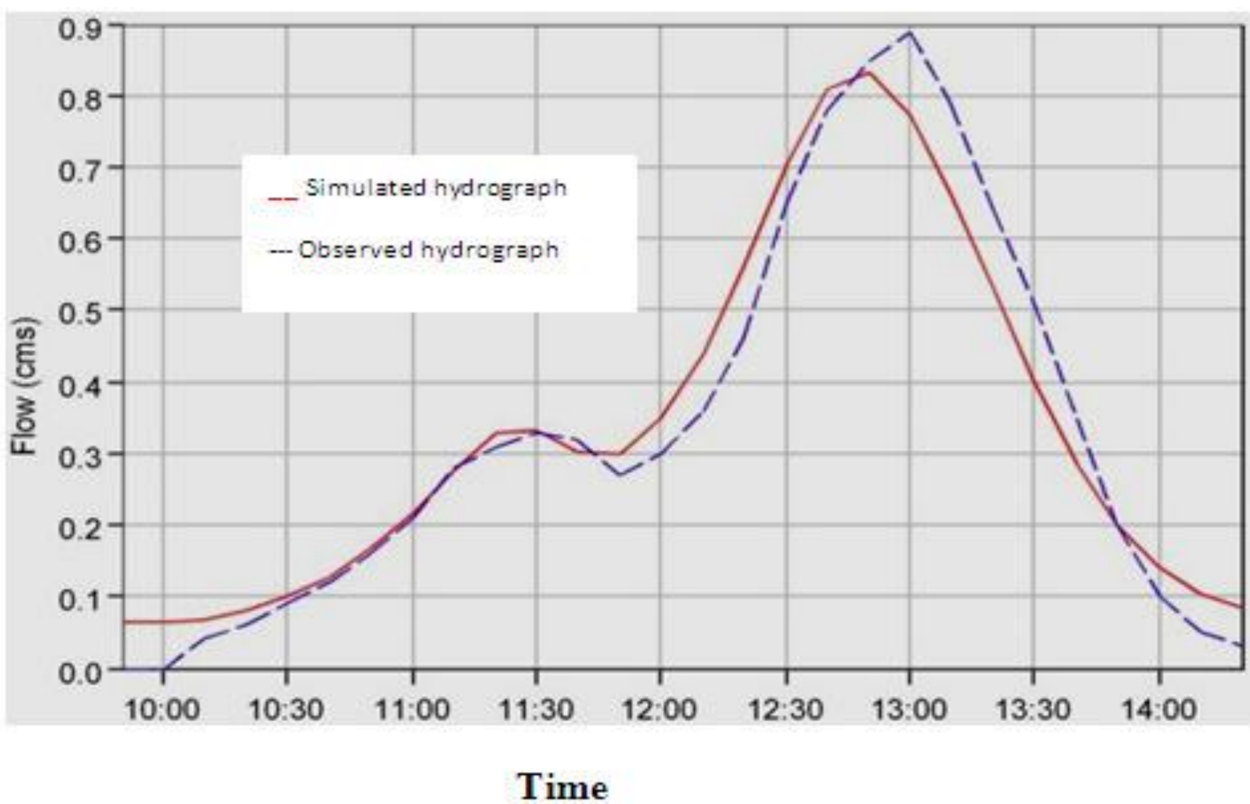


Figure C-4: Comparison of runoff hydrograph for 9th event (16th August, 2014)



Appendix D: Hydraulic Events, Outputs and Rating curves

Figure D-1: Culvert site 1 rating curve generated with HEC-RAS

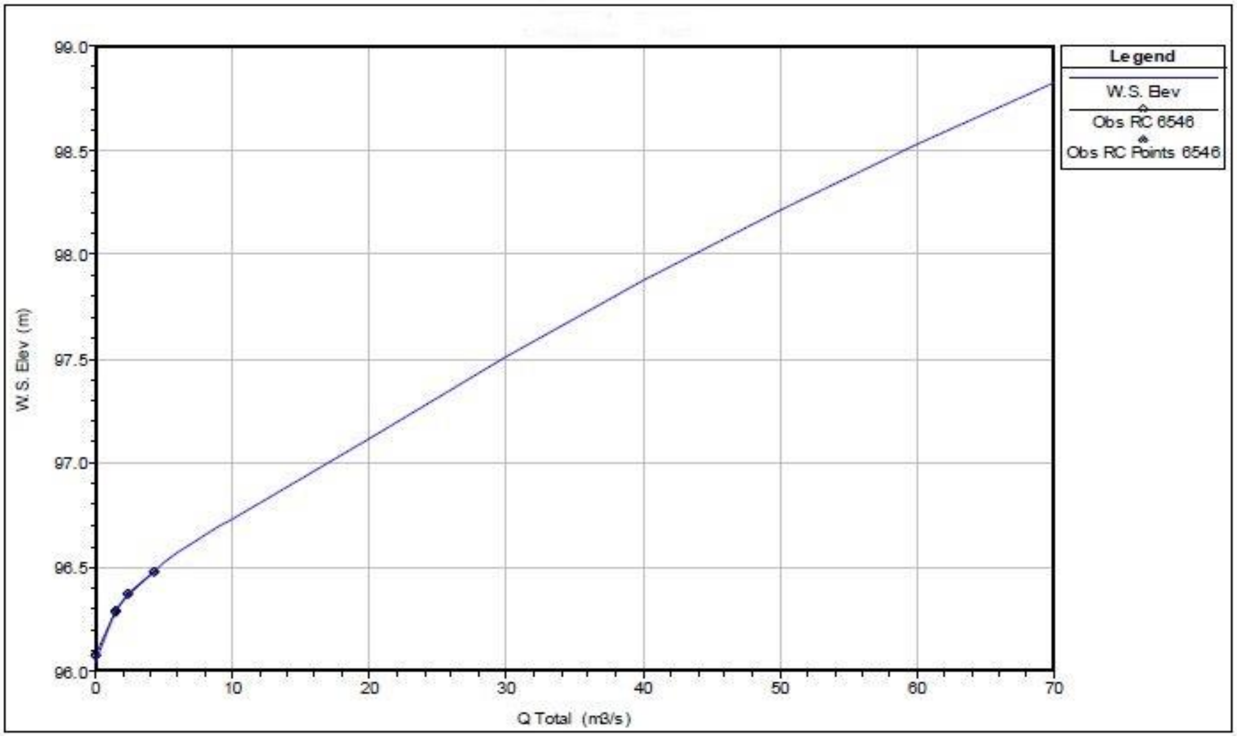


Figure D-2: Q/h relation – measured, manually extrapolated and modeled data for Gule

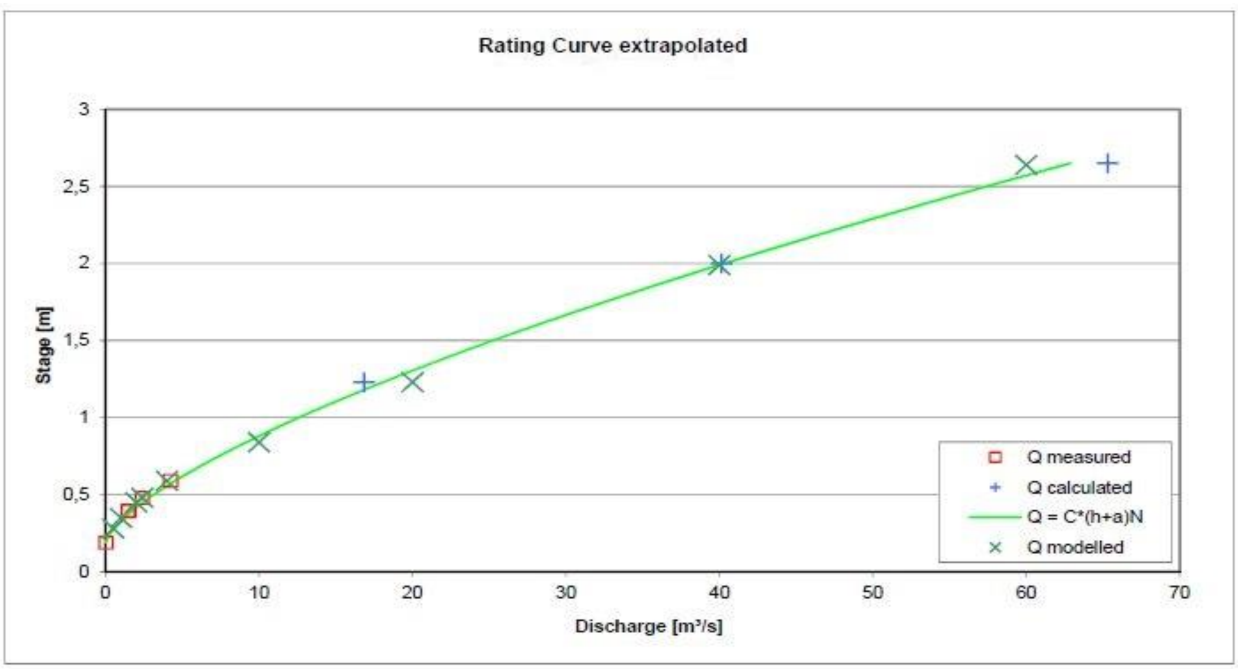


Figure D-3: Culvert site 2 Q/h relation – historic and new data

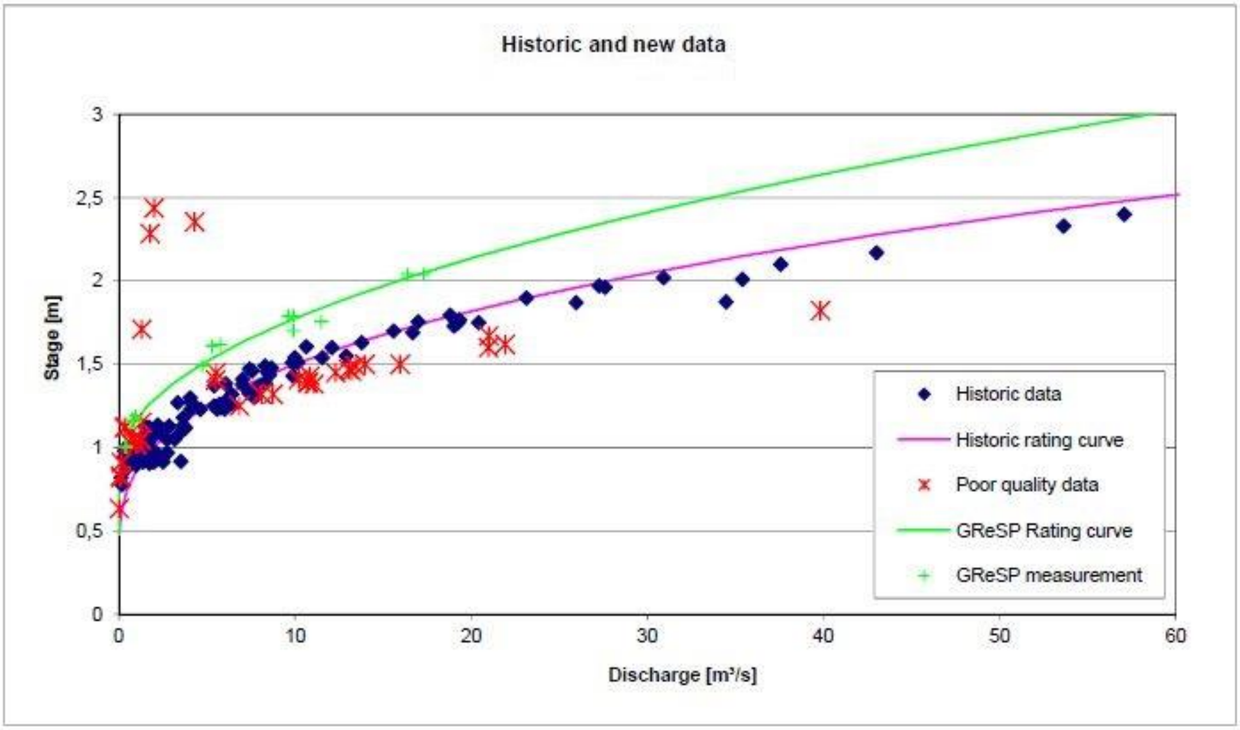


Figure D-4: Rating curve for Culvert site 3 from measured data

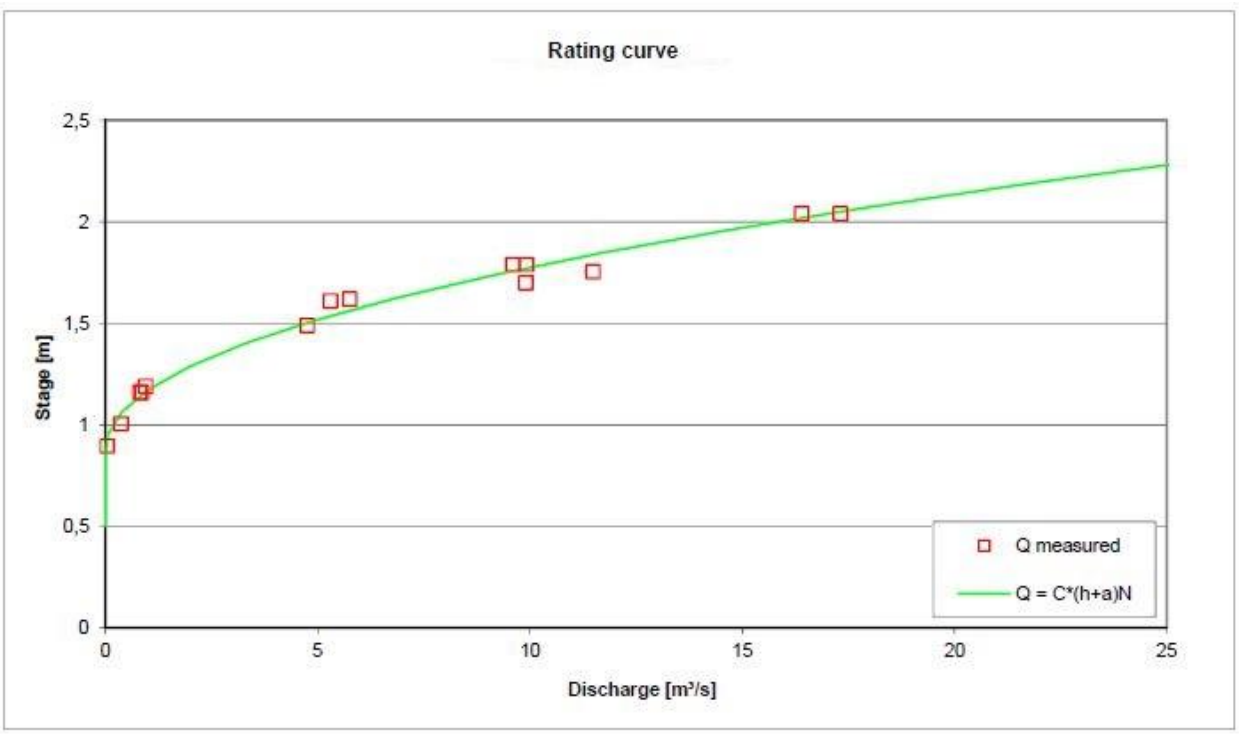


Figure D-5: Historic and new data from Culvert site 4

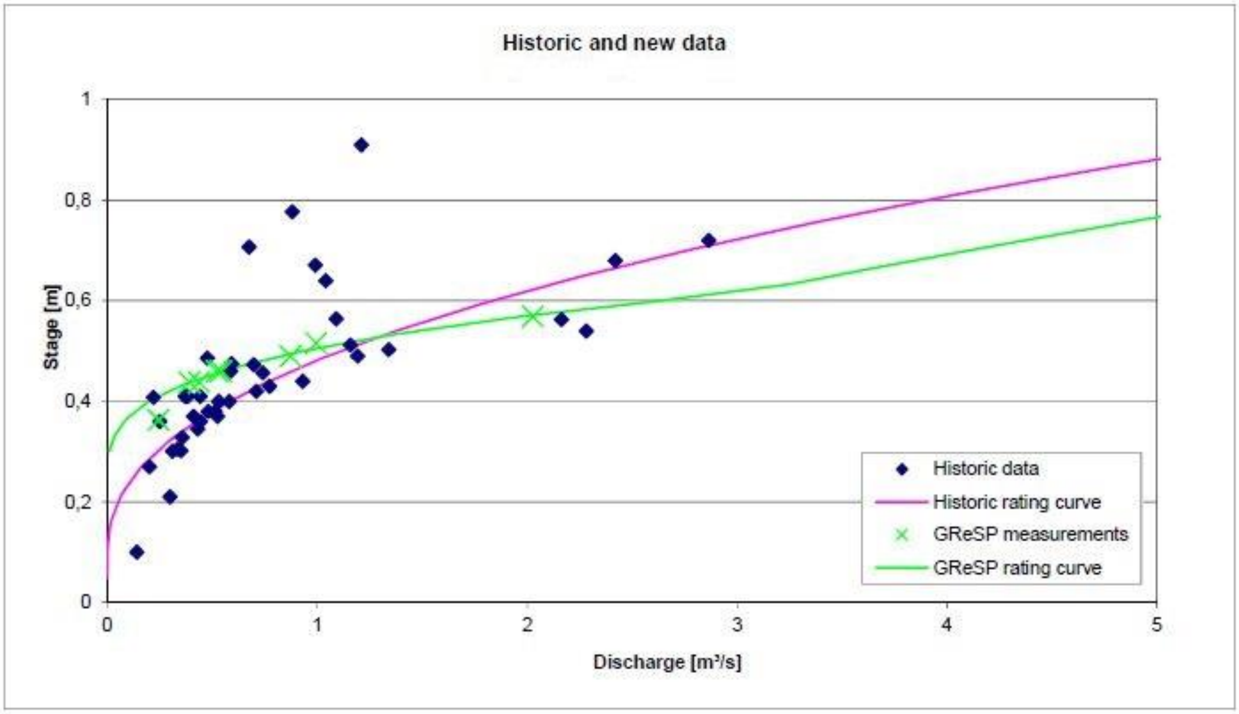


Figure D-6: Q/h relation - measured and calculated data at Culvert site 5 river

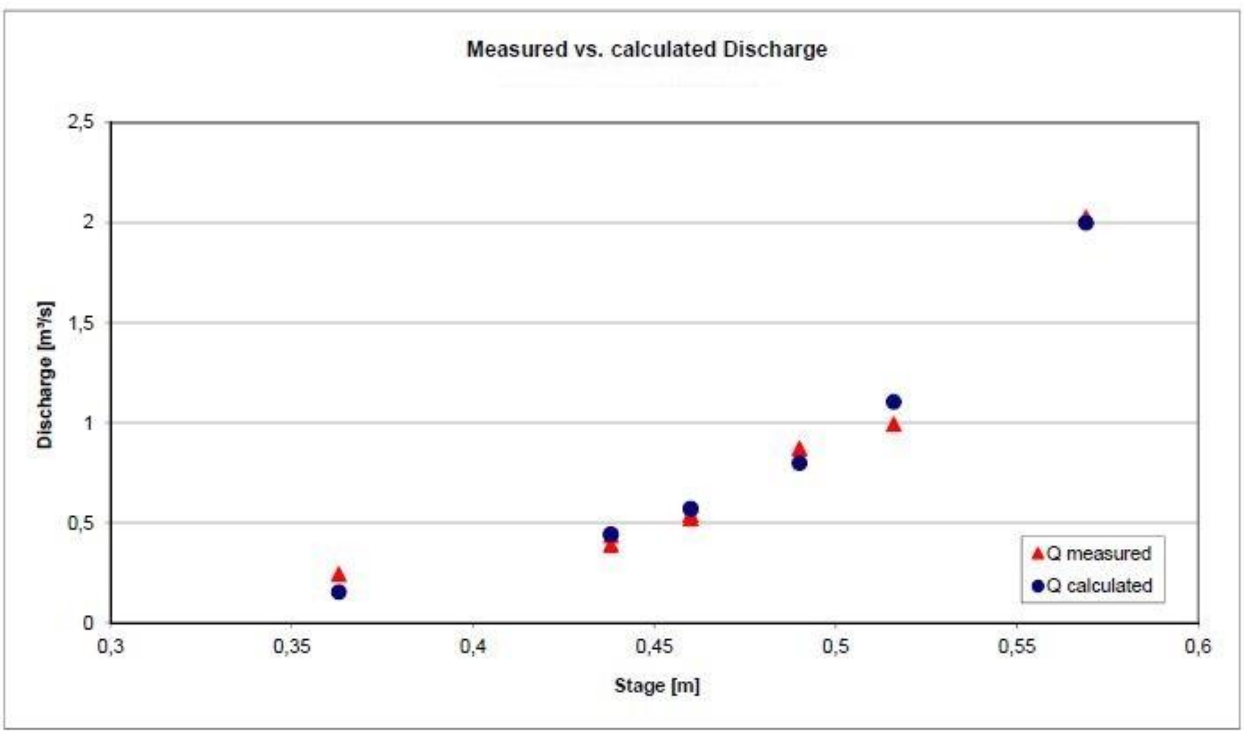


Figure D-7: Rating Curve for Culvert site 6 generated with HEC-RAS

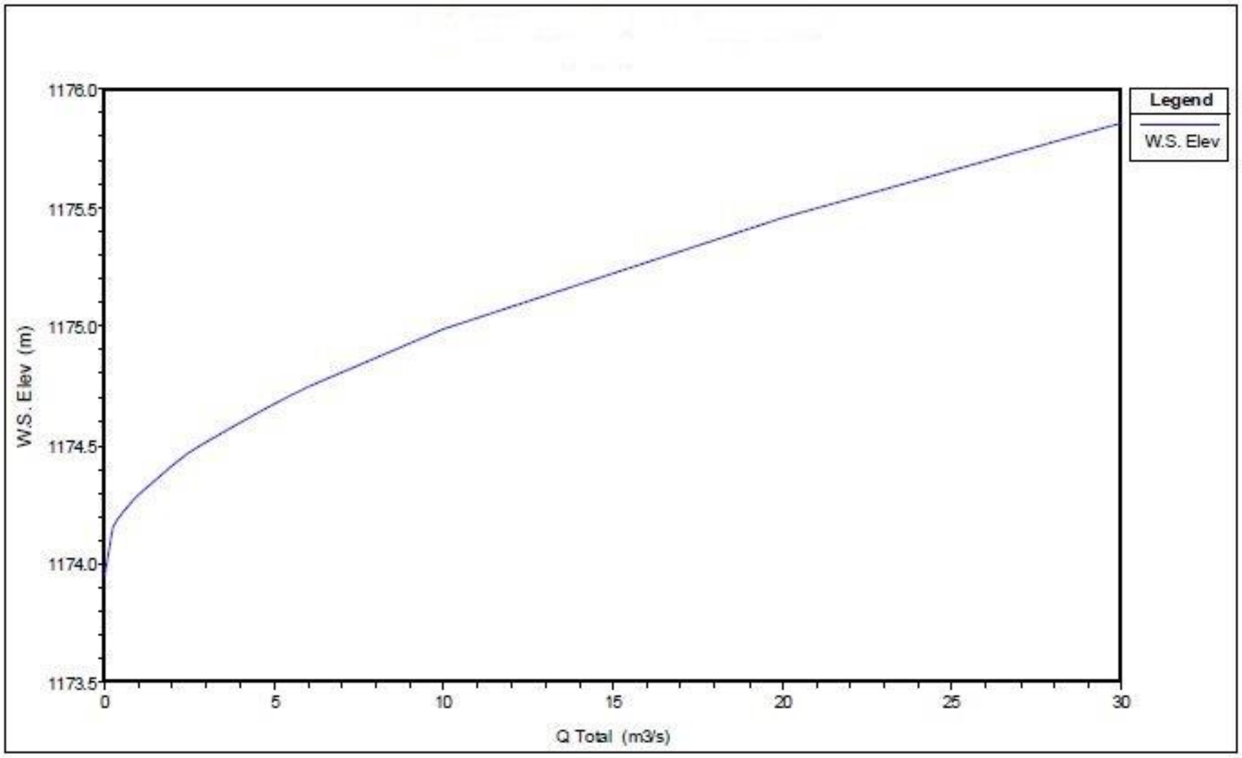


Figure D-8: Rating Curve for Culvert site 7

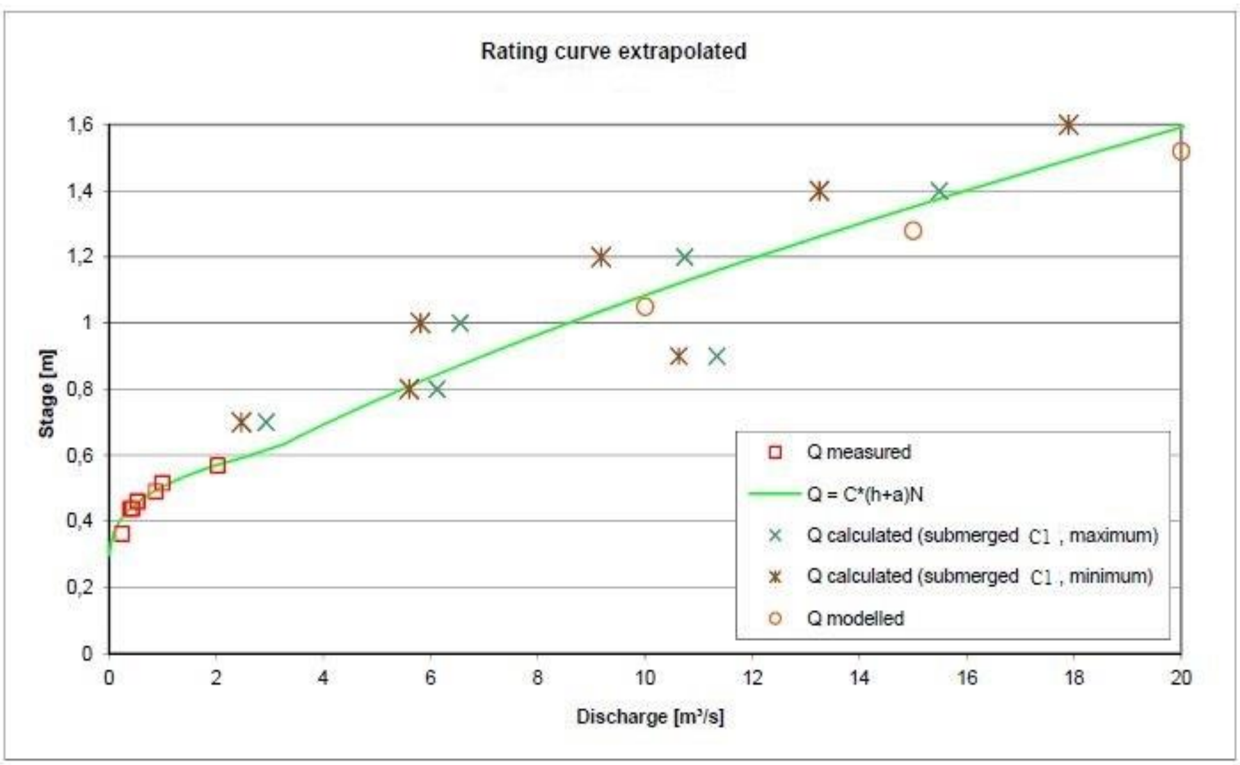


Figure D-9: Historic and new data from Culvert site 8

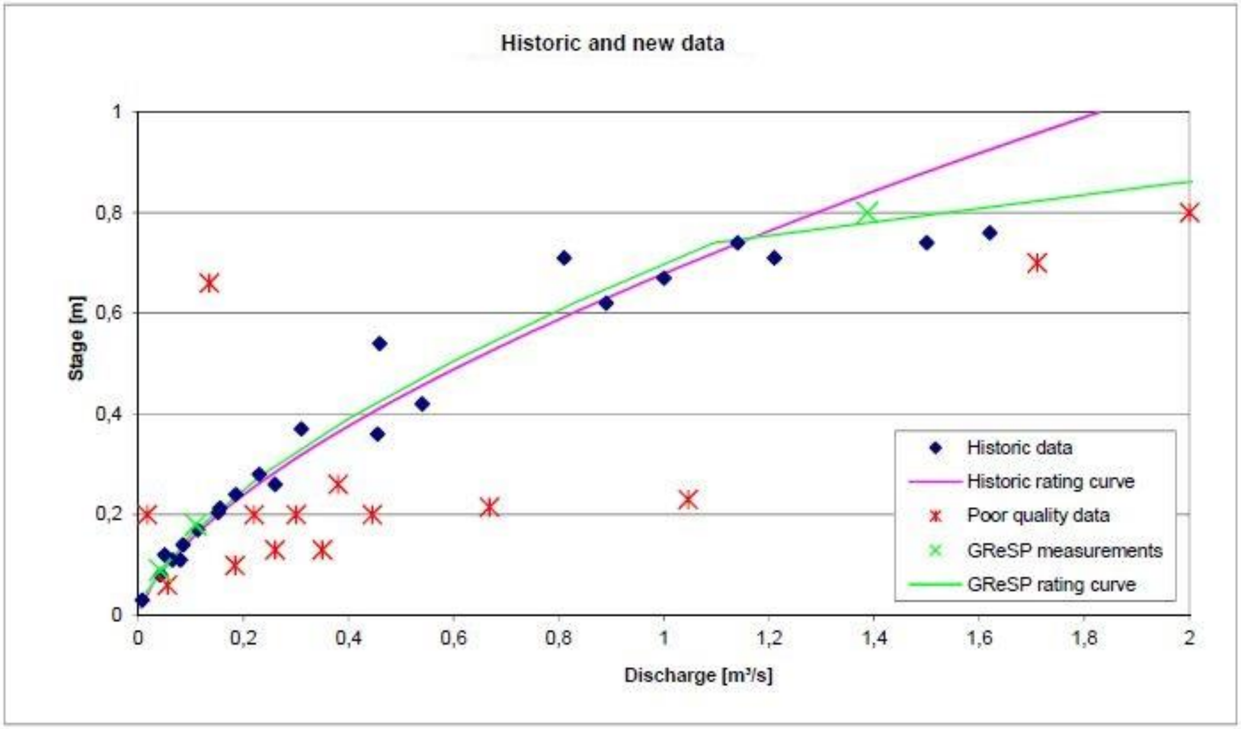


Figure D-10: Q/h relation - measured and calculated data at Culvert site 9

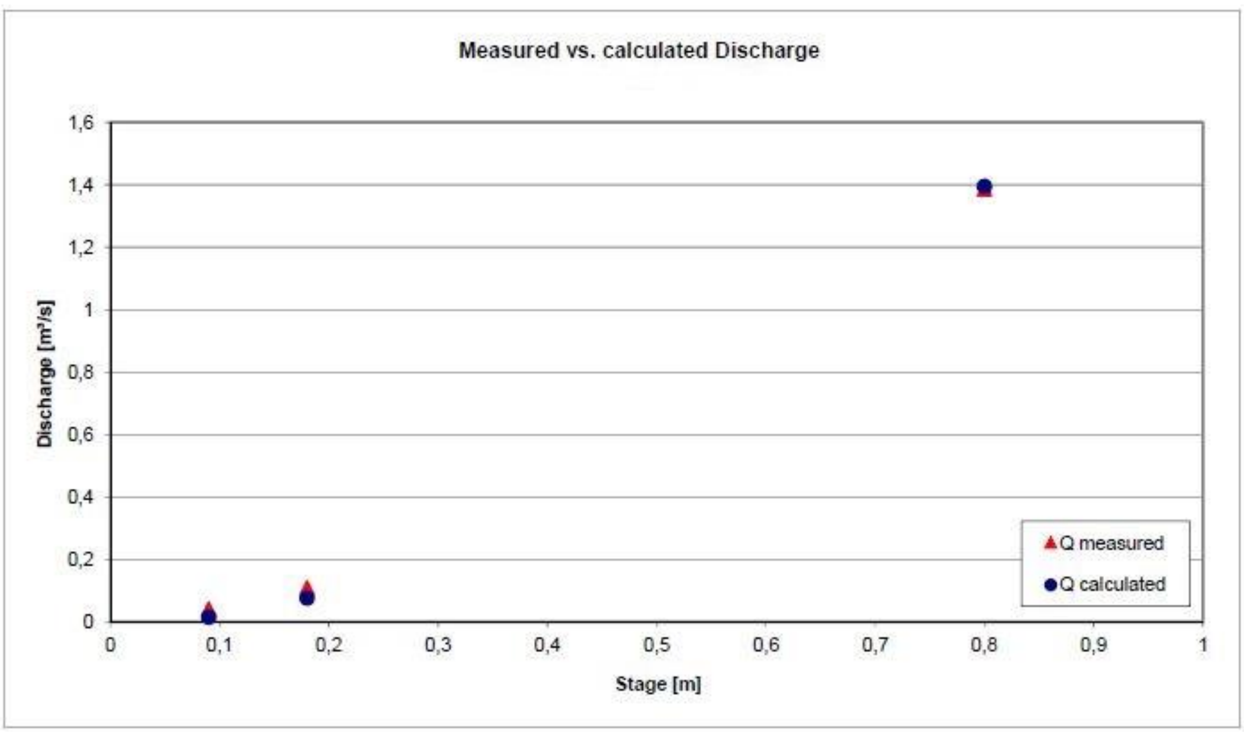


Figure D-11: Rating curve for Culvert site 10 generated with HEC-RAS from modeled data

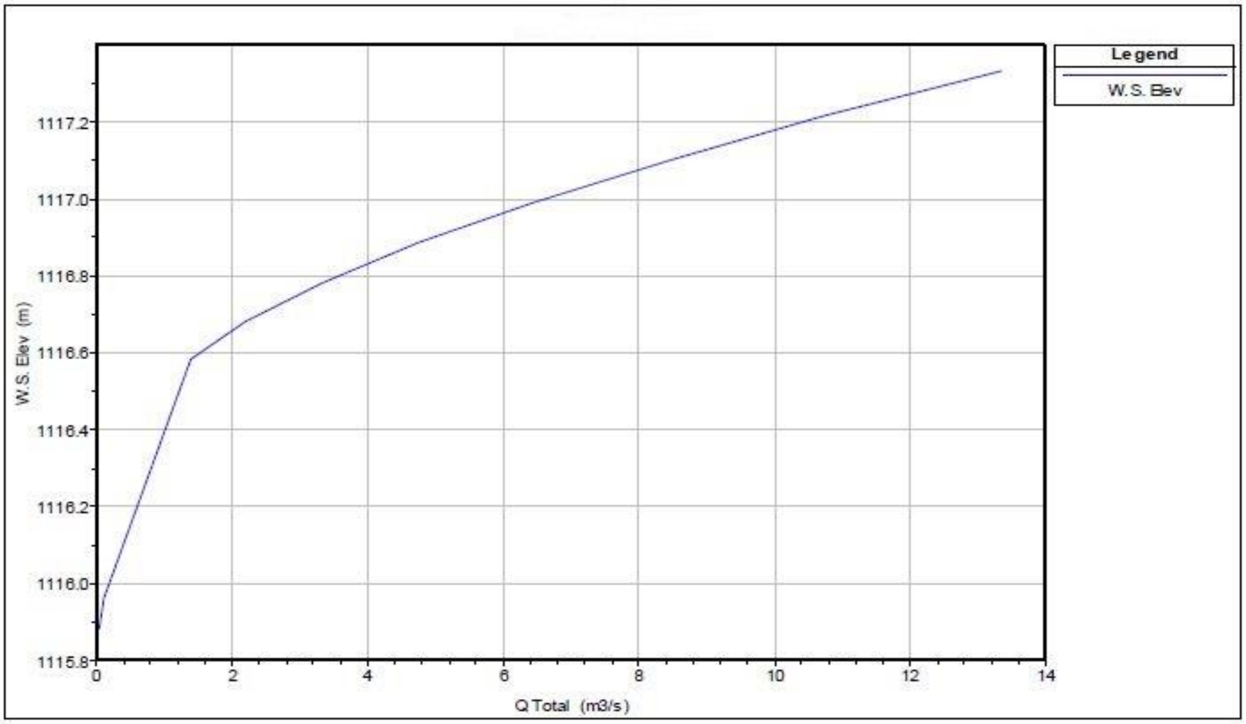


Figure D-12: Rating curve for Culvert site 11

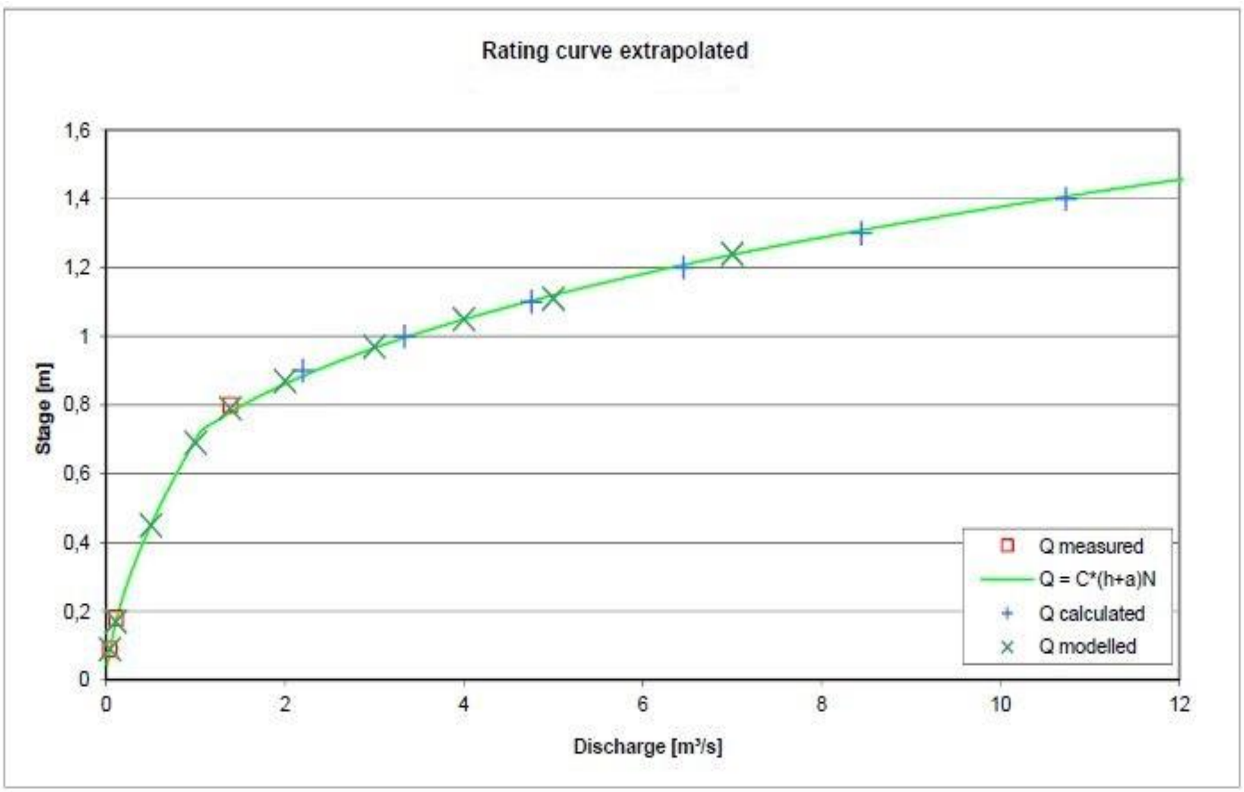


Figure D-13: Q/h relation at Gule gauging station

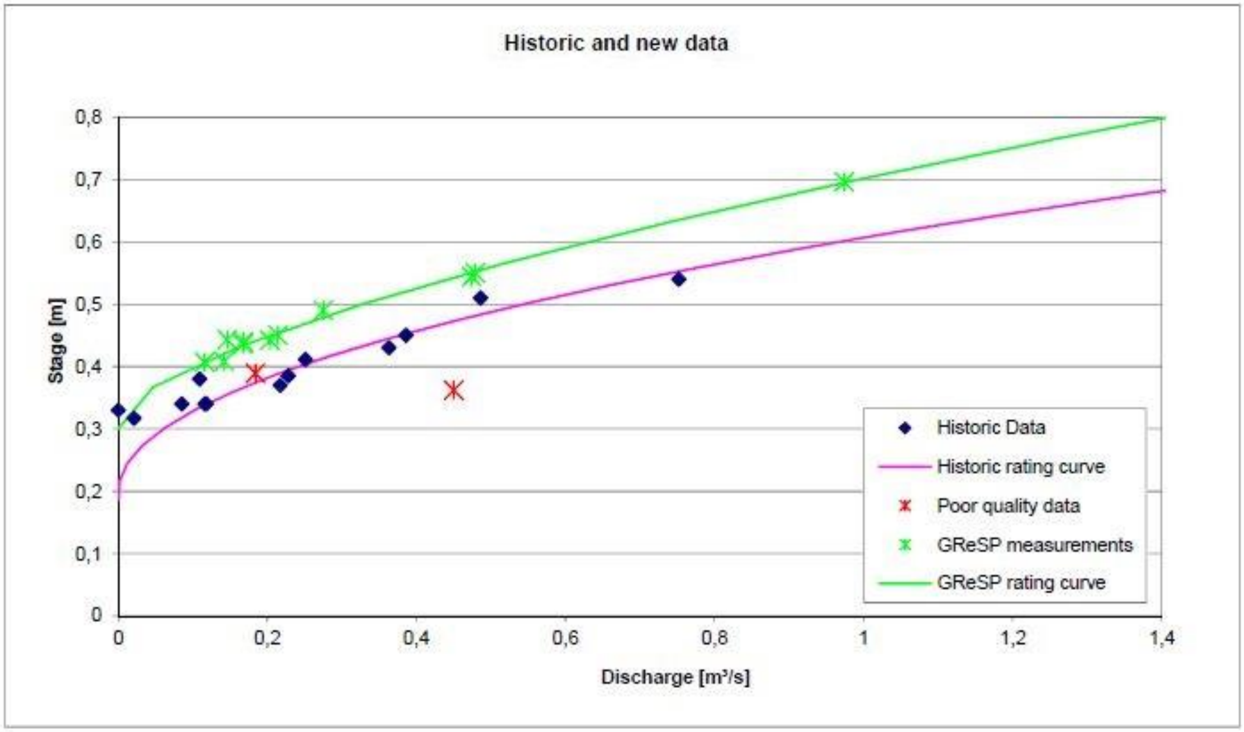


Figure D-14: Rating curve for Culvert site 12 generated with HEC-RAS from modeled data

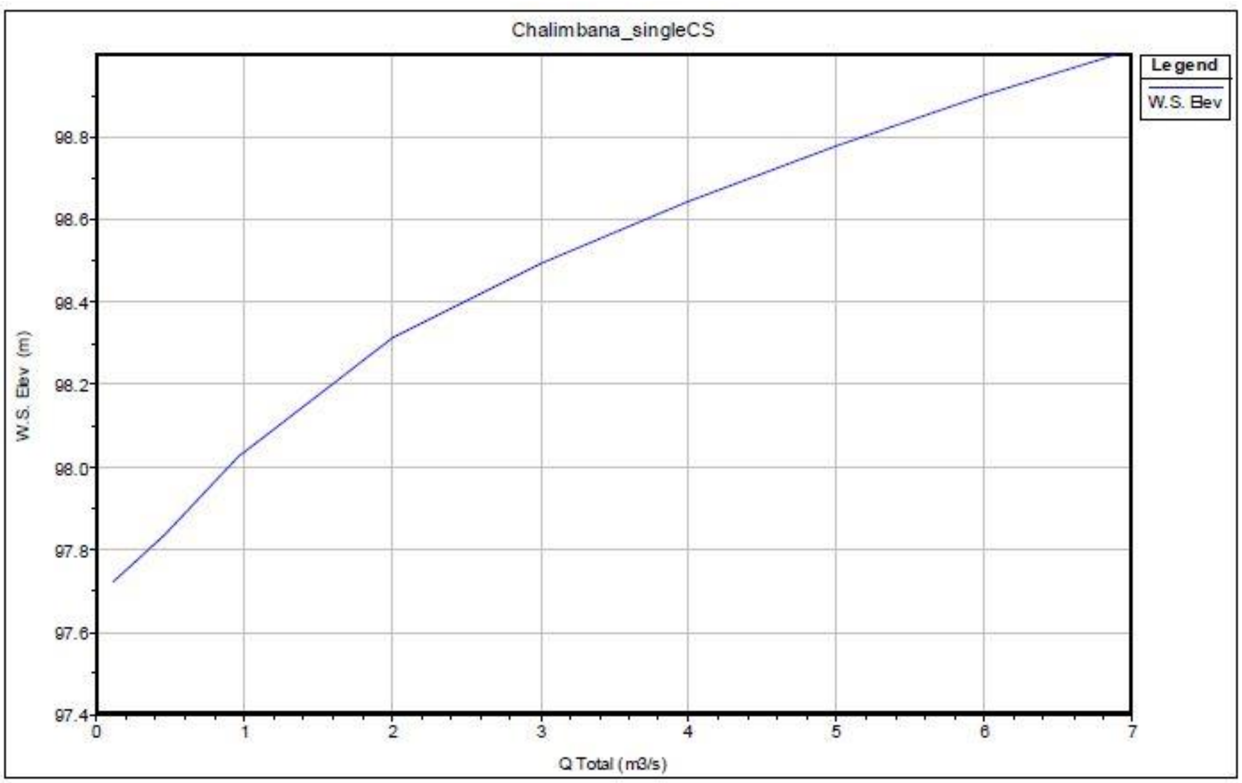


Figure D-15: Rating curve Culvert site 13 generated from measured and modeled data

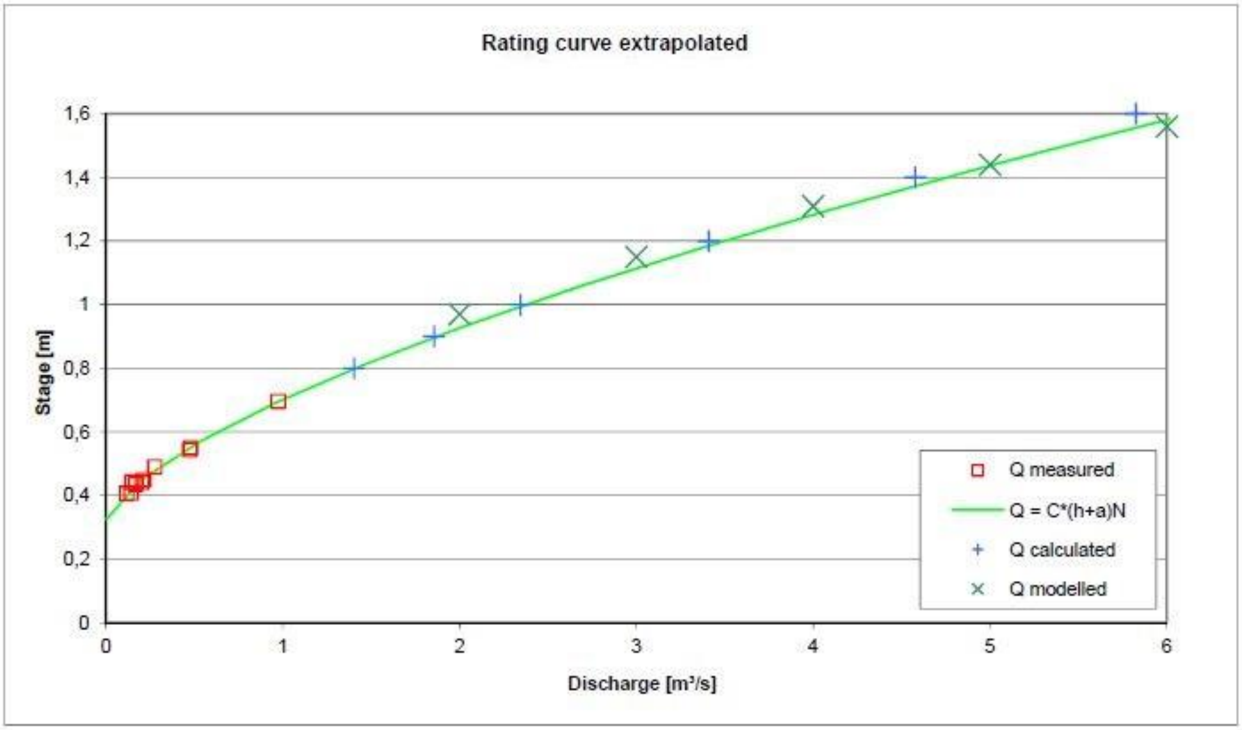


Figure D-16: Historic and new data at Gule gauging station

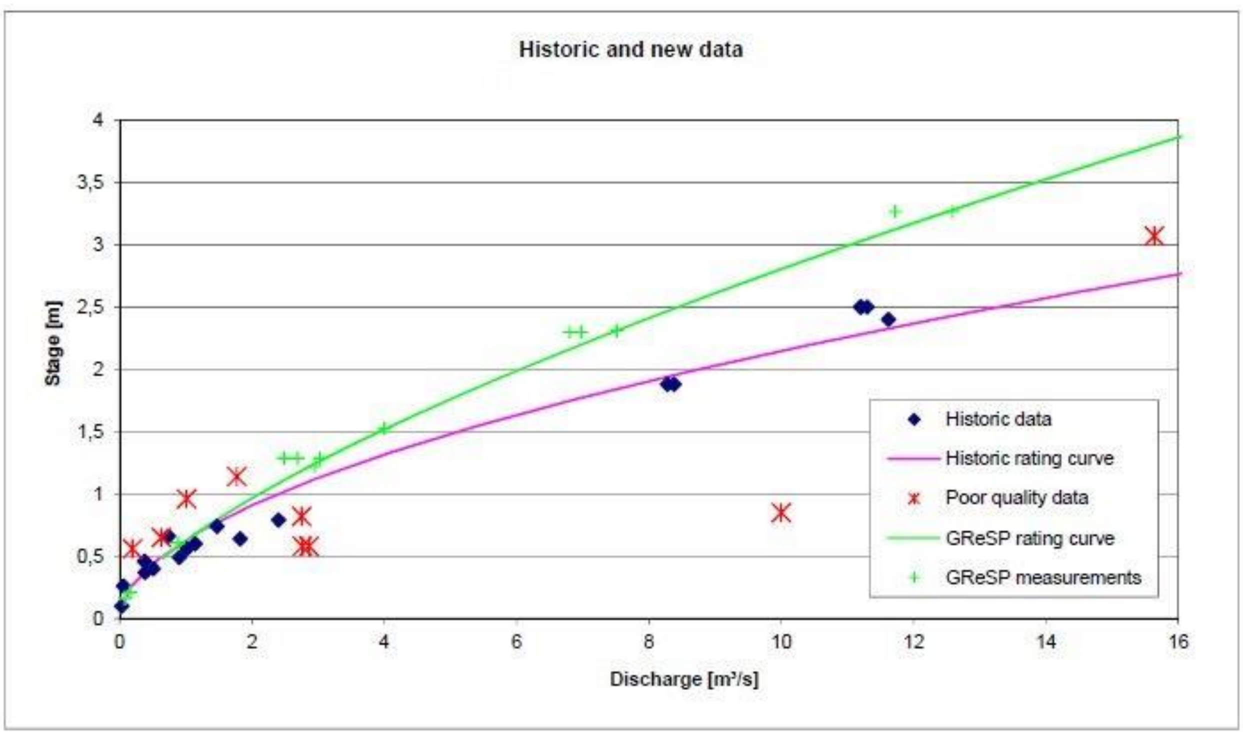


Figure D-17: Rating curve for Culvert site 14

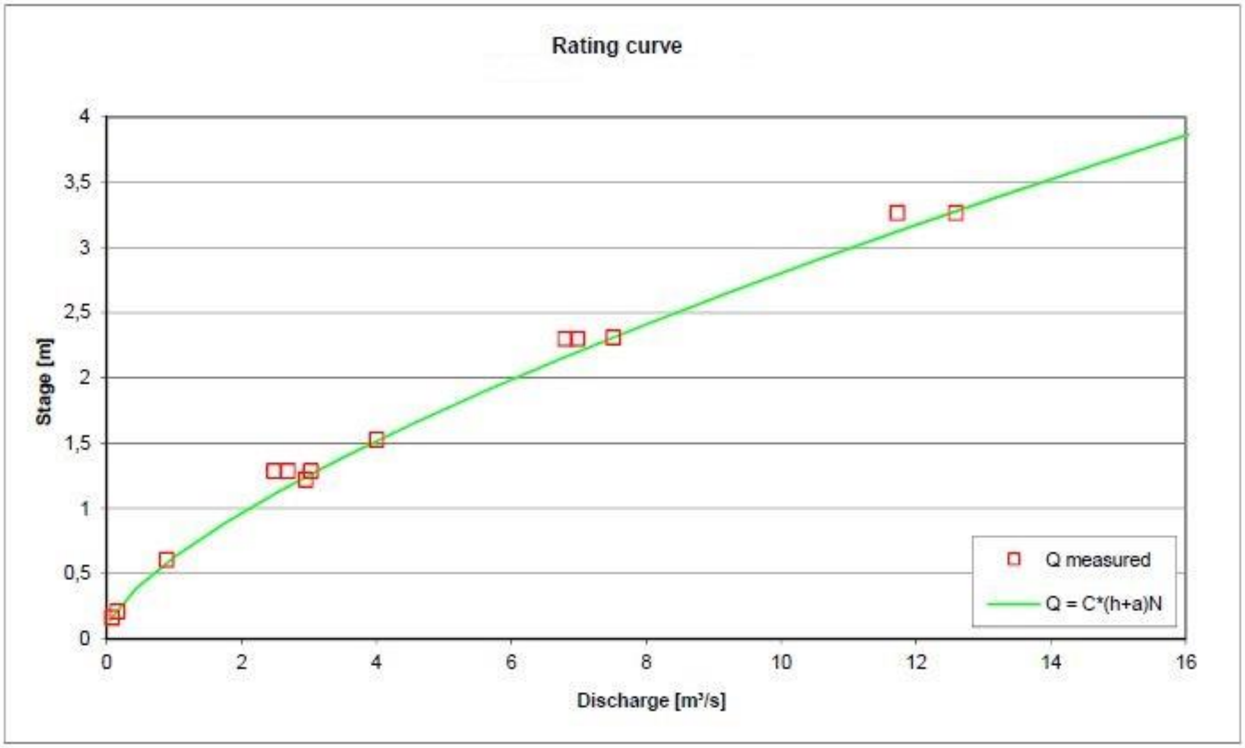
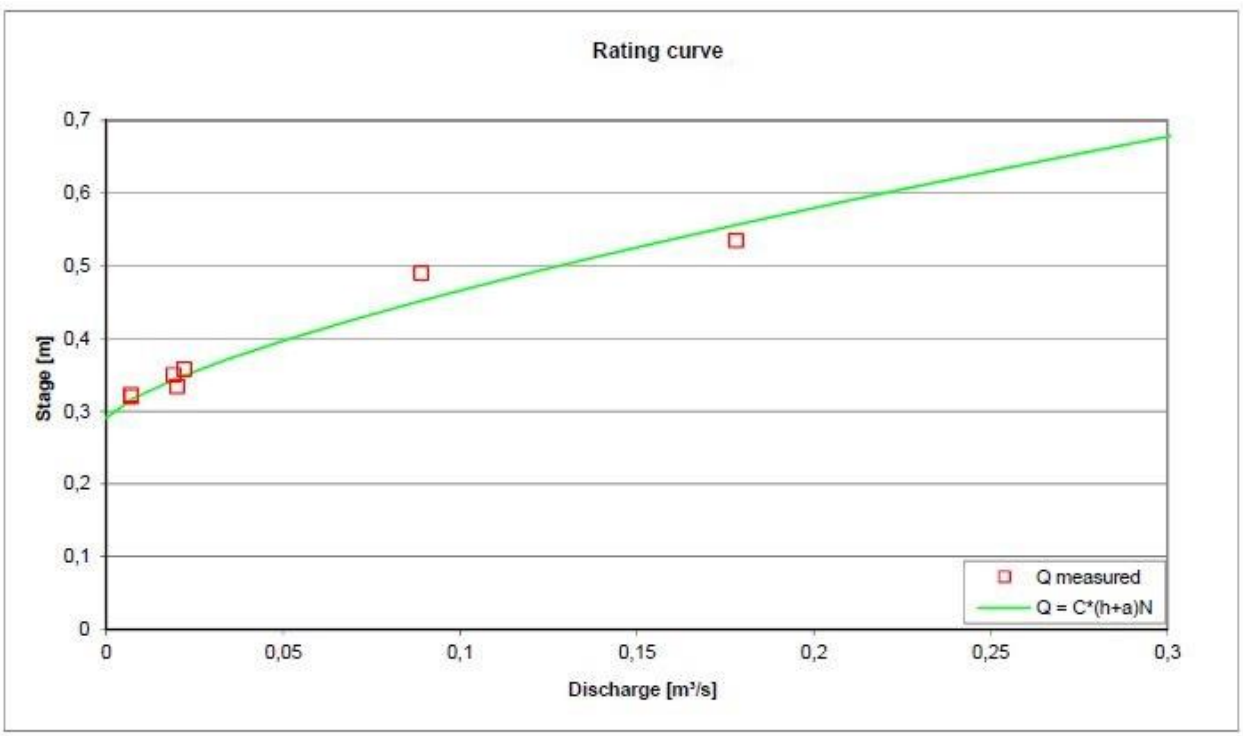


Figure D-18: Rating curve for Culvert site 15



Appendix E: Tabulated results of Scenarios and recommended solutions

Table E-1: Results on Scenarios and Options at U/S and D/S of the structure

Columns	Col 1	Col 2	Col 3	Col 4	Col 5
Scenarios / Culvert sites	**Key Findings**			**Sediment**	**Scenarios and options for the U/S and D/S of the culverts**
	(HEC-HMS) Discharge (m^3)	**(HEC-RAS) Depth of flow (m)**	**(HEC-RAS) Scour Velocity (m/s)**		
Culvert site 1	Max = 2.88 Ave = 1.96 Min = 1.04	Max = 1.53 Ave = 1.06 Min = 0.58	Max = 4.33 Ave = 3.26 Min = 2.18	Smooth flat bed representing soft clay sediment	Artificial expanding the U/S channel by 0.5m (0.25m each bank) and applying stone ripraps at each expanded banks will reduce the depth of flow to 0.98m then after the erosion reduces by 25% at D/S and the water to be harvested increases by 15%.
Culvert site 2	Max = 3.55 Ave = 2.86 Min = 2.16	Max = 1.37 Ave = 0.93 Min = 0.49	Max = 5.18 Ave = 4.55 Min = 3.92	uniform gravel	Scour at Culvert Inlets and Outlets as Influenced by the Turbulent Flow Structure so narrowing only the U/S channel by 0.3 m using stone riprap will reduce the turbulent flow to decrease scouring at D/S.
Culvert site 3	Max = 3.07 Ave = 2.53 Min = 1.98	Max = 1.34 Ave = 0.89 Min = 0.44	Max = 3.00 Ave = 2.58 Min = 2.15	non-cohesive gradations	The predicted and existing flow depth of the culvert is nearly the same, so the localized Scour hole on the culverts continues naturally at culvert outlets.

Culvert site 4	Max = 1.84 Ave = 1.35 Min = 0.85	Max = 1.82 Ave = 1.23 Min = 0.63	Max = 3.25 Ave = 2.43 Min = 1.61	uniform various sizes of sandy soils (angular)	Significant percentage of the cross-sectional flow had stream wise velocity lower than mean bulk velocity, so the channel continues safe.
Culvert site 5	Max = 2.49 Ave = 1.73 Min = 0.96	Max = 2.02 Ave = 1.34 Min = 0.65	Max = 5.22 Ave = 3.13 Min = 1.04	Small sizes of rounded and angular sandy gravels.	Velocity distributions and sediment transport is high through the two barrel box Culverts so the side banks are to be changed by reducing 0.7m both side (0.35m each) to reduce the turbulent velocity by 23% and sediment transport.
Culvert site 6	Max = 2.14 Ave = 1.52 Min = 0.90	Max = 1.34 Ave = 0.87 Min = 0.40	Max = 2.24 Ave = 1.79 Min = 1.34	Sediment size distributions and a well-graded mixture to test particle interlock	Water surface slopes and depth of flow have good correlation here with velocity distributions so the hydraulic flow is safe.
Culvert site 7	Max = 2.19 Ave = 1.56 Min = 0.92	Max = 1.37 Ave = 0.94 Min = 0.51	Max = 2.88 Ave = 1.96 Min = 1.04	Limited to sand and gravel sizes	Single-barrel culvert head-discharge Submerged inlet Condition @ U/S, so this culvert needs to expand the u/s channel by 1m to distribute the flow of water and then the depth decreases to 0.63m from the submergence.

Culvert site 8	Max = 4.65 Ave = 3.56 Min = 2.46	Max = 1.75 Ave = 1.18 Min = 0.60	Max = 4.31 Ave = 3.55 Min = 2.78	Fixed gravel roughness (not fixed at outlet)	Filled, the upstream and downstream degradation 0.8m from RB 0.5m from LB 0.75m from LB, expanding by 2m both the RB and LB of the upstream part using stone riprap gabion and expand downstream by 1.5m to have stagnant water at the culvert with safe hydraulic phenomena.
Culvert site 9	Max = 3.15 Ave = 2.63 Min = 2.10	Max = 1.60 Ave = 1.08 Min = 0.55	Max = 5.11 Ave = 4.84 Min = 4.56	Flat plains and irregular plains; with most of clay and sandy flat terrain is still covered by standing water.	The bed of inlet and outlet culvert is scoured with high flow velocity and the flow of depth, here distribute the water by expanding only the U/s channel by 1.2m will reduce velocity and avoid bottomless culvert bed geometries and Scour protection measures.
Culvert site 10	Max = 2.05 Ave = 1.47 Min = 0.88	Max = 1.19 Ave = 0.79 Min = 0.38	Max = 2.67 Ave = 2.16 Min = 1.64	Flat to gently rolling plains composed of only sandy soils.	Mean velocity Turbulence is moderate, then this culvert is faced to scour culvert hole geometry, so the culvert needs small improvement at u/s side increasing the width of u/s cannel by 0.75m.

Culvert site 11	Max = 2.63 Ave = 1.82 Min = 1.00	Max = 1.48 Ave = 1.00 Min = 0.51	Max = 3.08 Ave = 2.88 Min = 2.68	Nearly level to gently rolling silt clays glaciated till plains coming from hilly uplands.	Filled sediment at culvert bed, scour hole at US end of Culvert and scour geometry creates drop height. The culvert site 11 needs to reduce 0.5m from RB and expand 1.2m from LB to reduce scour at concave side to balance with convex side.
Culvert site 12	Max = 3.11 Ave = 2.60 Min = 2.08	Max = 1.53 Ave = 1.04 Min = 0.55	Max = 3.43 Ave = 3.21 Min = 2.99	Composed of rocky and sandy soils Nearly level to gently rolling.	Partially sediment filled at culvert u/s, sediment front slowly moving into culvert at high flows with slow flow velocity to only change the d/s culvert bed with the dissipater concrete.
Culvert site 13	Max = 2.58 Ave = 1.78 Min = 0.98	Max = 1.46 Ave = 0.98 Min = 0.50	Max = 3.08 Ave = 2.88 Min = 2.68	Extensive sandy outwash soils and aggregates	Significant scour at upstream end of culvert (to last culvert structure); 1/4 of the sediment filled from downstream end; only expand the d/s channel by 0.5m.
Culvert site 14	Max = 2.17 Ave = 1.56 Min = 0.94	Max = 1.30 Ave = 0.89 Min = 0.48	Max = 2.67 Ave = 2.16 Min = 1.64	Large till plains and with soft soils	Scour upstream end of culvert ; ½ filled from downstream end.
Culvert site 15	Max = 2.07 Ave = 1.49 Min = 0.90	Max = 1.25 Ave = 0.8 Min = 0.40	Max = 3.05 Ave = 2.13 Min = 1.21	Flat to gently rolling plains	Significant scour at upstream end of culvert (to end Culvert structure); ¾ filled from downstream end.

Table E-2: Recommended water harvesting solutions for the Scenarios

Solutions / Culvert sites	Culverts	River and Side Channel	Energy dissipater and Conveyances	Harvesting Solutions
Culvert site 1	Circular culvert with no overbanking @ both sides but high erosion and deposition at the concave side of the culvert	Steep, short river with meandering shape, short left side channel	Stone ripraps both @ inlet and outlet of the culvert to minimize erosion	Deep trench's, hand dug wells and well organized sand harvesting reservoirs
Culvert site 2	Circular culvert with no overbanking all sides	Steep, Long river with meandering shape, short left side channel	Drop structures near the culvert and stone ripraps.	Deep trench's, hand dug wells and well organized for harvesting reservoirs.
Culvert site 3	Box culvert with over banking @ both sides left and right way	Gentle, Long river with meandering shape	Stone ripraps near the culvert.	Deep trenches
Culvert site 4	Circular culvert with no overbanking @ both sides but high erosion and deposition.	Gentle, short river with uniform shape	Short drop structures using grass lining	Sand and water harvesting is good @ 200m d/s of the culvert.
Culvert site 5	Irish bridge with overbanking @ top but safe flow.	Steep, Short river with miscellaneous shape	Drop structures using stones and gabions.	Small harvesting potential but good for sand harvesting
Culvert site 6	Box culvert with overbanking @ both side	Gentle, long river Non Uniform shape	Stone ripraps both @ inlet and outlet of the culvert to minimize suspension	Hand dug wells but good for water harvesting
Culvert site 7	Circular culvert with no overbanking	Gentle, Short river with Non Uniform shape	Strong bondage @ both road side connections to control erosion of over banking.	Small amount of erosion and small harvesting potential

Culvert site 8	Irish bridge with overbanking @ top but flow with both sides	Steep, Long river with mindering shape, short left side channel	Arch shape guide bank stone riprap both @ inlet and outlet of the culvert.	Good d/s harvesting option
Culvert site 9	Circular culvert with no overbanking all sides	Steep, Short river with miscellaneous shape	High bed culvert control using massive stone riprap with gabions.	Hand dug wells
Culvert site 10	Box culvert with over banking @ only left side	Gentle, long river Non Uniform shape	Stone riprap @ the rotational flow with gabions.	Both deep trenches and hand dug wells.
Culvert site 11	Box culvert with overbanking @ both side	Gentle, long river Non Uniform shape	Stone ripraps only	Deep trenches
Culvert site 12	Circular culvert with overbanking @ all sides	Gentle, long river Non Uniform shape	Rotational flow is high that is why stone riprap @ the concave side	Small amount of erosion and good harvesting potential
Culvert site 13	Circular culvert with good flow	Gentle, long river Uniform shape	High bed culvert control using massive stone riprap with gabions	Artificial ponds for small harvesting potential
Culvert site 14	Box culvert with overbanking @ one side and erosion from one side deposition on the other side	Steep, Short river with miscellaneous shape	Drop structures using stones and gabions	Hand dug wells, Artificial pond and huge hand dug wells
Culvert site 15	Irish bridge with no overbanking and no side banking	Gentle, Short river with Non Uniform shape	Short drop structures using grass lining	Deep trenches

Table E-3: Recommended Sediment Reduction solutions for the Scenarios

Solutions / Culvert sites	Culverts		Catchments Slope	Energy dissipater and Conveyance allocation	Sediment reduction Solutions
	Before (U/S)	After (D/S)			
Culvert site 1	Erosion is High	Erosion is High	Steep Slope	@ U/S & D/S	Gabions @ the river
Culvert site 2	Erosion is High	Erosion is High	Medium Slope	D/S only	D/s ripraps
Culvert site 3	Acceptable Erosion	Acceptable Erosion	Medium Slope	D/S only	D/s ripraps
Culvert site 4	Erosion is Small	Erosion is Small	Gentle Slope	No dissipater only conveyance	Grass conveyance
Culvert site 5	Acceptable Erosion	Acceptable Erosion	Steep Slope	U/S only	U/S Small ponds
Culvert site 6	Erosion is Small	Erosion is Small	Medium Slope	D/S only	D/S Trenches
Culvert site 7	Erosion is Small	Erosion is Small	Gentle Slope	U/S only	U/S Small ponds
Culvert site 8	Erosion is High	Erosion is High	Steep Slope	@ U/S & D/S	Gabions @ the river
Culvert site 9	Erosion is High	Erosion is High	Steep Slope	@ U/S & D/S	Gabions @ the river
Culvert site 10	Acceptable Erosion	Acceptable Erosion	Gentle Slope	D/S only	D/s ripraps
Culvert site 11	Acceptable Erosion	Acceptable Erosion	Gentle Slope	D/S only	D/S Trenches
Culvert site 12	Acceptable Erosion	Acceptable Erosion	Steep Slope	U/S only	U/S Small ponds
Culvert site 13	Acceptable Erosion	Acceptable Erosion	Medium Slope	D/S only	D/s ripraps
Culvert site 14	Erosion is High	Erosion is High	Steep Slope	@ U/S & D/S	Gabions @ the river
Culvert site 15	Acceptable Erosion	Acceptable Erosion	Medium Slope	@ U/S & D/S	Gabions @ the river